

1. Report No. FHWA/TX-05/9-580/589-1	2. Government Accession No.	3. Recipient's Catalog No.	
4. Title and Subtitle PRODUCTION AND QUALITY CONTROL OF HIGH PERFORMANCE CONCRETE IN TEXAS BRIDGE STRUCTURES		5. Report Date December 1998	
		6. Performing Organization Code	
7. Author(s) John J. Myers and Ramón L. Carrasquillo		8. Performing Organization Report No. 9-580/589-1	
9. Performing Organization Name and Address Center for Transportation Research The University of Texas at Austin 3208 Red River, Suite 200 Austin, TX 78705-2650		10. Work Unit No. (TRAIS)	
		11. Contract or Grant No. 9-580 and 9-589	
12. Sponsoring Agency Name and Address Texas Department of Transportation Research and Technology Implementation Office P.O. Box 5080 Austin, TX 78763-5080		13. Type of Report and Period Covered Technical Report (9/96 — 8/97)	
		14. Sponsoring Agency Code	
15. Supplementary Notes Project conducted in cooperation with the Federal Highway Administration.			
16. Abstract To demonstrate the suitability of using high performance concrete (HPC) in highway structures, the FHWA initiated a series of projects in 1993 that included the complete incorporation of HPC — from design to long-term monitoring of the bridges in service. The report focuses on the design and construction of the first two of these projects in the United States, namely, the Louetta Road Overpass in Houston, Texas (Research Study 9-580), and the North Concho River Overpass in San Angelo, Texas (Research Study 9-589). This report summarizes the observations of, and conclusions obtained from, both laboratory and field investigations related to these two HPC bridges. It investigates HPC properties, mix design, applicability of current design requirements, applicability of current construction practices, predictability of short- and long-term performance, and the requirements for quality control and quality assurance testing and inspection. The performance-related test results of both mechanical and material durability testing of over 40,000 specimens from laboratory and plant-produced concrete is presented, including the statistical analysis. Related research findings pertaining to the early-age curing aspects of the prestressed/precast beams, the use of the maturity method for cast-in-place decks, and to a high performance concrete permeability correlation study (AASHTO T259 and AASHTO T277) are also presented.			
17. Key Words High performance concrete bridges, high strength concrete, quality control/quality assurance, compressive strength, elastic modulus, flexural strength, splitting tensile strength, concrete durability, air entrainment, freeze-thaw resistance, abrasion resistance, scaling resistance, creep, shrinkage		18. Distribution Statement No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161.	
19. Security Classif. (of report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of pages 564	22. Price

**PRODUCTION AND QUALITY CONTROL OF HIGH PERFORMANCE CONCRETE
IN TEXAS BRIDGE STRUCTURES**

by
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and
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Report Number 9-580/589-1

Project Numbers 9-580 and 9-589

Project title: *High Performance Concrete for Bridges*

Conducted for the

TEXAS DEPARTMENT OF TRANSPORTATION

in cooperation with the

**U.S. DEPARTMENT OF TRANSPORTATION
Federal Highway Administration**

by the

**CENTER FOR TRANSPORTATION RESEARCH
Bureau of Engineering Research
THE UNIVERSITY OF TEXAS AT AUSTIN**

December 1998

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ACKNOWLEDGMENTS

The authors wish to express their sincere gratitude to Mary Lou Ralls, P.E., (BRG), TxDOT project director for this study. We also thank the other members of the project monitoring committee, which included A. Cohen (DES), W. R. Cox (BRG), D. Harley (FHWA), G.D. Lankes (CST), L. Lawrence (CST), L. M. Wolf (BRG), Tom Yarbrough (RTI), and D. McDonnold (DES).

Research performed in cooperation with the Texas Department of Transportation and the U.S. Department of Transportation, Federal Highway Administration.

IMPLEMENTATION

This report summarizes the observations of, and conclusions obtained from, both laboratory and field investigations related to two high performance concrete (HPC) bridges. It addresses various aspects of HPC involving concrete properties, mix design, applicability of current design requirements, applicability of current construction practices, predictability of short- and long-term performance, and the requirements for quality control and quality assurance testing and inspection.

This research study encompassed several objectives. The first objective was to develop several HPC mix designs in the laboratory using local material for use in various components of two HPC bridges and to determine the range of engineering properties of the HPC. This included performance-related testing such as compressive strength with time, elastic modulus, splitting tensile strength,

permeability, freeze-thaw resistance, abrasion resistance, and scaling resistance. The use of local materials was stressed in order to develop economical mix designs. The objective of the FHWA was to create a database with these mechanical and material properties for HPC. In addition, the study examined the variability of HPC produced at local precast and batch plants as part of the concrete produced for the structure. Plant-produced concrete was also compared to concrete produced in the laboratory to better estimate associated loss in strength when a mix design developed in the lab is selected for use in the field. Several curing regimes were investigated, including moist-cured, member-cured, match-cured, and site-cured, to determine the variation between moist-cured cylinders and concrete that closely simulates concrete in the structure.

Another primary objective was to determine the effect of elevated heat curing temperatures at early ages on the two primary supporting element shapes. This included the Texas U-Beam and the standard AASHTO Type IV I-beam shape. A commercially available match-curing system was selected to aid in this study and was evaluated as a quality control/quality assurance (QC/QA) tool to be used in the precast industry.

A third objective was to develop a QC/QA testing program to evaluate the performance-related test results throughout the investigation and to provide recommendations for use in performance-based specifications. The study also examines the use of prescriptive-based specifications versus performance-based specifications from a durability standpoint.

Secondary objectives were addressed through the use of side studies or investigations associated with HPC and the two HPC bridges. One of these included evaluating the use of maturity method for HPC and conventional concrete in cast-in-place decks. A second study included a correlation study of AASHTO T259 and AASHTO T277 for HPC. Finally, a third study entitled "TxDOT Replacement Study for Non-HRWR HPC Mixes" examined fly ash replacement and minimum cement contents currently specified in many prescriptive specifications.

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CHAPTER 1. INTRODUCTION

1.1 Historical Perspective on the Evolution of Bridge Systems

1.1.1 The Evolution of Stone and Timber Bridges

Prior to the nineteenth century, the two most widely used and important materials selected for bridge construction were stone and timber. Bridges built of these materials date as far back as prehistoric times [102]. In Roman times, the arch that characterized stone bridges was the main spanning element, with piers used as intermediate supports. While many of these stone bridges were somewhat massive, the majority of master bridge builders or designers of the period designed these arch bridges in full compression. The fact that not one portion of the bridge was designed in tension partially explains why many of these structures remain standing to this day. Whether the master bridge builders realized it or not, the durability-related problems that may have occurred or been initiated in tension zones were avoided. Furthermore, because the bridges were made only of stone or unreinforced materials, there were no durability-related concerns associated with corrosion of reinforcing steels.

The simplest timber bridges of antiquity were little more than wooden beams supported on stone or masonry piers. Arches and simple inclined-leg frames were used for the construction of longer spans until the master Swiss builder, Hans Ulrich Grubenmann (1709–1783), designed intricate compound inclined-leg framed systems with spans up to 70 m (230 ft).

1.1.2 The Evolution of Iron and Wrought Iron (Steel) Bridges

With the industrial revolution came the use of materials such as industrialized irons preeminently used for railway bridge construction. The first iron bridge that crossed the Severn River in Coalbrookdale, England, was built in 1779 by Abraham Darby (1750–1791). The experience acquired by bridge builders in the construction of stone and timber bridges was of little use in the design of iron bridges because the advantages of iron could only be fully exploited by using radically different structural systems. In addition, the brittleness of the material limited the use of iron to compression members only.

The development of wrought iron and its ultimate successor, steel, together with structural statics, made it possible to design and construct a great variety of truss-based bridges spanning up to 500 m (1650 ft). Ultimately, industrialized iron transformed the hanging cable, which dated back to prehistoric times, into the modern suspension bridges. Suspension bridges date back to the early

nineteenth century in the United States, where the world's first wire cable bridge was built in 1816. American leadership in suspension bridge design and construction was established by John Roebling (1806–1869) and culminated in arguably one of the two most famous bridges in the United States, the Brooklyn Bridge. The bridge spans 488 m (1,600 ft) over the East River in New York and was without precedent at the time of its completion in 1883.

1.1.3 The Evolution of Reinforced Concrete Bridges

Iron and steel were not unqualified successes. Their strength, light weight, and speed of construction were offset by high maintenance costs and a high live-load-to-dead-load ratio. Furthermore, many nineteenth century railway bridges in which iron and steel found widespread use, had to be strengthened or reinforced owing to the development of heavier, faster trains. These disadvantages helped to bring about the application of another new material, namely, portland cement concrete. Compared to metals, concrete was viewed as maintenance free and afforded a reduced live-load-to-dead-load ratio. The first plain concrete bridges were arches built in Spain and France in the 1860s and resembled many of the masonry bridges built previously.

The discovery of reinforced concrete can be traced back to the work of American researcher Thaddeus Hyatt (1816–1901), who conducted investigations of reinforced concrete beams in 1850. However, it was Joseph Monier (1823–1906) of France who patented wire reinforced concrete in 1867 — an innovation that led to the spread of reinforced concrete throughout Europe. In 1893 François Hennebique (1842–1931) then introduced a complete system for construction of steel wire reinforced monolithic composite members. The reinforced bridges built by Hennebique were considerably cheaper than traditional masonry, stone, or iron bridges. While many early reinforced concrete bridges were efficient, their aesthetic beauty did not compete with many of the masonry, stone, or iron bridges until Robert Maillart (1872–1940) transformed reinforced concrete bridges into works of art that were not only aesthetically pleasing, but also efficient and economic. The world took notice and the use of reinforced concrete became more widespread. However, the single most important innovation in the history of reinforced concrete technology was the development of prestressed concrete.

1.1.4 The Evolution of Prestressed Concrete Bridges

The earliest investigations of prestressed concrete beams were conducted in the nineteenth century [102]. In 1888, a system was patented by German engineer W. Döring for the construction of slabs, planks, and beams, which were prestressed with wires to reduce crack widths. Prestress saw no further practical application until the efforts of Freyssinet of France, who worked on the

development of stressing systems with high-strength wires from 1926 to 1936. In Aue, Germany, the first prestressed concrete bridge was built in 1937. The designer, Dischinger, experienced significant prestress losses and, hence, cracking and large deformations because of his use of unbonded low tensile strength prestressing steel. His experience led him into investigations of prestress losses caused by creep and shrinkage.

By the end of the 1940s, many new prestressing systems were developed, including the Dywidag and Leoba system. However, all were based to some extent on Freyssinet's system. Over the next decade, many prestressed concrete bridges would be built throughout Europe. In 1949/1950, Walnut Lane Bridge, shown in Figure 1. 1, was built in Philadelphia, Pennsylvania.



Figure 1.1: Walnut Lane Bridge in Philadelphia, Pennsylvania [103]

The Walnut Lane Bridge was the first prestressed concrete bridge built in North America and had a specified concrete strength of 37. 2 MPa (5,400 psi) that was considered high strength at the time. Concrete producers at that time consistently produced concrete in the 20. 7 to 27. 6 MPa (3,000 to 4,000 psi) range. Was this the first high performance concrete (HPC) bridge? Many believe so.

As construction and labor costs rose in the late 1950s and early 1960s, more complicated systems with arches and labor-intensive falsework became less competitive and were only built in

exceptional cases. Precast/prestressed girders made rapidly built and economical short-span bridges possible. Precast segmental bridges with cantilever construction were built in the 1960s, making long-span systems in concrete once again competitive with short-span concrete bridges. Developments of the 1970s brought about the precast segmental box girder systems and the cable-stayed systems using a balanced cantilever method for economical construction. Along the way, all of these developments have continued to stimulate the advancement of concrete technology and optimization of concrete for efficient and cost-effective use in bridge structures.

1.2 FHWA Current Level of Bridge Performance in the United States

Before exploring the evolution and advantages of HPC in bridge structures, let us examine the performance of bridge structures in the United States. The Federal Highway Administration (FHWA) currently maintains a National Bridge Inventory (NBI) database that includes all public highway bridges. The data is submitted by states to the FHWA annually. The database includes prescriptive data on all bridges in the United States having a span length greater than 6.1 m (20 ft). This includes information such as span length and structure material, as well as the condition and appraisal rating of the bridge substructure and superstructure. The NBI includes 576,396 bridges to date. Of these, 471,430 have been classified as bridges, 104,886 have been classified as culverts, and 80 have been classified as tunnels. For the purposes of the data presented in Figures 1.2 through 1.4, *structurally deficient* has been defined by the following FHWA criteria:

1. Condition Rating ≤ 4
 - Deck
 - Superstructure
 - Substructure
2. Appraisal Rating ≤ 2
 - Structural Evaluation
 - Waterway Adequacy

Both of these rating scales are based on a numerical classification ranging between 0 to 10, with 0 being the worst performance rating and 10 being the best performance rating.

Figure 1.2 illustrates the structurally deficient bridges in the United States by mainspan superstructure material dating back to bridges constructed in 1950. The year 1950 was selected as a starting point because 1950 was the year the first prestressed concrete bridge was built in the United States (Figure 1.1).

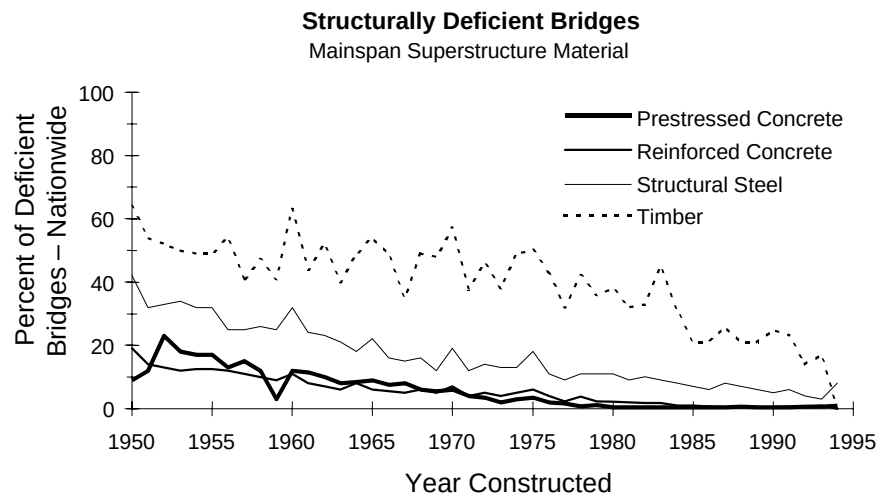


Figure 1.2: Structurally Deficient Bridges in the United States [104]

Figure 1.3 illustrates the structurally deficient bridge decks in the United States by deck material dating back to bridges constructed in 1950. Figure 1.4 illustrates the performance of bridge decks since 1950 by span condition. The following subsections discuss the performance of bridges in the United States based on material type.

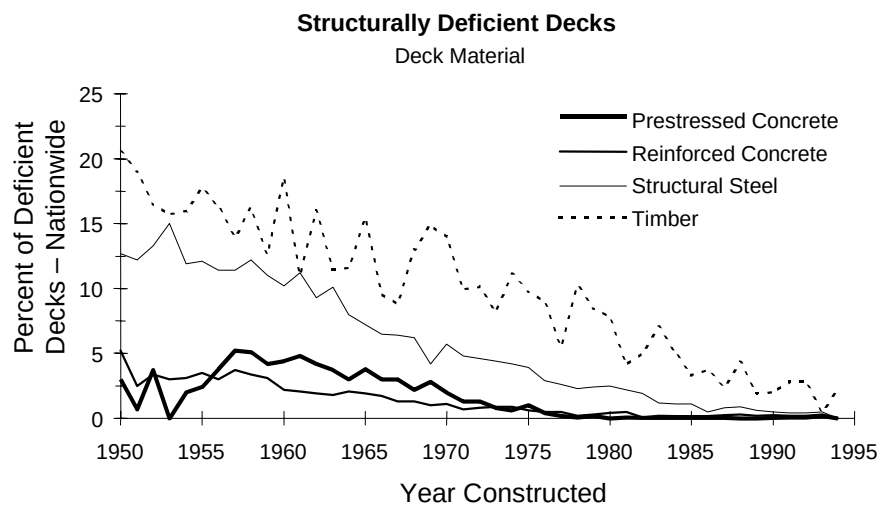


Figure 1.3: Structurally Deficient Decks in the United States [104]

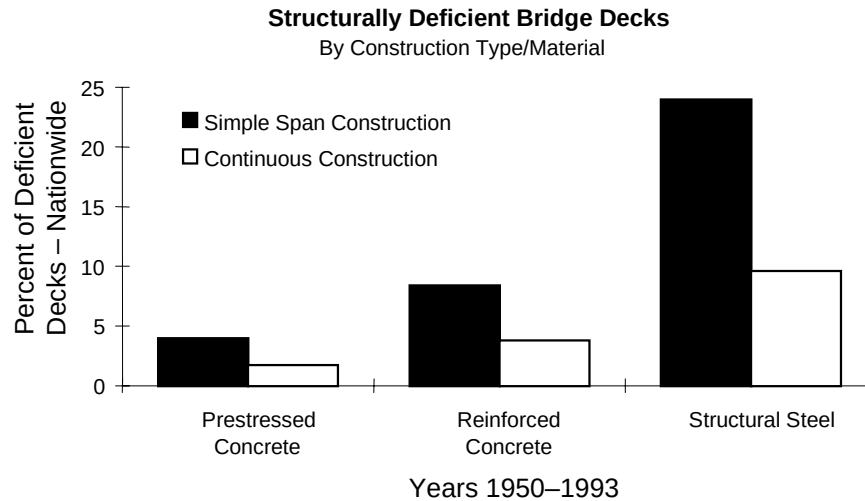


Figure 1.4: Structurally Deficient Decks in the United States by Span Condition [104]

1.2.1 Concrete Bridges

Incorporated within the database are both reinforced and prestressed concrete bridges. Both systems have demonstrated satisfactory performance for bridges based on a 40-year projected design life that many of the structures have been designed for.

1.2.1.1 Reinforced Concrete. Approximately 15 percent of mainspan structures constructed of reinforced concrete 40 years ago were found to be deficient as illustrated in Figure 1. 2. Some of the problems may have been associated with poor construction techniques or poor quality control and assurance. Overall, reinforced concrete decks have performed well when compared to other materials, as shown in Figure 1.3. As illustrated in Figure 1.4, continuous construction has demonstrated better performance when compared to simple span construction owing to the additional joints associated with simple span construction. Any time joints can be eliminated, there is one less avenue for chlorides and other harmful chemicals to ingress into the concrete, and better durability performance can be anticipated.

1.2.1.2 Prestressed Concrete. Approximately 18 percent of the mainspan structures constructed of prestressed concrete 40 years ago were found to be deficient, as illustrated Figure 1.2. Prestressed concrete decks have exhibited good performance (Figures 1.3 and 1.4). Prestressing applied to the decks limits the crack width of any surface cracks and increases the time for chlorides and other surface chemicals to ingress into the concrete. On the whole, concrete is still clearly the most durable bridge material to date.

1.2.2 Structural Steel Bridges

Over the years, structural steel bridges have provided several advantages, most notably their reduced construction time, long-span capability (Figures 1.5 and 1.6), and ability to conform within tight restrictions or design parameters. Their design flexibility allows for a tight radius through curved structural steel girders that would not be feasible or possible with concrete. Unfortunately, structural steel comes with a price, most notably the high cost of maintenance that is required to combat corrosion-related problems. Mainspan structures constructed of structural steel have been rated as structurally deficient at almost twice the rate of concrete bridges, as illustrated in Figure 1.2. Structural steel decks have also exhibited poor performance over the years owing to susceptibility to corrosion-related problems. The combination of a structural steel girder with a cast-in-place concrete deck, as shown in Figure 1.4, has exhibited the best performance with reference to structural steel bridges.



Figure 1.5: Structural Steel Truss Bridge in Buchanan, Texas



Figure 1.6: Structural Steel Girder Bridge with Composite Concrete Deck in Suwanee, Georgia

1.2.3 Timber Bridges

In general, timber bridges have been the poorest performing bridge structures to date. Over 40 percent of the mainspan superstructures constructed of timber that are more than 10 years old were found to be deficient, as illustrated in Figure 1.2. Even when the structure and deck surfaces are protected from environmental conditions, as illustrated in Figure 1.7, maintenance costs are high. The low load capacity of timber decks also presents modern-day problems for large and heavy vehicles. Perhaps the greatest value of many timber bridges is their aesthetic beauty and their reminder of our heritage.



Figure 1.7: Historic Covered Bridge Located in Reading, Pennsylvania

1.2.4 Other Bridges

Recent technological advances have led to the development of composites, such as carbon fiber reinforced plastic (carbon FRP), for use in infrastructure construction [105]. The intriguing advantage of this system is that the carbon FRP composite system is corrosion resistant. There has been a durability concern for both structural steel and reinforced concrete bridges. To date, several prototype pedestrian bridges [105] have been constructed in the United States to investigate the use of composites within the infrastructure.

1.3 The Evolution and Advantages of High Performance Concrete Bridges

Concrete technology has continued to advance throughout the years in order to meet the demands of designers and innovative structural systems. The definition of HPC has changed over the years as well. Initially, designers were concerned only with strength, and the term *high-strength concrete* (HSC) was widely used. In the 1950s, 34.5 MPa (5,000 psi) was considered to be HSC, while in the 1960s it was 41.4 to 48.3 MPa (6,000 to 7,000 psi). With the development of high range water reducers (HRWRs) and mineral admixtures, concrete strengths in excess of 69 MPa (10,000 psi) are readily available and provide enhanced properties (other than strength). Today, HPC not only encompasses strength, but also such material and durability properties as permeability, freeze-thaw resistance, abrasion resistance, and scaling resistance.

For many years, the driving force behind the development of HPC was economy, especially



**Figure 1.8: Interfirst Plaza
Located in Dallas, Texas [102]**

with respect to tall buildings such as represented by the Interfirst Plaza in Dallas, Texas, which was built in 1983 and is shown in Figure 1.8. In the 1980s, several small bridges were built in the state of Washington with 48.3 MPa (7,000 psi) concrete followed by the construction in 1990 of the Braker Lane Bridge in Austin, Texas, which used concrete with a specified design strength of 86.2 MPa (9,600 psi).

The benefits of HPC in the bridge industry have been well documented over the years. The higher strength associated with many HPC mix designs translates into longer spans with a reduced number of supports. The cost of the substructure on most bridges is approximately 50 percent of the total cost of the bridge (on average nationally). Therefore, if one support can be eliminated over five spans, for example, a 20 percent reduction in

substructure costs or 10 percent reduction in the total cost is attained. The higher strength can also be used to achieve wider beam spacings and, consequently, a reduced number of supporting members. These improvements were demonstrated in the construction of the North Concho River US 87 and S.O. RR Overpass in San Angelo, Texas, one of the bridges associated with this research study. It was demonstrated in the North Concho River US 87 and S.O. RR Overpass, illustrated in Figure 1.9, that the number of girders could be reduced from seven (Span 1— westbound bridge), using conventional concrete strengths currently produced in the precast industry, to four (Span 1— eastbound bridge), using high-strength/high performance concrete (HS/HPC). This translates not only to fewer girders to fabricate, but also to fewer precast panels and girders that the contractor has to transport and erect.

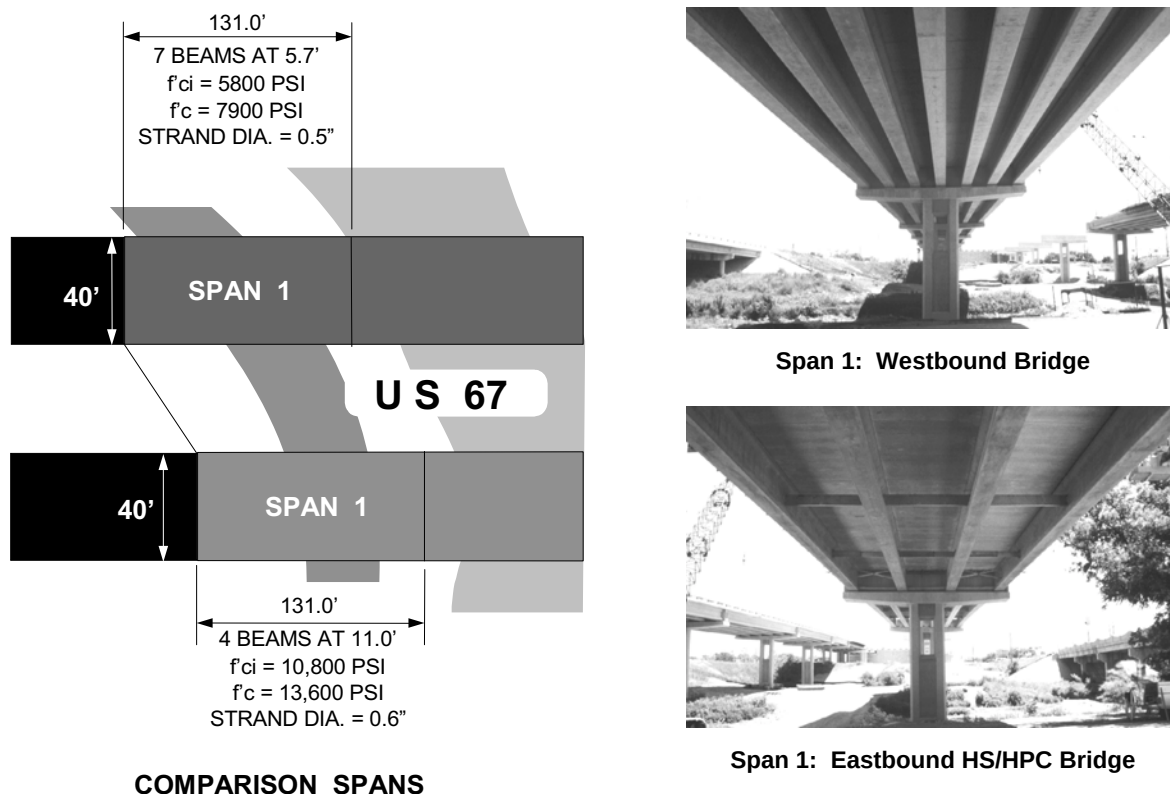


**Figure 1.9: North Concho River US 87 & S. O. RR Overpass
Comparison Spans of HS/HPC Girders and Conventional Strength Girders**

The added benefits include reduced complexity and construction time for the contractor and, thus, additional cost savings. In addition to the benefits of higher strength, HPC also provides increased durability compared to conventional concretes, which translates to reduced maintenance

costs and longer-life structures. These qualities are particularly attractive for use within the United States, where many of the bridges in our infrastructure are deficient. The typical design life of today's conventional concrete bridges is estimated at 40 years, while the design life of the HPC bridges presented in this study has been estimated at 75 to 100 years. This is where the real long-term cost savings and advantages lie.

1.4 Problem Statement

During the last 10 years, concrete having compressive strengths well in excess of 69 MPa (10,000 psi) has been successfully produced using conventional materials and concrete production techniques. Much research has been completed at The University of Texas at Austin (UT) and elsewhere regarding specific aspects of material selection, mix proportioning, concrete production, structural performance, and reinforcing steel detailing. However, little or no information is available that documents the complete construction process from design to service of a highway bridge structure fully utilizing the benefits of HPC. Based on the results of several parametric and theoretical studies, it is anticipated that the use of HPC in highway bridge structures will yield significant economical savings. This includes improved durability and impermeability of HPC, resulting in longer life and lower maintenance costs that are particularly relevant to bridge decks. Also, the utilization of HPC for prestressed concrete bridge girders results in longer spans and greater beam spacing. This translates to lighter superstructures and fewer supports in the substructure. Therefore, the overall benefits obtained with HPC are economical savings and greater flexibility for designers.

This report discusses the work performed by researchers at the Center for Transportation Research (CTR) of UT on two research projects cosponsored by the Texas Department of Transportation (TxDOT) and the FHWA. Project 9-580, "Design and Construction of Extra-High-Strength Concrete Bridges," and Project 9-589, "High Performance Concrete Bridges," are both part of the *FHWA HPC Showcase Program* that was designed for the purpose of advancing the use of HPC in highway bridges.

The research team was the same for both projects. The TxDOT project director was Mary Lou Ralls, P. E. The CTR principal investigators were Ramón L. Carrasquillo, Ph.D., P. E., Ned H. Burns, Ph.D., P. E., and David W. Fowler, Ph.D., P. E. The materials research was conducted by the Construction Materials Research Group (CMRG), and the large-scale structural component testing was conducted at the Phil M. Ferguson Structural Engineering Laboratory (FSEL). Both facilities are located at the J. J. Pickle Research Campus (PRC) of UT.

To demonstrate the suitability of HPC for use in highway structures, the FHWA initiated a series of projects in 1993 that included the complete incorporation of HPC from design to long-term

monitoring of the bridges in service. This report focuses on the design and construction of the first two of these projects in the United States, namely, the Louetta Road Overpass in Houston, Texas, and the North Concho River Overpass in San Angelo, Texas. The Louetta Road Overpass project included the use of pretensioned precast U-beams, while the North Concho River Overpass project included the use of pretensioned, precast American Association of State Highway and Transportation Officials (AASHTO) standard Type IV beams. These beams required very high initial prestressing forces and the use of 15.2-mm (0.6-in.) prestressing strands to take full advantage of the high performance concrete. The high initial prestressing forces required high early release strengths of 74.5 MPa (10,800 psi) and 56-day design strengths of 101.4 MPa (14,700 psi). The designers (TxDOT) also required a high modulus of elasticity at release and long-term 41.4 GPa (6,000 ksi) to meet serviceability requirements for the structures. Each bridge component had specific durability and/or mechanical property requirements similar to those of the beams. HPC mix designs were developed and optimized in the laboratory using locally available materials to produce the most economical mix designs for use in the various high performance concrete components associated with the two bridge projects.

This report investigates aspects of concrete properties, mix design, applicability of current design requirements, applicability of current construction practices, predictability of short- and long-term performance, and the requirements for quality control and quality assurance (QC/QA) testing and inspection. The performance-related test results of both mechanical and material durability testing of over 40,000 specimens from laboratory and plant-produced concrete are presented, including the statistical analysis. Related research studies, including early-age curing aspects of prestressed/precast beams, the use of maturity method for cast-in-place decks, and a high performance concrete permeability correlation study (AASHTO T259 and AASHTO T277), are also presented.

1.5 Research Objectives

This research study encompassed several objectives. The first primary objective was to develop several HPC mix designs in the laboratory using local material for use in various components of two HPC bridges, and to determine the range of engineering properties of the HPC. This included such performance-related testing as compressive strength with time, elastic modulus, splitting tensile strength, permeability, freeze-thaw resistance, abrasion resistance, and scaling resistance. Local materials were stressed in order to develop economical mix designs. The objective of the FHWA was to create a database with these mechanical and material properties for HPC. In addition, the study examined the variability of HPC produced at local precast and batch plants as part of the concrete produced for the structure. Plant-produced concrete was also compared to concrete produced in the

laboratory to better estimate associated loss in strength when a mix design developed in the lab is selected for use in the field. Several curing regimes were investigated, including moist cured, member cured, match cured, and site cured, to investigate the variation between moist-cured cylinders and concrete that closely simulates concrete in the structure.

Another primary objective was to determine the effect of elevated heat-curing temperatures at early ages on the two primary supporting element shapes. This included the Texas U-Beam and the standard AASHTO Type IV shape. A commercially available match-curing system was selected to aid in this study. This investigation also evaluated the use of a match curing system as a QC/QA tool to be used in the precast industry.

A third primary objective was to develop a QC/QA testing program to evaluate the performance-related test results throughout the investigation, and to provide recommendations for use in performance-based specifications. The study also examines the use of prescriptive-based specifications versus performance-based specifications.

Additional secondary objectives included several studies or investigations associated with HPC and the two HPC bridges. One of these included evaluating the use of the maturity method for HPC and conventional concrete in cast-in-place decks. A second study included a correlation study of AASHTO T259 and AASHTO T277 for HPC. Finally, a third study, *TxDOT Replacement Series*, examined fly ash replacement and minimum cement contents currently specified in many prescriptive specifications from a durability standpoint.

The overall research program associated with these two bridge projects (CTR studies 9-580 and 9-589) has incorporated several materials-related and structural performance-related research reports encompassing HPC to date. This report summarizes the objectives presented above as well as the previous materials-related studies associated with the construction and implementation of these HPC bridge projects. The previously submitted materials-related reports are as follows: Cetin and Carrasquillo [15] examined the effects of accelerated heat curing and mix characteristics on mechanical properties of HPC; Carlton and Carrasquillo [88] investigated several quality control issues of HPC, including curing effects and testing methods; and Touma and Carrasquillo [62] initiated and reported the short-term results of a long-term comparison study on the permeability of HPC. The long-term results of this last study are presented in this report.

A companion research report by Gross and Burns [136] summarizes the results of the structural performance-related issues pertaining to these bridges, including transfer and development length, laboratory creep and shrinkage testing, and instrumentation of the bridges for short-term and long-term monitoring of the bridges. The creep and shrinkage investigation was conducted under the structural performance-related investigations because the creep and shrinkage results were used for

time-dependent analyses and not for quality control purposes. The previously submitted structural performance-related reports are as follows: Farrington, Carrasquillo, and Burns [142] investigated the creep and shrinkage properties of the Louetta U-Beam concrete; Cordova, Braun, and Burns [143] and Gross and Burns [138] conducted transfer and development length tests on 15.2-mm (0.6-in.) diameter prestressing strands at a 50-mm (2-in.) grid spacing on large-scale test specimens; Byle and Burns [144] investigated the time-dependent deformation behavior effects on the Louetta Road Overpass beams; and Barrios and Burns [145] investigated the design and behavior of the prototype HPC U-Beams at transfer of prestress.

1.6 Scope of Materials-Related Research Presented

To pursue the above materials-related objectives, the following experimental program was undertaken:

1. The preparation of a series of laboratory trial mixes was undertaken, from which HPC mixes were developed for use in the structure. Individual mix constituent materials were optimized and a range of HPC properties produced with local materials was established.
2. The preparation of a series of field trial mixes was initiated to observe the performance of the laboratory-developed mix design under field conditions. This also gave plant personnel an opportunity to gain experience with the mix design prior to producing concrete for the structure.
3. The preparation of a series of laboratory trial mixes was undertaken to investigate the impact of coarse aggregates and accelerated heat curing on HPC properties.
4. The preparation of a series of laboratory trial mixes was undertaken to investigate fly ash replacement and minimum cement content requirements for mix designs without water reducers.
5. Plant-produced concrete was monitored at precast and ready-mix batch plants to investigate the statistical variation of HPC produced in the field. Plant-produced concrete was also monitored to develop empirical equations for the plant-produced HPC used in the Texas bridge structures.
6. Twenty precast/prestressed beams were instrumented to evaluate the use and applicability of match-curing technology as a QC/QA tool in the precast industry. The match-curing technology was also used in the field to determine the effect of elevated heat-curing temperatures at early ages on two precast/prestressed member shapes, the Texas U-Beam and the standard AASHTO Type IV beam.

7. A series of three microstructure studies was undertaken to investigate the impact of peak hydration temperature on microstructure cracking.
8. Six cast-in-place bridge decks were instrumented to evaluate the use and applicability of the maturity method in concrete bridge decks.
9. The preparation of a series of laboratory trial mixes was undertaken to investigate the correlation of AASHTO T259 and AASHTO T277 for HPC and to verify very low-permeable concrete using both methods. Also, the series of laboratory mixes was studied to investigate the effects of air entrainment on the permeability of HPC.

CHAPTER 2. LITERATURE SEARCH

2.1 Fundamental Mechanisms of High-Strength/High Performance Concrete

In the development of mix proportions for HS/HPC, the content of three components must be optimized to produce a mix design satisfying hardened or fresh concrete property requirements. These components include the strength of the coarse aggregate, the strength of the paste, and the strength of the paste-to-aggregate bond [1]. The load-bearing capacity of HS/HPC is determined by the weakest of these three components. This concept of the weakest component controlling the strength of the concrete is referred to as the *Three-Link Chain* concrete theory and is schematically presented in Figure 2.1. In producing the most efficient mix design utilizing readily available materials, it is important to optimize each component to maximize the strength potential. Furthermore, the compatibility among the constituents plays a very important role in the development of high-strength mixes as discussed herein.

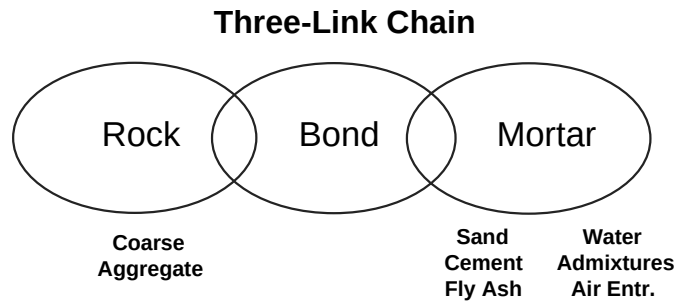


Figure 2.1: Three-Link Chain Concrete Theory

2.1.1 Aspects of Cement Hydration

Before discussing the individual constituents and the manner in which they affect the strength of the concrete, it is important to understand the chemical and physical aspects of cement hydration. Portland cement is composed of five major crystalline compounds. These components are tricalcium silicate (C_3S), dicalcium silicate (C_2S), tricalcium aluminate (C_3A), tetracalcium aluminoferrite (C_4AF), and gypsum (CSH_2). C_3S and C_2S are the most important ingredients that affect the strength development while the C_3A , C_4AF , CSH_2 , and alkaline compounds are important during the clinker production and regulation of setting and other rheological properties of fresh concrete. In the

presence of water, the silicates and aluminates hydrate and precipitate calcium silicate hydrate (C-S-H), calcium hydroxide (Ca[OH]₂), and calcium sulfoaluminate (C₆AS₃H₃₂). Mindness and Young [2] assert that cement hydration is an exothermic reaction in which the reaction kinetics may be followed by isothermal calorimetry. An isothermic curve for an ordinary portland cement is illustrated in Figure 2.2. The rate of strength gain of the cement is approximately proportional to the area under the rate of heat evolution curve.

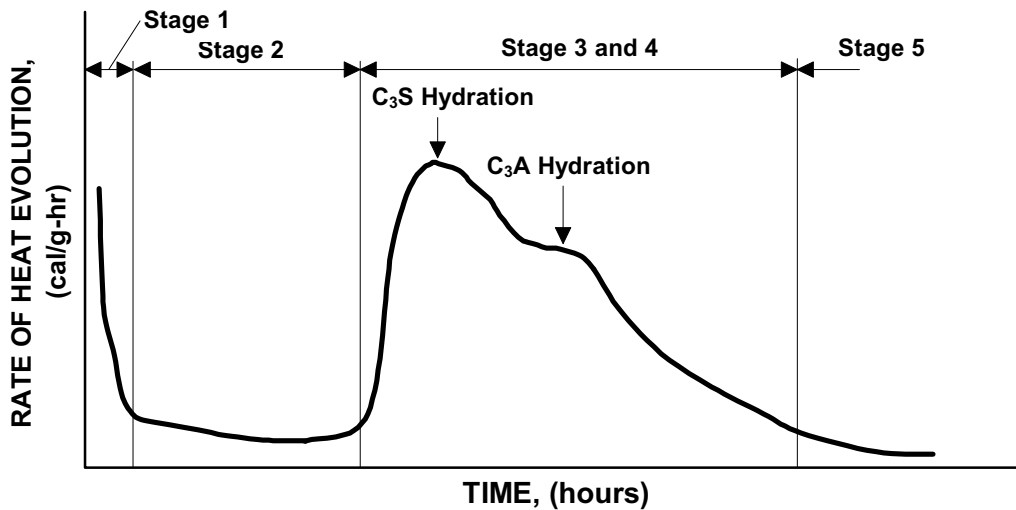


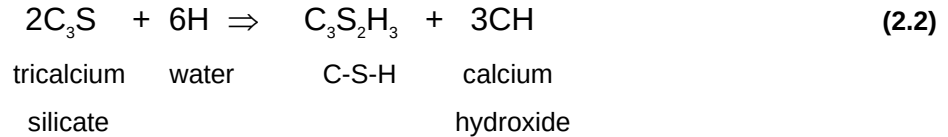
Figure 2.2: Rate of Heat Evolution during Hydration of Portland Cement

Stage 1 is a period of rapid evolution of heat and marks the rapid dissolution of C₃A, C₃S, and gypsum in water. A very alkaline solution results with a pH of over 12. Exothermic hydrolysis of ions slows down within approximately 10 minutes because of the formation of ettringite that resembles stubby rods on the outer surface of the gel layer surrounding the cement grains. Gypsum is added to form ettringite during the plastic state to curb the vigorous initial reaction of C₃A with water. The formation of ettringite is given by the following:

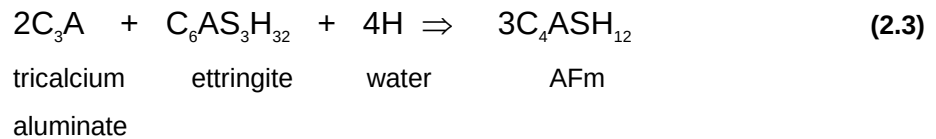


tricalcium gypsum water ettringite
aluminate

Stage 2 is a dormant period in which the hydrolysis of ions continues, but significant hydration does not occur for a period of 2 to 4 hours. This is because a certain ion concentration must be achieved in the solution before crystal nuclei can form. When calcium and hydroxide ions reach a certain concentration in the solution, hydration of C_3S proceeds rapidly and the initiation of the acceleration period (Stage 3) begins. This is characterized by a strong heat evolution and the following reaction:



Calcium hydroxide (CH) crystallizes from solution while the C-S-H develops around the cement grains forming a relatively porous shell initially. As hydration continues, the shell increases and forms a barrier through which water must flow to reach the cement particles and ions must diffuse to reach the growing crystals that nucleate on the ettringite rods. Hydration becomes diffusion-controlled at this juncture resulting in a deceleration of hydration of C_3S (Stage 4). Reactions that are diffusion-controlled are quite slow and become slower as the thickness of the diffusion barrier increases. When sulfate concentration is depleted by the formation of ettringite, ettringite further reacts with C_3A and water to form monosulfoaluminate (AFm). The formation of AFm is expressed by the following reaction:



The renewed growth of ettringite crystals is more acicular and their formation with C_3A is associated with this secondary peak. Once hydration is completely diffusion-controlled a steady-state period has been reached (Stage 5). This steady-state reaction stage is primarily relevant to the later-age strength gain of the concrete mix design.

2.1.2 Portland Cement Paste

The total porosity, pore-size distribution, and the presence of flaws within its structure are properties of portland cement paste, which have been found to have a significant effect on strength [1]. As the total porosity of the paste increases, the strength decreases. The pore size also affects

the strength of the paste. Large pores are considered to be more detrimental to the strength of the paste than the small pores. In some cases, lowering the water-to-cementitious material (cement + fly ash) ratio may reduce the strength of the concrete if the cement grains are not dispersed properly [3]. This may be caused by low slump of the concrete or poor compaction of the mix associated with the low slump of concrete. However, this is generally not a concern when a mid or high range water reducer is used in conjunction with a low water-to-cementitious material (w/cm) ratio. Figure 2.3 presents the volume relationships among the constituents schematically.

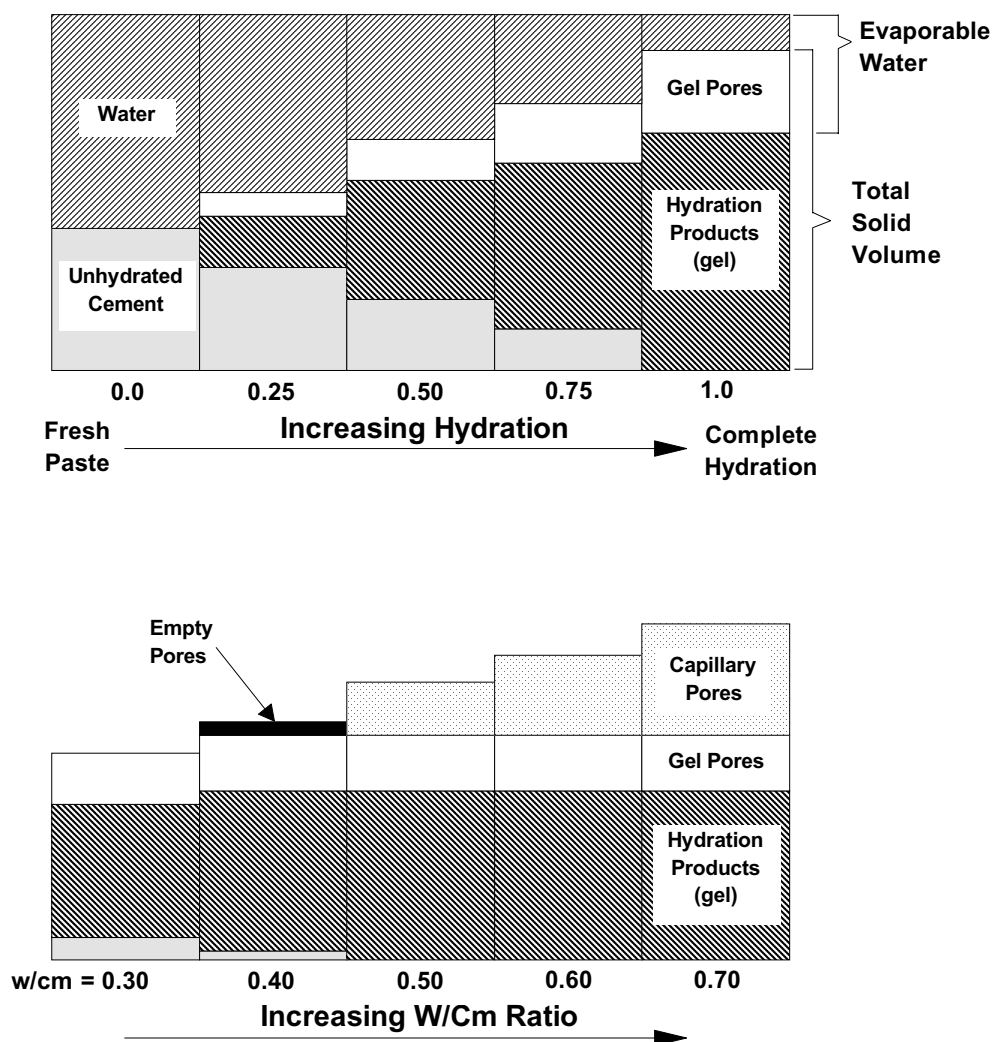


Figure 2.3: Volume Relationships among Constituents of Hydrated Pastes [2]

Capillary and gel pores are two types of pores in cement paste. Capillary pores represent the voids in the paste that have not yet been filled with hydration products, while the gel pores are the infinitesimal interconnected spaces that exist between the gel particles. The pore system plays a major role in determining the permeability of the concrete.

Immediately surrounding the aggregates is a layer of paste known as the *transition zone*. The transition zone may be 10 to 50 μm thick in conventional concretes. The thickness is dependent upon a number of variables, including the w/cm ratio and the aggregate size. In freshly placed conventional concrete, the bleed water deposits around the coarse aggregate particles forming a water film. The water film is thicker with larger aggregate sizes under identical conditions. This higher w/cm ratio at the transition zone causes the formation of larger crystals of ettringite and CH resulting in a more porous microstructure than in the bulk hydrated cement paste. Calcium hydroxide crystals grow with their c-axis perpendicular to the surface of the coarse aggregate, which affects the paste-to-aggregate bond in a detrimental manner.

The higher the w/cm ratio at the transition zone, the thicker the transition zone and the greater the number of CH crystals that are oriented perpendicular to the aggregate, resulting in weaker bond characteristics. Most experts [2, 4, 5, 6] agree that the transition zone is the weak link for conventional concretes. For the purposes of this report, conventional concretes are concretes without chemical or mineral admixtures generally less than 41.4 MPa (6,000 psi) in strength. The aggregate particles and paste components for conventional concretes are generally not the weak link because microcracking in the transition zones is already present owing to drying and cooling even before the concrete is subjected to loading. The weak nature of the transition zone and this presence of microcracking are mainly responsible for the inelastic behavior of concrete and its associated loss of stiffness relative to these two components.

For the case of HS/HPCs, the cement paste is denser when compared to conventional concretes [7]. Both conventional concretes and HS/HPC are composed of C-S-H and CH, but their proportions and morphologies are different. In HS/HPC, the CH content is significantly lower and less crystallized, which is attributed to a low w/cm ratio. The lack of water in HS/HPCs inhibits the formation of CH crystals because the solution is required to be supersaturated before CH crystals can form. Because the presence of CH in the transition zone is inhibited, C-S-H is in direct contact with the coarse aggregate particle resulting in an improved bonding between the aggregate particle and mortar. Therefore, in HS/HPCs, the transition zone may or may not be the weak link depending upon the composition and characteristics of the aggregate and mortar. Additionally, both the total porosity and the average pore size are reduced in HS/HPCs, resulting in low permeability as discussed in

Section 2.3.2. The generally low w/cm ratio of HS/HPC results in the improved durability performance as discussed in subsequent sections.

2.1.3 Effect of Chemical Admixtures

The use of chemical admixtures in the production of HS/HPCs can significantly improve the quality of the concrete. In order to select the best type and dosage rate of a chemical admixture for an HS/HPC, the admixture should be optimized and tested under the expected field conditions. Chemical admixtures may be separated into several general groups including water reducers and retarders, HRWRs, air-entraining admixtures, and accelerating admixtures.

2.1.3.1 Water Reducers and Retarders. The high cementitious material content and generally low w/cm ratio have made the use of water reducers and retarders a necessity for the production of high-strength concrete. Water reducers are an effective means of decreasing a high mixing water demand for proper workability. Water reducers lower a given w/cm ratio while maintaining a given slump resulting in improved concrete strength, impermeability, and durability. The use of retarding admixtures also reduces the mixing water requirement, but more importantly prolongs the plastic state of the concrete mix and aids in reducing the temperature rise of the fresh concrete owing to the heat of hydration. Most HS/HPCs incorporate very high contents of cement in which high heat of hydration temperatures are developed during hydration. These high temperatures can impact mechanical and material property development. By reducing the temperature rise during initial hydration, the later-age properties of the concrete including strength and elastic modulus can be improved. Longer set times, particularly during hot-weather concreting, can benefit the overall quality of the concrete. This aspect of controlling the peak heat of hydration is discussed in greater detail in Chapter 8.

2.1.3.2 High Range Water Reducers. The use of HRWRs, often referred to as a superplasticizers, is considered to be very important in the production of very high-strength concretes with compression strengths in excess of 68.9 MPa (10,000 psi). The use of a HRWR in a HS/HPC mix allows for a significant reduction in the w/cm ratio below a value of 0.40. HRWRs are high molecular weight anionic surfactants derived from sulfonated formaldehyde of naphthalene or melamine. HRWRs deflocculate the clumps of cement grains and fluidize the mixture. The higher dosage rates of HRWR over midrange or conventional water reducers allow for an increased and more uniform dispersion of hydration products and, thereby, a more effective use or optimization of the cementitious material. In effect, Philleo [8] explained that the water is used more efficiently because the cement particles are more evenly distributed, and their surface is more accessible to water for hydration to occur more expeditiously. The use of an HRWR lessens the sensitivity of the

concrete strength to low w/cm ratios [3]. This may be attributed to the more efficient use of the materials.

2.1.3.3 Accelerating Admixtures. Set-accelerating admixtures are used to increase the rate of hydration of C_3S , thereby reducing the length of the dormant period (Stage 2). Applications may include emergency concrete repair in which very rapid development of rigidity is required or cold-weather concreting in which these admixtures can offset delays caused by the cold temperatures. Set-accelerating admixtures are generally not recommended for use with HS/HPC because they increase the heat of hydration and detrimentally affect the long-term properties of the concrete. Furthermore, in many cold-weather concreting applications, temperature-related placement concerns are reduced owing to the high cement content of most HS/HPC mix designs. The high performance concrete beams and decks associated with this study are a prime example.

2.1.4 Effect of Mineral Admixtures

The most commonly used mineral admixtures are fly ash, slags, and silica fume. All of these materials are able to react with calcium hydroxide and are said to have pozzolanic activity and are therefore referred to as *pozzolans*. A pozzolan contains amorphous silica that is sufficiently reactive to combine with calcium hydroxide to form C-S-H in the presence of water. The pozzolanic reaction is given by:



When a pozzolan is combined with cement, it will react with the calcium hydroxide formed during hydration. The pozzolanic reaction does not begin until CH is produced by the cementitious reaction. The reaction with the pozzolan converts the CH crystals to the highly cementitious C-S-H. The strength of the paste is thereby increased while the permeability is decreased. However, because the pozzolanic reaction does not occur until CH becomes available for reaction, it does not contribute to the concrete strength until later ages. Since the cementitious and pozzolanic reactions occur at different times, the hydration temperature at any given time is generally lower than that of a concrete containing the same volume of portland cement. The control of temperature can be beneficial in most situations as previously discussed. The chemical and physical properties of these mineral admixtures determine the effect that each has on the properties of fresh and hardened concrete. While each mineral admixture will vary depending on its source, there are characteristics that generally apply to each type.

In terms of usage on a national basis, the material that is locally available is generally the mineral admixture of choice by concrete producers. When used as a replacement material by producers, the mineral admixture generally reduces the cost of the cementitious material. Along the northern rust belt, slags, fly ash, and silica fume are more typically used while fly ashes are used in the south and southwest. Therefore, it is mainly an economic criterion that drives the usage of a particular mineral admixture in a region.

2.1.4.1 Fly Ash. Fly ash is a by-product produced in the coal-burning process. When the coal is burned, the impurities in the coal that do not burn are suspended in the flue gases. These particles are called fly ash. Because of environmental concerns, the fly ash is removed from the flue gases by mechanical collectors and electrostatic precipitators. Fly ash particles are typically smooth and spherical in shape and range in diameter from 1 to 150 μm depending on the type of collection system.

Based on its composition, fly ash is classified into two groups: American Society for Testing and Materials (ASTM) Class C or high-calcium fly ash, and ASTM Class F or low-calcium fly ash. Class F fly ash is generally produced from the combustion of anthracite or bituminous coals. Because of their low calcium oxide content, these fly ashes require an external source of calcium for their reaction. Diamond and Lopez-Flores found [9] that mortars with a 30 percent Class F fly ash replacement achieved equal strength with reference mortars at 90 days. The pozzolanic reaction for Class F fly ash begins after approximately two weeks [10]. In some cases, Class F fly ash may contain approximately 5 percent by weight cenospheres or plerospheres [11, 12]. Cenospheres are hollow and empty, whereas plerospheres are hollow but contain smaller spheres inside. Class F fly ash also typically has a carbon content of 2 to 10 percent owing to incomplete combustion of the coal. A measure of its carbon content is indicated by the loss of ignition (LOI) of a fly ash. Generally, concretes having a high LOI level require a higher dosage of air-entraining admixtures for a given air content because the carbon particles absorb some of the air-entraining admixture.

Class C fly ash is produced primarily from the combustion of lignite or subbituminous coals. Class C fly ash has a relatively high calcium oxide content, between 15 and 30 percent. Class C fly ash exhibits both cementitious and pozzolanic behavior. It has been reported [9] that mortars with a 30 percent Class C fly ash replacement achieved equal strength with reference mortars at seven days. Class C fly ashes contain less than one percent carbon. Thus, the impact of a Class C fly ash on air-entraining dosage is minimal.

The rheological behavior of concrete is affected to a great extent by the addition of fly ash. When added as a replacement of cement, fly ash results in a more workable mix owing to its spherical shape and enables the producer to reduce the w/cm ratio for a given slump, thereby resulting in an

increase in strength and other mechanical properties. This characteristic, coupled with the reduction in hydration temperatures, makes fly ash a very desirable component in producing HS/HPC.

2.1.4.2 Slag. There are three main types of slag including two blast furnace slags (granulated and pelletized) and one steel slag. Nonferrous slag is not being used as an admixture in concrete [10].

Blast furnace slag is a by-product in the production of pig iron. The type of blast slag is determined by the method used for cooling the molten slag. If the liquid slag is quenched using water, granulated slag results. If the liquid slag is quenched with air and water, pelletized slag results. Both materials must be ground to a fineness of 400 to 600 m²/kg to exhibit satisfactory cementitious and pozzolanic characteristics [10]. Water demand for slags is generally higher than that for an equal volume of cement owing to its coarse texture and higher fineness. However, these slags can exhibit similar strengths equal to reference concrete at 7 days [13].

Steel slag is produced in the conversion of pig iron to steel. The liquid slag is quenched with water and ground fine. It exhibits both cementitious and pozzolanic characteristics similar to the granulated and pelletized slags.

Other slag materials such as nonferrous slags produced from metallic copper, nickel, and lead do exhibit pozzolanic characteristics, but are currently not being used as a mineral admixture.

2.1.4.3 Silica Fume. Silica fume is produced in electric arc furnaces during the production of metallic silicon and ferrosilicon alloys. These are smooth and spherical particles that are nearly pure silicon dioxide and 100 times finer than cement and fly ash particles. Silica fume is highly pozzolanic in nature and can achieve equal strengths with reference concrete at 14 days. Owing to the fineness of the particles, silica fume has an extremely high water demand and requires the use of a water reducer. To achieve adequate workability, the slump of mixes containing silica fume should be increased to offset the stickiness or cohesiveness caused by the abundance of fine particles. Mixes with silica fume also exhibit reduced bleeding characteristics that require special considerations in curing such as proper fogging to avoid plastic shrinkage cracks.

2.1.5 Effect of Air-Entraining Admixtures

Air-entraining admixtures entrain air in the concrete to provide an air void system for the movement of freezing water within the cement paste. An adequate well-distributed air void system provides a relief space for expanding water that minimizes the amount of microcracking damage owing to moist freeze-thaw cycles. The use of air-entraining admixtures for HS/HPC mix designs is only desirable for components subjected to freeze-thaw cycles in excess of the critical saturation threshold (91.7 percent). Air entrainment is not recommended in other cases because entrained air

can decrease a concrete's strength significantly at high dosages. The rule-of-thumb is that a 5 percent loss in strength may be anticipated for every 1 percent of air entrainment. Research at The University of Texas at Austin [14] and elsewhere has shown that a minimum of 3 percent air entrainment is required for HS/HPC, as illustrated in Figure 2.4. This value is slightly lower than that of conventional mix designs, primarily because of the pore structure of HS/HPC with low w/cm ratios.

HS/HPC mix designs are naturally more durable owing to their higher strength, lower permeability, and decreased porosity. While research studies to date have not specifically investigated the levels of porosity and time-to-saturation for HS/HPC mix designs with low w/cm ratios, it is safe to assume that the reduced porosity of these mixes requires a longer period of time to exceed the critical saturation threshold for a given moisture exposure. Air entrainment should be used in HS/HPC only where specifically required for exposure conditions and should always be tested under field conditions because of the high variability of air entrainments with other chemical and mineral admixtures.

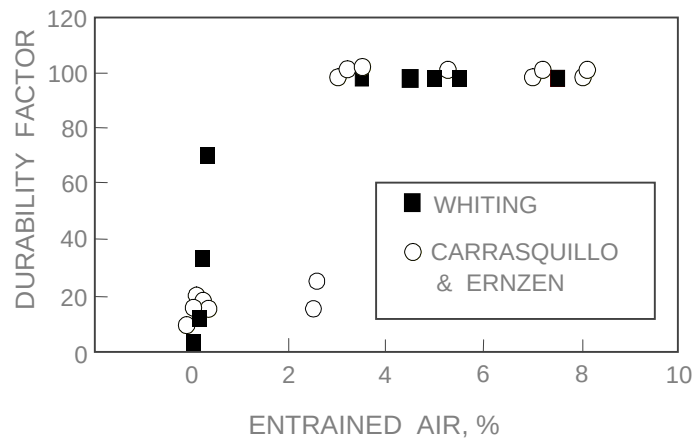


Figure 2.4: Durability Performance of HS/HPC Mixes with Entrained Air [14]

2.1.6 Coarse Aggregates

In conventional concretes, w/cm ratio is the major factor controlling most mechanical properties, including compressive strength. This ratio is achieved by altering the porosity of the cement paste. The aggregate plays a minor role in the performance of conventional concretes because the transition zone is typically the weak link, as discussed in Section 2.1.2. However, the situation is quite different in HS/HPC with the enhanced transition zone properties. Two components,

the aggregate and cement paste, behave more like a homogeneous material and share the stress associated with the load on the concrete. Under this situation, the aggregate can often be the limiting factor.

Several parameters of coarse aggregate that affect the behavior of the concrete include its shape, surface texture, mineralogy, strength, and elastic moduli. It has been well documented [15] that a smaller aggregate size produces higher compressive strength at a given w/cm ratio. The effect is more pronounced at lower w/cm ratios and higher cementitious contents. This is due to a reduced unit bond stress resulting from an increased surface area from the smaller aggregate size. It should be noted that the increased surface area from the smaller aggregate size requires a higher paste content to maintain the same workability. The elastic modulus of the concrete appears to be less affected by aggregate size for a given aggregate content, based on research at UT [15] and elsewhere.

While stiffer, denser aggregates improve the elastic modulus of the concrete, they also can act as a stress riser resulting in stress concentrations at the transition zone and subsequent microcracking at the bond interfaces reducing the compressive strength of the concrete [16, 17, 18]. As a result, the compatibility of materials in producing high-strength concrete is critical for microcrack development and compressive strength. To develop optimum strength characteristics out of a mix design, it is desirable to match the moduli of the aggregate with that of the mortar matrix. This will result in a more homogenous material. Because the aggregate type is largely responsible for the elastic modulus of the concrete, a higher aggregate content results in a higher elastic modulus. Therefore, the elastic modulus for HS/HPC may be “engineered” by selecting a specific aggregate type and content within a mix design.

Surface texture and particle shape of the coarse aggregate are responsible for the bond between the matrix and the aggregate owing to mechanical interlocking. Crushed aggregates result in superior bond when compared to smooth, rounded gravel. The bond characteristics appear to be more pronounced at low w/cm ratios [19]. Additionally, tensile strengths are even more sensitive to the aggregate surface texture because the tensile strength is dependent on the aggregate-matrix bond strength [20].

Recent studies [6, 19, 21] also indicate that the aggregate type plays a role in the bond characteristic. In the case of limestone, dolomite, and siliceous aggregates, a chemical reaction between the rocks and cement matrix can enhance the bond characteristics for HS/HPC.

2.2 Mix Proportioning Guidelines for Optimization Of HS/HPC

In the development of mix proportions for HS/HPC, there are basic guidelines that can be applied to optimize each of the materials selected. The exact proportions are dependent on the chemical and physical properties of the constituents.

2.2.1 Required Concrete Strength

The initial step in the proportioning of a HS/HPC mix is to determine the required strength, the age at which the concrete strength should be achieved, and the manner in which the specimens will be tested.

2.2.1.1 Test Age. High-strength concrete generally has a high early strength, as well as significant strength gain at later ages, particularly when mineral admixtures are used. Carrasquillo and Myers [22] have shown that HS/HPC mix designs with a 35 percent Class C fly ash replacement can experience a 15 percent strength gain from 28 to 56 days. Therefore, it is reasonable to specify a later age for acceptance such as 56 or 90 days unless construction loads require the design strength at an earlier time. Traditionally, most design engineers specify a 28-day acceptance criterion for design strength for both precast and cast-in-place (CIP) concrete.

It is important to note that in many applications a later-age acceptance criterion imposes no additional restrictions or detriment for a fabricator or contractor. Production and construction sequences can typically be adjusted so that a later test age does not impact or reduce productivity. Additionally, a later test age may be beneficial to the concrete producer in two ways. Firstly, a later test age may allow for more efficient use of materials and, thereby, result in reduced material and construction costs. Secondly, production concerns may be lessened regarding achieving higher end strengths, which are typical for the primary load-resisting elements such as columns, piers, and beams.

2.2.1.2 Laboratory versus Field Test Results. Strength test results obtained from laboratory trial mixes are generally 10 percent higher than those obtained in the field for identical mix proportions [23]. Therefore, if laboratory trial mixes are to be used for strength evaluation, the strength achieved by these mixes should be a minimum of 10 percent higher than that desired in the field. A more accurate and cost-effective approach is to investigate the actual strength loss associated with a mix design in the field prior to production. Once a mix design has been optimized in the laboratory, a plant trial mix phase should be conducted to investigate the concrete properties produced under field conditions. This process is schematically represented in Figure 2.5. This process not only indicates what can be anticipated under field conditions, but also allows plant

personnel the opportunity to gain experience with production of the mix design. This procedure also permits the opportunity to revise and rebatch mix designs in the laboratory if plant trial mixes are excessively high or low in strength. This concrete production sequence was used for both of the Texas HPC bridges.

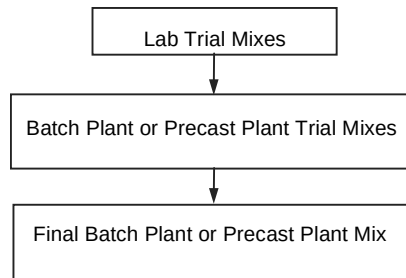


Figure 2.5: QC/QA Mix Design Evolution

2.2.1.3 Curing and Strength Development. The strength development with time is a function of the curing techniques and constituent materials. Curing may be defined as the process of maintaining a satisfactory moisture content and a favorable temperature in concrete during the hydration period of cementitious materials so that desired properties of the concrete can be developed [4]. Primary curing methods include water curing and sealed curing. Water curing, either by ponding, spraying, or sprinkling, supplies additional moisture to the concrete surface and assists in preventing moisture loss. Sealed curing, either by plastic sheeting, waterproof paper, or curing membranes, is applied to solely prevent moisture loss from the concrete. Special curing situations may be required for hot weather, cold weather, and mass concrete. Precast/prestressed concrete may require curing at elevated temperatures through steam curing to satisfy release strengths. Table 2.1 illustrates the effect of limited moist curing on the development of compressive strength for a conventional concrete mix design at 180 days.

Table 2.1: Influence of Moist Curing on Strength Development [2]

Curing Condition of Specimen	Relative Strength at 180 Days
Continuous Moist Curing (Control Specimen)	1.00
28-Day Moist Curing followed by Lab Air Curing	0.91
14-Day Moist Curing followed by Lab Air Curing	0.85
7-Day Moist Curing followed by Lab Air Curing	0.71
3-Day Moist Curing followed by Lab Air Curing	0.62
Stored Continuously in Laboratory Air	0.43

An adequate supply of moisture is necessary to ensure that hydration is sufficient to reduce porosity to such a level that a desired strength and durability can be attained.

In the case of HS/HPC mixes with low w/cm ratios, proper curing techniques play a very important role in strength development. There is enough water in concrete to ensure complete hydration without additional water being supplied if the w/cm ratio is 0.42 or greater [2]. Below this value, self-desiccation occurs preventing further hydration unless water is supplied externally. This reduction of water stops hydration and arrests the strength development. Hydration cannot occur when the vapor pressure in the capillary pores falls below 80 percent relative humidity [4]. As a result, moist curing is essential for strength development of concretes with low w/cm ratios while sealing is usually adequate for concretes with higher w/cm ratios owing to the availability of water within the mix to develop complete hydration. According to Carrasquillo et al. [24], high strength concretes show a larger reduction in compressive strength than conventional concretes when allowed to dry before completion of curing.

Experts [25, 26, 27] generally agree that fly ash concretes are more sensitive to poor curing. This sensitivity increases with increasing fly ash content because the fly ash relies on the presence of CH for the pozzolanic reaction to occur. This is similar to cement hydration in that pozzolanic reaction requires a minimum vapor pressure of 80 percent relative humidity [25]. Therefore, longer curing periods are needed for fly ash concrete to exhibit similar strength development to that of control mixes.

One area of construction in the United States in which inspection agencies and contractors have not placed a proper emphasis is in proper curing techniques, particularly when strength requirements have been satisfied. Proper curing is one of the most inexpensive means with which to improve the later-age strength gain of all concretes. More importantly, it can improve the long-term durability and reduce the maintenance costs of a structure. Aitcin [28] and many others have concluded that the effect of moist curing is more pronounced on concretes with lower w/cm ratios. More attention must be paid to curing for HS/HPC, including concrete that is exposed to an environment where durability is an important consideration.

2.2.2 Water-to-Cementitious Ratio

The production of HS/HPC requires the use of low w/cm ratios. In order to produce concrete with 28-day compressive strengths ranging from 41.4 MPa (6,000 psi) to 96.5 MPa (14,000 psi), a w/cm ratio in the range of 0.25 to 0.52 is required by weight. In addition, an HRWR is required when the very low w/cm ratios are used to produce HS/HPC.

2.2.3 Cement

The cement selected for the production of HS/HPC should have good strength-producing properties. The fact that cements produced at different plants, all of which meet the same ASTM specification for a given type of cement, does not mean that concretes produced with each of the different brands of cements will perform similarly. It is difficult to predict the interaction of a particular cement with other constituents within a mix design. As Figure 2.6 illustrates, concretes produced with different brands of cement can have significantly different compressive strengths.

For this reason, it is suggested that trial mixes be prepared that vary locally available cements in order to investigate which cement produces optimum strength and, thus, the most cost-effective mix design. An ASTM Type III cement should only be specified where early-age strength gain is critical as is the case with many precast/prestressed elements. A Type III cement does not provide the same beneficial later-age strength gain characteristics that are typical of Type I and Type II cements. Additionally, Type III cements generally experience higher concrete temperatures during hydration owing to the higher blaine (fineness) of the cement particles. HS/HPC mixes, in general, develop higher concrete temperatures owing to the high cement contents, which typically vary from 390 to 558 kg/m³ (658 to 940 lb/cy). The use of coarser ground cements can help control the concrete temperature rise during hydration, but may also result in incomplete hydration owing to the larger particle size. A good balance between these two concerns has been to use a mineral admixture such as fly ash as a replacement material that can be used not only to reduce the concrete temperature during hydration, but also to benefit the ultimate strength capacity of the mix design.

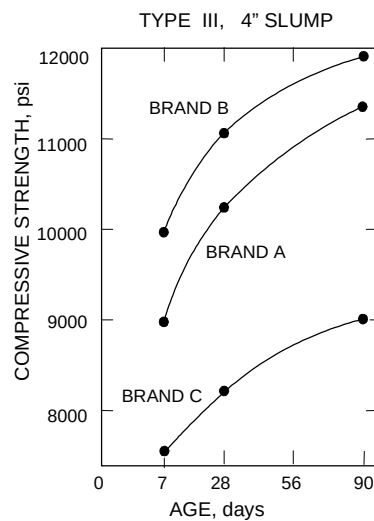


Figure 2.6: Cement Brand Variation versus Compressive Strength

Trial mixes are required to determine the optimum cement content of an HS/HPC mix. The optimum cement content is the content in excess of which further addition of cement does not result in increased strength and the point at which all aggregates are completely surrounded by cement particles. Figure 2.7 illustrates this optimum cement content concept and how the aggregate type and surface texture might affect it. For example, the 13-mm (0.5-in.) limestone aggregate has a rough surface texture and, therefore, a higher cement demand for a given aggregate content when compared to the smooth textured river gravel aggregate. In this case, the optimum cement content is higher for the limestone aggregate based on its surface texture, shape, and size.

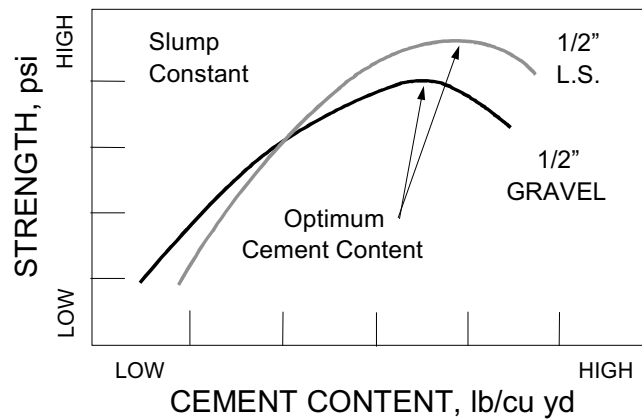


Figure 2.7: Optimum Cement Content for Two Aggregates

2.2.4 Mineral Admixtures

The rates of addition of mineral admixtures are different for each type. American Concrete Institute (ACI) Committee 201 has established some guidelines for the replacement of portland cement with other cementitious materials, as illustrated in Table 2.2. Silica fume, at additional rates in the range of 7 to 15 percent by weight of cement, has been used in the production of high-strength concrete. No specific guidelines or recommendations are given in terms of an optimum percentage of replacement material to use. This may be determined through trial mixes until specification requirements are satisfied.

Table 2.2: ACI 211 Guidelines for Portland Cement Replacement

Other Cementitious Material	Portland Cement Replacement Percent by Weight
Fly Ash	
Class F	15–25
Class C	20–35
Blast Furnace Slag	30–50

2.2.5 Coarse Aggregate Type

Of great importance is the selection of coarse aggregates for use in the production of HS/HPC. Recommended guidelines for the selection of aggregates include aggregates with a maximum gradation size less than 25 mm (1 in.) with good compactability. Furthermore, crushed aggregates are better than smooth as discussed in Section 2.1.6. Because of the effect of surface area on the water requirement, coarser gradations result in the achievement of higher compressive strengths as a result of being able to reduce the amount of mixing water as illustrated in Figure 2.8. In addition, the use of lower percent void coarse aggregates results in the production of concrete of higher compressive strength owing to a lower water demand for a given workability.

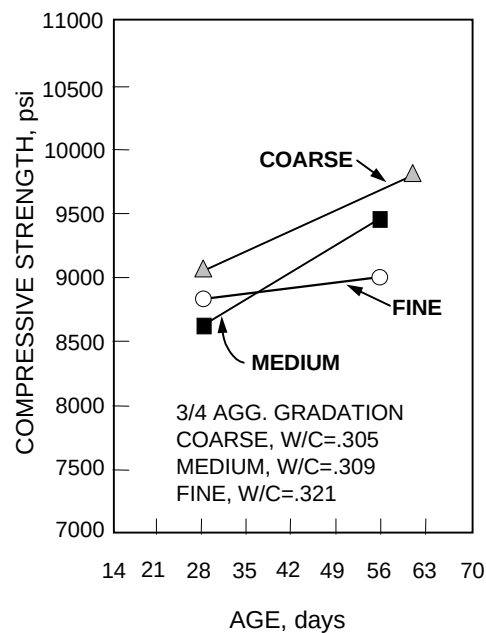


Figure 2.8: Aggregate Gradation versus Compressive Strength

2.2.6 Coarse Aggregate Content

Generally speaking, HS/HPC mixes contain a larger volume of coarse aggregate than conventional normal-strength mixes. The higher aggregate content in HS/HPC mixes helps to reduce the water demand and increase the elastic modulus of the concrete. Because many HS/HPC elements are thinner and have longer span lengths, the elastic modulus of the concrete becomes more critical in addressing serviceability requirements.

2.2.7 Fine Aggregate

Because HS/HPC mix designs have a high content of cementitious materials, the gradation of the fine aggregate is not so important as in conventional normal-strength concrete. It is recommended that fine aggregates be used that will minimize the mixing water and provide good workability. Generally, sands meeting ASTM C-33 requirements and having a fineness modulus between 2.6 and 3.2 perform well in HS/HPC mixes. Figure 2.9 illustrates the low variability in compressive strength for sands with a moderate fineness modulus. Sands having a low fineness modulus may be too cohesive.

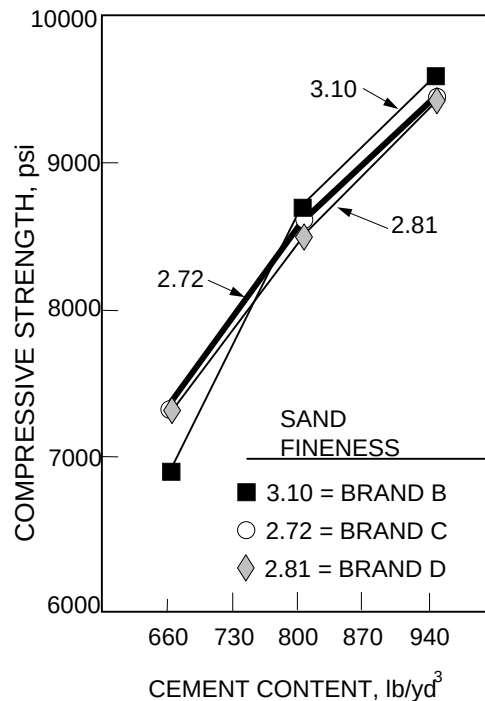
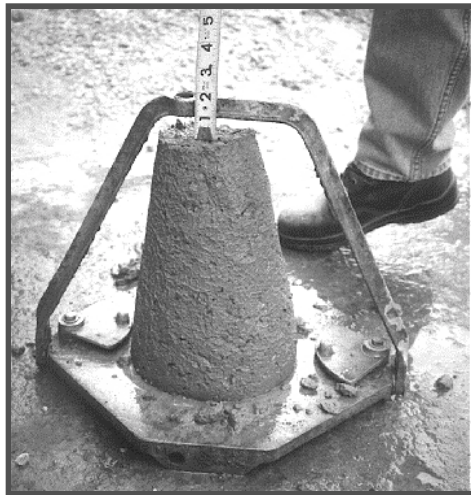


Figure 2.9: Fineness Modulus Variation versus Compressive Strength

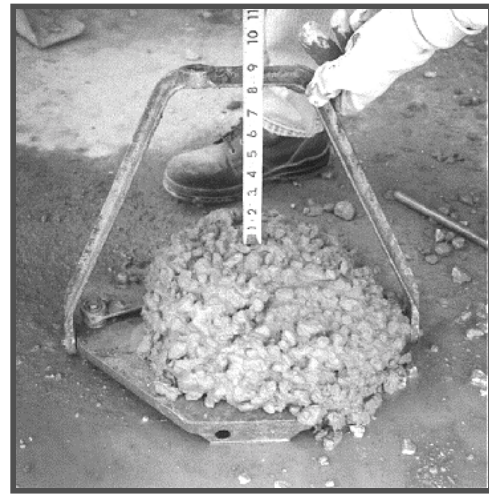
Manufactured sands are generally not recommended for use in HS/HPC mixes owing to their high absorption capacities. Fine aggregates with high absorption capacity make it more difficult for a concrete producer to consistently produce an HS/HPC mix with a low standard deviation. This is because the variability of mixing water absorbed on a daily basis by the fine aggregate in conjunction with the low w/cm ratio.

2.2.8 Recommended Slump of Concrete

It is recommended that the slump of concrete produced without the addition of a midrange water reducer or HRWR be 50 to 100 mm (2 to 4 in.). The minimum slump of concrete before the addition of a water reducer is recommended to be 1 inch. Requiring this 1-inch slump prior to the addition of a water reducer ensures that the mix contains sufficient water to provide adequate workability. Satisfying this guideline will also reduce the potential of plastic shrinkage cracking. Furthermore, most HRWR manufacturers recommended that the slump of concrete after the addition of an HRWR be 200 to 250 mm (8 to 10 in.) as illustrated in Figure 2.10. This allows the HRWR to fully distribute the cement particles and maximize the strength potential of the concrete. Section 3.2.3.2 of this report further discusses concrete slump from a specification and field performance perspective.



Before Addition of HRWR



After Addition of HRWR

Figure 2.10: Concrete Slump before and after Addition of HRWR

2.2.9 Chemical Admixtures

In the production of HS/HPC, it is generally advantageous to use certain chemical admixtures, such as retarders, water reducers, and HRWRs. Dosage rate recommendations are provided in most cases by the manufacturer. However, the optimum dosage rate of any chemical admixture must be determined through field trial batches conducted under the expected job conditions. In general, the production of HPC will result in the use of higher than recommended dosages of chemical admixtures. As illustrated in Figure 2.11, the use of increasing dosages of chemical admixtures above the manufacturer's recommended maximum dosage rate of 591 milliliters per 45.4-kilograms (20 oz per 100 lb) of cement resulted in higher concrete strengths without any detrimental effect on any of the performance characteristics of the concrete. This is based on research conducted at UT by Carrasquillo et al. [29].

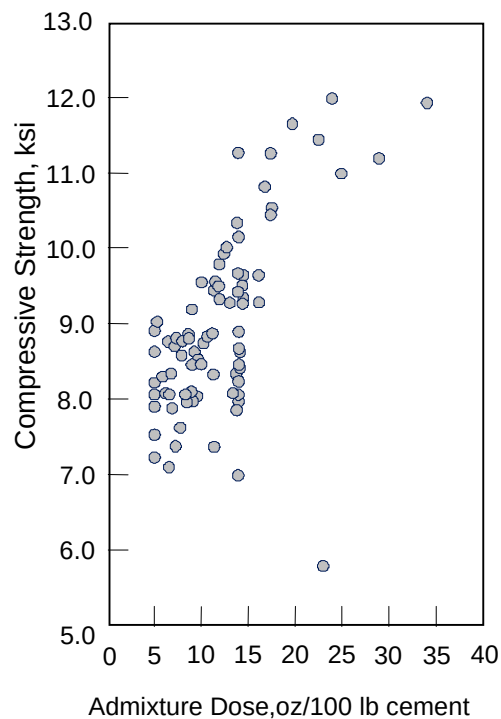


Figure 2.11: Effect of HRWR Dosage Rate on Compressive Strength

2.3 DURABILITY OF CONCRETE

Durability of concrete may be defined as the concrete's ability to resist cracking and deterioration owing to chemical and/or physical attack. Concrete is inherently a durable material if properly designed and produced for the environment that it will be exposed to. However, concrete is potentially vulnerable to attack in different exposures unless certain precautions are taken. Deterioration of concrete can be caused by the adverse performance of the aggregate, paste, or reinforcing owing to chemical or physical attack. Although one particular environmental factor may initiate distress in a structure, other factors may contribute and aggravate the situation. One of the most difficult aspects of durability is predicting concrete behavior and performance decades into the future on the basis of short-term tests. Much of our understanding of durability has come through years of studies on actual field problems. First, let us examine the primary causes of most durability-related problems.

2.3.1 Damage and Deterioration of Concrete

The following section outlines several forms of chemical and physical attack including sulfate attack, freeze-thaw damage, alkali-aggregate attack, and corrosion of reinforcement.

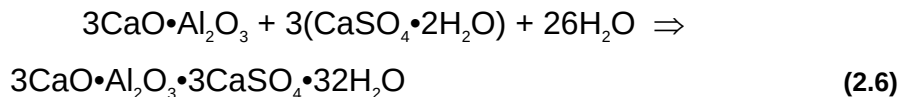
2.3.1.1 Sulfate Attack. Sulfate attack is perhaps the most widespread form of chemical attack and is dependent on sulfates to occur. It is caused by the reaction of soluble sulfates with calcium hydroxide and calcium aluminate hydrate. The sources of the soluble sulfates may include soils, seawater, groundwater, or materials selected and utilized in the production of cement and concrete. Sulfate attack is a complex process that may involve a number of secondary processes, although a correlation has been established between the C_3A content of a portland cement and its susceptibility to sulfate attack. The major cause is sulfoaluminate corrosion [3] in which ettringite is formed from monosulfoaluminate as characterized by the following expression:



This reaction is accompanied by a very large increase in solid volume that causes a volume expansion within the paste. The resulting internal stresses lead to cracking. Sulfoaluminate corrosion is initiated by a reaction referred to as gypsum corrosion. Only at high concentrations of sulfate ions (> 4,000 PPM) does gypsum corrosion directly contribute to concrete deterioration. Generally, it exists at low levels and encourages the penetration of sulfate ions into the concrete and concentrates

them into a form in which they can directly react with monosulfoaluminate. The use of mineral admixtures [51] such as a Class F fly ash or blast furnace slag can improve sulfate resistance. Type II or Type V cements can also improve sulfate resistance by controlling the level of C_3A within the portland cement. HS/HPCs can also help control sulfate attack because of a reduced porosity resulting from the low w/cm ratio. It should be noted that this will only offset the initiation of damage, but not prevent it.

2.3.1.1.1 Delayed Ettringite Formation (Late Ettringite Formation). Delayed ettringite formation (DEF), often referred to as late ettringite formation (LEF), is a form of sulfate attack that is a recently discovered phenomenon. Within the last decade, DEF has been attributed to premature concrete deterioration in the United States and abroad. DEF involves the production of the compound ettringite in concrete months or years after casting. Ettringite naturally forms during the early hydration process of portland cement while the concrete is in the plastic state (see Section 2.1.1), but the natural reaction process of ettringite differs from that of DEF. DEF is characterized by the following reaction:



The formation of ettringite, as illustrated in Figure 2.12, is an expansive reaction that can cause cracking when the concrete hardens or attains rigidity. Previous investigations of DEF have concluded that it may be caused by any of the following [30, 146]:

- ◆ Steam curing of the concrete at temperatures sufficient to decompose ettringite, which forms during the initial stages of the cement hydration.
- ◆ Heat developed in mass concrete above the decomposition temperature for ettringite generally considered to be above 71.1° Celsius (160° Fahrenheit). However, cases have been noted as low as 60° Celsius (140° Fahrenheit) [146].
- ◆ Excessive amounts of sulfate in the clinker phase of portland cement.
- ◆ Excessive amounts of sulfate in the cement.

The terms *delayed ettringite* or *late ettringite formation* are used to characterize ettringite that has formed at a later stage when the concrete has hardened. As noted above, ettringite may decompose and reform at a later stage. This may occur owing to excess sulfates in the cement composition. The development of DEF in the paste matrix proceeds in four stages:

1. Localized ettringite growth in situ or in nest.
2. A further growth of ettringite accompanied by characteristic crack patterns and formation of gaps around aggregate particles.
3. The further growth of ettringite or filling of ettringite within the developed cracks.
4. The further growth of ettringite or filling of ettringite within the developed gaps.

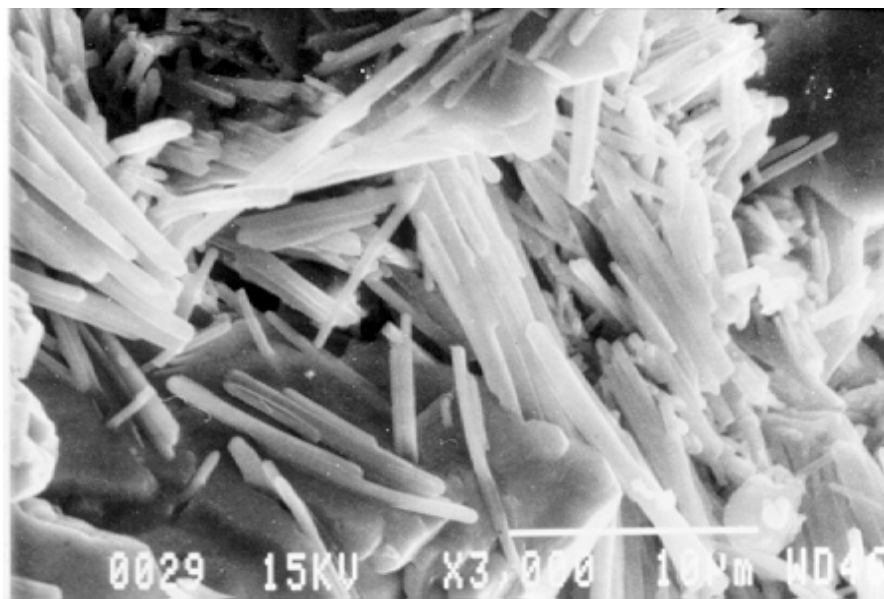


Figure 2.12: Ettringite Growth under SEM Examination at X3000 Magnification

When the second stage of DEF is reached, severe cracking of the concrete may result. Severe concrete cracking, which was attributed to DEF, is illustrated in Figure 2.13. At this time, it is very difficult to address how large the problem of DEF could be. It has been suggested that previous cases of DEF attributed to alkali-silica reaction (ASR) could have been misdiagnosed under low magnification petrography. Many designers such as TxDOT engineers are just beginning to try to address in-service structures with this problem. Methods such as epoxy injection to seal all cracks are really not practical unless one can define the level of deterioration that will take place. It certainly is not practical, if not impossible, to modify the exposure condition of a bridge structure to keep

moisture away. To date, the industry has attempted to address this problem by some quality control measures such as recommending air entrainment for example.



Figure 2.13: Severe Concrete Cracking Attributed to Delayed Ettringite Formation

Some contend [146] that the air voids provided by air entraining can act as relief valves or reservoirs for ettringite to grow or expand into, thereby reducing the concrete damage owing to the expansive reaction. Figure 2.14 illustrates growth of ettringite in an air void. The use of air-entrained concrete might reduce damage owing to DEF; however, it does little to prevent further concrete damage in severe exposures owing to freeze-thaw.

One of the conditions that has been reported to drastically influence the development of DEF is the curing temperature or condition [146]. Reports throughout Europe recommended trying to maintain low curing temperatures without accelerated steam curing. In Germany, the specifications recommend concrete temperatures below 60 °C (140 °F). This may prevent optimum conditions for DEF to occur, but certainly does not guarantee that DEF will not develop as has been seen in several of the case studies within the state of Texas [30].

Naturally, one area that should be closely monitored is the sulfate content in the clinker and cement. Currently, ASTM C150 limits the total sulfate content of the gypsum and clinker. However,

the specification currently does not control the level of SO_3 within the clinker, which is critical in the development of DEF. Perhaps this is one area that should be more closely investigated.

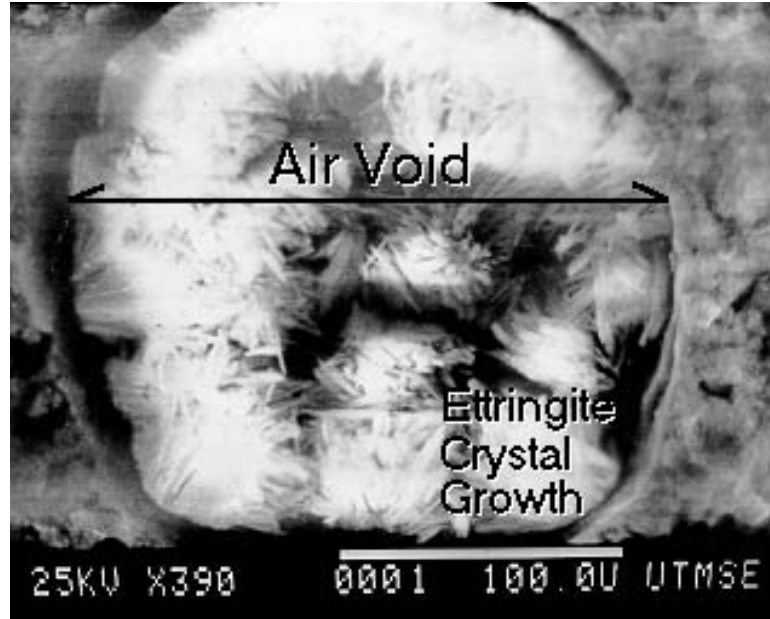


Figure 2.14: Ettringite Growth in an Air Void at X390 Magnification

2.3.1.2 Freeze-Thaw Damage. Frost damage to concrete is the result of successive freeze-thaw cycles of the cement paste and aggregate. Hardened cement paste is particularly susceptible to damage owing to its high relative porosity. Fortunately, air entrainment, as discussed in Section 2.1.5, has proven to be an effective means of protecting concrete from frost attack. The number of freeze-thaw cycles to denote good performance has been defined by the measured change in length of a specimen at a specified number of cycles. ASTM C-666, *“Resistance of Concrete to Rapid Freezing and Thawing,”* has been used frequently to predict freeze-thaw performance in the field.

Stark found a good correlation using a failure criterion of 0.035 percent length change at 350 cycles at 2 cycles per day [31]. Specimens that have less than 0.035 percent dilation after 350 cycles generally show good freeze-thaw performance in the field. However, it has been difficult to extrapolate the number of freeze thaw cycles in laboratory tests to the number of cycles in the field because the laboratory specimens are much smaller than actual field elements and do not experience similar cooling rates (gradients) to those in the field.

Figure 2.15 illustrates the average number of freeze-thaw cycles per year in the state of Texas. In order for a freeze-thaw cycle to be detrimental to the concrete, the concrete must reach an

internal temperature of at least -5°C (23°F) for a minimum of 6 hours and exceed a critical saturation threshold during this time period. An internal concrete temperature of -5°C (23°F) is equivalent to an ambient air temperature of -9°C (15°F) for concrete 75 mm (3 in.) below the surface. This is approximately the center of most bridge decks that are 190 to 203 mm (7.5 to 8 in.) in thickness.

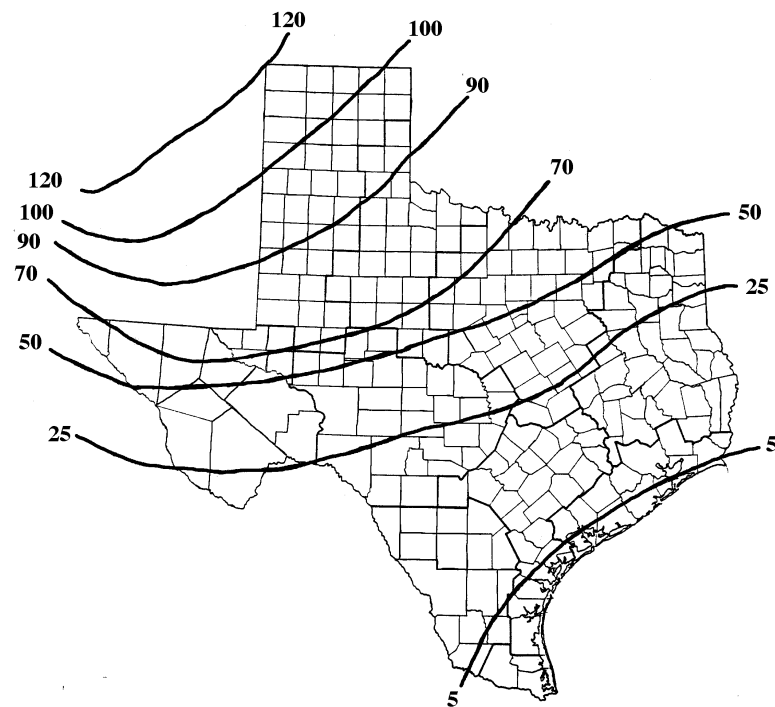


Figure 2.15: Number of Freeze-Thaw Cycles per Year [32]

Members that are not subjected to becoming saturated above the critical saturation threshold of 91.7 percent do not warrant air entrainment for freeze-thaw protection. These would include members such as beams, pier-caps, or other substructure elements not subjected to a splash zone and protected from vertical saturation by a bridge deck or other element. In the context of a bridge, the only components that generally warrant air entrainment are the bridge decks themselves.

Additionally, components such as decks that are open to air circulation on many sides generally experience a higher number of freeze-thaw cycles. Figure 2.16 illustrates the contours indicating the number of moist freeze-thaw cycles below a -9°C (15°F) ambient temperature in the state of Texas.

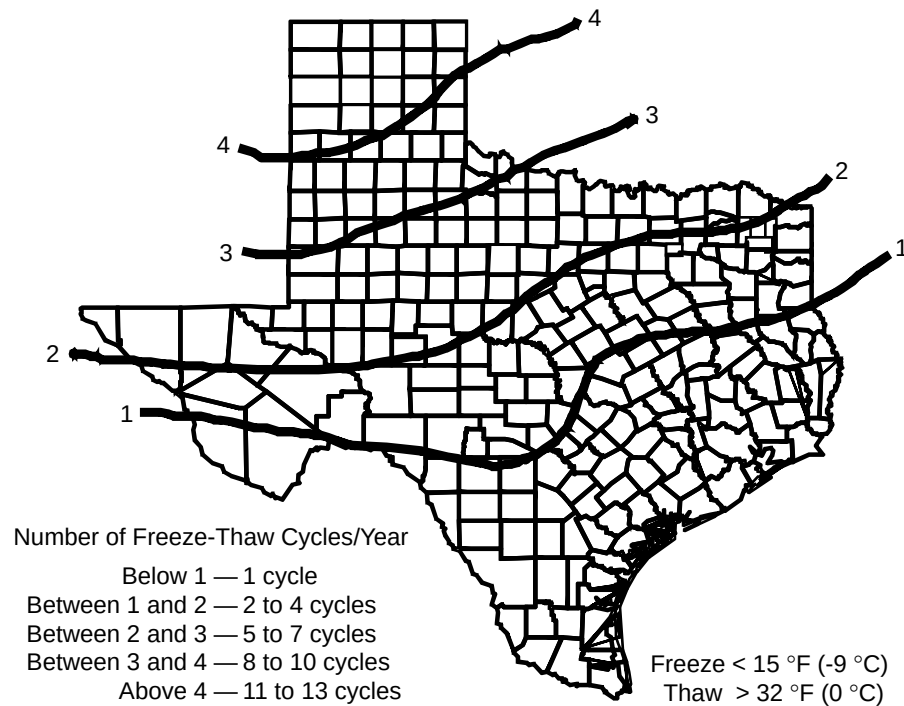


Figure 2.16: Effective Freeze-Thaw Cycles per Year [33]

2.3.1.3 Alkali-Aggregate Reaction. The degradation of concrete owing to alkali-aggregate reaction was identified for the first time in the United States in 1940 [34]. Disorders occur at various ages of structures from 2 to 10 years or even at later ages. The signs of distress may include the following [34]:

- ◆ Map cracking.
- ◆ Exudation of calcite and alkaline silicate gel.
- ◆ Pop-outs owing to superficial reactive aggregates.
- ◆ Movements due to displacement of structures such as swelling of a dam crest.
- ◆ Coloration or discoloration mainly along cracks.

The main factors responsible for alkali-aggregate reactions are a humid environment, a high concentration of alkalis in the interstitial liquid phase of concrete, and alkali-reactive particles in the aggregates. Reactions that are complex and heterogeneous occur between the liquid localized in the capillary pores and solid particles irregularly distributed in the material. Chemical and physical

processes interact in the formation of silicate gel, which can swell and crack the concrete. The alkali-silica reaction is the most frequent case.

2.3.1.3.1 Alkali-Silica Reaction. ASR is a chemical reaction between the reactive siliceous components of an aggregate and the hydroxyl ions (OH^-) of the concrete pore solution. The product of this reaction, called alkali-silica gel (also referred to as ASR gel), can lead to expansion and cracking of the concrete. For ASR to occur, the following preconditions must exist:

- ◆ the presence of reactive silica,
- ◆ a high alkalinity system composed of OH^- availability,
- ◆ the presence of water or a relative humidity above 80 percent, and
- ◆ the presence of calcium hydroxide.

Generally, silicas that are poorly crystallized or amorphous and porous are the most reactive. However, case studies have shown that even dense crystalline silicas can cause ASR under unfavorable conditions of concrete production and use. The development of ASR is considered to occur in four stages [3]:

1. initial alkaline depolymerization and dissolution of reactive silica,
2. the formation of a hydrous alkali silicate gel,
3. the attraction of water by the gel, and
4. the formation of a fluid sol (a dilute suspension of colloidal particles).

The comprehension of the factors affecting alkali-aggregate reaction can be used to control its effects in concrete. The following approaches can be used [3]:

1. control of the pH in the concrete,
2. control of alkali concentrations,
3. control of the amount of reactive silicas,
4. control of moisture, and
5. alteration of alkali-silica gel.

In addition, mineral admixtures are commonly known to control the expansions associated with alkali-aggregate reaction. Pozzolans react with the calcium hydroxide in the paste and, thus, lower the pH of the pore structure and concrete. Susceptible aggregates should be avoided based on service records and petrographic examinations. For exterior structures it is impractical, if not

impossible, to prevent moisture migration to the structure as required for the reaction to precipitate and progress. However, low w/cm ratio concretes such as HS/HPC may help limit the supply of water needed to cause alkali-silica gel to swell, owing to its very impermeable nature. Keep in mind that this approach will only slow down the reaction, not prevent it.

2.3.1.4 Corrosion of Reinforcement. The annual cost of corrosion in the United States has been estimated at \$30 billion while corrosion of bridge decks alone has been estimated at \$500 million [52]. Corrosion not only represents a tremendous economic loss, but also, oftentimes little is done to prevent or reduce its effects in concrete structures. By its very nature, concrete is a high alkaline environment that protects and passivates the carbon steel reinforcement against corrosion. However, when concrete exists in an aggressive environment, this protection can be destroyed, especially if the pH of the concrete reaches 11 or lower [53]. The passive iron oxide layer of the carbon steel reinforcement is destroyed when the pH is reduced below 11 causing the porous oxide layer (rust) to form during corrosion.

The formation of this oxide layer (rust) on the surface of the carbon steel reinforcement exerts a force on the surrounding concrete initiating cracking and eventually spalling of the concrete. This critical reduction of pH occurs in the concrete when calcium hydroxide is converted to calcium carbonate (calcite) by atmospheric carbonation. The calcium hydroxide is responsible for maintaining the high pH in the cement paste and, thereby, the concrete. In well-cured concrete with a low w/cm ratio, the depth of the carbonated zone is unlikely to exceed 25 mm (1 in.), and therefore concrete cover of 25 to 40 mm (1 to 1.6 in.) over reinforcing bars should provide adequate protection from corrosion in most instances [3]. Research studies have shown that low w/cm ratio concretes have an increased time-to-corrosion owing to the reduced porosity over conventional concretes [2].

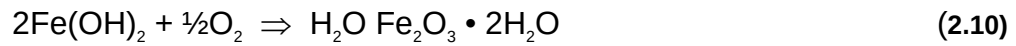
Corrosion in concrete can take the form of a galvanic or an electrochemical reaction. For galvanic corrosion to occur, two dissimilar metals must come in electrical contact in the presence of oxygen and moisture to form an anode and a cathode. Oxidation-reduction processes can then take place at the surface of the two metals, and under normal conditions, iron becomes the anode and oxidation of the iron occurs. This anodic reaction is represented by the following reaction:



The electrons released by the anodic reaction are transported by either the steel rebars or the pore solution (electrolyte) to the cathode where an equivalent quantity of hydrogen ions forms a protective film around the steel. The destruction of the hydrogen film is achieved by the reduction of the oxygen. This cathodic reaction is represented by the following reaction:



The hydroxide ions are carried back to the anode through the rebar where they will react with Fe^{++} to form ferrous hydroxide. In the presence of excess oxygen, the ferrous hydroxide is unstable and oxidizes further into ferric salt or red rust:



This corrosion process in concrete is represented schematically in Figure 2.17.

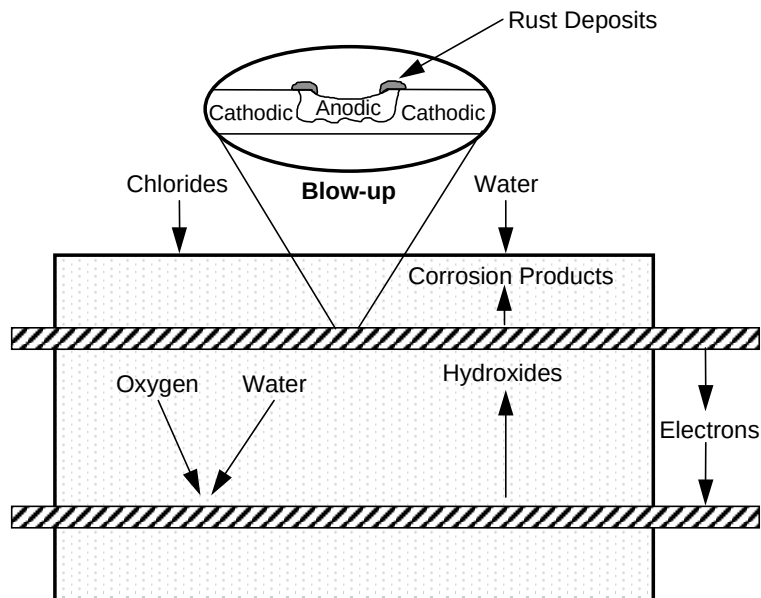


Figure 2.17: Schematic Representation of the Corrosion Cell

In terms of corrosion protection, interruption of the chloride ions, oxygen, water, or electron flow will stop the entire corrosion process. The following corrosion protection systems approaches have been used to reduce or eliminate corrosion damage to the bridge reinforcing steel [54]:

1. concrete barrier system,
2. reinforcing steel coating,
3. chemical stabilization,
4. surface sealers, and
5. electrochemical systems.

The most direct approach to preventing or reducing corrosion-related damage is to prevent the chlorides in the concrete surface from reaching the reinforcing bars. This approach is represented by the use of overlay systems such as dense concrete, asphalt-based membranes, latex-modified, and polymer concrete overlays. Other methods such as low w/cm ratio, use of mineral admixtures, and providing increased cover have also proven to be effective. Each of these methods has proven performance in the absence of surface cracking that allows for a more direct path for the chlorides to ingress toward the steel reinforcing.

The primary steel coating method specified over the last decade has been the epoxy-coated reinforcing bar. Many studies have investigated epoxy-coated reinforcing [55, 56, 57, 58, 59] and have found the overall performance to be good. The major weakness of epoxy-coated bars is their susceptibility to surface coating damage. These problems are best avoided through care in fabrication and construction, repair of surface damage, and education for the installers. An estimated increase in 1 to 3 percent in the overall project costs can be anticipated when using epoxy-coated rebar.

Chemical stabilization additives such as calcium nitrite have been used to modify the environment within the concrete. Calcium nitrite, traditionally used as an accelerator, acts to stabilize the passivating layer that surrounds uncorroded steel in concrete. It does not influence the permeability of the concrete, but rather it competes with the chloride to react with the steel and subsequently reduces the corrosion rate by “more than an order of magnitude” [60]. One of the drawbacks with calcium nitrite is that a retarder must be used in conjunction with it to offset its accelerating properties.

Sealer protection methods include silane and siloxane sealers, high molecular weight methacrylates, and linseed oil. Successful sealers are those that are “breathable” and allow water vapor to exit the concrete while preventing liquid water from penetrating. If water vapor cannot pass through a sealer to escape from the concrete, high vapor pressure can build up inside the concrete resulting in blistering and peeling of the sealer. These systems can be effective at preventing water and chloride migration, but often require re-application every 3 to 5 years, increasing the maintenance costs of the structure.

Electrochemical systems such as cathodic protection have also been selected as a means of corrosion protection. Cathodic protection works by providing an external electric potential that opposes the “natural” corrosion cell [54]. The electric potential established by the cathodic protection system prevents the flow of electrons associated with the reinforcement corrosion and, thereby, stops the corrosion process. Drawbacks of this system include the high initial expense and maintenance of the system, the additional dead load applied to the structure because of the system, the external

power supply that is required to operate the system, and the technical personnel who are required to monitor the system. The system's main merit is that it is the only corrosion protection method to date that completely stops the corrosion process.

Perhaps the most cost-effective system to combat corrosion in developing bridges for a 75 to 100-year design life is a combination of these systems. As described in subsequent sections of this report, the long-term durability study of the North Concho River Overpass project will investigate the combination of epoxy-coated reinforcing with a low permeable low w/cm ratio HPC, and a surface sealer. This system will be compared to conventional concrete with and without epoxy-coated reinforcing.

2.3.2 Permeability of Concrete

Permeability has long been considered one of the most important aspects in durability performance because the permeability of the concrete controls the rate of entry of moisture that may contain aggressive chemicals and the movement of water during heating and freezing.

2.3.2.1 Mechanisms of Concrete Permeability. The permeability of concrete can be measured by determining the rate of flow through a concrete slab. The mechanisms involved in the permeability of concrete are dependent upon the penetrating solutions (water, fluids, gases, or ions), pressure head, temperature, and the initial state of the concrete. Whiting [61] has listed seven stages to describe the movement of water through porous materials that are as follows:

Stage 1: Absorption and surface diffusion.

Stage 2: Vapor diffusion.

Stage 3: Film transfer.

Stage 4: Surface creep plus vapor diffusion.

Stage 5: Partially saturated liquid flow.

Stage 6: Liquid flow.

Stage 7: Ionic diffusion.

The first two stages consist of initial surface absorption followed by water vapor movement, which is diffusion controlled. Diffusion is mathematically expressed by Fick's first law [61] given in Equation 2.11:

$$J = -D \, dc/dx \quad (2.11)$$

where:

$$\begin{aligned} J &= \text{Flux (g/cm}^2\cdot\text{s)} \\ D &= \text{Diffusion coefficient (cm}^2\text{/s)} \\ dc/dx &= \text{Concentration gradient (g/cm}^3\text{/cm)} \end{aligned}$$

As relative humidity increases, water begins to condense within the capillary spaces (Stage 3). Owing to the pressure differential across the meniscus in the capillary, the resulting force induces a flow through the capillary, the rate of which is given by Washburn's equation [61] as expressed in Equation 2.12:

$$v = (rV_L/4dn) \cos \theta \quad (2.12)$$

where:

$$\begin{aligned} v &= \text{Flow rate (cm/s)} \\ r &= \text{Capillary radius (cm)} \\ V_L &= \text{Surface tension (Pa}\cdot\text{cm)} \\ d &= \text{Depth of penetration (cm)} \\ n &= \text{Fluid viscosity (Pa}\cdot\text{s)} \\ \theta &= \text{Contact angle} \end{aligned}$$

Upon complete saturation of the pore system (Stage 6), water flows as a fluid governed by Darcy's law [61] expressed in Equation 2.13:

$$Q = -KA \Delta h/L \quad (2.13)$$

where:

$$\begin{aligned} Q &= \text{Volume outflow (cm}^3\text{/s)} \\ A &= \text{Area (cm}^2\text{)} \\ \Delta h &= \text{Fluid head (cm)} \\ L &= \text{Path length (cm)} \\ K &= \text{Permeability constant (cm/s)} \end{aligned}$$

Chloride ions and other ionic species can migrate through free water within the concrete pore structure. This phenomenon may occur because of diffusion at low velocity owing to ionic interaction with the electric double layer on the cement paste surface during Stages 3 and 4, or more effectively, during Stage 7 in the fully saturated state. The diffusion process of an ionic species into a porous

material is altered when an electric potential is applied across an electrolytic solution [62]. Ions will migrate toward the opposing sign electrode with an ionic mobility (μ) given by the following expression:

$$\mu = x/t(dE/dx) \quad (2.14)$$

where:

μ	=	Ionic mobility ($\text{cm}^2/\text{V}\cdot\text{s}$)
x	=	Distance (cm)
t	=	Time (s)
dE/dx	=	Electric field strength (V/cm)

2.3.2.2 Effect of W/Cm Ratio on Permeability. The single parameter that has the largest influence on durability is the w/cm ratio of the concrete [2]. As the w/cm ratio is decreased, the porosity of the paste is decreased resulting in improved impermeable concrete. In fact, the w/cm ratio plays a dual role in concrete durability. The resulting increase in strength owing to a lower w/cm ratio also improves the concrete's resistance to cracking from the internal stresses that may be generated by adverse reactions. The permeability coefficient, K , is dependent upon the w/cm ratio, the age of the paste, and the capillary porosity.

According to Mindness and Young [3], the interconnecting network of capillary pores is what governs the permeability of the cement paste. As hydration proceeds, the formation of C-S-H breaks the capillary network and blocks the interconnected pores resulting in a decrease in K and in the concrete permeability. Table 2.3 illustrates the curing time required to produce a discontinuous system of capillaries. For a fixed w/cm ratio, the permeability coefficient, K , decreases and the capillary pores become more discontinuous as the cement paste age increases as illustrated in Table 2.4. The capillary pores are the smallest at complete hydration (w/cm = 0.30) as illustrated previously in Figure 2.3. By their very nature, HS/HPCs are very durable concretes with generally very low permeability.

The w/cm ratio selected (typically 0.27 to 0.40) to develop the high strengths are often very low, resulting in discontinuous capillary systems at very early ages (less than 3 days). In fact, in precast/prestressed concrete where very high early strengths are required for release of the prestressing strands, a discontinuous capillary system occurs within the first 24 hours when the majority of the cement hydration occurs. This, coupled with the prestressing forces applied, which reduce any crack widths, explains the excellent durability performance of precast/prestressed elements over the years. Neville [63] also illustrates in Figure 2.18 how the permeability coefficient

increases with an increase in the w/cm ratio. It may be inferred from this discussion that one of the most significant means to impact the durability performance of a mix design is the w/cm ratio.

Table 2.3: Curing Time Required to Produce a Discontinuous System of Capillaries [3]

W/Cm Ratio	Curing Time (days)*
0.40	3
0.45	7
0.50	28
0.60	180
0.70	365
>0.70	Not Possible

* Continuous Moist Curing

Table 2.4: Effect of Age of Cement Paste on Its Permeability Coefficient (W/Cm = 0.51) [3]

Age (days)	Kp (m/s)	
Fresh Paste	1E-05	Independent of W/Cm Ratio
1	1E-08	Capillary Pores Interconnected
3	1E-09	
4	1E-10	
7	1E-11	
14	1E-12	
28	1E-13	
100	1E-16	Capillary Pores Discontinuous
240	1E-18	

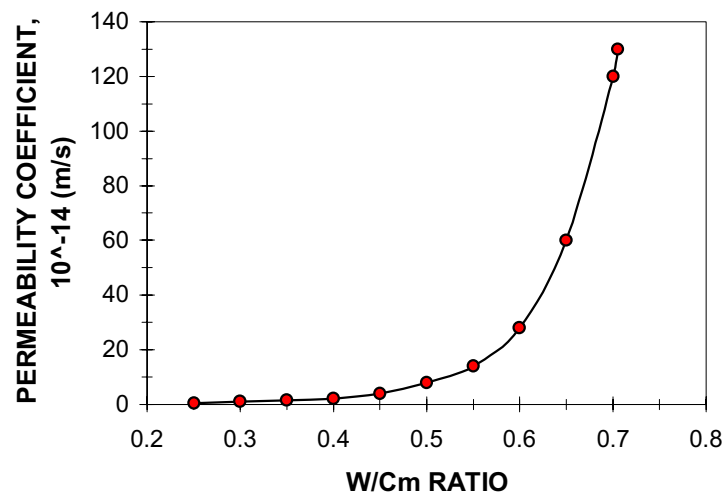


Figure 2.18: Effect of Water-to-Cement Ratio on the Permeability of Concrete [63]

2.3.2.3 Effect of Curing and Concrete Temperature on Permeability. Curing of concrete is a procedure used for promoting the hydration of cement and is not only used to control the temperature and moisture diffusion from and into concrete, but also to provide the necessary environment for complete hydration to occur. As previously discussed in Section 2.2.1.3, moist curing not only significantly influences the strength development, but also impacts the permeability of the concrete. The longer the moist-curing period, the higher the strength development and the lower the permeability [63].

The curing temperature has also been reported to influence the pore size distribution of a cement paste. According to Young [64], at higher curing temperatures there is an increase in the volume of large pores, which increases the permeability of the cement paste. The influence of moist curing and concrete temperatures on permeability is further investigated in Chapters Six and Eight of this report.

2.3.2.4 Effect of Coarse Aggregates on Permeability. The selection of coarse aggregates also plays a role in the permeability of concrete. The use of highly porous aggregates will increase the permeability of the concrete because substances can flow more easily through the aggregate pores than through the smaller pores of the cement paste [63, 65]. Many soft limestones that are high in calcite are generally considered porous. Many of these limestones can be found in South Texas and throughout parts of Mexico. Low permeable or nonporous, hard, dense aggregates such as river gravels will reduce the effective area over which substance flow can take place and, therefore, increase the effective flow path length.

As a result, the permeability of the concrete will test lower when compared to a bulk cement paste with the same w/cm ratio and degree of maturity [63]. However, Metha [66] indicated that in practice, the permeability of concrete can exceed that of a cement paste with the same w/cm because of the microstructure of the transition zones and the bleed water that is trapped under the coarse aggregate particles. This bleed water can cause regions of locally high w/cm ratios resulting in additional calcium hydroxide crystal growth. The formation of new hydration products may result in microscopic transition zone cracking, which can facilitate the formation of an interconnected capillary network, increasing the permeability of the system [65].

As discussed in Section 2.1.2, the transition zones are quite different for HS/HPC when compared to conventional concretes. Microcracking in the transition zones is often not an issue for HS/HPCs because the transition zones are not the *weak link* and microcracking does not occur at low-to midrange stress levels, which may be the case in conventional concretes. The selection of aggregates for a mix design is an important constituent in which permeability is considered an important parameter of concrete performance.

2.3.2.5 Effect of Fly Ash on Permeability. Several studies have demonstrated that the incorporation of fly ash in the concrete matrix will reduce its total porosity and result in a finer pore structure compared to the matrix without any fly ash [68, 69]. Other studies have suggested that the pozzolanic reaction of the fly ash has the tendency to break the interconnected pore system more readily [70]. Hooton at the University of Toronto, among others [71], contends that the fly ash binds a significant amount of the free chloride ions that can ingress into the concrete from the surface. The binding of these chloride ions reduces the amount of free chloride ions that are available to reach the level of the reinforcing steel to initiate corrosion.

The permeability of concrete containing fly ash is greatly affected by the curing process. Studies at UT by Tikalsky [51] have shown that the use of fly ashes increases the concrete permeability at early ages, but improves it at later ages. This is due to the slow process by which the pozzolanic reaction occurs.

2.3.2.6 Effect of Air Entrainment on Permeability. As discussed in Section 2.3.2.2, a continuous capillary system will result in a higher permeability, while a discontinuous system will prevent the diffusion of fluids into the concrete and result in a lower permeability. In conventional concretes, an adequately designed air-entrained system will create discontinuities in the capillary pores within the paste and result in decreased permeability. However, research studies have shown [14, 62] that for HS/HPCs, the addition of air entrainment does not drastically affect the permeability of the concrete. HS/HPCs typically have a very low capillary porosity owing to their low w/cm ratio as previously illustrated in Figure 2.3. Because a capillary pore system is very limited at best in HS/HPC's, the addition of an air void system does not create any new discontinuities within the system to reduce the permeability. It should also be noted that unless the air void system is properly distributed, it can result in creating a continuous system of voids that will increase the permeability of the concrete. The presence of entrained air will reduce bleeding water and increase the workability owing to its spherical particle shape. As a result, a lower w/cm ratio is allowed, which results in a lower permeability [63].

2.3.2.7 Effect of HRWR on Permeability. The use of HRWRs has been shown to reduce the permeability of concrete by allowing the reduction in the w/cm ratio and the distribution of cement particles. The HRWR helps disperse the cement grains more uniformly within the paste resulting in a pore structure with fewer coarse pores. This results in a reduced permeability. However, research conducted at The University of Texas at Austin [67] has shown that the use of an HRWR can increase the permeability of the concrete when compared to a control mix without an HRWR during hot-weather concreting under certain conditions. The research was conducted on bridge decks that were cast during both cold and hot weather. No associated increase was noted during the cold weather

placement. The increase in permeability during the hot weather placement was attributed to difficulty in finishing. The problems associated with finishing the low w/cm ratio mix resulted in deep surface cracks and, thereby, a more permeable deck.

2.3.3 Freeze-Thaw Resistance of Concrete

Section 2.3.1.2 introduced and discussed the mechanism of freeze-thaw damage. To date, standard freeze-thaw tests using ASTM C666/AASHTO T161 have been used to predict the freeze-thaw performance of concrete. A parameter known as the durability factor, DF, was introduced to characterize concrete performance. The DF is expressed in the following expression:

$$DF = \frac{(\% \text{ of Initial Modulus})(\# \text{ Cycles})}{(\text{Duration of Test})} \times 100 \quad (2.15)$$

The relative modulus is the dynamic modulus, determined by using ASTM C215, at a given number of cycles divided by the original dynamic modulus of the specimen before testing. The duration of the test is typically 300 cycles.

2.3.3.1 Factors Affecting Freeze-Thaw Resistance. There are two conditions that must be present for freezing and thawing damage to be a concern as discussed in Section 2.3.1.2. Concrete must undergo sufficient freezing and thawing temperature cycles in the presence of adequate moisture. If either of these two conditions is not met, no damage will occur. Moisture is present in concrete in one of three forms or transitions of these forms. These include chemically bonded water, adsorbed water, or capillary pore water.

The form of moisture within the concrete determines its freezing properties. Chemically bonded water or interlayer water is water that is chemically bonded to the C-S-H lattice. Adsorbed water is a film of nonfreezable water adsorbed on the C-S-H particles, which is physically bound on or between surfaces by adsorptive forces. Adsorbed water can become freezable if it desorbs to become capillary water. Capillary pore water is freezable water that is located in the capillary pores of the concrete between hydrated cement grains or in the capillary pores of the aggregate transition zones. It is free to move throughout the pore system and is similar to bulk water. However, the size of the pore in which it is located and the concentration of alkalis affect the freezing point and vapor pressure of the capillary pore water.

Deicing chemicals are also important criteria to consider because they have been known to greatly accelerate the amount of damage in concrete. The use of deicing salts on exposed surfaces can create freeze-thaw effects by changing the freezing point of the water within the concrete. The

deicing agents cause the ice to melt at the surface of the concrete and the thawed water to pond on the surface. The thawed water at the surface of the concrete can now saturate into the concrete which can create a depression in the freezing point to as low as -50°C (-58°F) [33]. Damage caused from deicing salts is typically in the form of popouts or scaling. The chlorides present in the deicing chemicals can also lead to corrosion of the reinforcing, which manifests itself as severe spalling and cracking of the concrete.

The size of the concrete mass and location in the structure affect how many times the element will freeze in a given winter. Elements with large dimensions and a large volume make it less susceptible to freeze-thaw cycles than concrete members with smaller properties because of the insulating properties of the larger sections. Concrete structures freeze from the outside to the inside. Structures that are open to air flow or circulation on many sides will generally experience a higher number of freeze-thaw cycles. A faster rate of freezing will cause more damage because the pressure from the freezing water builds very quickly before expanding water can migrate to relief voids. The magnitude of hydraulic pressure in the paste is affected by the permeability of the paste, the distance to the nearest unfilled void, the rate of freezing, and the degree of saturation.

2.3.3.2 Recommendations for Freeze-Thaw Resistant Concrete In Texas. A recent research study undertaken by Watkins [33] at UT provides recommendations for freeze-thaw resistant concrete based on exposure conditions and member types. The results from the analysis of member exposure conditions based on the member's susceptibility to moisture, freeze-thaw cycles, and deicing salts are presented in Table 2.5. The exposure conditions have been broken down into three exposures (mild, moderate, and severe) based on the previous criteria.

Similarly, Watkins [33] has ranked the environmental conditions in Texas as severe, moderate, or mild based on the combined effects of freeze-thaw cycles, moisture, and deicing chemicals, as shown in Figure 2.19. Taking into account both the environmental conditions and the member exposure, a recommendation on the amount of air entrainment required for freeze-thaw durability was developed in this study. Table 2.6 summarizes these recommendations based on environmental and member exposure conditions. Table 2.7 presents the total air requirement based on the maximum aggregate size of the mix design.

2.3.4 Abrasion Resistance of Concrete

Surface wear of concrete can take one of several forms. These forms of wear include cavitation, erosion, and abrasion. Both cavitation and erosion usually occur on dams, spillways, pipes, and tunnels where fluids are present. Cavitation is caused by the sudden formation and

collapse of low-pressure bubbles in liquids flowing at high velocities [35]. Erosion is caused by the action of abrasive materials carried by fluids. A third form of wear and most relevant to bridge decks is abrasion. Abrasion is the result of rubbing, scraping, skidding, or sliding of objects on concrete surfaces.

Table 2.5: Member Exposure Condition Ratings [33]

Exposure Condition	Member/Element	
Mild Exposure	• Anchors • Backup Walls • Culverts • Drilled Shafts • Driveways • Headwalls • Manholes • Prestressed Concrete Beams	• Prestressed Boxes • Prestressed Piling • Retards • Railroad Structures • Slurry Shafts • Road Signs and Anchors • Wing Walls
Moderate Exposure	• Bridge Piers* • Bridge Railing* • Bridge Substructure • Cast-in-Place Traffic Barriers • Columns	• Drilled Shafts in Water* • Inlets • Precast Traffic Barriers* • Riprap
Severe Exposure	• Approach Slabs • Bents* • Bridge Slabs* • Concrete Overlay • Concrete Pavement* • Curbs	• Dense Concrete Overlay • Direct Traffic Culvert • Gutters • Seal Concrete • Sidewalks

* Indicates that these members are currently required by TxDOT to have air entrainment [73]

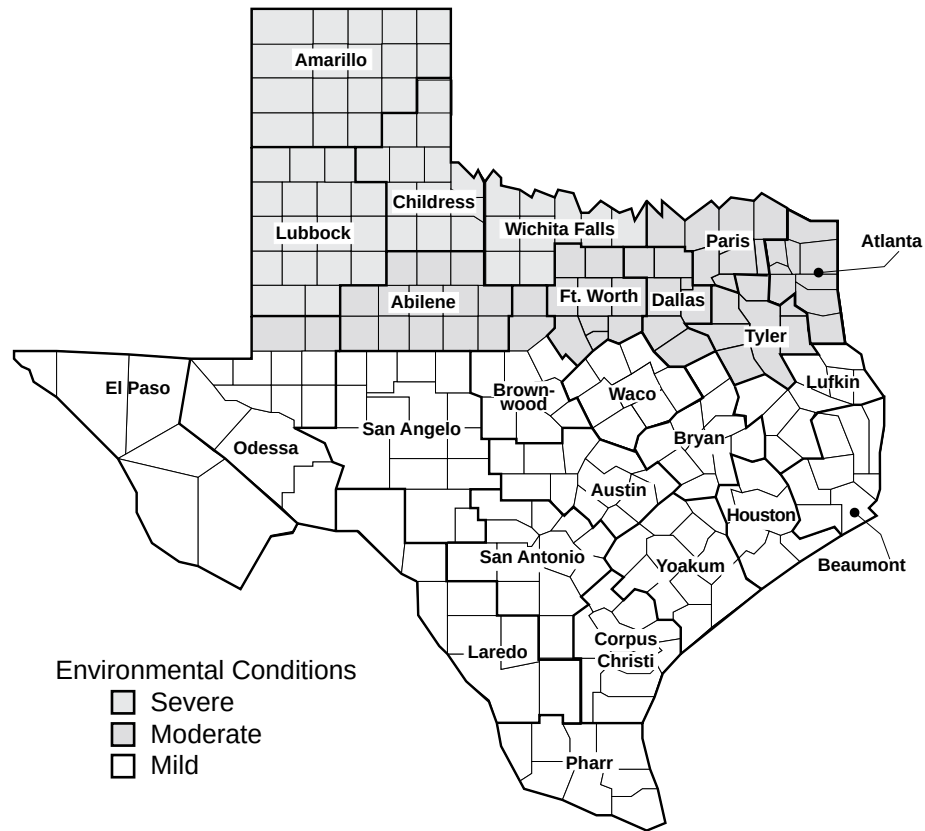


Figure 2.19: Summary of Macro-Exposure Conditions for Texas [33]

Table 2.6: Average Total Air Requirements Based on Exposure Conditions [33]

Member Exposure	Environmental Exposure		
	Mild	Moderate	Severe
Mild	None	None	None
Moderate	None	Medium Air 4.0% to 6.0%	Maximum Air 5.0% to 8.0%
Severe	None	Maximum Air 5.0% to 8.0%	Maximum Air 5.0% to 8.0%

Table 2.7: Recommended Average Total Air Content

Grade	7 & 8	6	5	4	2 & 3	1
Max Aggregate Size	3/8"	1/2"	3/4"	1"	1-1/2"	2"
Medium Air	6.0%	5.5%	5.0%	5.0%	4.5%	4.0%
Maximum Air	8.0%	7.0%	6.5%	6.0%	5.5%	5.0%

Conversion Factor: 1 inch = 25.4 mm

Until the mid 1970s, there was no specific standard test for determining the abrasion resistance of concrete. The American Society for Testing and Materials then adopted ASTM C779, “*Standard Test Method for Abrasion Resistance of Horizontal Concrete Surfaces*” [36]. In 1980, ASTM C-944, “*Standard Test Method for Abrasion Resistance of Concrete or Mortar Surfaces by the Rotating-Cutter Method*,” was issued and has been successfully used as a quality control method for concrete used in highway structures subjected to traffic.

2.3.4.1 Factors Affecting Abrasion Resistance. The primary factors affecting the abrasion resistance of concrete are:

- ◆ compressive strength,
- ◆ aggregate properties,
- ◆ surface finishing,
- ◆ curing, and
- ◆ use of surface hardeners or toppings.

Previous research has shown that the major factor influencing the abrasion resistance of concrete is the compressive strength. Air entrainment in concrete typically reduces the compressive strength and, therefore, the abrasion resistance. Backstrom and Witte [37] investigated the effects of air entrainment on the abrasion resistance of concrete. Their test results revealed that for equal strengths, air-entrainment concrete is as resistant as concrete without air entrainment. As the voids (water plus air) to cement ratio increases, both the compressive strength and abrasion resistance decreased, as illustrated in Figure 2.20.

The abrasion resistance of aggregate is also important in determining the abrasion resistance of concrete [35]. This is particularly true when an exposed aggregate surface is used. Aggregates, which are stiffer, are more wear resistance.

Proper finishing and curing significantly impact the abrasion resistance of concrete. In the case of conventional concrete, best results are obtained when finishing is delayed until all the bleed water has evaporated and the concrete has initially set. Fentress [38] investigated the effect of finishing techniques and curing on the wear resistance of concrete from the same batch. Three finishing techniques were investigated including wood float, steel trowel, and hard steel trowel. The curing techniques included moist burlap and curing compound at various stages. It can be seen from Figure 2.21 that when proper curing techniques are practiced in conjunction with a hardened finish,

improved wear resistance results. Generally, the longer wet mats and moist curing are specified, the better the wear resistance.

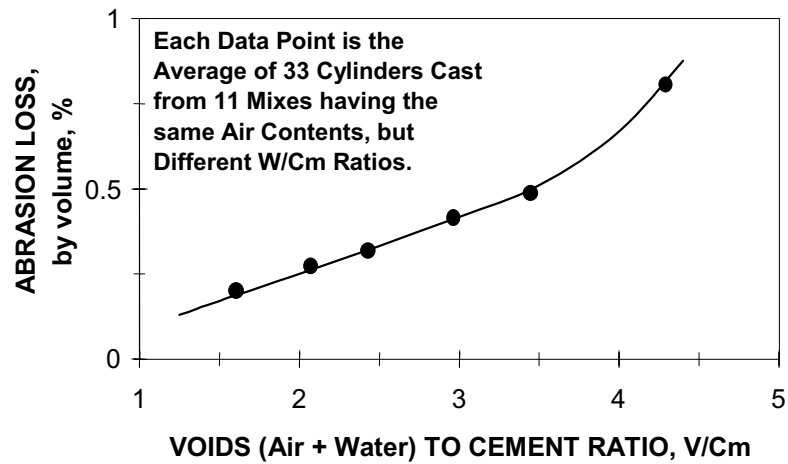


Figure 2.20: Void to Cement Ratio versus Abrasion Loss [37]

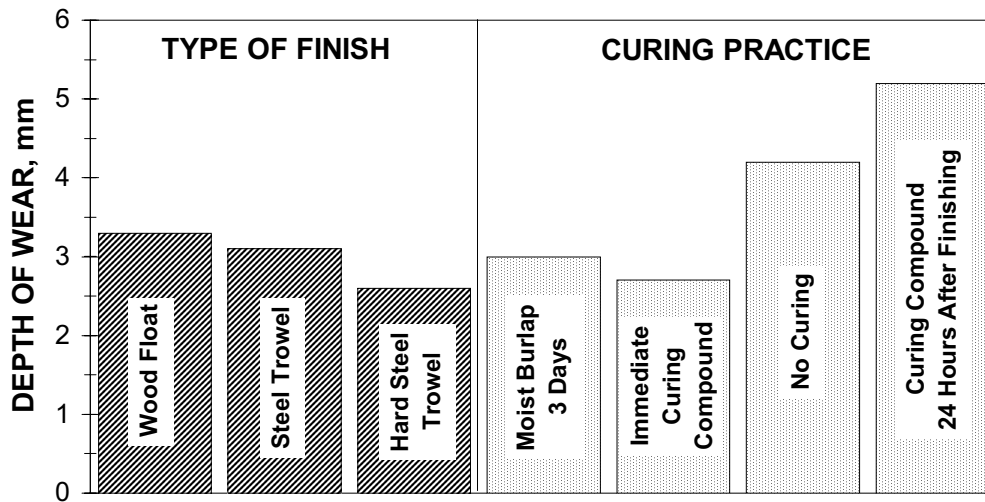


Figure 2.21: Influence of Finish Type and Curing Practice on Wear Resistance [38]

The application of special toppings or the incorporation of fiber reinforcing has also been shown to improve the abrasion resistance of concrete [39, 40]. Both Hoff and Vondran found that for concretes of equal strength, steel fiber-reinforced concrete exhibited the most resistance to wear followed by polypropylene fiber-reinforced concrete. Extensive research at The University of Texas at Austin [41] has shown that the incorporation of fly ash replacement does not reduce or improve the wear resistance of the concrete. Concrete strength is the governing factor affecting the abrasion resistance of concrete as illustrated in Figure 2.22.

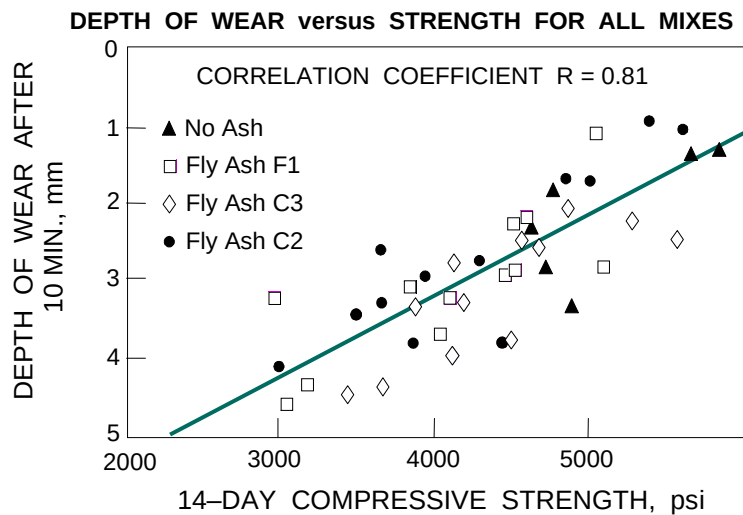


Figure 2.22: Depth of Wear versus Replacement Type [41]

One would expect that since the major factor influencing the abrasion resistance of concrete is the compressive strength, HS/HPCs would exhibit enhanced abrasion resistance over conventional concretes. This is indeed the case. High-quality aggregates are traditionally used in HS/HPC mixes in order to develop the high-strength characteristics that are desired. In addition, stiff aggregates are generally selected to satisfy elastic modulus requirements, which will also result in improved abrasion resistance. With respect to w/cm ratio and porosity, Almeida [43] found that the abrasion resistance of HS/HPC varied inversely with w/cm ratio, cement paste volume, and porosity of concretes. He also showed that the use of an HRWR improved the abrasion resistance for a given mixture by 25 percent. Almeida concluded that the superplasticizing admixtures allowed a substantial reduction in the w/cm ratio and, therefore, a substantial improvement in the abrasion resistance of the concrete.

Following with proper surface finishing and curing techniques can only improve the abrasion resistance of HS/HPC, just as with conventional concretes. However, there are some minor differences between the two types of concretes. Because bleed water is typically not a concern for

low w/cm ratio concretes, the timing of surface finishing and techniques used are less critical when compared to conventional concretes. In fact, some experts [44] feel that less finishing for HS/HPCs provides a reduced surface disruption and is actually better for the quality of the concrete.

Proper curing techniques are very critical for HS/HPCs. In climates such as Texas where environmental conditions with high temperatures, high winds, and low humidity often exist, surface moisture loss is always a concern. In this case, finishing should take place as soon as possible to avoid evaporation of surface moisture. Provisions for fogging should always be available when low w/cm ratio concretes are selected. According to Neville and Brooks [42] HS/HPC should be exposed to curing earlier than conventional concrete, because surface hydration segments the pores preventing further benefit from prolonged curing. As curing water cannot penetrate the concrete mass, it is not able to induce additional hydration. Moist curing of HS/HPC may not influence hydration as much as moist curing of conventional concrete, but it does improve the quality of the concrete. It aids in keeping hydration temperatures down and thereby improving the long-term strength gain of the concrete.

2.3.4.2 Recommendations for Abrasion-Resistant Concrete. To obtain abrasion resistance, the following measures should be followed for conventional concretes or high performance concretes with moderate w/cm ratios (0.40 to 0.50) [35]:

1. The concrete should have adequate strength levels; 27.6 MPa (4,000 psi) minimum at time of exposure to services is recommended. This may be achieved through proper mix proportioning by keeping the w/cm ratio and air content low.
2. The finishing operation of the concrete surface should be delayed until all the bleed water has evaporated and the concrete is stiff enough to withstand some pressure.
3. Adequate curing should begin immediately after the finishing operation is completed and continued for a minimum of 7 days.
4. Where the concrete is likely to be subjected to excessive abrasion, the use of hard, tough aggregates or special topping may be recommended to lengthen the service life of the concrete.

To obtain abrasion resistance, the following measures should be followed for high-strength/high performance concretes with low w/cm ratios (< 0.40):

1. The concrete should have adequate strength levels; 27.6 MPa (4,000 psi) minimum at time of exposure to services is recommended.
2. Proper fogging, as required, is recommended during concrete placement and finishing.

3. The finishing operation of the concrete surface should occur as soon as possible. Excessive finishing is not recommended as it may adversely affect the top surface of the concrete.
4. Adequate curing should begin immediately after the finishing operation is completed and should be continued for a minimum of 10 days.

2.3.5 Scaling Resistance of Concrete

Scaling occurs when the top surface of the concrete flakes or peels off. It is mainly caused by repeated freezing and thawing in the presence of deicing salts or chemicals. The scaling mechanism of concrete subjected to salt solution is very complex although there is a general consensus that the mechanism involved is mostly physical in nature rather than chemical. According to Mindness and Young [2], the consumption of heat required to melt ice when salt is applied causes a rapid drop in the temperature of the concrete just below the surface, which may cause damage either from the effects of rapid freezing or stress caused by differential thermal strains. The addition of free moisture now present at the surface of the concrete may encourage the growth of microscopic or macroscopic ice lenses near the surface, where ice formation can still occur. Osmotic pressure has also been suggested as a scaling mechanism [3, 44].

When salt is used to melt ice, the salt solution slowly penetrates the concrete surface and fills the pores of the cement paste. The capillary pores, which are filled much faster than the cement gel pores, attain a higher salt concentration. Upon freezing, ice builds up in the capillary pore and the salt concentration of the unfrozen water is further increased. As the concentration of the solution keeps increasing, the capillary pores are transformed into osmotic pressure cells capable of attracting weaker solutions from the surrounding cement gel pores. This causes the capillaries to grow and exert pressure on the cement gel, which can eventually rupture. The larger the difference in concentration between the capillary pore solution and the cement gel pore solution, the greater the pressure exerted on the cement paste. In the case of HS/HPC (w/cm ratio < 0.40), the capillary pore structure is discontinuous within 3 days after placement [3]. Because the capillary pore system is discontinuous at an early age and the differential in pore solutions would be reduced owing to a lower porosity, HS/HPCs can be expected to demonstrate improved scaling resistance over conventional concretes when finished and cured under similar conditions.

Others, such as Fagerlund [46], suggest that deterioration by scaling could be the result of the combination of hydraulic and osmotic pressures. Upon cooling, ice formation takes place first in the coarser pores. This generates localized hydraulic pressures which are influenced by the amounts of

salts dissolved in the pore solution. The higher the salt concentration, the lower the amount of freezable water at a given temperature. The hydraulic pressure should therefore decrease with an increase in the salt concentration because less ice is formed, while the osmotic pressures should increase with an increase in the salt concentration. While the imperfect understanding of mechanisms involved clearly underlines the very complex nature of the problem, many of the factors affecting scaling resistance are understood.

2.3.5.1 Factors Affecting Scaling Resistance. The primary factors affecting the scaling resistance of concrete are [41, 47]:

- ◆ materials,
- ◆ mixture characteristics,
- ◆ curing and curing procedures,
- ◆ use of sealers,
- ◆ aging effects,
- ◆ concentration of salt solution, and
- ◆ air entrainment.

The proper materials selected for mix proportioning are essential in order to obtain proper scaling resistance. The use of good sound aggregates is necessary to ensure proper durability. The cement can also affect the scaling resistance. It has been found that the use of finer cements can improve the scaling resistance [48, 49]. This can most likely be related to the reduction in the average size of the capillary pores that is observed from finer cements. Faster hydration rates caused by finer hydration products or by an increased volume of cementitious materials also reduce the risks of surface damage. Fagerlund [49] also suggests that cements with low alkali/low C_3A contents exhibit reduced scaling. Fagerlund attributes this to the fact that high alkali/high C_3A cements can have detrimental effects on air entrainment. The use of these cements can result in larger spacing factors for the entrained air bubbles. Air entrainment has proven to enhance the scaling resistance of concrete because it acts as a pressure relief point and provides a path for water osmosis. A tighter spacing factor can be expected to result in reduced damage. This is not to say that air entrainment by itself will always effectively prevent scaling. The best protection against scaling is reduced porosity [41].

The most fundamental characteristic of any concrete mixture is the w/cm ratio. The ratio influences the porosity of the paste and, thus, many of the concrete properties including, most notably, the strength and durability. Most field and laboratory data for normal-strength concrete show that as

the w/cm ratio decreases, the scaling resistance increases [49]. Since HS/HPCs generally have w/cm ratios in the 0.35 to 0.25 range, improved scaling resistance over traditional concretes can be expected. As the w/cm ratio decreases, so does the freezable water content. At a certain critical w/cm value for a given cement, air entrainment is not required for protection against freezing in the presence of deicer salts [47].

2.3.5.2 Recommendations for Scaling Resistant Concrete. To obtain scaling resistance, the following measures should be followed for conventional concretes or high performance concretes:

1. Good, sound aggregates that have a documented proven performance record should be selected.
2. Proper and timely curing of the concrete will result in improved scaling resistance. Excessive finishing is not recommended as it may adversely affect the top surface of the concrete.
3. The use of sealers or surface hardeners is recommended to enhance scaling resistance, when applicable.
4. The specification of air entrainment is recommended to enhance scaling resistance for conventional mix designs in which strength gain is not critical.

CHAPTER 3. DEFINITION OF HIGH PERFORMANCE CONCRETE AND SPECIFICATION REQUIREMENTS

3.1 Definition of High Performance Concrete

The definition of HPC has changed significantly over the years, and a consensus has not yet been reached as to its definition. According to Zia [20], HPC can be defined as any concrete that satisfies the criteria proposed to overcome limitations of conventional concretes. In many cases, the enhanced property of an HPC is strength, although this may not always be the case. In some situations the enhanced property may be elastic modulus, flexural strength, tensile strength, durability (permeability, freeze-thaw resistance, abrasion resistance, or scaling deicing resistance) constructability, or economics.

The performance requirements for the intended use of the concrete clearly dictate the definition of HPC. For the purposes of this report, when the primary enhanced property is strength, the concrete is referred to as a high-strength/high performance concrete (HS/HPC). When the primary enhanced property is something other than strength, the concrete is referred to as an HPC. The two definitions are differentiated because of the differences in production of the concretes. The production of HS/HPC involves the optimization of all materials within the mix design to optimize the strength and produce the most economical mix design. The production of HPC may only involve the incorporation or variation of one or two constituents to enhance a specific property such as durability.

3.1.1 Current ACI High-Strength and High Performance Concrete Definitions

In order to differentiate between the definition of HS/HPC and HPC, the ACI has instituted two current working definitions. The following working definition of high-strength concrete that was adopted by ACI Committee 363 is presented [73]:

Concretes that have specified strengths for design of 40 MPa (6,000 psi) or greater, but for the present time, considerations shall not include concrete made using exotic materials or techniques.

The word *exotic* was included in the definition to indicate that concretes such as polymer-impregnated concretes, epoxy concrete, or concrete made with artificial normal weight and heavy weight aggregates would not be considered high-strength concrete by ACI Committee 363 at this time. Although 40 MPa (6,000 psi) was selected as the lower limit, it was not intended to imply that there is a drastic change in mechanical properties, production and inspection techniques, or testing methods

that occur above this strength level. In reality, all changes that occur above 40 MPa (6,000 psi) are a progressive process.

ACI Special Publication SP-140, *High-Performance Concrete in Severe Environments*, has defined HPC as concrete that meets special performance and uniformity requirements that cannot always be obtained using conventional ingredients, normal mixing procedures, and typical curing practices. These requirements may include the following enhancements:

- ◆ ease of placement and consolidation without affecting strength,
- ◆ long-term mechanical properties,
- ◆ high early strength,
- ◆ toughness,
- ◆ volume stability, and
- ◆ longer life in severe environments.

For high-strength concrete, performance-based specifications are preferred to prescriptive specifications for the reasons outlined below.

3.1.2 Current Strategic Highway Research Program HPC Definition for Highway Structures

The Strategic Highway Research Program (SHRP) has investigated more than sixty concrete and structural products in order to develop an HPC definition based on performance characteristics. Previous definitions developed by Zia, Ahmad, and Leming [20] under the SHRP program initiated a trend in defining HPC by more than strength alone. Zia et al. classified HPC as concrete meeting one of three strength classifications and two durability-related criteria. The strength classifications include very high strength (VHS), high early strength (HES), and very early strength (VES) concretes as illustrated in Table 3.1. The durability criteria required a minimum durability factor of 80 percent based on the ASTM C666 Freeze-Thaw Resistance test and a maximum w/cm ratio of 0.35.

Table 3.1: Categories of HPC based on SHRP Definition by Zia et al. [20]

Category	Minimum Compressive Strength	Age
Very Early Strength (VES)	2,000 psi	6 hours
High Early Strength (HES)	5,000 psi	24 hours
Very High Strength (VHS)	10,000 psi	28 days
1 ksi = 1,000 psi = 6.895 MPa		

Following this study, an investigation conducted and summarized by Goodspeed et al. [74], investigated performance characteristics including eight mechanical and durability properties in the hopes of establishing a clear understanding of HPC. The FHWA proposed to define HPC by using long-term performance criteria. The proposed definition consists of four durability and four strength parameters as illustrated in Table 3.2. Performance is determined by using standard test methods. The specific details of the recommended test method are presented in Table 3.3. By restricting the definition to long-term performance parameters, designers who develop concrete mixture proportions may be more willing to incrementally modify mixture designs, change concrete curing procedures, and use admixtures and alternative hydraulic cements.

According to Mather [76] both the designer of concrete mix proportions and the bridge engineer play a key role. He states that for the bridge engineers to adopt an HPC performance definition, the definition must include adequate durability and strength parameters. The performance-based approach by SHRP has tried to focus on this approach. Using grades to represent performance, a bridge design engineer can specify a mixture to yield a desired concrete service life. Each parameter can be independently specified by a designated grade based on desired performance.

An example of this could be a bridge deck in San Angelo, Texas similar to the North Concho River US 87 and S.O. RR Overpass bridge presented in this report. In a climate such as San Angelo's, the bridge deck is subjected to deicing salts and a moderate number of freeze-thaw cycles. Based on the current working definition presented in Table 3.2, this may be specified by a high grade (2) to resist freezing and thawing distress, a medium grade (2) to resist scaling, abrasion, and chloride penetration, and a low grade (1) to obtain strength and elastic modulus. Naturally, the use of a performance definition alone cannot address all deterioration mechanisms.

Table 3.2: Grades of Performance Characteristics for High Performance Structural Concrete

[75]

Performance Characteristic ²	Standard Test Method	FHWA HPC Performance Grade ³				
		1	2	3	4	N/A
Freeze-Thaw Durability⁴ (χ = relative dynamic elastic modulus after 300 cycles)	AASHTO T161 ASTM C666 Proc. A	$60\% \leq \chi < 80\%$	$\chi \geq 80\%$			
Scaling Resistance⁵ (χ = visual rating of the surface after 50 cycles)	ASTM C672	$\chi = 4,5$	$\chi = 2,3$	$\chi = 0,1$		
Abrasion Resistance⁶ (χ = avg. depth of water in mm)	ASTM C944	$2.0 > \chi \geq 1.0$	$1.0 > \chi \geq 0.5$	$\chi < 0.5$		
Chloride Penetration⁷	AASHTO T277	$3000 \geq \chi >$	$2000 \geq \chi >$	$\chi \leq 800$		

Performance Characteristic ²	Standard Test Method	FHWA HPC Performance Grade ³				
		1	2	3	4	N/A
(χ = coulombs)	ASTM C1202	2000	800			
Strength (χ = compressive strength)	AASHTO T2 ASTM C39	$6 \geq \chi > 8$ ksi ($41 \geq \chi > 55$ MPa)	$8 \geq \chi > 10$ ksi ($55 \geq \chi > 69$ MPa)	$10 \geq \chi > 14$ ksi ($69 \geq \chi > 97$ MPa)	$\chi > 14$ ksi ($\chi > 97$ MPa)	
Elastic Modulus ¹⁰ (χ = modulus of elasticity)	ASTM C469	$4 \geq \chi > 6 \times 10^6$ psi ($28 \geq \chi > 40$ GPa)	$6 \geq \chi > 7.5 \times 10^6$ psi ($40 \geq \chi > 50$ GPa)	$\chi \geq 7.5 \times 10^6$ psi ($\chi \geq 50$ GPa)		
Shrinkage ⁸ (χ = microstrain)	ASTM C157	$800 \geq \chi > 600$	$600 \geq \chi > 400$	$\chi < 400$		
Creep ⁹ (χ = microstrain / pressure unit)	ASTM C512	$0.52 \geq \chi > 0.41$ / psi ($75 \geq \chi > 60$ / MPa)	$0.41 \geq \chi > 0.31$ / psi ($60 \geq \chi > 45$ / MPa)	$0.31 \geq \chi > 0.21$ / psi ($45 \geq \chi > 30$ / MPa)	$\chi \leq 0.21$ / psi ($\chi \leq 30$ / MPa)	

1. This table does not represent a comprehensive list of all characteristics that good concrete should exhibit.
2. All tests to be performed on concrete samples moist- or submersion-cured for 56 days. See Table 3.3 for exceptions.
3. A given HPC mix design is specified by a grade for each desired performance characteristic. For example, a concrete may perform at Grade 4 in strength and elasticity, Grade 3 in shrinkage, and Grade 2 in other categories.
4. Based on SHRP C/FR-91-103, p. 3.52.
5. Based on SHRP S-360.
6. Based on SHRP C/FR-91-103.
7. Based on *PCA Engineering Properties of Commercially Available High-Strength Concretes*.
8. Based on SHRP C/FR-91-103, p. 3.25.
9. Based on SHRP C/FR-91-103, p. 3.30.
10. Based on SHRP C/FR-91-103, p. 3.17

Table 3.3: Details of Test Methods for Determining HPC Performance Grades [75]

Performance Characteristic	Standard Test Method	Notes ¹
Freeze-Thaw Durability	AASHTO T161 ASTM C666 Proc. A	1. Test specimen 3 x 3 x 11 in. (76.2 x 76.2 x 279.4 mm) as cast or cut from 6 x 12 in. (150 x 300 mm) cylinder. 2. Acoustically measure dynamic modulus until 300 cycles.
Scaling Resistance	ASTM C672	1. Test specimen to have a surface area of 72 in. ² (46,451 mm ²). 2. Perform visual inspection after 50 cycles.
Abrasion Resistance	ASTM C944	1. Concrete shall be tested at three different locations. 2. At each location, 22 pounds (98 Newtons) for three, 2 minute, abrasion periods shall be applied for a total of 6 minutes of abrasion time per location. 3. The depth of abrasion shall be determined per ASTM C799 Procedure B.
Chloride Penetration	AASHTO T277 ASTM C1202	1. Test per standard test method.
Strength	AASHTO T2 ASTM C39	1. Molds shall be rigid metal or one-time use rigid plastic. 2. Cylinders shall be 4 in. dia. x 8 in. long (100 x 200 mm) or 6 in. dia. x 12 in. long (150 x 300 mm).

Performance Characteristic	Standard Test Method	Notes ¹
		3. Ends shall be capped with high strength capping compound, ground parallel, or placed onto neoprene pads per AASHTO specifications for concrete. 4. Use of neoprene pads on early-age testing of concrete exceeding 10 ksi (70 MPa), should use neoprene pads at 56-day test. 5. The 56-day strength is recommended.
Elastic Modulus	ASTM C469	1. Test per standard test method.
Shrinkage	ASTM C157	1. Use 3 x 3 x 11.25 in. (76.2 x 76.2 x 285 mm) specimens. 2. Shrinkage measurements are to start 28 days after moist curing and be taken for a drying period of 180 days.
Creep	ASTM C512	1. Use 6 x 12 in. (150 x 300 mm) specimens. 2. Cure specimens at 73°F (22.8 °C) and 50 percent RH after 7 days until loading at 28 days. 3. Creep measurements to be taken for a creep loading period of 180 days.

1. See Table 3.2 for curing period to be used before testing.

As discussed in Chapter 2, there is insufficient experience to relate laboratory test results to resistance to the wide range of field conditions encountered nationally. For this very reason, the developers of Table 3.2 recommend that information gained from local experience receive careful consideration when selecting a performance grade. The current performance definition is considered a working definition by the authors. As quality control/quality assurance methods for HPC and concrete in general improve and long-term performance results are readily available, modifications to the definition can be anticipated.

3.2 Specification Requirements for High Performance Concrete

The approach in the transportation industry has always been to use concrete with characteristics at appropriate levels to ensure satisfactory performance for the intended service life. In the past, many have tried to accomplish this through prescriptive specifications rather than performance-based specifications (Table 3.2). The reasons vary as to why the vast majority of designers/specifiers select prescriptive specifications. Many designers/specifiers want to avoid additional QC/QA requirements or the presence of additional constraints placed upon the concrete producer or contractor. The reason that is cited most often by the designers/specifiers is that the owner is unwilling to pay the additional project costs associated with the performance-related requirements.

Additionally, many design engineers and owners are shortsighted and most interested in front end costs, and they do not place a great amount of emphasis on the long-term durability performance

of exposed structures. The current state of the infrastructure serves as a living example of this. With the use of mineral and chemical admixtures in today's industry, many specification requirements for HPC not only need to be questioned, but also need to be addressed and modified. The following sections address several of the key specification requirements, which are impacted by the use of mineral and chemical admixtures.

3.2.1 Preconstruction Meeting

The sensitivity of HS/HPCs is greater than in standard conventional concretes. Small variations in mixture proportions and deviations from good testing practices can have a greater effect on the strength and other properties than in conventional concretes. Prior to construction, all of the participants should meet to clarify contract requirements, special project requirements, placement conditions, and to review the planned inspection and testing programs of the various parties. The effects of time, environmental conditions (temperature, wind, relative humidity, etc.), placing, curing, and acceptance criteria should be reviewed. The capabilities of the contractor's work force, inspection staff, and the testing and batching facilities should also be reviewed as HPC often requires additional work force needs for placement and QC/QA.

The meeting should establish lines of communication and identify responsibilities. It is especially important to review the procedures the inspector will follow when noncompliance with contract requirements is determined or suspected. This advance understanding will minimize future disputes, and contribute to a quality process. Timely and accurate reporting is a necessity for all parties involved. Trial production batches should have established a workable mixture, but it may become necessary to make adjustments due to site conditions such as weather.

Guidelines for field addition of materials should be well clarified, including pre-existing specification requirements, due to the often-low w/cm ratio of the mixes. The ready-mix producer is essential to this discussion. Only designated individuals should have the authority for the addition of any materials at the site, particularly water. No water in excess of the approved mixture proportions should be added to the HS/HPC mixtures.

3.2.2 Prequalification of Concrete Suppliers/Trial Field Mixes

ACI Committee 363 recommends that all bidders should be prequalified prior to the award of a concrete supply contract with a specified strength of 70 MPa (10,000 psi) or higher, or at least 7 MPa (1,000 psi) higher than previously produced in the local market [73]. Naturally, there is no substitution for experience, particularly in the area of HPC production. However, based on location and the

experience of the concrete producer, a more detailed prequalification process is warranted. To date, many concrete producers who are located in rural areas within the United States have little or no experience with mineral and chemical admixtures. In this situation, a more detailed prequalification process should be carried out. This would culminate with the production of a trial batch of the proposed mix proportions.

It is strongly recommended that the concrete producer attend a preconstruction meeting to clearly understand the special project needs and responsibilities. It is also recommended that plant personnel conduct a series of trial mixes to familiarize themselves with the mix design and to verify that the mixture satisfies all of the specification requirements. Quite often special aggregates, cements, or admixtures will be required to satisfy specification requirements. Preparation of laboratory trial mixes may be warranted to establish a promising mixture when historical records of established plant mixes do not satisfy specification requirements. The laboratory trial mixes shall be prepared according to ASTM C192, "Standard Method of Making and Curing Concrete Test Specimens in the Laboratory," and whenever possible, timing, handling, and environmental conditions similar to those that are likely to be encountered in the field should be approximated [77].

When formulating a mix design in the laboratory, it is necessary to bear in mind that later-age compressive strength values may vary by as much as 10 percent higher when batching in the laboratory compared to batching at the plant (Section 2.2.1.2). This may be attributed to more controlled proportioning as well as batching conditions. Once a desirable mixture has been formulated in the laboratory, field trial mixes are recommended with production-size batches at the ready-mix or precast plant. Practicality of production and of quality control procedures has been better evaluated when production-size trial batches have been prepared using the equipment and personnel that are to be used in the actual work.

3.2.3 Fresh Concrete-Related Specification Issues

3.2.3.1 Transporting. Most often concrete is transported to the job site using a transit mixer or ready-mix truck. Typically, a transit mixer can transport a maximum of 7.6 cubic meters (10 cubic yards) of concrete on open roads safely. However, when high slump concrete (>150 mm or 6 in.) is used, the specifications should address a limitation on the maximum volume of concrete that can be transported to the site. This limitation is recommended at 6.9 cubic meters (9 cubic yards) to prevent spillage due to the higher fluidity of the higher slump concretes.

Transporting time may also be an issue. Low w/cm ratio concretes are very cohesive in nature. While retarding agents and redosing of HRWRs can be used effectively to extend preset

times and, thereby, transportation times, the time interval between the addition of cement to the batch and the placing of concrete in the forms should be considered when the specifications are written.

3.2.3.2 Slump Requirements. In addition to transportation limitations, specifications need to address limitations on slump. Provisions for slump prior to and after the addition of an HRWR may be addressed as follows, based on ACI Committee recommendations as illustrated in Table 3.4.

Recommended slump values after the addition of an HRWR may depend on several conditions. These conditions may include the casting conditions or the environmental conditions at the time of placement. For example, a 203.2-mm to 254-mm (8-in. to 10-in.) slump is generally used in the precast industry when concrete is placed almost immediately after batching in preset forms that generally require limited finishing. This would also be consistent with cast-in-place drilled shafts, piers, and pier caps.

Table 3.4: ACI Committee 201 Slump Recommendations with and without HRWR

Concrete Made Using High Range Water Reducers	
Slump before adding HRWR	1 in. to 2 in. (25.4 mm to 50.8 mm)
Concrete Made without High Range Water Reducers	
Slump	2 in. to 4 in. (50.8 mm to 101.6 mm)

However, for a cast-in-place deck where a significant amount of finishing is required, a lower slump in the 152.4-mm to 177.8-mm (6-in. to 7-in.) range may be warranted if the deck is cast on a grade. Limitations on slump after the addition of an HRWR should generally not be specified, but rather all parties should discuss placement requirements at the preconstruction meeting. Should the project require a lower placement slump in the 101.6-mm to 152.4-mm (4-in. to 6-in.) range, a mid-range water reducer should be selected in lieu of an HRWR unless manufacturer recommendations dictate otherwise. The majority of HRWRs are not designed to hold slump in the mid-slump ranges and serious placement problems can occur when they are used in this capacity.

3.2.3.3 Temperature Requirements. Specification requirements related to placing temperatures for mix designs including HRWR may also need to be modified for many standard specifications. HS/HPC mix designs typically contain high contents of cementitious materials on the order of 390 to 502 kilograms of cementitious material per cubic meter (658 to 846 pounds per cubic yard) of concrete. These high contents of cementitious materials generate higher than normal heat of hydration during batching and placement. Several methods can be used to reduce the initial concrete temperature during hydration. These include the following:

- ◆ cooling material stock piles with water,
- ◆ using chilled mixing water,
- ◆ using ice as a portion of the mixing water, and
- ◆ using mineral admixtures as cement replacements.

While many of these methods may be used successfully, the ready-mix producer may still have trouble satisfying the maximum concrete temperature limit. Many specifications [72] currently specify a maximum concrete temperature of 27 °C (80 °F) for concrete delivered to the site. Temperature-related issues are a larger concern for HS/HPCs due to their higher cement contents. Preconstruction discussions should address proposed solutions or options in order to avoid concrete temperatures in excess of specification limits.

3.2.4 Curing of High Performance Concrete

The full potential strength and durability of high-strength and/or HPC will be fully developed only if it is properly cured for an adequate period prior to being placed in service or subjected to construction loading. Many acceptable methods for curing are available as outlined in ACI 308, but HS/HPCs pose additional concerns due to their inherent nature. HS/HPCs are often very dense and impermeable. They usually do not exhibit significant bleeding, and without protection from loss of surface moisture, plastic shrinkage cracking may be a concern. Fogging or fog misting can be used to alleviate plastic shrinkage concerns during placement and initial set.

According to Klieger, it is more advantageous to supply additional surface water during curing for concretes with low w/cm ratios than for concretes with high w/cm ratios [78]. Because many HS/HPC mix designs incorporate mineral admixtures such as fly ash, which contribute to later-age strength gain through pozzolanic action, moist curing is strongly recommended for a minimum of 10 days. It has been widely documented in many publications that strength gain and durability performance are the benefits of proper curing (Section 2.2.1.3). Several moist-curing methods exist including wet mat curing, water spray, and ponding. Whichever method is specified or selected for moist curing, it is important to address this issue in specifications and to monitor compliance.

3.2.5 Performance-Related Testing Issues

The general consensus is that HS/HPC is more sensitive to testing variables than normal-strength concrete [73]. Measurements of mechanical and material properties during construction provide the basic information required to evaluate the acceptability of the concrete. Factors having

little or no effect on 21 MPa (3,000 psi) concrete can have a significant effect on HS/HPC, particularly on compressive strength. Proper methods to cast, cure, prepare specimen ends, and test specimens were developed based on concretes with compressive strengths below 41.4 MPa (6,000 psi). Over the past several years researchers have begun to investigate many of these issues. The following discussion presents several of these key performance-related testing issues.

3.2.5.1 Testing Personnel and Equipment. Properly trained and experienced testing personnel, both in the field and laboratory, are of primary importance. They should have documented prior experience with HS/HPC testing and have demonstrated the capabilities necessary for certification. Testing equipment should conform to ASTM standards and be properly calibrated. The laboratory should conform to the requirements of ASTM C1077, "Standard Practice for Laboratories Testing Concrete and Concrete Aggregates for Use in Construction and Criteria for Laboratory Evaluation," and be regularly inspected by the Cement and Concrete Reference Laboratory (CCRL). Any deficiencies should be corrected promptly. In many cases, inadequate testing techniques and interlaboratory inconsistencies have been found to cause more problems than have actually occurred with the concrete. Hester [79] found differences in measured compressive strengths between laboratories to be as high as 10 percent, depending upon the mix and laboratories used. Adequate planning with review of personnel and laboratory qualifications and establishing acceptance criteria should help to prevent questions or disputes once construction has begun.

3.2.5.2 Sampling. Statistical methods are an excellent means to evaluate HS/HPC, as discussed in Chapter 5. For statistical procedures to be valid, the data should be derived from samples obtained by means of a random sampling plan. Random sampling may be defined as any portion of the material being presented has an equal chance of being selected. The samples should represent the quality of concrete supplied.

Due to the limitation of the capacity of many testing machines, 100-mm x 200-mm (4-in. x 8-in.) cylindrical specimens may be considered for the job control cylinders in lieu of larger more traditional 150-mm x 300-mm (6-in. x 12-in.) specimens when testing HS/HPCs. However, it should be documented that based on many recent research studies, the smaller 100-mm x 200-mm (4-in. x 8-in.) cylinders generally test slightly higher than companion 150-mm x 300-mm (6-in. x 12-in.) cylinders. Ost [80] found that the 150-mm (4-in.) diameter cylinders tested within about 1 percent higher than companion 150-mm (6-in.) diameter cylinders for concrete strengths in the range of 70 MPa (10,000 psi) to 140 MPa (20,000 psi). Carino et al. [81] suggest that the differences between the two cylinder sizes may be related to differences in density. Recent unpublished reports by Burg et al. [80] have suggested that where rigid upper platens are used, the test results for both sizes of test

specimens are the same. Regardless of the specimen size, the size used should be consistent with testing machine capabilities and specification requirements.

3.2.5.3 Frequency of Testing. Because many HS/HPCs contain mineral admixtures, it may be necessary for acceptance criteria to be required at later ages such as 56 or 90 days. Tests for air content, slump, temperature, and unit weight should always be made on the first truckload each day to establish conformance with previously batched trial mixes. In addition, criteria should be established for sampling a determined number of truckloads or cubic meters (cubic yards) delivered to the site. These fresh concrete tests are perhaps the easiest method for early detection that something is amiss with the mix design. It is recommended that each of these tests be clearly outlined in the specifications.

3.2.5.4 Mold Types. The choice of mold material can have a significant effect on the measured compressive strength. Most notably, the choice of molds includes steel, plastic, and cardboard. While the steel molds are reusable, only the use of rigid single-use plastic molds with a wall thickness of 6 mm (0.25 in.) or greater can ensure a cylindrical shape [82]. Plastic molds with less than a 6-mm (0.25-in.) wall thickness should have a cap to maintain a circular shape. The caps also help to prevent moisture loss, which should be promoted in all cases. Other mold materials such as cardboard molds are strongly not recommended for HS/HPC. Even high quality cardboard molds have resulted in compressive strengths about 13 percent lower than steel molds [83]. The specifications should clearly document which mold types are acceptable for QC/QA.

3.2.5.5 Specimen Preparation. Specimen preparation can be separated into three general categories: capping, end grinding, and unbonded cap systems. The first of these, capping of specimens, is required according to ASTM C39 so that the loading surfaces are plane to within 0.05 mm (0.002 in.). The thickness and strength of the cap are more important for HS/HPC than for lower strength concrete. A cap thickness for sulfur mortar caps in excess of 1.5 mm (0.0625 in.) was found to result in reduced measured compressive strength by Lobo et al. [84]. A uniform cap thickness of 1.5 mm (0.0625 in.) or less was found to be necessary for HS/HPCs. When concrete strengths are in excess of 70 MPa (10,000 psi), lower and more viable compressive strengths may be obtained due to confinement effects. Above 70 MPa (10,000 psi), tests should be performed to demonstrate the suitability of the capping material compared with cylinders prepared by end grinding.

Problems associated with capping can be resolved by end grinding. Recommended plainness of 0.025 mm (0.001 in.) and 0.3 degrees perpendicularity for concretes exceeding 10,000 psi have been specified by Pistilli et al. [85]. End grinding produces less variation in test results. Above 70 MPa (10,000 psi), end grinding also results in a higher average strength [86]. The major disadvantage of end grinding is the time factor in grinding and preparing the specimens, particularly

when a large number of specimens is to be tested. For this reason, unbonded cap systems have become quite attractive to testing laboratories and testing personnel.

Unbonded cap systems have grown in popularity because of the ease of use and preparation. The effect of end conditions on the compressive strength was investigated by Carrasquillo et al. [87]. More than 500 cylinders with strengths from 17.2 MPa to 110 MPa (2,500 to 16,000 psi) were tested with either unbonded caps or sulfur mortar caps. The study concluded that for strengths below 76 MPa (11,000 psi), the neoprene pads yielded average test results within 3 percent of the sulfur-capped specimens. The unbonded caps resulted in a lower standard deviation of within-test results (variability) and provided a cleaner, safer, cost-effective alternative.

If specifications allow the use of a neoprene unbonded capping system, they should address or provide recommendations for acceptable neoprene durometer nominal hardness. For HS/HPC strengths over 70 MPa (10,000 psi), an acceptable minimum nominal durometer hardness of 70 to 80 is recommended. Neoprene pads with a lower durometer hardness have exhibited excessive wear after a limited number of tests, resulting in a less cost-effective system. Additionally, when the softer neoprene pads (hardness = 40) have been used for testing cylinders with strengths exceeding 70 MPa (10,000 psi), they have resulted in compressive strengths about 1 percent higher than companion specimens tested with neoprene pads exceeding a hardness of 70 [88].

3.2.6 Additional Considerations: Prescriptive Specifications versus Performance-Based Specifications

Before the advent of many mineral and chemical admixtures, many design engineers tried to satisfy mechanical and material performance (durability) concerns through prescriptive specifications. These included specified requirements such as a maximum w/cm ratio and a minimum cement content to address durability concerns or a required compressive strength to satisfy not only strength requirements, but also serviceability concerns (elastic modulus, flexural strength, etc.). For conventional concretes, empirical equations have been developed over the years to relate strength to elastic modulus, modulus of rupture, and tensile strength, just as w/cm relationships have been developed to estimate durability performance. Table 3.5 illustrates how prescriptive guidelines are currently being used today to address durability concerns.

While this methodology has been successfully used for conventional concretes, the same cannot be said for HPC. It has been widely accepted that in the case of elastic modulus for example, there is not one exact empirical equation for HPC that can reliably predict the elastic modulus for all HS/HPCs.

Table 3.5: Maximum Water-to-Cement Ratios (by Weight) [89]

Exposure	W/Cm by Weight
No Freeze-Thaw, No Delcers	
Watertight Air-Entrained Concrete	
In Fresh Water	0.50
In Seawater	0.45
Frost Resistant A-E Concrete	
Thin sections, any section with less than 2-in. cover over reinforcement, and any concrete exposed to deicers	0.45
All other structures	0.50
Exposed to Sulfates (A-E)	
Moderate	0.50
Severe	0.45
Placed under Water Floor on Grade	At least 650 lb. cement per cy. Selected for strength, minimum cement requirements

Because of this, ACI Committee 363 has adopted a lower bound empirical equation that was developed by Carrasquillo while at Cornell University to provide a *conservative* approach to predicting the elastic modulus of HS/HPCs. This variability in the elastic modulus of HS/HPC, like most mechanical and material properties, is due to the quality of the constituents and their interaction. HPC has often been referred to as an *engineered concrete* for this reason. *Engineered*, because when given the required mechanical or material parameters, a mix design can be developed to satisfy said requirements. This concept of a performance-based specification was utilized on both of the HPC bridges as presented in Chapter 5 of this report. Several investigations conducted during Phase I of this research program discuss why prescriptive specifications are no longer favored over performance specifications. These are presented in Chapter 6 of this report.

CHAPTER 4. EXPERIMENTAL PROGRAM

4.1 Introduction

The experimental program of research for this project has been divided into four primary phases. Phase I describes the objectives of the laboratory trial mixes. Phase II describes the monitoring of the precast plant and ready-mix batch plant-produced concrete and the objectives of this investigation. Phase III describes the objectives and specifics of the long-term durability study of the two bridges including the variables within the study. Phase IV describes four related research studies associated with this report. These include the effects of early-age curing on the precast beams fabricated for the two bridge projects, a concrete maturity study of the cast-in-place decks, a permeability correlation study, and a study investigating the influence of air entrainment on permeability.

4.2 Phase I Research Program: Laboratory Trial Mixes

The principal objective of the Phase I Research Program was to investigate and optimize the performance of several mix designs that would be selected for use in the bridge structures. The laboratory study that was conducted looked at variables including aggregate type, cement type, fly ash replacement, air entrainment, accelerated heat curing, and proportioning. The most promising of these mixes were selected for a series of field trial mixes. Several of these mix designs were also incorporated into related research studies as described throughout this report.

In an effort to develop HPC mix designs that were economical, the use of locally available materials was emphasized. Only if the required material properties could not be attained using readily available materials in Texas would outside sources be considered. Representative stock piles of coarse aggregate and fine aggregate were delivered to CMRG at UT from a variety of sources to investigate the interaction of the various constituents. These included materials locally used by concrete producers in the Houston and San Angelo areas where the bridges would be constructed. A laboratory trial phase was initiated at the CMRG to develop and optimize the materials selected for the most promising mix designs. The mix designs for the various components would have to satisfy both mechanical property requirements and material property guidelines specified by TxDOT and FHWA as described in Section 5.4 of this report.

4.3 Phase II Research Program: Precast Plant and Batch Plant-Produced Concrete

Phase II involved the monitoring and testing of all concrete produced for both the Louetta Road Overpass Bridges in Houston and the North Concho River US 87 and S.O. RR Overpass Bridges in San Angelo. A location map of these two sites relative to The University of Texas at Austin, CTR, and TxDOT is illustrated in Figure 4.1.

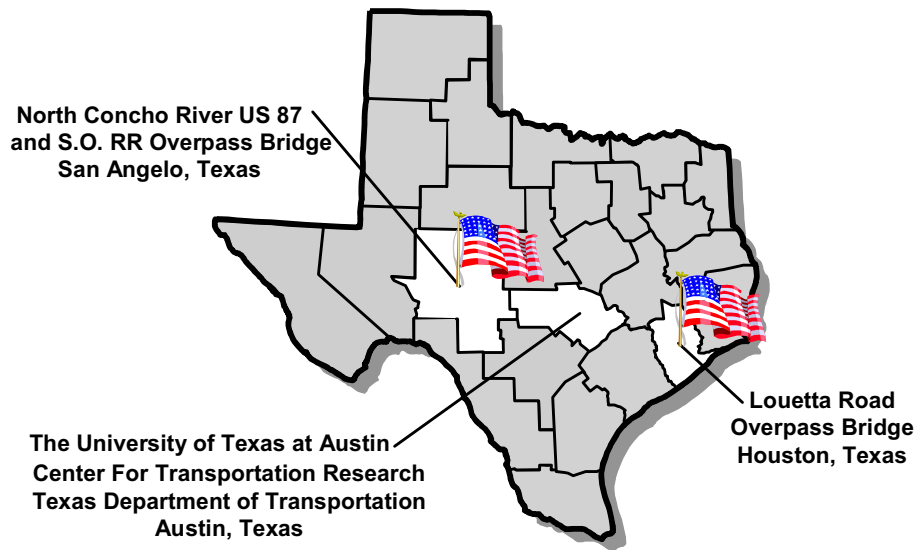


Figure 4.1: Location Map of Bridge Sites in Texas

The objective of the Phase II Research Program was twofold: 1) to provide TxDOT with additional QC/QA services to verify that project specifications and guidelines were being satisfied as described in Chapters 5 and 6 of this report; and 2) to develop a database of HPC mechanical and material properties for both the FHWA and TxDOT. The study was also to examine the statistical analysis and field performance of the HPC mix designs (Chapter 7) used in the structures.

An innovative aspect of both of these bridges is that in order to take advantage of the HS/HPC and its benefits, higher initial prestressing forces for the precast/prestressed beams would be required. To accomplish this, a series of laboratory tests on trial beams incorporating the larger 15.2-mm diameter (0.6-in.) prestressing strands was initiated to investigate transfer and development

length of the strands as well as any other associated concerns with the larger prestressing strands on the 50.8-mm x 50.8-mm (2-in. x 2-in.) standard grid system. Prior to this investigation, the FHWA did not allow larger than 12.7-mm diameter (0.5-in.) prestressing strands on the standard 50.8-mm x 50.8-mm (2-in. x 2-in.) prestressing grid system. Results of the trial tests on the beams at UT were favorable [138, 145] and resulted in the moratorium on the 15.2-mm (0.6-in.) strands being lifted by the FHWA. As shown in Figure 4.2, the 15.2-mm (0.6-in.) diameter strands have a cross-sectional area 40 percent larger than the 12.7-mm (0.5-in.) diameter strands and, therefore, replacement on a one-to-one basis results in a 40 percent higher prestressing force capacity. More detailed project descriptions are provided in the subsections below.

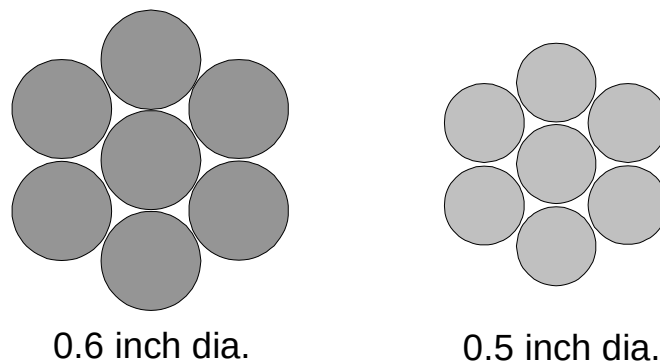


Figure 4.2: Cross Section of 0.6-in. and 0.5-in. Diameter Seven Wire Steel Strands

4.3.1 Louetta Road Overpass Bridge Project

In February 1994, the first HPC bridge project in the United States was let in the state of Texas. This was one of a series of bridge projects initiated by the FHWA to demonstrate the suitability of HPC as a means of rehabilitation and to stimulate the use of higher quality concretes in highway structures in the United States. The Louetta Road Overpass Bridge, shown in Figure 4.3, was the first of these projects to fully incorporate the use of HPC in the substructure and superstructure.

The Louetta Road Overpass Bridge consists of two three-span highway bridge structures that are part of a 4.8 km (3 mile) improvement project to Texas State Highway 249 in Northwest Harris County. A location map relative to downtown Houston is illustrated in Figure 4.4.



Figure 4.3: Louetta Road Overpass Bridge located in Houston, Texas

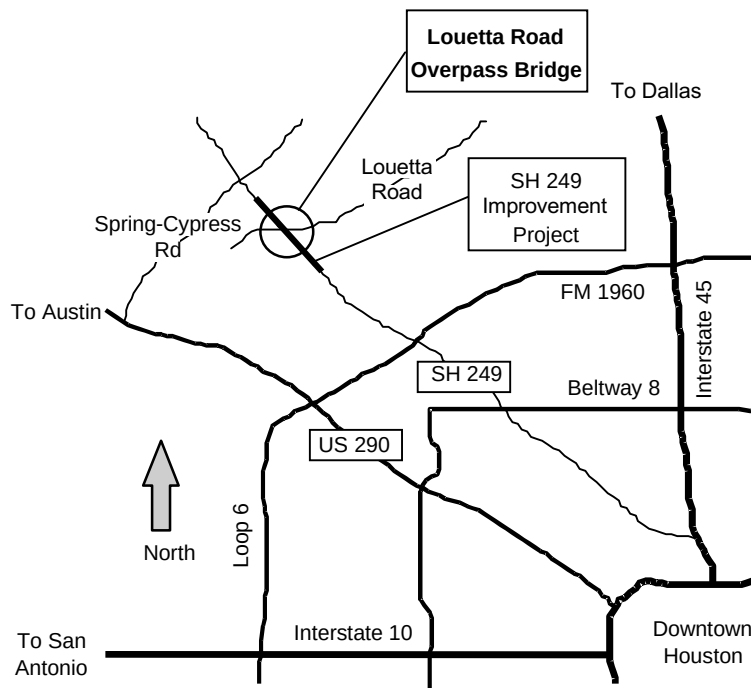


Figure 4.4: Location Map of Louetta Road Overpass Bridge in Northwest Houston, Texas

The northbound overpass was designed to carry three lanes, while the southbound overpass was designed to carry three lanes with an additional exit ramp as illustrated in Figure 4.5. After construction of the northbound and southbound overpasses was completed, the bridges were

expanded by one lane in each direction (interior location) to accommodate a projected increase in traffic flow. The expansion of these two lanes was initiated in December 1997 and completed in June 1998. The bridges were opened to traffic in both directions in June 1998.

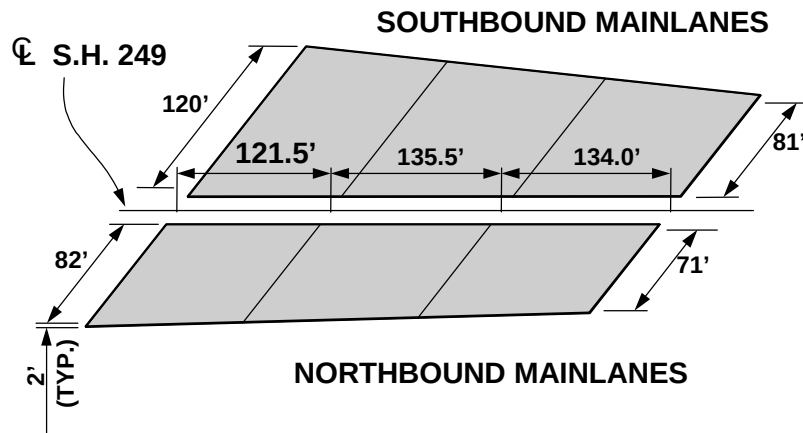


Figure 4.5: Plan View of Louetta Road Overpass Bridge

The project involved the incorporation of a nonstandard U-shaped precast beam member in lieu of a traditional AASHTO I-shaped precast member, which is widely used throughout Texas. TxDOT conceived the U-shaped member, commonly referred to as the U-Beam by TxDOT personnel, in the late 1980s. While aesthetics was one of the driving forces behind the conception of the U-beam, it was not the only design consideration. Economy, durability, function, and safety were also very important. Advantages of the U-shaped member over more traditional I-shaped members were recognized early in the project. These included reduced production requirements at the precast plant, fewer members to erect and transport to the site, and members that provide much greater stability during erection and shipping.

Figure 4.6 illustrates the cross-sectional dimensions of both the U54A and U54B. Two bottom flange widths were developed for various span conditions. The U54A (158-mm () flange thickness) allows for two vertical layers of prestressing strands in the bottom flange, while the U54B (208-mm () flange thickness) allows for three vertical layers of prestressing strands. The HPC U-Beams utilized the U54B section detailed below.

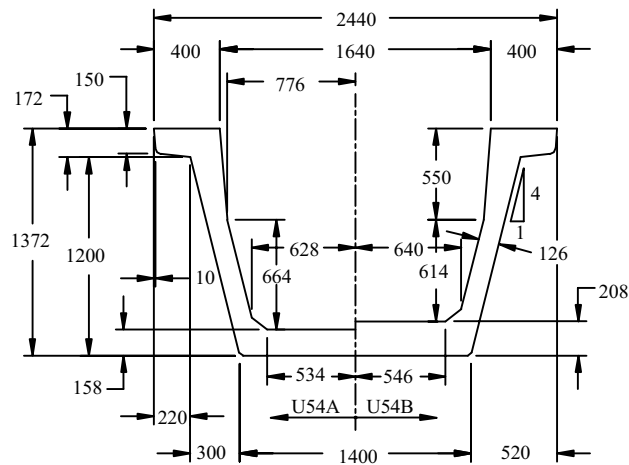


Figure 4.6: Cross-Sectional Dimensions of Precast U-Beam Section (in mm)

A hollow core 101.6-mm (4-in.) thick precast post-tensioned column element was selected as the support mechanism for the U-Beams in lieu of a traditional pier and pier cap element. The precast column element is illustrated in Figure 4.7. The use of this section allowed for a lighter, more economical substructure with improved aesthetics and a more open appearance as illustrated in Figure 4.8. Due to concerns about a reduction in substructure redundancy, the lower 1.52 m (5 ft) of the hollow column element was infilled with solid concrete. This would provide additional substructure stability in the event of impact by a vehicle.



Figure 4.7: Hollow Core Precast Column Element



Figure 4.8: Underdeck Perspective of Louetta Road Overpass Bridge

4.3.2 North Concho River US 87 and S.O. RR Overpass Bridge Project

In June 1995 the second HPC bridge project in the United States was let, also in Texas. The North Concho River US 87 and S.O. RR Overpass Bridge, shown in Figure 4.9, is located in San Angelo. The project is located due west of central downtown San Angelo, as illustrated in Figure 4.10, and intersects the two major thoroughfares in the San Angelo metropolitan region.



Figure 4.9: North Concho River US 87 and S.O. RR Overpass Bridge in San Angelo, Texas

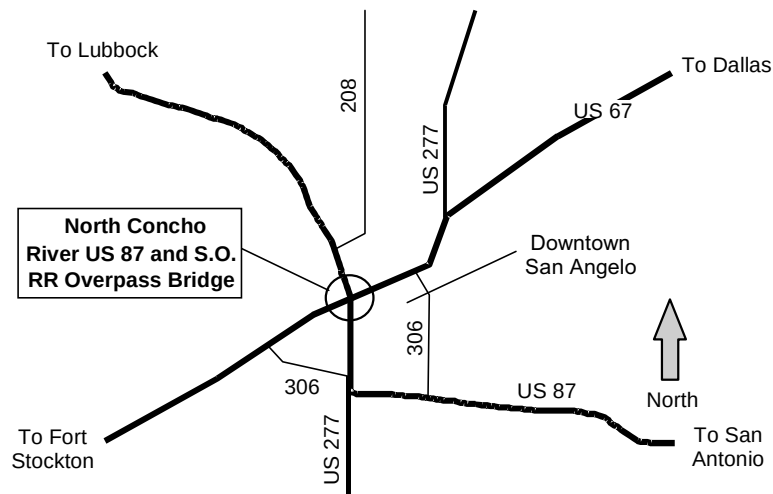


Figure 4.10: Location Map of North Concho River US 87 and S.O. RR Overpass Bridge in San Angelo, Texas

The site was ideal to demonstrate the use of HPC because it contained design constraints including the North Concho River, a city park, and the South Orient Railroad. HPC provided the design engineers at TxDOT with greater design flexibility over conventional concretes.

The North Concho River US 87 and S.O. RR Overpass Bridge utilizes a standard AASHTO Type IV precast/prestressed beam shape with a cast-in-place decorative pier. The decorative pier, shown in Figure 4.11, was selected to enhance the aesthetics of the bridge within the surrounding park area. The window opening allowed for a reduction in mass and gave the appearance of a significantly lighter substructure.

The North Concho River US 87 and S.O. RR Overpass Bridge project allowed TxDOT to demonstrate the applicability of HPC on the most widely utilized bridge system within the state of Texas. The North Concho River US 87 and S.O. RR Overpass Bridge consists of two adjacent bridge structures designed to accommodate two lanes of traffic in each direction.



Figure 4.11: North Concho River Overpass Decorative Pier

Figure 4.12 illustrates a plan view of the San Angelo eastbound HPC mainlanes and westbound mainlanes. The westbound mainlanes were designed as a conventional concrete bridge with the exception of the cast-in-place decks for Spans 1 through 5, which were HPC. These portions of the westbound mainlanes were designated as HPC as part of the durability study described in Section 4.4.2 of this report. Spans 1 through 6 of the westbound overpass and spans 1 through 5 of the eastbound overpass are considered mainspans while the remaining spans for each overpass were

controlled by geometric constraints and clearance requirements associated with the South Orient Railroad at the north end of the project.

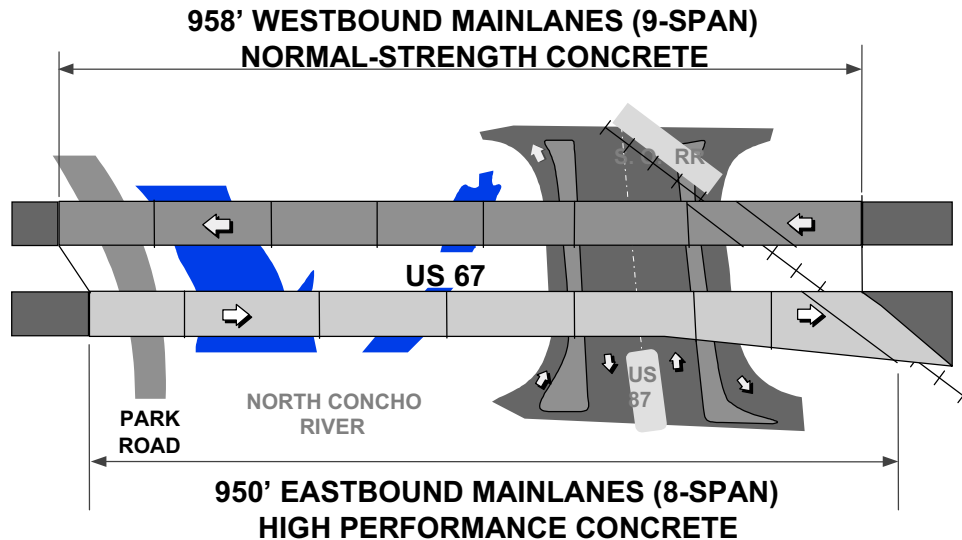


Figure 4.12: Plan View of North Concho River US 87 and S.O. RR Overpass Bridge

4.4 Phase III Research Program: Long-Term Durability Study

The Phase III Research Program involved an in situ durability study to compare the long-term performance of various concretes and deck reinforcing steels. As previously noted, both the Louetta Road Overpass Bridge and San Angelo North Concho River Overpass Bridge projects involved the construction of two multispan bridges. Each project provided an excellent opportunity to compare the long-term durability performance of HPCs and conventional concretes under identical environmental exposures and field conditions.

The improved durability performance of HPC compared to normal-strength or conventional concretes has been widely documented based on laboratory tests. However, this project provided the first opportunity for a long-term comparison to be conducted in an actual bridge structure in which the decks were constructed simultaneously under similar construction techniques and by the same personnel. As discussed previously, while laboratory durability testing provides the design engineer with some expectations on anticipated durability performance in the field, the specimen sizes and exposure conditions never truly predict identical field conditions. The following subsections describe

the variation of concrete mix designs selected for the respective bridges and the deck reinforcing study.

4.4.1 Louetta Road Overpass Bridge Project

Two HPC mix designs were selected for the deck durability study for the Louetta Road Overpass Bridge project as described below.

4.4.1.1 Class S Modified Cast-in-Place Deck. The Class S modified mix design, as presented in Chapter 6, incorporated a 28 percent fly ash replacement for reduced permeability and, thereby, improved durability performance without the use of a high or mid-range water reducer. The mix design was selected for Spans 1 through 3 of the northbound mainlanes and is similar to the standard TxDOT Class S mix design with fly ash replacement. The concrete may be classified as an HPC.

4.4.1.2 Class K Cast-in-Place Deck. The Class K mix design, as presented in Chapter 6, incorporated a 32 percent fly ash replacement and an HRWR for a reduced w/cm ratio and reduced permeability for enhanced durability performance. The mix design was selected for Spans 1 through 3 of the southbound mainlanes. The concrete may be classified as an HS/HPC.

4.4.2 North Concho River US 87 and S.O. RR Overpass Bridge Project

One conventional mix design and two HPC mix designs were selected for the deck durability study for the North Concho River and S.O. RR Overpass Bridge project as described below.

4.4.2.1 Class S Cast-in-Place Deck. The Class S mix design, as presented in Chapter 6, is a standard TxDOT deck mix with a required minimum of 6.5 sacks of cement and a maximum w/cm ratio of 0.45. The mix design was selected for Spans 6 through 9 of the westbound mainlanes. The concrete may be classified as conventional concrete.

4.4.2.2 Class S Modified Cast-in-Place Deck. The Class S modified mix design, as presented in Chapter 6, incorporated a 30 percent fly ash replacement for reduced permeability and thereby improved durability performance without the use of a high or mid-range water reducer. The mix design was selected for Spans 1 through 5 of the westbound mainlanes and is similar to the standard TxDOT Class S mix design without the fly ash replacement. The concrete may be classified as an HPC.

4.4.2.3 Class K Cast-in-Place Deck. The Class K mix design, as presented in Chapter 6, incorporated a 30 percent fly ash replacement and an HRWR for a reduced w/cm ratio and reduced

permeability for enhanced durability performance. The mix design was selected for Spans 1 through 8 of the eastbound mainlanes. The concrete may be classified as an HS/HPC.

4.4.3 Deck Reinforcing Durability Investigation

The North Concho River and S.O. RR Overpass Bridge project incorporated both epoxy-coated reinforcing bars and plain reinforcing (uncoated) bars as illustrated in Figure 4.13. The various reinforcing bars were selected to evaluate the high-durability, low-permeability aspects of the HPC and to investigate the advantages of epoxy-coated reinforcing related to corrosion control. Spans 2 through 9 of the westbound bridge deck incorporated epoxy-coated reinforcing. All other westbound bridge deck spans and eastbound bridge decks incorporated plain reinforcing bars.

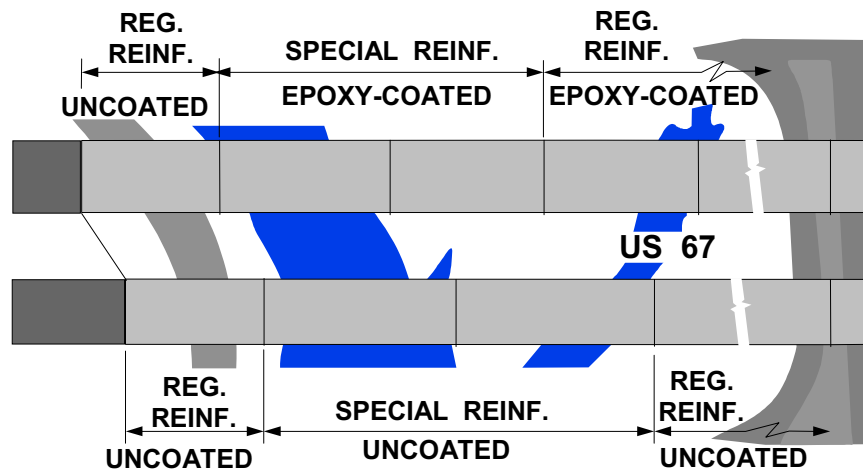


Figure 4.13: North Concho River US 87 and S.O. RR Overpass Bridge Deck Reinforcing

The study also included the use a special reinforcing bar developed at the University of Kansas. The special reinforcing bar replaced the standard deck reinforcing on a one-to-one basis. The special reinforcing bar, shown at the top of Figure 4.14 provides both an angular lug pattern and tighter spacing for improved bond characteristics. The improved bond characteristics of the special reinforcing bars resulted in a reduced development length (reduced splice length) over traditional lug patterns reducing the overall required quantity of steel for the respective spans. Long-term performance of these bars is also under investigation both from a durability and structural performance standpoint. Figure 4.14. also shows the plain and conventional epoxy-coated reinforcing bars.

The San Angelo project was selected for this portion of the durability study instead of the Louetta project because of the more severe climate and environmental conditions throughout the calendar year. The San Angelo project also provided additional spans that allowed for incorporation of several of the presented variables.

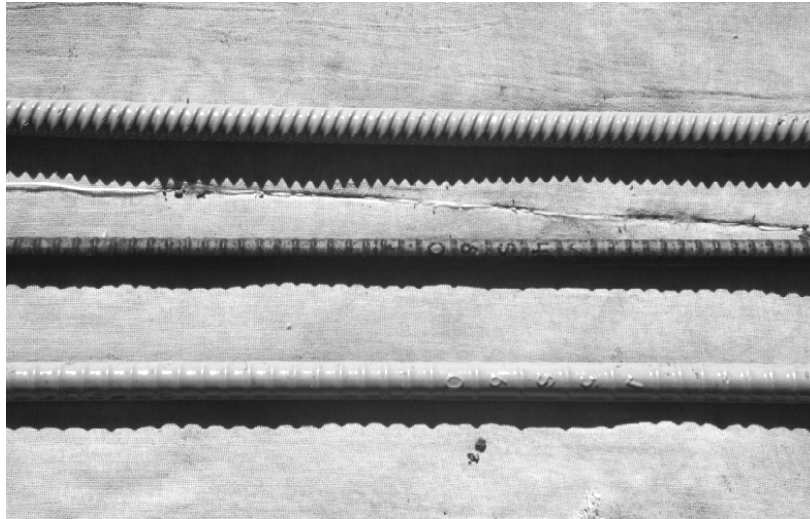


Figure 4.14: Variation in Reinforcing Bar Types and Coatings

4.5 Phase IV Research Program: Related Research Studies

In addition to the performance-related testing and QC/QA monitoring associated with the bridge projects, several related studies have been incorporated within the scope of this report. They are described in the following sections.

4.5.1 Effects of Early-Age Curing Temperatures on Precast Beams

The effects of the early-age temperatures and strength development on both the U-Beam and AASHTO Type IV precast beams were investigated during fabrication of the members. The objective of this investigation was to monitor the temperature development in various locations in the members and to determine the effects on the short- and long-term mechanical and material properties of the members. The temperatures were recorded using thermocouples in the members while the mechanical and material properties were investigated through the use of a match-curing system as detailed in Section 5.3.3.1 of this report. The investigation also provided the opportunity to evaluate

the match-curing system and provide recommendations for its use as a quality control/quality assurance tool in the precast industry.

4.5.2 Maturity Investigation of Cast-in-Place Deck Curing

A common means of strength prediction is the maturity method. This method uses an empirical base to relate the time-temperature profile of a particular mix design to its strength at early-ages as described in Chapter 9. Previous research conducted at UT by Cetin [15] illustrated how the maturity method underestimated the early strength (release strengths) and overestimated the long-term strength for accelerated heat-cured specimens such as the prestressed/precast U-Beam. The high cementitious content of the HPC mix and the mass sections of concrete created a *burn-out* effect (high initial strength gain followed by limited long-term strength gain) as described by Cetin. He concluded that the traditional maturity-strength relationship based on standard-cured cylinders does not appear to be representative of strengths of concretes subjected to accelerated heat curing.

The objective of this study was to investigate the applicability of the maturity method for use in HS/HPC and HPC bridge decks. While these mix designs have a large amount of cementitious materials similar to the precast beams discussed herein, the decks generally have a surface area to volume (SA/V) ratio twice as large as their counterpart beams. Because of the larger SA/V, the decks can dissipate heat much more quickly limiting the development of high internal concrete temperatures and, thereby, the maturity method may be applicable for HPC bridge decks. This study looks at the applicability of the maturity method for cast-in-place HPC bridge decks.

4.5.3 Correlation Study between AASHTO T-277 and AASHTO T-259

Concrete permeability may be the most relevant concrete property affecting its durability, especially under exposure to aggressive environment. As a result, permeability is an aspect of concrete performance that must be specified, designed for, and monitored in the production of concrete. This is especially true in the case of bridge decks, parking garages, pavement overlays, and offshore structures subjected to adverse conditions in which durability and life cycle costs are a critical issue. To determine the chloride permeability of concrete, many engineers have increasingly been using AASHTO T277, "Rapid Determination of the Chloride Permeability of Concrete," and specifying a coulomb rating of 2,000 or less to ensure low-permeability concrete.

Current research studies dealing with normal to mid-range concrete strengths have questioned the validity of AASHTO T277. As a sole indicator of concrete permeability, it has been suggested [122] that for specific concrete mixes, AASHTO T277 should not be used solely without

developing initial correlation data with AASHTO T259, "Resistance of Concrete to Chloride Ion Penetration."

The investigation reported herein summarizes the results of a long-term research study that was initiated in the fall of 1995 at The University of Texas at Austin. The main objective of the study was to investigate the correlation between AASHTO T277 and AASHTO T259 for HS/HPC. The variables investigated included entrained air content, fly ash content, and aggregate type. Statistical data on compressive strength, modulus of elasticity, splitting tensile strength, chloride percentage, and rapid ion permeability are presented.

4.5.4 Air Entrainment and Permeability Investigation

The main objective of the study was to investigate the influence of air entrainment on permeability of HPC. It has been previously reported in Section 2.3.2.6 that air entrainment does not create any new discontinuities for HPC within the capillary pore system to reduce permeability as is the case for conventional concretes. This study examines this aspect of permeability and was conducted concurrently with the correlation study described in Section 4.5.3 of this report.

CHAPTER 5. QUALITY CONTROL/QUALITY ASSURANCE TESTING PROGRAM

5.1 Introduction

The following sections outline the materials, the test procedures, and the performance-based specification system selected by TxDOT for design and construction of the two HPC bridges. The materials and procedures used throughout this study conform to procedures approved by one or more accepted standards. These include AASHTO “Standard Specification for Methods of Sampling and Testing” [91], the American Society for Testing and Materials “1994 Annual Book of Standards” [92, 93], and the Texas Department of Transportation, “Standard Specifications for Construction and Maintenance of Highways, Streets and Bridges” [72]. Prior to presenting the materials, methods, and procedures used throughout this study, it is important to understand quality control and the factors that contribute to the variability in properties.

5.1.1 Factors Affecting Quality Control/Quality Assurance

Many factors are involved in the production of high-quality concrete including materials, proportioning, handling, placing, curing, and testing. Like any other material, concrete is inherently a variable material. In general, the factors that contribute to this variability may be grouped as follows [2]:

1. *Materials*: This includes variability in cement, in the grading, moisture content, mineral and chemical composition, particle shape and texture of the aggregates; and the admixtures used.
2. *Production*: Type of batch plant and equipment, method of transporting concrete, and the procedures and workmanship used to produce and place the concrete.
3. *Testing*: The sampling procedure, making and curing of test specimens, and the test procedures used.

The term *quality control/quality assurance* (QC/QA) is frequently used in the construction industry today. Quality control applies to each action used to measure the properties or components of the concrete in such a way as to control them within established specifications. Quality assurance refers collectively to all of the steps taken to ensure adequate confidence that the concrete will perform satisfactorily in service.

5.1.2 Measurement of Variability

The concept of variability related to concrete strength can best be approximated by the normal (Gaussian) distribution that is defined by two parameters, the mean, μ , and the standard deviation, s . The equation of the normal distribution curve is given by the following expression:

$$y = \frac{1}{s\sqrt{2\pi}} \exp \left[\frac{-(x - \mu)^2}{2s^2} \right] \quad (5.1)$$

The mean is the arithmetic mean of all of the values. If the value of compressive strength is represented by x , then the mean is represented by the following expression:

$$\mu = \frac{\sum x}{n} \quad (5.2)$$

The standard deviation, often referred to as the root-mean-square deviation, is a measure of the dispersion or variability of the values. It is expressed as follows:

$$s = \sqrt{\frac{\sum (x - \mu)^2}{n - 1}} \quad (5.3)$$

The normal distribution curve is represented in Figure 5.1 for a region of interest within three standard deviations from the mean that contains 99.73 percent of all expected values. This curve is very useful as a means to characterize concrete properties.

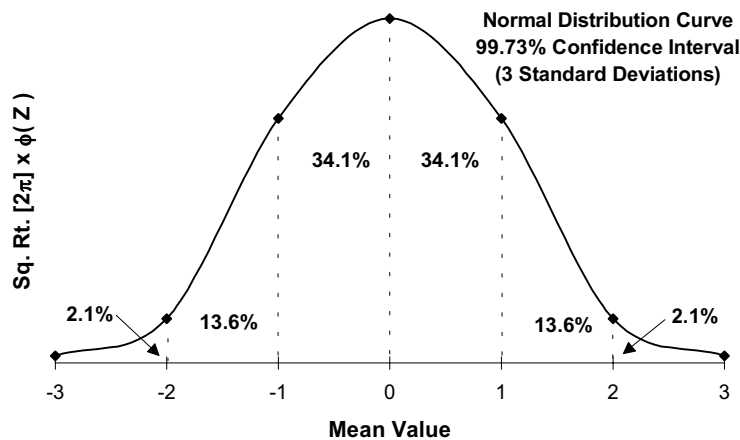


Figure 5.1: Area under the Normal Distribution Curve

Statistical description of concrete strength in structures may be derived from two separate random quantities, as shown in Figure 5.2. Given a deterministic value of f'_c , the average in-place strength of a structure, $f_{c,is}$, is a random quantity that depends on many factors as discussed above. The relationship between mean in-place strength and specified strength may be derived from two independent random quantities. The quality of material produced for a particular specified strength is checked using 28-day-old cylinders (typically) that are molded, cured, capped, and tested in accordance with standard procedures [96]. Factor F_1 can be characterized as the ratio of the average strength of the standard cylinders, f_{cyl} , to f'_c as illustrated in Equation 5.4.

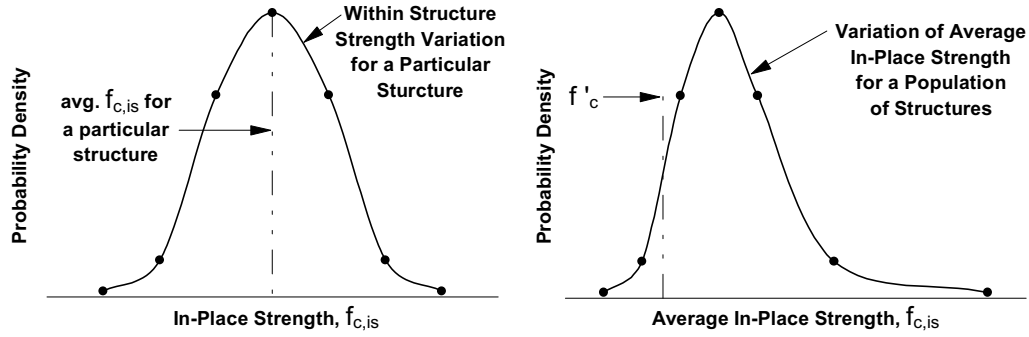


Figure 5.2: Statistical Descriptions of Concrete Strength In Structures

$$F_1 = f_{cyl} / f'_c \quad (5.4)$$

Typically, F_1 exceeds 1 because the concrete producer develops a mix design that exceeds the specified strength requirements to assure the specifications are satisfied. The actual quality of the concrete in the structure may differ from the representative cylinder because of age, consolidation, or in-place curing. Therefore, the material in the structure may not be well represented by the standard test cylinder. Factor F_2 is the ratio of the average in-place strength of concrete, $f_{c,is}$, to average cylinder strength.

$$F_2 = f_{c,is} / f_{cyl} \quad (5.5)$$

The value of F_2 may vary depending on the size and type of member. The relationship between average compressive strength of concrete in structures and specified strength is obtained by elimination of f_{cyl} from Equation 5.4 and 5.5 resulting in Equation 5.6.

$$f_{c,is} = F_1 F_2 (f'_c) \quad (5.6)$$

The approach of ACI and its Committee 214 to the variability inherent in concrete strength is to require an average concrete strength sufficiently in excess of the specified design strength so that only an allowable proportion of low-strength concrete will result. The required average strength is computed from the following expression:

$$\sigma_{cr} = \sigma'_c + tS \quad (5.7)$$

where: σ_{cr} = required average strength
 σ'_c = specified design strength
 t = constant based on proportion of tests allowed below σ'_c as presented in Table 5.1
 S = the predicted value of standard deviation

Table 5.1: Values of t Used in Equation 5.7 [100]

Percentages of Tests Falling Within the Limits $\mu \pm ts$	Chances of Falling Below Lower Limit	t
40	3 in 10	0.52
50	2.5 in 10	0.67
60	2 in 10	0.84
68.27	1 in 6.3	1.00
70	1.5 in 10	1.04
80	1 in 10	1.28
90	1 in 20	1.65
95	1 in 40	1.96
95.45	1 in 44	2.00
98	1 in 100	2.33
99	1 in 200	2.58
99.73	1 in 741	3.00

A minimum of 30 samples is recommended to improve the reliability of the standard deviation for the sampled population. This approach provides a great deal of flexibility allowing for different probabilities of low-strength concrete for different classes of concrete or type of structure and the economic or other consequences of failure. Regarding the required strength for normal-strength or

conventional concrete, ACI Committee 318 [101] has specified a value of 1.34 for t based on documented performance and experience. The provisions are outlined in the following expressions:

$$f'_{cr} = f'_c + 1.34s \quad (5.8)$$

$$f'_{cr} = f'_c + 2.33s - 500 \quad (5.9)$$

Once the standard deviation has been determined, the required average strength is obtained from the larger of Equations 5.8 and 5.9. Equation 5.8 provides a probability of 1 in 100 that averages of three consecutive tests will be below the specified strength of f'_c . Equation 5.9 provides a similar probability of individual tests more than 500 psi below the specified strength of f'_c . These equations assume that the standard deviation used is equal to the population value for a large number of tests (> 30).

Although there are no current guidelines for the determination of the required field strength of concrete for highway construction relevant to HS/HPC, ACI has developed such guidelines for building construction. The main variation between the values presented in Equations 5.10 and 5.11 and the values presented for conventional concrete (Equations 5.8 and 5.9) is the utilization of a 10 percent reduction factor in the design strength. This is used as the allowable underdesign strength factor compared to 500 psi (Equation 5.9). ACI Committee 211 has also suggested recommendations for selecting the required average field strength of concrete on the basis of laboratory field trials as illustrated in Equation 5.12.

$$f'_{cr} = f'_c + 1.34s \quad (5.10)$$

$$f'_{cr} = 0.90f'_c + 2.33s \quad (5.11)$$

$$f'_{cr} = \frac{(f'_c + 1400)}{0.90} \quad (5.12)$$

It should be clear that whatever acceptance criteria are adopted for HS/HPC and highway construction, concrete producers gain a significant economic benefit in achieving the lowest coefficient of variation possible and, thereby, the tightest quality control. Chapter 7 investigates and discusses the statistical variation of the plant-produced HS/HPC throughout this study.

5.1.3 Epigrammatic Summary of the Quality Control/Quality Assurance Program

The QC/QA program for this research study was developed to monitor, verify, and document both early- and later-age concrete properties during plant production of concrete. In addition to creating a database for the FHWA and TxDOT on HPC properties, the QC/QA program provided mechanical property results under plant/field-curing conditions that could be utilized in the time-

dependent analyses to better predict the short- and long-term behavior of the HPC bridges. The early-age mechanical properties that were closely monitored included compressive strength, elastic modulus, flexural strength, and splitting tensile strength. Additional performance-related testing incorporated within the QC/QA program included concrete material (durability) testing and monitoring including abrasion resistance, freeze-thaw resistance, scaling resistance, and chloride permeability. Generally, the concrete material testing was performed on superstructure concrete elements such as CIP decks where durability performance was considered most critical. Only chloride permeability was investigated for substructure elements. The QC/QA program also investigated mechanical and material concrete properties under a variety of curing conditions. These included ASTM moist-cured cylinders, TxDOT member-cured cylinders, match-cured cylinders, and site/field-cured cylinders.

Because of the high strength and modulus of elasticity required at release and 56 days for the precast/prestressed beams, it was very important to determine the concrete properties of the member as accurately as possible. This meant trying to duplicate the curing conditions of the test cylinders with respect to the actual curing condition of the precast/prestressed beams. A match-curing system was selected to accomplish this objective. The specifics of the QC/QA program are further detailed in the following sections.

5.2 Materials

Commercially available materials were used throughout this investigation for laboratory and ready-mix batch plant-produced concrete. Portland cement, coarse aggregate, fine aggregate, fly ash, and chemical admixtures were consistent with those used by local suppliers and were approved by TxDOT [72] for use in Texas highways, streets, and bridges. The following subsections outline the materials used throughout the laboratory trial series and associated laboratory investigations contained within this study.

5.2.1 Portland Cement

Commercially available Type I cements designated as B-I/II and C-I/II, and Type III cements designated as A-III, C-III, and L-III were used. They conformed to ASTM C150-94, "Standard Specification for Portland Cement." Their physical and chemical properties are given in Tables 5.2 and 5.3.

Table 5.2: Physical Properties of Portland Cements

Physical Property			Portland Cements				
Specific Surface, cm ² /gm Blaine Wagner			L-III	B-I/II	A-III	C-III	C-I/II
			5740	3530	6360	5730	3430
			2880	1970	2933	2926	1823
Percentage Passing No. 325 Sieve			N/A	N/A	99.7	98.6	93.7
Compressive Strength, psi							
1 Day			3700	2190	4230	4545	N/A
3 Day			5400	3650	5076	5910	4085
7 Day			N/A	4670	5930	6750	5115
28 Day			N/A	6210	N/A	N/A	N/A
Setting Time, min.							
Vicat	Initial		N/A	100	N/A	75	115
	Final		N/A	210	N/A	120	180
Gilmore	Initial		125	130	62	135	185
	Final		265	230	118	255	350

Table 5.3: Chemical Properties of Portland Cements

Chemical Property	Portland Cement*				
	L-III	B-I/II	A-III	C-III	C-I/II
SiO ₂	19.90	21.30	18.95	19.66	20.24
Al ₂ O ₃	6.10	4.60	5.60	5.38	5.66
Fe ₂ O ₃	2.40	4.10	2.97	2.06	2.11
CaO	64.60	64.60	64.57	64.05	64.63
MgO	1.20	0.90	0.71	1.26	1.27
SO ₃	3.80	2.60	3.79	4.09	3.16
Loss on Ignition	1.60	1.10	1.57	2.64	2.06
Insoluble Residue	0.06	0.20	0.24	0.27	0.19
Free Lime	N/A	0.70	1.70	N/A	N/A
C ₃ S	56.00	57.00	61.10	60.58	59.23
C ₃ A	12.00	5.20	1.80	10.77	11.43
Total Alkali	0.67	0.60	0.71	0.60	0.60

* (I-ASTM Type 1 I/II-ASTM Type 1/2 III-ASTM Type 3)

5.2.2 Coarse Aggregate

Seven types of coarse aggregates were used in the overall study, namely two crushed river gravels [CG, CGS, Nominal Maximum Size of Aggregate (NMSA) = 19 mm (0.75 in.)], trap rock [TR, NMSA = 19 mm (0.75 in.)], dolomitic limestone [LS and LL, NMSA = 13 and 19 mm (0.50 and 0.75 in.)], calcitic limestone [CL, NMSA = 19 mm (0.75 in.)], and a Mexican limestone [ML, NMSA = 19 mm (0.75 in.)]. Their moisture conditions, specific gravities, gradations, and mineral and chemical compositions are given in Tables 5.4 through 5.7, respectively. Figure 5.3 illustrates three aggregates predominately used throughout this investigation.

**Table 5.4: Moisture Conditions and Specific Gravities of Aggregates
Used in Concrete Production**

Property	Coarse Aggregates					
	CG	CGS	TR	DLS	DLL	CL
Absorption, %	0.72	0.37	0.62	0.50	0.57	3.80
Specific Gravity	2.55	2.63	3.14	2.68	2.82	2.50
DRUW, pcf	105.1	106.7	111.0	101.2	104.9	100.8

Table 5.5: Sieve Analysis of Coarse Aggregates

Sieve Size	Percentage Passing							ASTM C33-94	
	CG	CGS	TR	DLL	DLS	CL	ML	3/4"	1/2"
3/4"	100	100	100	100	100	100	96.2	90-100	100
1/2"	68	78.8	77	64	99.7	83	79.7	-	90-100
3/8"	30	39.4	11	39	65	47	46	20-55	40-70
1/4"	6.2	0.1	1.2	13	12	6.4	9.5	-	-
No. 4	1.6	-	0.6	5.2	1	2.1	4.2	0-10	0-15
Pan	-	-	-	-	-	-	-	-	-

Table 5.6: Mineral Composition of Coarse Aggregates

Aggregate**	Mineral Composition*
Crushed Gravel, (CG, CGS)	Quartz, Calcite
Trap Rock, (TR)	Diopside, Forsterite, Epidote
Dol. Limestone, 3/4" (DLL)	Dolomite, Quartz, Calcite
Dol. Limestone, 1/2" (DLS)	Dolomite, Quartz, Calcite
Calcitic Limestone, (CL)	Calcite, Dolomite, Quartz
Mexican Limestone, (ML)	Calcite, Dolomite, Quartz

* By X-ray Diffraction Analysis (In order of decreasing abundance)

** Mineral Composition of ML Not Available

Table 5.7: Chemical Composition of Coarse Aggregates

Oxides*	Coarse Aggregate**				
	CG	TR	DLL	DLS	CL
CaO	-	11.86	-	-	-
CaCO ₃	2.78	-	3.78	2.91	41.58
SiO ₂	92.71	36.53	5.73	5.39	7.50
Al ₂ O ₃	0.74	11.03	0.48	0.71	0.02
Fe ₂ O ₃	1.02	11.91	0.09	0.11	0.07
MgO	0.19	14.46	-	-	-
MgCO ₃	-	-	38.92	41.52	22.62
Na ₂ O	0.10	3.01	0.08	0.12	0.02
K ₂ O	0.30	0.94	0.23	0.43	0.02
MnO	0.01	0.19	0.01	0.01	0.01
P ₂ O ₅	0.13	1.35	0.10	0.13	0.10
TOTAL	97.98	91.26	95.56	100.61	98.62

* By inductively-coupled plasma atomic emission spectrometry (% by weight)

** Chemical composition of CGS and ML not available



Crushed River Gravel



Burnet Limestone



Trap Rock

**Figure 5.3: Coarse Aggregates used throughout the Investigation
(Crushed River Gravel, Burnet Limestone, Trap Rock)**

5.2.3 Fine Aggregate

Two locally available concrete fine aggregates from the Colorado River were used in the overall study. One was from the lower Colorado River (S-I) and a second was from the Colorado River north of the Highland Lakes chain (S-II). They had a specific gravity of 2.63 (S-I) and 2.64 (S-II), respectively, and a fineness modulus of 2.45 (S-I) and 2.85 (S-II), respectively. The fine aggregate gradations are presented in Table 5.8. Figure 5.4 illustrates a representative fine aggregate used throughout the study.

Table 5.8: Sieve Analysis of Fine Aggregates

Sieve Size	Percentage Passing		ASTM C33-94
	S-I	S-II	
3/8"	100	100	100
No. 4	96	99.9	95-100
No. 8	85	85.2	80-100
No. 16	75	66.8	50-85
No. 30	56	46.8	25-60
No. 50	39	14.7	10-30
No. 100	3.8	2.8	2-10
Pan	0	0	-



Fine Aggregate S-II

BSG = 2.65 FM = 2.85

Figure 5.4: Fine Aggregate used throughout the Investigation

5.2.4 Fly Ash

Class C fly ash from three different sources (J, D, and L) and a Class F fly ash (R) were used in varying amounts. They conformed to ASTM C618-94a, "Standard Specification for Fly Ash and Raw and Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete." Their physical and chemical properties are presented in Tables 5.9 and 5.10, respectively.

Table 5.9: Physical Properties of Fly Ashes

Physical Property	Fly Ash			
	J	D	L	R
Fineness (Amount Retained #325 Sieve), %	16.57	14.70	12.60	22.60
Specific Gravity	2.63	2.75	2.69	2.16
Autoclave Expansion, %	0.09	N/A	N/A	N/A
Water Reqmt., % Control	94	N/A	N/A	N/A

Table 5.10: Chemical Properties of Fly Ashes

Chemical Property	Fly Ash			
	J	D	L	R
SiO ₂	34.99	34.60	34.70	57.90
Al ₂ O ₃	20.25	20.90	19.70	26.30
Fe ₂ O ₃	5.24	6.10	5.60	3.90
Sum of SiO ₂ (Al ₂ O ₃ and Fe ₂ O ₃)	61.48	61.60	60.00	88.10
MgO	4.65	4.80	5.50	2.10
SO ₃	1.74	1.60	1.80	0.52
Moisture Content, %	0.06	0.10	0.10	0.09
Loss of Ignition	0.28	0.20	0.30	0.17
Total Alkali	1.18	N/A	N/A	N/A
CaO	26.12	24.80	26.60	9.60

5.2.5 Chemical Admixtures

Three chemical admixtures were used throughout the scope of the research program including an HRWR, a set retarder, and an air-entraining admixture. Four HRWRs were used including a naphthalene-based superplasticizer conforming to ASTM C494-92 (Type A/F), a

naphthalene-based superplasticizer conforming to ASTM C494-92 (Type A), and two recently developed water reducers including 1) a superplasticizer consisting of water-soluble acrylic graft copolymers with functional sulfonic and carboxyl groups, and 2) a molecular-designed graft copolymer with functional anion group. A retarder conforming to ASTM C494-92 (Type B) and an air-entraining agent conforming to ASTM C260 were used.

5.2.6 *Mixing Water*

Potable city of Austin water was used throughout the laboratory batching series and was assumed to have a specific gravity of 37 kg/m³ (62.4 lb/cy).

5.3 Performance-Related Testing and Procedures

5.3.1 *Mixing Procedures*

All laboratory mixing was conducted in a 0.17 cubic meter (6 cubic foot) capacity rotary drum, shown in Figure 5.5, using 0.1 cubic meter (3.5 cubic foot) batches. ACI standard practice [96] was followed for batching and making test specimens in the laboratory. All materials reached room temperature (equilibrium) at 21.1 to 22.8 °C (70 to 73 °F) prior to batching. Mixing water was adjusted accordingly based on the moisture conditions of the coarse and fine aggregates approximately one-half hour prior to batching.

All batching procedures satisfied ACI Committee 211 guidelines and manufacturer recommendations. The ideal duration of the batching procedure and casting of specimens was approximately 1 hour to simulate a worst-case scenario for plant-produced concrete and associated slump loss for the HPC mixes. It was felt that 1 hour was representative of the amount of time it would take to batch, deliver, and place concrete in the field. The ready-mix concrete plant in Houston was approximately 5 minutes from the site while the San Angelo plant was approximately 10 minutes from the site. All precast plant-produced concrete was located several hundred yards from the precasting beds in the respective precasting yards.



Figure 5.5: Rotary Mixing Unit used for Laboratory-Produced Concrete

5.3.2 Test Specimens

QC/QA test specimens were cast throughout this investigation with laboratory and plant-produced concrete. Figure 5.6 illustrates the standard QC/QA test specimens that were cast for an HPC CIP deck pour. The following sections detail the various test specimens.



Figure 5.6: Representative QC/QA Test Specimens Cast for a Standard Deck Pour

5.3.2.1 Cylinder Specimens. The standard control specimens throughout the laboratory and batch plant phases of this study were 100-mm diameter x 200-mm long (4-in. x 8-in.) cylinders cast from single-use plastic molds. The cylinders were made in accordance with ASTM C192, “Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory” [96] and capped with a plastic lid. Laboratory-produced concrete was cured in the lab at room ambient temperature varying between 21.1 to 22.8 °C (70 to 73 °F) for 24 hours. Similarly, field- or plant-produced concrete was also cured at the respective site or precast plant for 24 hours as described in Section 5.3.6. After 24 hours, the cylinders were then demolded and subjected to the various curing procedures as describe in Section 5.3.6 of this report.

5.3.2.2 Chloride Ponding Block Specimens. Four 304.8-mm x 304.8-mm x 88.9-mm (12-in. x 12-in. x 3.5-in.) ponding block specimens were cast for each pour in which long-term chloride resistance was to be determined. Three specimens were subjected to ponding while the fourth served as the control specimen in accordance with the requirements of AASHTO T259 “Resistance of Concrete to Chloride Ion Penetration.” Concrete was placed and vibrated in two layers. The surface was screeded and a trowel finish was applied prior to covering with wet burlap and plastic sheeting. The blocks were demolded at 24 hours and subjected to curing in accordance with AASHTO T259. Figure 5.7 illustrates the ponding blocks before and after a light sandblasting was performed. Section 5.3.5.2.6 provides a comprehensive description of the testing methods used and sampling procedures followed for the chloride ion penetration specimens.

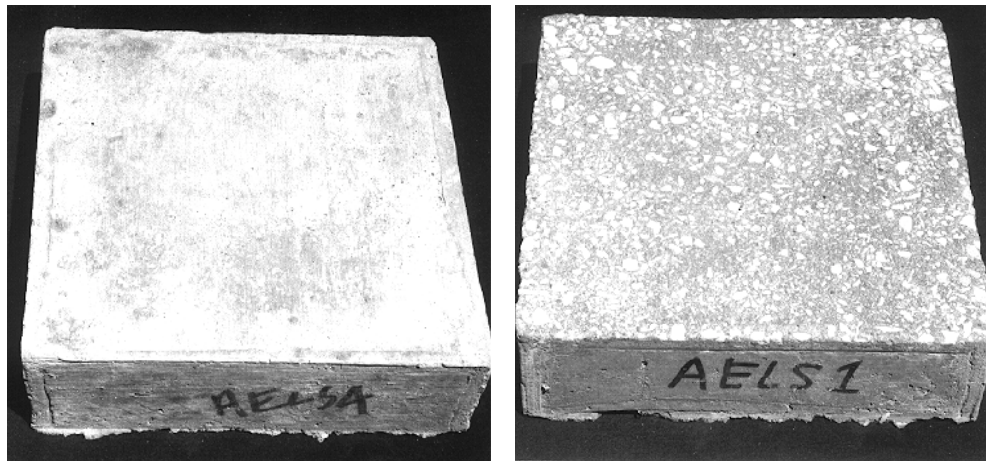


Figure 5.7: Ponding Block Specimens before and after Sandblasting

5.3.2.3 Freeze-Thaw Specimens. Three 76.2-mm x 101.6-mm x 406.4-mm (3-in. x 4-in. x 16-in.) freeze-thaw specimens were cast for each pour in which freeze-thaw resistance was to be determined. Concrete was placed and vibrated in two layers. The surface was screeded and a trowel finish was applied prior to covering with wet burlap and plastic sheeting. The blocks were demolded at 24 hours and subjected to curing in accordance with ASTM C666, “Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing.” Figure 5.8 illustrates the freeze-thaw specimens after demolding. Section 5.3.5.2.7 provides a comprehensive description of the testing methods used.



Figure 5.8: Freeze-Thaw Specimens

5.3.2.4 Abrasion Resistance Specimens. Four 304.8-mm x 304.8-mm x 76.2-mm (12-in. x 12-in. x 3-in.) abrasion resistance test specimens were cast for each pour in which abrasion resistance was to be determined. Concrete was placed and vibrated in two layers. The surface was screeded and a trowel finish was applied prior to covering with wet burlap and plastic sheeting. The blocks were demolded at 24 hours and subjected to curing in accordance with ASTM C944, “Standard Test Method for Abrasion Resistance of Concrete or Mortar Surfaces by the Rotary Cutting Method.” Section 5.3.5.2.8 provides a comprehensive description of the testing methods used.

5.3.2.5 Scaling Deicing Specimens. Four 304.8-mm x 304.8-mm x 76.2-mm (12-in. x 12-in. x 3-in.) scaling deicing test specimens were cast for each pour in which scaling resistance was to be

determined. Concrete was placed and vibrated in two layers. The surface was screeded and a trowel finish was applied prior to covering with wet burlap and plastic sheeting. The blocks were demolded at 24 hours and subjected to curing in accordance with ASTM C672, "Standard Test Method for Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals." Section 5.3.5.2.9 provides a comprehensive description of the testing methods used.

5.3.3 Frequency of Testing

Guidelines for sampling frequency were developed for fresh and hardened concrete properties as presented herein. The fresh and hardened performance-related testing was conducted on each laboratory- and plant-produced concrete batched. The following sections outline the testing dates on which specimens were evaluated.

5.3.3.1 Laboratory-Produced Concrete (Lab Trial Mixes). Beginning in the summer of 1993 through the fall of 1996, approximately 175 trial batches were made using locally available materials at the laboratory of CMRG at UT. The test results of over seventy-five of these trial batches were investigated and reported by Cetin and Carrasquillo in a report entitled "Effect of Accelerated Heat Curing and Mix Characteristics on the Heat Development and Mechanical Properties of High Performance Concrete." [15].

Tests on the remaining trial batches were conducted by the author and are reported herein. As an initial starting point, guidelines provided by ACI Committee 211 [95] were followed to come up with an initial set of mix proportions for the HPC. The mix designs were later adjusted through laboratory trial mixes designed to optimize the performance of the constituent materials used. Cylinder specimens were cast in 100-mm x 200-mm (4-in. x 8-in.) plastic cylinder molds and moist cured in accordance with ASTM C192-90a "Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory" until the test age. The specimens were tested in compression and elastic modulus was determined at the ages of 1, 3, 7, 28, and 56 days. The average of three tests was reported as the compressive strength and two as the elastic modulus.

5.3.3.2 Plant-Produced Concrete (Field Trial Mixes). Following the successful development and optimization of the laboratory mix designs, a field trial mix phase was initiated to batch the mix designs at the respective plants where concrete would later be produced. Cylinder specimens were cast in 100-mm x 200-mm (4-in. x 8-in.) plastic cylinder molds in accordance with ASTM C31/C31M-95 "Standard Practice for Making and Curing Concrete Test Specimens in the Field." Table 5.11 summarizes the concrete producers and the plant locations where the concrete was produced for the structure, and the bridge components for which the trial mixes were prepared.

Table 5.11: Summary of Plant-Produced Concrete

Concrete Producer	Plant Location	Bridge Component Produced or Fabricated
Texas Concrete Company	Victoria, Texas	Louetta Precast/Prestressed U-Beams NCRO Precast/Prestressed Type IV's
Williams Brothers Constr. Co. Inc. - Precast Plant	Houston Texas	Louetta Precast Column Elements Louetta Precast Panels
Lopez-Gloria Ready-Mix Concrete Plant	Houston, Texas	Louetta Cast-in-Place Decks
Bexar Concrete Works Inc.	San Antonio, Texas	NCRO Precast Panels
Concho Concrete Company	San Angelo, Texas	NCRO Cast-in-Place Piers and Pier Caps NCRO Cast-in-Place Decks

The purpose of this phase was two-fold: 1) to investigate the strength loss of the mix designs, which typically occurs when a laboratory mix design is utilized in the field, and 2) to enable plant personnel to gain added experience with HRWRs and the use of mineral admixtures by batching the field trial mixes at the precast plants.

5.3.3.3 Precast Plant-Produced Concrete (Prestressed/Precast Concrete). Specimens were tested in compression and elastic modulus was determined at the ages of 1 (Release) 7, 28, and 56 days for concrete sampled from the precast/prestressed beams and precast deck panels. The average of three tests was reported as the compressive strength and the average of two tests as the elastic modulus. Table 5.12 presents the performance-related tests and measurements performed on the precast plant-produced concrete. Various curing conditions for these specimens were investigated as described in Section 5.3.6.1.

**Table 5.12: Louetta Road Overpass Bridge and North Concho River Overpass Bridge
Performance-Related Tests and Measurements**

Member/Component	Louetta Road Overpass Bridge		North Concho River Overpass Bridge	
	Northbound	Southbound	Eastbound	Westbound
Cast-in-Place Decks	1-7, 11-12	1-7, 11-12	1-12	1-12
Precast Panels	1-6, 11		1-6, 11	
Precast Beams	1-6, 11-12		1-7, 11-12	1-6, 11-12
Precast Columns	1-3, 6, 11		-	-
Cast-in-Place Piers	-		1-3, 6	1-3, 6
Cast-in-Place Pier Caps	-		1-3, 6	1-3, 6

1. Compressive Strength
2. Elastic Modulus
3. Splitting Tensile Strength
4. Shrinkage
5. Creep
6. Chloride Ion Penetration
7. Ponding Chloride Ion
8. Freeze-Thaw Resistance
9. Abrasion Resistance
10. Scaling Resistance
11. Temperature
12. Maturity

5.3.3.4 Ready-Mix Batch Plant-Produced Concrete (Cast-in-Place Concrete). Concrete samples from the CIP decks, piers, and pier caps were tested in compression and elastic modulus was determined at the ages of 7, 28, and 56 days. Specimens were also tested in compression and elastic modulus was determined at the ages of 1 and 14 days in addition to 7, 28, and 56 days for CIP concrete in which concrete maturity was investigated. This was necessary in order to develop the maturity strength gain curve for the mix design in conjunction with the CIP deck maturity study (Chapter 9). The average of three tests was reported as the compressive strength and the average of two tests as the elastic modulus. Table 5.12 presents the performance-related tests and

measurements performed on the CIP concrete. Various curing conditions for these specimens were investigated as described in Section 5.3.6.2.

5.3.4 Test Apparatus

The following systems were selected for use in testing and monitoring of concrete throughout this study in addition to the standard testing equipment described in Section 5.3.5.

5.3.4.1 Match-Curing System. A commercially available match-curing system was selected to produce the match-cured cylinders. The system incorporates steel molds with internal coils that keep the cylinders at the same temperature as the thermocouple location in the member. The match-curing steel molds were located in a room where the temperature of the environment could be controlled. The room was maintained at a cool temperature because the match-curing system could not cool the specimens, only heat them. This system was selected to more closely simulate the actual concrete temperature profile in various locations of the member. Figure 5.9 illustrates these match-curing molds in the climate-controlled environment. This system was only used to investigate plant-produced concrete at the Texas Concrete Company in Victoria, Texas, because production of beams was continuous beginning in the fall of 1994 and continuing through the spring of 1997.



Figure 5.9: Cylinder Molds of the Match-Curing System

Prior to use in the field, the system was used to investigate the effects of higher curing temperatures on both the compressive strength and elastic modulus of the concrete. It was anticipated that higher curing temperatures would be encountered at the end block locations in the

precast/prestressed beams, which would impact the concrete properties. It was important to closely simulate the concrete in the member to verify that the project specifications and guidelines were being met. Higher end block temperatures within the precast/prestressed beams were noted during casting of the beams and are discussed in further detail later in Chapter 8. Using a temperature-matched curing system, four of the laboratory trial mixes were subjected to accelerated heat curing. The curing regime applied to these trial mixes consisted of an approximate 4- to 5-hour preset time followed by an 8-hour soaking time as described in Chapter 6.

5.3.4.2 Hydration Chamber. The hydration chamber, which is part of the match-curing system, is a two-channel unit containing a heat sensor inside the chamber, and heating coils around the chamber and in the lid used to monitor the heat development of concrete due to hydration. The first channel is used to monitor the temperature of freshly mixed concrete in the chamber, and the second one is used to control the temperature of the hydration chamber itself so that the chamber will follow the temperature of the concrete inside. As a result, the concrete is perfectly insulated and does not lose heat to the surrounding environment. The temperature curve obtained from the hydration chamber is the cumulative heat development curve for that particular batch.

5.3.4.3 Maturity Monitoring System. A commercially available four-channel maturity monitoring system, shown in Figure 5.10, was selected as the data acquisition system for concrete maturity development. The data acquisition system scanned internal concrete temperatures through the use of thermocouples, which were embedded in the concrete at various locations and compiled concrete maturity every 30 minutes during the first 48 hours of casting and every 60 minutes thereafter.



Figure 5.10: Maturity Monitoring System

5.3.5 Test Procedures

5.3.5.1 Fresh Concrete Testing. The fresh concrete performance-related tests that were conducted on the precast/prestressed concrete and CIP concrete included air content, slump (before and after addition of HRWR), unit weight, and concrete temperature.

5.3.5.2 Hardened Concrete Testing. The hardened concrete performance-related tests that were conducted on the precast/prestressed concrete and CIP concrete included compressive strength gain with time, elastic modulus, flexural strength, splitting tensile strength, rapid chloride permeability, and concrete temperature rise. Additional durability testing was conducted on CIP deck concrete in addition to the aforementioned testing. Specimens tested included ASTM Moist Cured, Member Cured, Match Cured, and Site Cured as described in Section 5.3.5. Table 5.12 summarizes the details of the hardened concrete performance-related tests.

5.3.5.2.1 Compressive Strength ASTM C39-94. The compressive strength tests were performed in accordance with ASTM C39-94 "Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens" using a 600 kip capacity hydraulically operated compression test machine as shown in Figure 5.11. The specimens were loaded at a rate of 241 ± 103 kPa per second (35 ± 15 psi per second) until complete failure. Special care was taken to test the specimens in a moist condition if they were moist cured cylinders to eliminate any variation associated with their moisture condition at the time of testing.



**Figure 5.11: Compressive Strength and Elastic Modulus
Forney Hydraulically Operated 600 kip Capacity Multiple Loading Rate Machine**

No attempt was made to use sulfur mortar caps. Neoprene pads inserted in steel retaining caps were used throughout this study. Neoprene pads used for HS/HPC had a manufacturer's stated minimum hardness of 70 durameters.

5.3.5.2.2 Modulus of Elasticity ASTM C469-94. Elastic moduli were determined in accordance with ASTM C469-94 "Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression." The elastic modulus test set-up is illustrated in Figure 5.12.

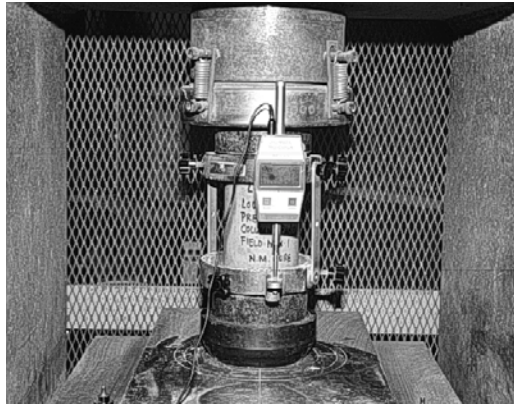


Figure 5.12: Static Elastic Modulus Testing Ring and Set-Up

5.3.5.2.3 Flexural Strength ASTM C78-94. The flexural strength test (modulus of rupture) was performed on a limited number of 152.4-mm x 152.4-mm x 508-mm (6-in. x 6-in. x 20-in.) prisms loaded on a 457.2-mm (18-in.) beam tester, shown in Figure 5.13. The tests were done according to ASTM C78-94 "Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)."

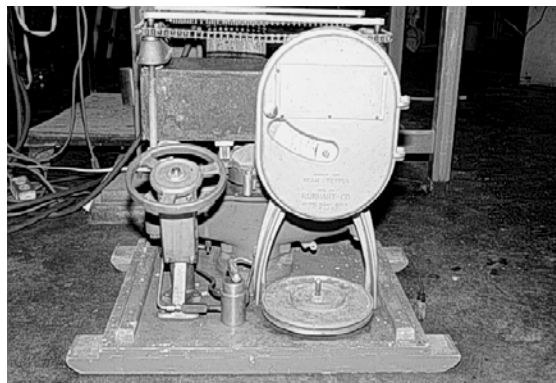


**Figure 5.13: Flexural Strength Testing Machine and Set-Up
Rainhardt Series 416 Beam Tester**

5.3.5.2.4 *Splitting Tensile Strength ASTM C496-96.* The splitting tensile strength tests were performed on 100-mm x 200-mm (4-in. x 8-in.) specimens in accordance with ASTM C496-96 “Splitting Tensile Strength of Cylindrical Concrete Specimens” using a 120 kip capacity hydraulically operated test machine as shown in Figure 5.14. The specimens were loaded at a continuous rate of 517 ± 103 kPa per minute (75 ± 15 psi per minute) until failure.

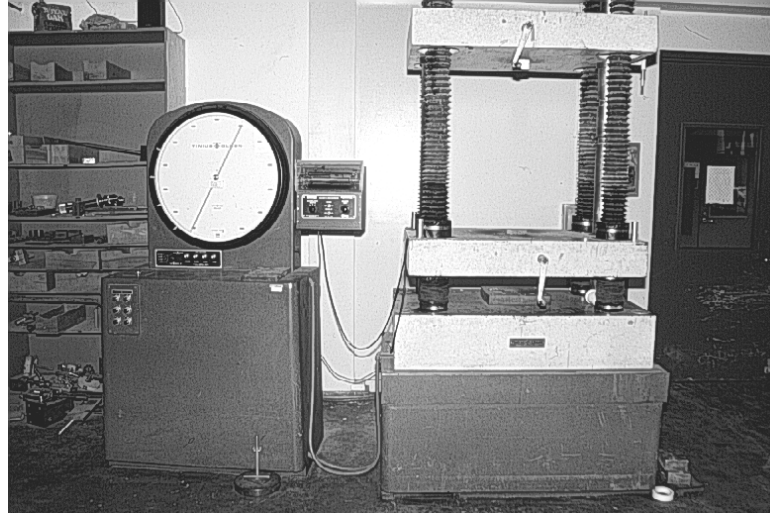


**Figure 5.14: Splitting Tensile Strength
Tiinius-Olsen Hydraulically Operated 120 kip Capacity Testing Machine**

5.3.5.2.5 *Rapid Chloride Permeability AASHTO T277.* Rapid chloride permeability testing was performed in accordance with AASHTO T277 “Standard Method of Test for Rapid Determination of the Chloride Permeability of Concrete.” The test measured, in coulombs, the amount of current that flowed through a slice of saturated concrete when placed between two electrolytic solutions and subjected to a 60-volt DC voltage. The permeability results were averaged based on a minimum of four slices for any one casting date under any one curing condition. Sampling for rapid chloride ion permeability focused on all concrete produced throughout this study.

5.3.5.2.6 *Chloride Ponding AASHTO T259.* Diffusion of chloride ions into the ponding block specimens was determined using a CL-1000 chloride test system by James Instruments, shown in Figure 5.15, in accordance with AASHTO T259 “Resistance of Concrete to Chloride Ion Penetration.” Following 14 days of moist curing, the surfaces of the blocks were prepared with a light sandblasting to simulate a deck surface subjected to vehicular traffic (Figure 5.7). The specimens were air dried for 28 days and then ponded with a 3 percent NaCl solution in a controlled environment.

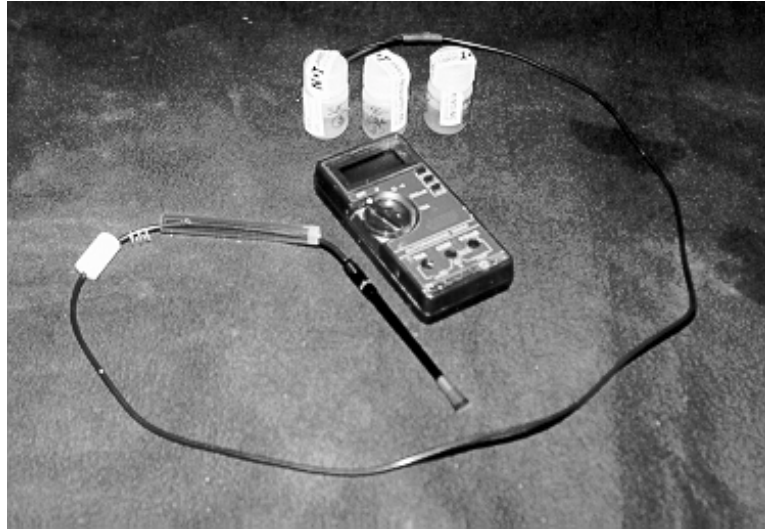


Figure 5.15: Chloride Testing Equipment and Calibration Solutions

After 90 days of continuous ponding in a climate-controlled room, powdered samples were collected from each ponding block as well as the control block by drilling three holes to various depths as follows: 1.6 mm to 12.7 mm (0.065 in. to 0.5 in), 12.7 mm to 25.4 mm (0.5 in. to 1.0 in.), and 25.4-mm to 38.1 mm (1.0 in. to 1.5 in.). The concrete powdered samples from each depth range were mixed and then divided into two 1.5-gram (0.0529-oz.) samples. This resulted in six ponding block samples and two control block samples for each casting date, sampled at each depth. The powder was then mixed with a 15 percent acetic acid extraction solution that reacted with the chloride ions and produced an electrochemical reaction, the degree of which was measured by the electrode. All drilled holes were then closed and sealed using a rapid setting epoxy prior to additional ponding stages. Figure 5.16 illustrates the ponding blocks in the controlled environment as well as the drilling, sampling, and epoxy application stages.

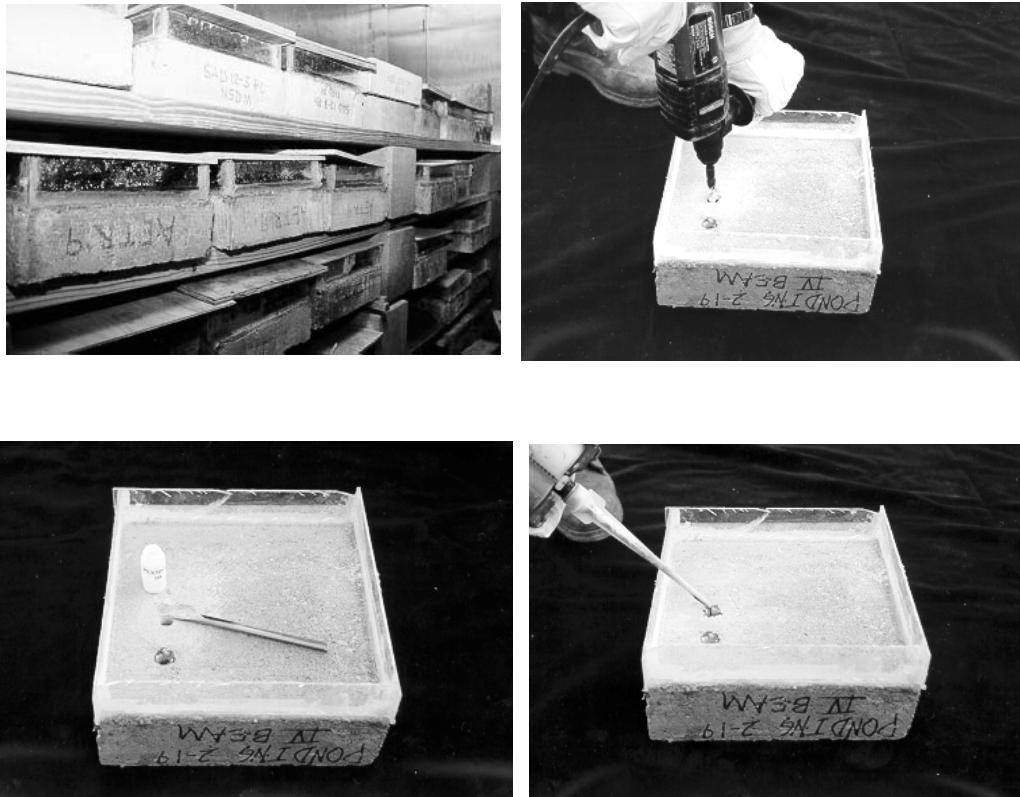


Figure 5.16: Chloride Ponding Curing Regimes and Sampling Technique

Ponding periods for the mix designs, which were investigated as part of the comparison study presented in Chapter 10, were continued for a long-term ponding period of 365 days due to the slow intrusion of chlorides related to these HS/HPC mix designs. Figure 5.17 illustrates the ponding blocks after two stages of ponding.



Figure 5.17: Chloride Ponding Blocks after 90-Day (left block) and 365-Day (right) Cycles

It may be noted that a core has been taken from the 365-day ponding block as part of the correlation study presented in Chapter 10. Measurements from the testing were converted into percent chloride ion content using Procedure C presented in AASHTO T260 “Sampling and Testing for Chloride Ion in Concrete and Concrete Raw Materials.” Sampling for chloride ion penetration focused on concrete produced for both the North Concho River Overpass Bridge and Louetta CIP decks where the use of chloride salts was anticipated.

5.3.5.2.7 Freeze-Thaw Testing ASTM C666. The test for resistance to freezing and thawing of the concrete was initiated at 14 days on three companion specimens in accordance with ASTM C666 Procedure A “Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing.” Subsequent weight and frequency measurements were made at least every 20 cycles thereafter until either 300 cycles or the squared value of the fundamental frequency was reduced to 60 percent of the square of the initial frequency value. The final freeze-thaw resistance values were obtained. Values including the number of cycles, mass, and resonant frequency were determined in accordance with ASTM C215 “Impact Resonance Method.” The relative dynamic modulus, final mass change, and durability factors for each set of specimens after 300 cycles were determined in accordance with ASTM C666, Sections 9.1 and 9.2, respectively.

Sampling for freeze-thaw resistance focused on concrete produced for the North Concho River Overpass Bridge CIP decks where moist saturation and freeze-thaw cycles were anticipated. Due to the mild Houston climate, concrete produced for the Louetta CIP decks was not sampled. Testing services for all of the freeze-thaw specimens were provided by the FHWA in McLean, Virginia. A typical freeze-thaw testing machine is shown in Figure 5.18.



Figure 5.18: Freeze-Thaw Testing Machine

5.3.5.2.8 Abrasion Resistance ASTM C944. The abrasion resistance test was initiated at 14 days on three companion specimens in accordance with ASTM C944 “Standard Test Method for Abrasion Resistance of Concrete or Mortar Surfaces by the Rotating Cutter Method.” The testing machine, shown in Figure 5.19, consisted of a drill press rotating at a speed of 200 rpm and exerting a force of 10 kgf (98N) on the surface being tested. The rotating cutter consisted of twenty-four grinding dressing wheels. A total of 6-minutes of abrasion were performed on every surface tested. The depth of wear on the surface of the specimen was measured to determine the abrasion resistance of the concrete. The average of three readings at each measuring time was used as the depth of wear for each specimen. Sampling for abrasion resistance focused on concrete produced for the North Concho River Overpass Bridge CIP decks where conventional concrete, HPC, and HS/HPC could be compared. Testing services for all of the abrasion resistance specimens were provided by the Materials and Tests Division of TxDOT in Austin.

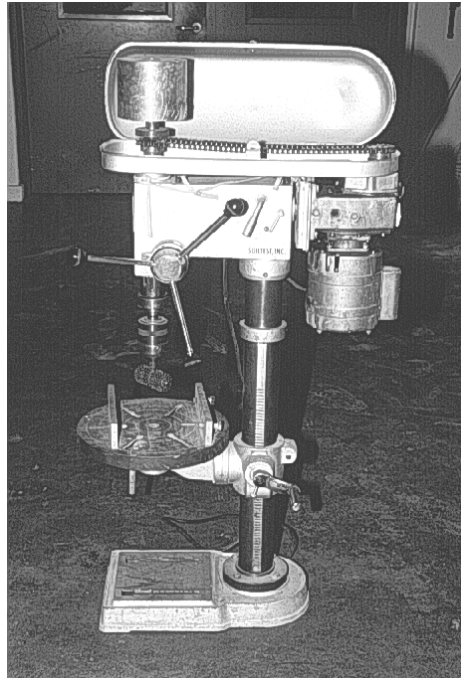


Figure 5.19: Abrasion Resistance Testing Machine

5.3.5.2.9 Scaling Deicing ASTM C672. The scaling resistance test was initiated at 28 days on three companion specimens in accordance with ASTM C672 “Standard Test Method for Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals.” Prior to initiating the test, a caulking dike was built around the edges of the specimens to hold the salt solution in place. The deicing agent that was used consisted of a 3 percent solution of calcium chloride. Visual inspection of the test specimens was done after 50 cycles. The damage was assessed according to ASTM C672 on a scale from 0 to 5 with 0 indicating no deterioration and 5 indicating severe damage.

Sampling for scaling resistance focused on concrete produced for the North Concho River Overpass Bridge CIP decks where moist saturation, freeze-thaw cycles, and salt usage were anticipated. Testing services for all of the scaling resistance specimens were provided by the Materials and Tests Division of TxDOT in Austin.

5.3.6 Variation in Curing Conditions under Investigation

The curing histories were formulated to represent a range of different scenarios that occur in the construction industry for both precast and CIP concrete. The study explored the variations in strength and other mechanical and material properties that occur due to different curing regimes for HPC.

5.3.6.1 Precast Concrete. The following curing conditions were investigated for the precast concrete as part of the research study on HPC.

5.3.6.1.1 ASTM Moist Cured—Precast. This group was cured according to the ASTM C31/C31M-95 specification and was intended to serve as the control group. The cylinders were kept in a 21.1 °C (70 °F) room for approximately 24 hours after casting at which time they were demolded and cured in a moist-curing room (Figure 5.20) until testing.



Figure 5.20: ASTM Moist-Curing Room

5.3.6.1.2 TxDOT Member Cured. These specimens are the standard TxDOT control specimens in the precast/prestress bridge industry in Texas and are used to verify that release and design strengths have been met for the precast/prestressed members. These cylinders were kept along the beds of the precast elements until the forms for the member were stripped (typically 22 to 26 hours) at which time three representative cylinders were tested to verify release strength before approved release of the prestressing strands. If a member were cast on a Saturday, the cylinders would remain alongside the member for approximately 48 hours, until Monday when the forms and strands would be released. The member-cured cylinders were intended to be representative of the concrete in the member because they were cured at a similar temperature profile as the concrete in the member. After testing for release strength (Release of Tension Cylinders), the additional specimens were then placed in a moist-curing room until testing for verification of specified design strengths (Design Strength Cylinders) as described in the TxDOT Manual of Testing Procedures [135] as follows:

1. Release of Tension Cylinders, Partial Tensioning Cylinders, and Tensioning Cylinders shall be cured identically to, and along with, the members they represent, and shall be placed where the most unfavorable conditions are offered. If elevated temperature curing is used, the cylinders shall be cured at the coolest point. If post tensioned members are partially tensioned and removed from the casting bed to the curing area, the tensioning cylinders shall also be moved to this area and again resume curing identical to the members. If curing requirements of the members are completed prior to final tensioning, curing of the tensioning cylinders shall also cease at this point. However, prior to testing cylinders that have completed this curing period, the cylinders shall be moisture conditioned by submerging them in water for a minimum period of 40 hours. This is to assure a uniform moisture condition of the cylinder.
2. Design Strength Cylinders shall be cured identically to, and along with, the members they represent until release of stress or partial tensioning strength is obtained. At this time they will be inspected carefully to determine if the cylinder has any inaccuracies of molding or any visible evidence of damage, marked with a serial number and date of casting, and placed in a tank or moist cabinet or room.

5.3.6.1.3 *Member Cured.* This group of cylinders was very similar to the TxDOT member cured-cylinders, with one exception. After removing the precast forms and releasing the prestressing strands for the precast member, the cylinders were stored in an ambient condition adjacent to the precast member until testing. In the case of the U-beams, the cylinders were stored inside the web of the member. It was felt that these specimens would be more representative of concrete in the member than the TxDOT member-cured cylinders because they were cured continuously alongside the member and not subjected to continuous moist curing after release of prestressing strands. Figure 5.21 shows a member-cured cylinder that is located on the precast forms along the precasting bed line. The member-cured cylinder illustrated in Figure 5.21 is instrumented with a standard shielded thermocouple that is tied into the data acquisition system in order to monitor the temperature profile during hydration of the concrete. Further specifics of the temperature-related investigation are discussed in Chapter 8 of this report.

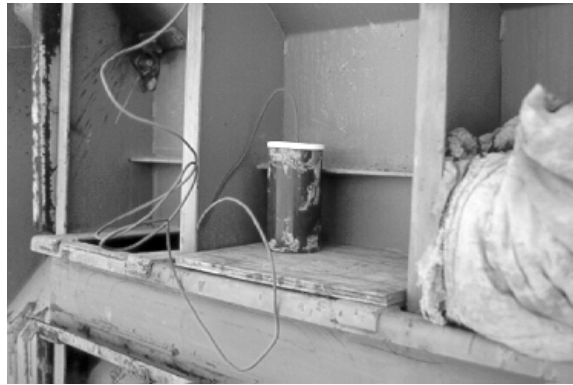


Figure 5.21: Member-Cured Cylinder

5.3.6.1.4 Match Cured. Finally, this group of cylinders was selected to more precisely simulate the actual concrete in various locations of the member by curing cylinders at the exact temperature profile as the member. While member-cured cylinders did a good job at simulating concrete in the member, the specimens did not cure at the exact temperature profile as documented in Chapter 8. These cylinders were cured in steel match-curing molds (Figure 5.9) at the exact temperature profile of the member until release of the strands, at which time the cylinders were stored in an ambient condition adjacent to the precast member until testing.

5.3.6.2 Cast-in-Place Concrete. The following curing conditions were investigated for the CIP concrete as part of the research study on HPC.

5.3.6.2.1 ASTM Moist Cured—CIP. This group was cured according to the ASTM C31/C31M-95 specification and was intended to serve as the control group for the CIP concrete. The cylinders were cured at the site for approximately 24 hours after casting at which time they were demolded and cured in a moist-curing room until testing. The specimens were protected from both high and low temperatures during the initial 24-hour period.

5.3.6.2.2 Site/Field Cured. This group was cured at the site for approximately 24 hours at which time the plastic molds were stripped. The specimens were then left exposed to ambient weather conditions at the site until they were tested. It was felt that these cylinders would best represent the CIP concrete in the structure. It should be noted that there was one modification to this procedure for concrete sampled from the CIP concrete decks. After stripping off the molds, these specimens were placed under the wet-mats on the surface of the concrete decks for the duration of wet-mat moist curing, followed by exposure to ambient weather conditions at the site until they were tested. It was felt that in this manner the specimens would benefit from the heat of hydration, the daily

temperature gradient profile of the deck, and the wet-mat curing associated with the concrete deck and, thereby, better represent the CIP concrete in the decks.

5.3.6.2.3 ASTM/Maturity Moist Cured. This group was cured according to the ASTM C1074-93 (ASTM C511) specification for estimating concrete strength by the maturity method and was intended to serve as the control group to develop the concrete maturity curve of the mix design. The maturity cylinders that were cast at the respective bridge sites were kept in a climate-controlled environment adjacent to the site at 21.1 °C (70 °F) for approximately 24 hours after casting at which time they were demolded and cured in a moist-curing room until testing. Concrete maturity curves were developed in the laboratory at CMRG for specimens that were kept in a climate-controlled room at 21.1 °C (70 °F) for approximately 24 hours after casting at which time they were demolded and cured in a moist-curing room until testing.

5.3.6 Additional QC/QA Testing Services

Additional testing services were provided by the FHWA Mobile Concrete Laboratory at both the Louetta Road Overpass Bridge and North Concho River US 87 and S.O. RR Overpass Bridge sites as illustrated in Figure 5.22. This mobile laboratory is capable of performing in situ mechanical and durability testing.



Figure 5.22: Federal Highway Administration (FHWA) Mobile Concrete Laboratory

5.4 Performance-Based Specifications

TxDOT used a performance-based specification system for both mechanical and material properties. It was felt that in this manner, concrete mix designs could be “engineered” and optimized in the laboratory to address and satisfy the particular concrete property that was considered critical for the respective component. For example, compressive strength, flexural strength, and elastic modulus would be critical for a beam while concrete permeability, abrasion resistance, freeze-thaw resistance, and scaling resistance would be more critical for a bridge deck. The preceding sections present the requirements of the performance-based specifications for the two bridge projects.

5.4.1 Specification Requirements

Tables 5.13 and 5.14 outline the required compressive strengths for the respective components of the bridges, which were designated as HPC in the project specifications.

Table 5.13: Required Compressive Strengths for Precast Concrete Components

HPC Component	High Performance Concrete Bridge Project	
	Louetta Road Overpass Bridge	North Concho River Overpass Bridge
Precast Prestressed Beams	(U-Beam Shape)	(AASHTO Type IV Shape)
at Release	6,900 - 8,800 psi (47.6 - 60.7 MPa)	8,900 - 10,800 psi (61.4 - 74.5 MPa)
at 56-Days	9,800 - 13,100 psi (67.6 - 90.3 MPa)	10,900 - 14,700 psi (75.2 - 101.4 MPa)
Precast Deck Panels		
at Release	6,000 psi (41.4 MPa)	5,000 psi (34.5 MPa)
at 28-Days	8,000 psi (55.2 MPa)	6,000 psi (41.4 MPa)
Precast Column Element	(Post-tensioned Precast Element)	Not Applicable
at Release	6,000 psi (41.4 MPa)	
at 28-Days	10,000 psi (68.9 MPa)	

Table 5.14: Required Compressive Strengths for Cast-in-Place Concrete Components

HPC Component	High Performance Concrete Bridge Project	
	Louetta Road Overpass Bridge	North Concho River Overpass Bridge
Cast-in-Place Piers at 28-Days	Not Applicable	6,000 psi (41.4 MPa)
Cast-in-Place Pier Caps at 28-Days	Not Applicable	8,000 psi (55.2 MPa)
Cast-in-Place Decks at 28-Days	8,000 psi (55.2 MPa)	6,000 psi (41.4 MPa)

In addition to the required compressive strength, the prestressed beams for both projects required a minimum elastic modulus of 41.4 GPa (6,000 psi) at release and at 56 days to satisfy serviceability requirements. The specifications also required a minimum flexural strength of $830 \sqrt{f'_c}$, kPa ($10 \sqrt{f'_c}$, psi) at release and at 56 days for the prestressed beams.

As discussed in Section 3.1, concrete may be classified as HPC based on strength and durability properties as illustrated by the current SHRP HPC definition for highway structures. For the purposes of this report, when the primary enhanced property is strength, the concrete is referred to as high strength/high performance concrete (HS/HPC). When the primary enhanced property is something other than strength such as one or several durability aspects, the concrete is referred to as high performance concrete (HPC). Table 5.15 further classifies the precast and CIP mix designs utilized in the two bridge projects in terms of strength and durability.

Table 5.15: HPC Classifications for Strength and Durability

	HPC for Strength	HPC for Durability
<i>Precast Produced Concrete</i>		
Louetta Road Overpass NB/SB HS/HPC Prestressed Beams	Yes	No
NCRO EB HS/HPC Prestressed Beams	Yes	No
NCRO WB Non-HPC Prestressed Beams*	No	No
Louetta Road Overpass NB/SB HS/HPC Precast Deck Panels	Yes	Yes
NCRO EB HS/HPC Precast Deck Panels	Yes	Yes
NCRO WB NSC Precast Deck Panels	No	No
<i>Cast-in-Place Produced Concrete</i>		
Louetta Road Overpass NB HPC Class S Modified Deck	No	Yes
Louetta Road Overpass SB HS/HPC Class K Deck	Yes	Yes
NCRO EB HS/HPC Class K Deck	Yes	Yes
NCRO WB HPC Class S Modified Deck	No	Yes
NCRO WB NSC Class S Deck	No	No
* The WB North Concho River Overpass (NCRO) prestressed beam concrete was not designated as HPC because it was made with standard concrete produced in the precast/prestressed industry in Texas. By some definitions presented in Chapter 3, its design compressive strength of 61.4 MPa (8,900 psi) would classify it as HPC.		

5.4.2 Additional Project Guidelines

In order to meet the desired 75- to 100-year design life of these bridges, the FHWA and TxDOT imposed guidelines for a maximum value of 1,500 coulombs passed at 56 days for all HPC mix designs developed based on AASHTO T277 Rapid Chloride Permeability Test (RCPT). While this guideline was not a specification requirement for either bridge project, mix designs satisfying this criterion were felt to result in higher quality, more durable, longer-lasting concrete.

CHAPTER 6. TEST RESULTS AND DISCUSSION

6.1 Laboratory-Produced Concrete (Lab Trial Mix Designs)

Since the summer of 1993, approximately 175 trial batches were made using locally available materials at the laboratory of CMRG at UT. This trial phase was initiated by A. Cetin who conducted over 75 of these trial mixes and who developed the original laboratory mix design recommended for use in the prestressed/precast U-beams. His report, "Effect of Accelerated Heat Curing and Mix Characteristics on the Heat Development and Mechanical Properties of High Performance Concrete," may be referenced for further documentation on these trial mixes [15].

Trial laboratory and trial field mix designs for all other HPC elements were conducted at the laboratory of CMRG at UT by the author as part of this research study. Tables 6.1 through 6.10 present the mix properties for the laboratory trial mixes. Materials, test procedures, and performance-related testing of these laboratory trial mixes is described in Chapter 5 of this report.

Table 6.1: Mix Properties for Laboratory Trial Batches

MIX DESIGNATION	CG1	CG2	CG3	CG4	CG5	CG6	CG7	CG8
Mix Proportions								
Coarse Aggregate, Type (ASTM Grade 6) Quantity	CGS 3/4" max. 1967 pcy	CGS 3/4" max. 1782 pcy	CGS 3/4" max. 1946 pcy	CGS 3/4" max. 1946 pcy	CGS 3/4" max. 1946 pcy	CGS 3/4" max. 1838 pcy	CGS 3/4" max. 1838 pcy	CGS 3/4" max. 1838 pcy
Fine Aggregate, Type (Natural River Sand, FM=2.45) Quantity	S-I 882 pcy	S-I 969 pcy	S-I 884 pcy	S-I 884 pcy	S-I 884 pcy	S-I 1291 pcy	S-I 1291 pcy	S-I 1291 pcy
Water, (City of Austin) Quantity	262 pcy	290 pcy	305 pcy	288 pcy	246 pcy	238 pcy	230.5 pcy	200.5 pcy
Cement, Type Quantity	Type A-III 888 pcy	Type A-III 1000 pcy	Type B-I/II 850 pcy	Type B-I/II 638 pcy	Type B-I/II 552 pcy	Type B-I/II 611 pcy	Type B-I/II 458 pcy	Type B-I/II 397 pcy
Fly Ash, (ASTM Class C) Quantity Pct. Replacement by Weight	None	None	None	212 25%	298 35%	None	153 25%	214 35 %
Retarder, (ASTM Type B) Quantity	28.3 oz/cy	31.9 oz/cy	27.1 oz/cy	26.2 oz/cy	21.7 oz/cy	18.2 oz/cy	17.6 oz/cy	15 oz/cy
HRWR, (ASTM Type F) Quantity	142 oz/cy	160 oz/cy	58 oz/cy	66 oz/cy	136 oz/cy	None	None	None
Air Entrainment, (ASTM C260) Quantity	None	None	None	None	None	None	None	None
Fresh Concrete Properties								
W/Cm (C+ FA), by weight	0.30	0.29	0.36	0.34	0.29	0.39	0.28	0.33
Total Air	NR	NR	NR	NR	NR	NR	NR	NR
Slump, before HRWR	1"	1"	4"	2"	4"	2"	6.5"	3"
Slump, after HRWR	7.5"	10"	9"	7"	9.5"	-	-	-

Table 6.2: Mix Properties for Laboratory Trial Batches (continued)

MIX DESIGNATION	CG9	CG10	CG11	CG12	CG13	CG14	CG15	CG16	CG17
Mix Proportions									
Coarse Aggregate, Type (ASTM Grade 6) Quantity	CGS 3/4" max. 1946 pcy	CGS 3/4" max. 1946 pcy	CGS 3/4" max. 1946 pcy	CGS 3/4" max. 1946 pcy	CGS 3/4" max. 1946 pcy	CGS 3/4" max. 1946 pcy	CGS 3/4" max. 1946 pcy	CGS 3/4" max. 1946 pcy	CGS 3/4" max. 1946 pcy
Fine Aggregate Type (Natural River Sand, FM=2.45) Quantity	S-I 884 pcy	S-I 884 pcy	S-I 884 pcy	S-I 884 pcy	S-I 884 pcy	S-I 884 pcy	S-I 884 pcy	S-I 884 pcy	S-I 884 pcy
Water, (City of Austin) Quantity	246 pcy	246 pcy	246 pcy	279 pcy	246 pcy	246 pcy	332 pcy	278 pcy	259 pcy
Cement, Type Quantity	Type A-III 850 pcy	Type A-III 638 pcy	Type A-III 552 pcy	Type A-III 850 pcy	Type A-III 638 pcy	Type A-III 552 pcy	Type A-III 850 pcy	Type A-III 638 pcy	Type A-III 552 pcy
Fly Ash, (ASTM Class C) Quantity Pct. Replacement by Weight	None	212 25%	298 35%	None	212 25%	298 35%	None	212 25%	298 35%
Retarder, (ASTM Type B) Quantity	27 oz/cy	27 oz/cy	27 oz/cy	27 oz/cy	27 oz/cy	27 oz/cy	27 oz/cy	27 oz/cy	27 oz/cy
HRWR, (ASTM Type F) Quantity	175 oz/cy	129 oz/cy	156 oz/cy	175 oz/cy	129 oz/cy	156 oz/cy	None	None	None
Air Entrainment, (ASTM C260) Quantity	None	None	None	None	None	None	None	None	None
Fresh Concrete Properties									
W/Cm (C+ FA), by weight	0.29	0.29	0.29	0.33	0.29	0.29	0.39	0.27	0.30
Total Air	NR	NR	NR	NR	NR	NR	NR	NR	NR
Slump, before HRWR	0.5"	1"	1.75"	0.5"	0.75"	0.75"	3"	5"	5.5"
Slump, after HRWR	7.5"	8.5"	9"	7"	9.5"	7.5"	-	-	-

Table 6.3: Mix Properties for Laboratory Trial Batches (continued)

MIX DESIGNATION	LSC1	LSC2	LSC3	LSC4	LSC5	RGC1	RGC2	RGC3	RGC4	RGC5
Mix Proportions										
Coarse Aggregate, Type (ASTM Grade 6) Quantity	DLL 3/4" max. 1838 pcy	DLL 3/4" max. 1823 pcy	DLL 3/4" max. 1808 pcy	DLL 3/4" max. 1793 pcy	DLL 3/4" max. 1778 pcy	CGS 3/4" max. 1838 pcy	CGS 3/4" max. 1823 pcy	CGS 3/4" max. 1808 pcy	CGS 3/4" max. 1793 pcy	CGS 3/4" max. 1778 pcy
Fine Aggregate, Type (Natural River Sand, FM=2.45) Quantity	S-II 1294 pcy	S-II 1272 pcy	S-II 1508 pcy	S-II 1562 pcy	S-II 1618 pcy	S-II 1294 pcy	S-II 1272 pcy	S-II 1508 pcy	S-II 1562 pcy	S-II 1618 pcy
Water, (City of Austin) Quantity	270 pcy	300 pcy	230 pcy	224 pcy	216 pcy	250 pcy	250 pcy	230 pcy	224 pcy	216 pcy
Cement, Type Quantity	Type A-III 611 pcy	Type A-III 564 pcy	Type A-III 362 pcy	Type A-III 329 pcy	Type A-III 296 pcy	Type CI/II 611 pcy	Type CI/II 564 pcy	Type CI/II 362 pcy	Type CI/II 329 pcy	Type CI/II 296 pcy
Fly Ash, (ASTM Class C) Quantity Pct. Replacement by Weight	None	None	155 30%	141 30%	127 30%	None	None	155 30%	141 30%	127 30%
Retarder, (ASTM Type B) Quantity	19.5 oz/cy	18 oz/cy	16.5 oz/cy	15 oz/cy	13.5 oz/cy	19.5 oz/cy	18 oz/cy	16.5 oz/cy	15 oz/cy	13.5 oz/cy
HRWR, (ASTM Type F) Quantity	None	None	None	None	None	None	None	None	None	None
Air Entrainment, (ASTM C260) Quantity	None	None	None	None	None	None	None	None	None	None
Fresh Concrete Properties										
W/Cm (C+ FA),by weight	0.44	0.53	0.44	0.48	0.51	0.41	0.44	0.44	0.48	0.51
Total Air	NR	NR	NR	NR	NR	NR	NR	NR	NR	NR
Slump, before HRWR	3"	5"	3"	3"	3"	3"	4"	4.5"	4.25"	3.75"
Slump, after HRWR	-	-	-	-	-	-	-	-	-	-

Table 6.4: Mix Properties for Laboratory Trial Batches (continued)

MIX DESIGNATION		LS1	LS2	LS3	LS4	Heat Curing Time Schedule			
Mix Proportions									
Coarse Aggregate, Type (ASTM Grade 6)	CGS 3/4" max. 1803 pcy	CGS 3/4" max. 1803 pcy	CGS 3/4" max. 1803 pcy	CGS 3/4" max. 1803 pcy					
Quantity									
Fine Aggregate, Type (Natural River Sand, FM=2.45)	S-I 991 pcy	S-I 991 pcy	S-I 991 pcy	S-I 991 pcy	Hour	LS1 & LS4	LS2	LS3	
Quantity					4	70 °F	70 °F	70 °F	
Water, (City of Austin)	Quantity	268 pcy	268 pcy	268 pcy	268 pcy	5	100 °F	100 °F	100 °F
						6	130 °F	130 °F	130 °F
Cement,	Type A-III 797 pcy	Type C-III 797 pcy	Type C-III 797 pcy	Type C-III 797 pcy	8	160 °F	190 °F	140 °F	
									7
Fly Ash, (ASTM Class C)	Quantity	200	200	200	200	9	160 °F	190 °F	140 °F
						16	160 °F	190 °F	140 °F
Pct. Replacement by Weight	20%	20%	20%	20%	18	70 °F	70 °F	70 °F	
Retarder, (ASTM Type B)	Quantity	32 oz/cy	32 oz/cy	32 oz/cy	32 oz/cy	Notes: Preset Time = 4 hours Control Mixes Without Accelerated Heat Curing: LS2A, LS3A, DLS40 [15] deg. F = (9/5)(deg. C) + 32 deg. C = (5/9)(deg. F - 32)			
HRWR, (ASTM Type F)	Quantity	223 oz/cy	223 oz/cy	223 oz/cy	223 oz/cy				
Air Entrainment, (ASTM C260)	Quantity	None	None	None	None				
Fresh Concrete Properties									
W/Cm (C+ FA), by weight	0.27	0.27	0.27	0.27					
Total Air	NR	NR	NR	NR					
Slump, before HRWR	0.5"	0.25"	0.5"	0.5"					
Slump, after HRWR	9"	9.5"	9"	9"					

Table 6.5: Mix Properties for Laboratory Trial Batches (continued)

MIX DESIGNATION	MLS1	MLS2	MLS3	MLS4	MLS5	MLS6	MLS7	MLS8
Mix Proportions								
Coarse Aggregate, Type (ASTM Grade 6)	ML 3/4" max.	ML 3/4" max.	ML 3/4" max.	ML 3/4" max.	ML 3/4" max.	ML 3/4" max.	ML 3/4" max.	ML 3/4" max.
Quantity	1803 pcy	1803 pcy	1803 pcy	1890 pcy	1718 pcy	1547 pcy	1547 pcy	1547 pcy
Fine Aggregate, Type (Natural River Sand, FM=2.45)	S-II	S-II	S-II	S-II	S-II	S-II	S-II	S-II
Quantity	991 pcy	911 pcy	991 pcy	811 pcy	988 pcy	1166 pcy	1468 pcy	1692 pcy
Water, (City of Austin)	Quantity	294 pcy	283 pcy	278 pcy	316 pcy	294 pcy	294 pcy	247 pcy
Cement, Type	Type C-III	Type C-III	Type C-III	Type C-III	Type C-III	Type C-III	Type C-III	Type C-III
Quantity	997 pcy	797 pcy	697 pcy	1006 pcy	1006 pcy	1006 pcy	752 pcy	564 pcy
Fly Ash, (ASTM Class C)	Quantity	None	212	298	None	None	None	None
Pct. Replacement by Weight		25%	35%					
Retarder, (ASTM Type B)	Quantity	31.8 oz/cy	31.8 oz/cy	31.8 oz/cy	32.1 oz/cy	32.1 oz/cy	32.1 oz/cy	24 oz/cy
HRWR, (ASTM Type F)	Quantity	168 oz/cy	223 oz/cy	223 oz/cy	167 oz/cy	146 oz/cy	137 oz/cy	125 oz/cy
Air Entrainment, (ASTM C260)	Quantity	None	None	None	None	None	None	None
Fresh Concrete Properties								
W/Cm (C+ FA), by weight	0.29	0.28	0.28	0.31	0.29	0.29	0.33	0.38
Total Air	NR	NR	NR	NR	NR	NR	NR	NR
Slump, before HRWR	0.5"	0.5"	0.5"	0.25"	0.5"	0.5"	0.5"	0.75"
Slump, after HRWR	10"	8.5"	10"	9.75"	9.25"	10"	8.5"	8"

Table 6.6: Mix Properties for Laboratory Trial Batches (continued)

Mix Designation	MFA1	MFA2	MFA3	RG1	RG2	RG3	SSP-3	SSP-32	HP-3	HP-32
Mix Proportions										
Coarse Aggregate, Type (ASTM Grade 6)	ML 3/4" max.	ML 3/4" max.	ML 3/4" max.	CG 3/4" max.	CG 3/4" max.	CG 3/4" max.	CGS 3/4" max.	CGS 3/4" max.	CGS 3/4" max.	CGS 3/4" max.
Quantity	1823 pcy	1823 pcy	1823 pcy	1547 pcy	1718 pcy	1890 pcy	1946 pcy	1946 pcy	1946 pcy	1946 pcy
Fine Aggregate, Type (Natural River Sand, FM=2.45)	S-II	S-II	S-II	S-II	S-II	S-II	S-II	S-II	S-II	S-II
Quantity	272 pcy	1272 pcy	1272 pcy	1166 pcy	988 pcy	811 pcy	884 pcy	884 pcy	884 pcy	884 pcy
Water, (City of Austin)	Quantity	320 pcy	288 pcy	286 pcy	282 pcy	282 pcy	282 pcy	210 pcy	169.2 pcy	210 pcy
Cement, Type	Type CI/II	Type CI/II	Type CI/II	Type CI/II	Type CI/II	Type CI/II	Type CI/II	Type CI/II	Type CI/II	Type CI/II
Quantity	611 pcy	564 pcy	362 pcy	1006 pcy	1006 pcy	1006 pcy	549.9 pcy	549.9 pcy	549.9 pcy	549.9 pcy
Fly Ash, (ASTM Class C)	Quantity	None	113	152	None	None	None	296.1	296.1	296.1
Pct. Replacement by Weight		20%	27%					35%	35%	35%
Retarder, (ASTM Type B)	Quantity	16.8 oz/cy	16.2 oz/cy	13.8 oz/cy	32 oz/cy	32 oz/cy	32 oz/cy	27 oz/cy	27 oz/cy	27 oz/cy
HRWR, (ASTM Type F)	Quantity	None	None	None	64 oz/cy	64 oz/cy	64 oz/cy	166 oz/cy	679 oz/cy	180 oz/cy
Air Entrainment, (ASTM C260)	Quantity	None	None	None	None	None	None	6.95 oz/cy	6.95 oz/cy	3.86 oz/cy
										3.86 oz/cy
Fresh Concrete Properties										
W/Cm (C+ FA),by weight	0.57	0.51	0.51	0.28	0.28	0.28	0.25	0.20	0.25	0.20
Entrained Air							0.3%	1.1%	3.3%	3.3%
Total Air	NR	NR	NR	NR	NR	NR	1.5%	2.3%	4.5%	4.5%
Slump, before HRWR	3"	4"	4.5"	1"	0.5"	0.25"	0.25"	0.25"	0.25"	0.25"
Slump, after HRWR	-	-	-	10"	9.5"	7.5"	10"	8.5"	9.5"	8.75"

Table 6.7: Mix Properties for Laboratory Trial Batches (continued)

Mix Designation	AELS1	AELS2	AELS3	AELS4	AELS5	AELS6	AELS7	AELS8	AELS9
Mix Proportions									
Coarse Aggregate, Type (ASTM Grade 6)	DLL 3/4" max.	DLL 3/4" max.	DLL 3/4" max.	DLL 3/4" max.	DLL 3/4" max.	DLL 3/4" max.	DLL 3/4" max.	DLL 3/4" max.	DLL 3/4" max.
Quantity	1946 pcy	1946 pcy	1946 pcy	1946 pcy	1946 pcy	1946 pcy	1946 pcy	1946 pcy	1946 pcy
Fine Aggregate, Type (Natural River Sand, FM=2.45)	S-II	S-II	S-II	S-II	S-II	S-II	S-II	S-II	S-II
Quantity	884 pcy	884 pcy	884 pcy	884 pcy	884 pcy	884 pcy	884 pcy	884 pcy	884 pcy
Water, (City of Austin)	250 pcy	250 pcy	250 pcy	224 pcy	224 pcy	224 pcy	188 pcy	188 pcy	188 pcy
Cement, Type	Type C-I/II	Type C-I/II	Type C-I/II	Type C-I/II	Type C-I/II	Type C-I/II	Type C-I/II	Type C-I/II	Type C-I/II
Quantity	846 pcy	846 pcy	846 pcy	488.8 pcy	488.8 pcy	488.8 pcy	564 pcy	564 pcy	564 pcy
Fly Ash, (ASTM Class C)	None	None	None	263.2	263.2	263.2	188	188	188
Percent Replacement by Weight				35%	35%	35%	25%	25%	25%
Retarder, (ASTM Type B)	27.7 oz/cy	27.7 oz/cy	27.7 oz/cy	25.0 oz/cy	27.8 oz/cy	27.8 oz/cy	27.8 oz/cy	27.8 oz/cy	27.8 oz/cy
HRWR, (ASTM Type F)	111 oz/cy	111 oz/cy	111 oz/cy	64 oz/cy	33.2 oz/cy	33.2 oz/cy	69.4 oz/cy	69.4 oz/cy	69.4 oz/cy
Air Entrainment, (ASTM C260)	None	5.78 oz/cy	9.64 oz/cy	None	5.78 oz/cy	9.87 oz/cy	None	6.16 oz/cy	10.32 oz/cy
Fresh Concrete Properties									
W/Cm (C+ FA), by weight	0.30	0.30	0.30	0.30	0.30	0.30	0.26	0.27	0.27
Entrained Air	0.0%	1.2%	7.3%	0.0%	1.4%	4.5%	0.0%	2.1%	5.9%
Total Air	1.2%	2.0%	8.5%	1.3%	2.7%	5.8%	1.9%	4.0%	7.8%
Slump, before HRWR	0.25"	0.25"	0.5"	0.5"	1"	0.5"	0.25"	0.25"	0.25"
Slump, after HRWR	9.75"	9.5"	10"	10"	8"	9"	7"	10"	10"

Table 6.8: Mix Properties for Laboratory Trial Batches (continued)

Mix Designation	AETR1	AETR2	AETR3	AETR4	AETR5	AETR6	AETR7	AETR8	AETR9
Mix Proportions									
Coarse Aggregate, Type (ASTM Grade 6)	TR 3/4" max.	TR 3/4" max.	TR 3/4" max.	TR 3/4" max.	TR 3/4" max.	TR 3/4" max.	TR 3/4" max.	TR 3/4" max.	TR 3/4" max.
Quantity	1946 pcy	1946 pcy	1946 pcy	1946 pcy	1946 pcy	1946 pcy	1946 pcy	1946 pcy	1946 pcy
Fine Aggregate, Type (Natural River Sand, FM=2.45)	S-II	S-II	S-II	S-II	S-II	S-II	S-II	S-II	S-II
Quantity	884 pcy	884 pcy	884 pcy	884 pcy	884 pcy	884 pcy	884 pcy	884 pcy	884 pcy
Water, (City of Austin)	Quantity	250 pcy	250 pcy	250 pcy	224 pcy	224 pcy	224 pcy	224 pcy	224 pcy
Cement, Type	Type C-I/II	Type C-I/II	Type C-I/II	Type C-I/II	Type C-I/II	Type C-I/II	Type C-I/II	Type C-I/II	Type C-I/II
Quantity	846 pcy	846 pcy	846 pcy	488.8 pcy	488.8 pcy	488.8 pcy	564 pcy	564 pcy	564 pcy
Fly Ash, (ASTM Class C)	Quantity	None	None	None	263.2	263.2	263.2	188	188
Percent Replacement by Weight				35%	35%	35%	25%	25%	25%
Retarder, (ASTM Type B)	Quantity	27.7 oz/cy	27.7 oz/cy	27.7 oz/cy	27.6 oz/cy	27.6 oz/cy	27.6 oz/cy	24 oz/cy	24 oz/cy
HRWR, (ASTM Type F)	Quantity	86 oz/cy	86 oz/cy	86 oz/cy	64 oz/cy	64 oz/cy	64 oz/cy	112 oz/cy	112 oz/cy
Air Entrainment, (ASTM C260)	Quantity	None	5.00 oz/cy	6.95 oz/cy	None	3.32 oz/cy	7.76 oz/cy	None	2.47 oz/cy
	6.17 oz/cy								
Fresh Concrete Properties									
W/Cm (C+ FA), by weight	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30
Entrained Air	0.0%	2.0%	5.7%	0.0%	1.5%	4.8%	0.0%	3.1%	5.9%
Total Air	0.7%	2.7%	6.4%	1.2%	2.7%	6.0%	1.9%	5.0%	7.8%
Slump, before HRWR	0.25"	0.25"	0.25"	0.5"	0.25"	0.5"	0.25"	0.25"	0.5"
Slump, after HRWR	9"	9"	9.5"	9.5"	9"	9.5"	8.75"	10"	9"

Table 6.9: Mix Properties for Laboratory Trial Batches (continued)

Mix Designation	AERG1	AERG2	AERG3	AERG4	AERG5	AERG6	AERG7	AERG8	AERG9
Mix Proportions									
Coarse Aggregate, Type (ASTM Grade 6) Quantity	CGS 3/4" max. 1946 pcy	CGS 3/4" max. 1946 pcy	CGS 3/4" max. 1946 pcy	CGS 3/4" max. 1946 pcy	CGS 3/4" max. 1946 pcy	CGS 3/4" max. 1946 pcy	CGS 3/4" max. 1946 pcy	CGS 3/4" max. 1946 pcy	CGS 3/4" max. 1946 pcy
Fine Aggregate, Type (Natural River Sand, FM=2.45) Quantity	S-II 884 pcy	S-II 884 pcy	S-II 884 pcy	S-II 884 pcy	S-II 884 pcy	S-II 884 pcy	S-II 884 pcy	S-II 884 pcy	S-II 884 pcy
Water, (City of Austin) Quantity	250 pcy	241 pcy	210 pcy	220 pcy	224.5 pcy	224 pcy	188 pcy	188 pcy	188 pcy
Cement, Type Quantity	Type C-I/II 846 pcy	Type C-I/II 634.5 pcy	Type C-I/II 549.9 pcy	Type C-I/II 488.8 pcy	Type C-I/II 488.8 pcy	Type C-I/II 488.8 pcy	Type C-I/II 564 pcy	Type C-I/II 564 pcy	Type C-I/II 564 pcy
Fly Ash, (ASTM Class C) Quantity Percent Replacement Weight	None	211.5 25%	296.1 35%	263.2 35%	263.2 35%	263.2 35 %	188 25%	188 25%	188 25%
Retarder, (ASTM Type B) Quantity	27 oz/cy	27 oz/cy	27 oz/cy	24 oz/cy	24 oz/cy	24 oz/cy	28 oz/cy	28 oz/cy	28 oz/cy
HRWR, (ASTM Type F) Quantity	118 oz/cy	76 oz/cy	72 oz/cy	50 oz/cy	56 oz/cy	41.6 oz/cy	83.2 oz/cy	83.2 oz/cy	83.2 oz/cy
Air Entrainment, (ASTM C260) Quantity	None	5.00 oz/cy	6.94 oz/cy	None	6.10 oz/cy	10.3 oz/cy	None	2.45 oz/cy	6.17 oz/cy
Fresh Concrete Properties									
W/Cm (C+ FA), by weight	0.33	0.32	0.28	0.29	0.29	0.29	0.27	0.27	0.27
Entrained Air	0.0%	3.6%	1.6%	0.0%	3.0%	5.3%	0.0%	1.1%	3.0%
Total Air	0.9%	4.5%	2.5%	1.2%	4.2%	6.5%	1.1%	2.1%	4.0%
Slump, before HRWR	0.5"	0.5"	0.5"	0.75"	0.5"	0.75"	0.25"	0.25"	0.5"
Slump, after HRWR	9.75"	10"	7"	9.5"	10"	9.5"	9.5"	9.5"	10"

Table 6.10: Mix Properties for Final North Concho River Overpass Bridge Laboratory Trial Batches

Mix Designation	SA-K1	SA-K2	SA-S1	SA-S2	SA-S3
Mix Design Classification	Class K	Class K	Class S	Class S Mod	Class S Mod
Mix Proportions					
Coarse Aggregate, (ASTM Grade 6)	Crushed Gravel, 1-1/4" max. 1965 pcy	Crushed Gravel, 3/4" max. 1965 pcy	Crushed Gravel, 1-1/4" max. 1856 pcy	Crushed Gravel, 1-1/4" max. 1856 pcy	Crushed Gravel, 3/4" max. 1856 pcy
Fine Aggregate, (Natural River Sand, FM=2.45)	S-II 928 pcy	S-II 928 pcy	S-II 1573 pcy	S-II 1355 pcy	S-II 1355 pcy
Water, (City of Austin)	246 pcy	246 pcy	238 pcy	200 pcy	200 pcy
Cement,	Type C-I/II 552 pcy	Type C-I/II 552 pcy	Type C-I/II 611 pcy	Type C-I/II 397 pcy	Type C-I/II 397 pcy
Fly Ash, (ASTM Class C)	298	298	None	214	214
Percent Replacement by Weight	35 %	35 %		35 %	35 %
Retarder, (ASTM Type B)	27 oz/cy	27 oz/cy	19.5 oz/cy	19.5 oz/cy	19.5 oz/cy
HRWR, (ASTM Type F)	127 oz/cy	127 oz/cy	None	91 oz/cy	91 oz/cy
Air Entrainment, (ASTM C260)	None	None	5 oz/cy	None	None
Fresh Concrete Properties					
W/Cm (C+ FA), by weight	0.29	0.29	0.39	0.33	0.33
Total Air	NR	NR	6.5 %	NR	NR
Unit Weight	146.5 pcf	147.6 pcf	158.4 pcf	149.0 pcf	149.1 pcf
Slump, before HRWR	0.5"	0.5"	3.5"	0.5"	0.5"
Slump, after HRWR	7"	9"	-	9"	9"
Concrete Temperature, deg. F	95	94	96	94	95
Ambient Temperature, deg. F	99	98	94	95	95

6.2 Plant-Produced Concrete (Field Trial Mix Designs)

Following the successful development and optimization of the laboratory mix designs, a field trial mix phase was initiated to batch production-size trial mixes at the respective plants. This phase included the precast plants where the precast members would be fabricated, and the ready-mix batch plants where the CIP concrete would be produced. The purpose of this phase was twofold: First, we investigated the strength loss of the mix design that typically occurs when a laboratory mix design is utilized in the field. The field trial phase provided an opportunity to modify the mix design in the field as required prior to full-scale production if the mix design was not meeting the specification requirements under field conditions. Second, the field trial mixes were prepared at the precast and ready-mix batch plants so that plant personnel could gain additional experience with HRWRs and with the use of mineral admixtures. Undoubtedly during production, the various mix designs would require “tweaking” or adjustments to constituents, owing to environmental changes and/or placement constraints. The more experience the concrete producers had with a particular mix design, the less likely that placement concerns would become an issue. This was also the first opportunity to mix production-size batches [6.88 cubic meters (9 cubic yards)] prior to fabrication and construction of the bridges. Tables 6.11 through 6.13 illustrate the trial mix properties used at the respective precast and ready-mix batch plants, as outlined in Table 5.11 for the Louetta Road Overpass Bridge and for the North Concho River Overpass Bridge Projects.

6.3 Plant-Produced Concrete (Precast and Ready-Mix Designs)

Following the verification of concrete property requirements in the plant trial phase, production of concrete for the bridge and its components was at hand. Tables 6.14 through 6.34 present the mix properties for the precast plant and ready-mix batch plant produced concrete for both the Louetta Road Overpass Bridge and North Concho River Overpass Bridge Projects. This includes the individual batch tickets on the casting dates sampled for each of the CIP concrete components.

Table 6.11: Mix Properties for Louetta Precast Plant and Batch Plant Field Trial Batches

MIX DESIGNATION	LPC1	LSB1	LSB2	LNB1
Member/Component	Precast Column & Precast Panel Trial	HS/HPC Deck-Trial 1 (Southbound Deck)	HS/HPC Deck-Trial 2 (Southbound Deck)	HPC Deck-Trial 1 (Northbound Deck)
Mix Proportions				
Coarse Aggregate, Type (ASTM Grade 6) Quantity	Crushed Gravel, 3/4" max. 1983 pcy	Crushed Limestone, 1" max. 1811 pcy	Crushed Limestone, 1" max. 1811 pcy	Crushed Limestone, 1-1/2" max. 1856 pcy
Fine Aggregate, Type (Natural River Sand, FM=2.45) Quantity	Natural River Sand FM=2.60 1110 pcy	Natural River Sand FM=2.60 1304 pcy	Natural River Sand FM=2.60 1302 pcy	Natural River Sand FM=2.60 1243 pcy
Water, (City of Austin) Quantity	227 pcy	245 pcy	246 pcy	230 pcy
Cement, Type Quantity	Type A-III 564 pcy	Type H-I 546 pcy	Type H-I 473 pcy	Type H-I 423 pcy
Fly Ash, (ASTM Class C) Quantity Percent Replacement by Weight	163 22 %	158 22 %	221 32 %	115 21 %
Retarder, (ASTM Type B) Quantity	23 oz/cy	30 oz/cy	23 oz/cy	46 oz/cy
HRWR, (ASTM Type F) Quantity	223 oz/cy	165 oz/cy	429 oz/cy	None
Air Entrainment, (ASTM C260) Quantity	None	None	None	11.5 oz/cy
Fresh Concrete Properties				
W/Cm (C+ FA), by weight	0.31	0.35	0.35	0.43
Total Air	2.0 %	NR	NR	3.8 %
Unit Weight	149.9 pcf	150.6 pcf	150.2 pcf	143.2 pcf
Slump, before HRWR	1"	1"	1"	0.5"
Slump, after HRWR	9.5"	9.5"	9.5"	-
Concrete Temperature, Deg. F	NR	NR	NR	80
Ambient Temperature, Deg. F	87	82	NR	84

Table 6.12: Mix Properties for N.C.R.O. Batch Plant Field Trial Batches

Mix Designation	K-1	K-2	K-3	F-1	F-2
Mix Design Classification	Class K	Class K	Class K	Class F	Class F
Mix Proportions					
Coarse Aggregate, Type (ASTM Grade 6)	Crushed Gravel, 1-1/4" max.	Crushed Gravel, 1-1/4" max.	Crushed Gravel, 1-1/4" max.	Crushed Gravel, 1-1/4" max.	Crushed Gravel, 1-1/4" max.
Quantity	1965 pcy	1900 pcy	2004 pcy	NR	2000 pcy
Fine Aggregate, Type (Natural River Sand, FM=2.45)	Nat. River Sand FM=2.70	Nat. River Sand FM=2.70	Nat. River Sand FM=2.70	Nat. River Sand FM=2.70	Nat. River Sand FM=2.70
Quantity	928 pcy	928 pcy	928 pcy	NR	928 pcy
Water, (City of Austin)	Quantity	246 pcy	200 pcy	245 pcy	NR
Cement, Type	Type LS-II	Type LS-II	Type LS-II	Type LS-II	Type LS-II
Quantity	552 pcy	490 pcy	490 pcy	NR	526 pcy
Fly Ash, (ASTM Class C)	Quantity	298	210	210	NR
Percent Replacement by Weight	30 %	30 %	30 %	30 %	226 30 %
Retarder, (ASTM Type B)	Quantity	27 oz/cy	22 oz/cy	22 oz/cy	NR
HRWR, (ASTM Type F)	Quantity	127 oz/cy	156 oz/cy	156 oz/cy	None
Air Entrainment, (ASTM C260)	Quantity	None	None	None	NR
Fresh Concrete Properties					
W/Cm (C+ FA), by weight	0.29	0.29	0.35	NR	0.30
Total Air	NR	NR	NR	5.0 %	5.3 %
Unit Weight	146.5 pcf	NR	153.6 pcf	NR	NR
Slump, before HRWR	0.5"	0.5"	0.5"	NR	3.75"
Slump, after HRWR	10"	8.5"	8.75"	-	-
Concrete Temperature, Deg. F	95	94	75	75	73
Ambient Temperature, Deg. F	99	98	76	76	68

Table 6.13: Mix Properties for N.C.R.O. Batch Plant Field Trial Batches for Cast-In-Place Decks

Mix Designation		S2 10-18	S1 10-25	K1 4-17	K2 5-2	K3 5-2
Member/Component		Class S Mod	Class S	Class K	Class K	Class K
Mix Proportions						
Coarse Aggregate, (ASTM Grade 6)	Type Quantity	Crushed Gravel, 1-1/4" max. 1854 pcy	Crushed Gravel, 1-1/4" max. 1848 pcy	Crushed Gravel, 1-1/4" max. 1900 pcy	Crushed Gravel, 1-1/4" max. 1893 pcy	Crushed Gravel, 1-1/4" max. 1900 pcy
Fine Aggregate, (Natural River Sand, FM=2.45)	Type Quantity	Nat. River Sand FM=2.70 1316 pcy	Nat. River Sand FM=2.70 1316 pcy	Nat. River Sand FM=2.70 1424 pcy	Nat. River Sand FM=2.70 1067 pcy	Nat. River Sand FM=2.70 960 pcy
Water, (City of Austin)	Quantity	272 pcy	268 pcy	242 pcy	302 pcy	270 pcy
Cement, Quantity	Type	Type LS-II 427.1 pcy	Type LS-II 611 pcy	Type LS-II 492.8 pcy	Type LS-II 530 pcy	Type LS-II 563 pcy
Fly Ash, (ASTM Class C) Percent Replacement by Weight	Quantity	186.4 30 %	None	210 30 %	231.6 30 %	243.3 30 %
Retarder, (ASTM Type B)	Quantity	24.7 oz/cy	24.7 oz/cy	35 oz/cy	38 oz/cy	40 oz/cy
HRWR, (ASTM Type F)	Quantity	None	None	NR	NR	NR
Air Entrainment, (ASTM C260)	Quantity	6.57 oz/cy	3.3 oz/cy	2.7 oz/cy	3.33 oz/cy	3.67 oz/cy
Fresh Concrete Properties						
W/Cm (C+ FA), by weight		0.45	0.44	0.34	0.40	0.34
Total Air		3.9 %	5.3 %	5.4 %	4.8 %	7.0 %
Unit Weight		150.8 pcf	145.3 pcf	NR	NR	NR
Slump, before HRWR		4"	3.25"	1"	1"	1"
Slump, after HRWR		-	-	5.5"	4"	5.5"
Concrete Temperature, Deg. F		76	70	75	74	71
Ambient Temperature, Deg. F		64	62	54	62	60

Table 6.14: Mix Properties for Precast/Prestressed Concrete Beams

	Louetta NB/SB HS/HPC Beams	San Angelo EB HS/HPC Beams	San Angelo WB HPC Beams
Mix Proportions			
Coarse Aggregate, Type Quantity	Crushed Dolomitic Limestone, 1/2" max. ASTM Grade 7 1918 pcy	Crushed Dolomitic Limestone, 1/2" max. ASTM Grade 7 1863 pcy	Crushed River Gravel, 3/4" max. ASTM Grade 6 1997 pcy
Fine Aggregate, Type Quantity	Natural River Sand FM = 2.60 1029 pcy	Natural River Sand FM = 2.60 1062 pcy	Natural River Sand FM = 2.60 1160 pcy
Water, Type Quantity	City of Victoria Potable Water 247 pcy	City of Victoria Potable Water 246 pcy	City of Victoria Potable Water 196 pcy
Cement, Type Quantity	Type C-III 671 pcy	Type C-III 671 pcy	Type C-III 526 pcy
Fly Ash, Type Quantity % Replacement by Weight	ASTM Class C 316 pcy 32%	ASTM Class C 312 pcy 32%	ASTM Class C 196 pcy 27%
Retarder, Type Quantity	ASTM Type B 27 oz/cy	ASTM Type B 28 oz/cy	ASTM Type B 15.7 oz/cy
HRWR, Type Quantity	ASTM Type F 200 oz/cy	ASTM Type F 200 oz/cy	ASTM Type F 159 oz/cy
Air Entrainment, Type Quantity	ASTM C260 None	ASTM C260 None	ASTM C260 None
Fresh Concrete Properties			
W/Cm (C+FA), by weight	0.25	0.25	0.27
Slump	8 to 10 inches	7 to 9 inches	7 to 8 inches
Total Air ⊗	0.9%	0.9%	0.9%
Unit Weight	154.0 pcf	152.9 pcf	149.3 pcf
Required Release and Design Compressive Strengths			
Release Strength	6,900 - 8,800 psi	8,900 - 10,800 psi	4,000 - 6,600 psi
Design Strength ⊕	9,800 - 13,100 psi	10,900 - 14,700 psi	5,000 - 8,900 psi
⊗ No entrained air ⊕ 56-day strengths for HS/HPC mixes, 28-day strengths for HPC mixes $1 \text{ lb/yd}^3 = 27 \text{ lb/ft}^3 = 0.5933 \text{ kg/m}^3$ $1 \text{ ksi} = 1,000 \text{ psi} = 6.895 \text{ MPa}$ $1 \text{ oz/yd}^3 = 0.03868 \text{ L/m}^3$ $1 \text{ inch} = 25.4 \text{ mm}$			

Table 6.15: Mix Properties for Precast/Prestressed Concrete Deck Panels

	Louetta NB/SB HS/HPC Deck Panels	San Angelo EB HPC Deck Panels	San Angelo WB NSC Deck Panels
Mix Proportions			
Coarse Aggregate, Type	Crushed River Gravel, 3/4" max. ASTM Grade 6	Crushed Limestone, 1" max. ASTM Grade 5	Crushed Limestone, 1" max. ASTM Grade 5
Quantity	1983 pcy	1844 pcy	1889 pcy
Fine Aggregate, Type	Natural River Sand FM = 2.60	Natural River Sand FM = 2.63	Natural River Sand FM = 2.63
Quantity	1110 pcy	1355 pcy	1457 pcy
Water, Type	City of Houston Potable Water	Bexar County Well Water	Bexar County Well Water
Quantity	227 pcy	250.8 pcy	275 pcy
Cement, Type	Type A-III	Type C-III	Type C-III
Quantity	564 pcy	657.7 pcy	563.8 pcy
Fly Ash, Type	ASTM Class C	None	None
Quantity	163 pcy		
% Replacement by Weight	22%		
Retarder, Type	ASTM Type B	ASTM Type D	ASTM Type D
Quantity	23.2 oz/cy	79 oz/cy	49 oz/cy
HRWR, Type	ASTM Type F	ASTM Type D	ASTM Type D
Quantity	170.1 oz/cy	300 oz/cy	257 oz/cy
Air Entrainment, Type	ASTM C260	ASTM C260	ASTM C260
Quantity	None	None	None
Fresh Concrete Properties			
W/Cm (C+FA), by weight	0.31	0.38	0.49
Slump	9 to 10 inches	5 to 6 inches	6 to 7 inches
Total Air ⊗	2.0%	1.5%	1.5%
Unit Weight	149.9 pcf	150.9 pcf	150.7 pcf
Required Release and Design Compressive Strengths			
Release Strength	6,000 psi	4,000 psi	4,000 psi
Design Strength ⊕	8,000 psi	6,000 psi	5,000 psi
⊗ No entrained air ⊕ 28-day strengths for HS/HPC, HPC, and NSC mixes ∅ Mid-range water reducer (WR) used for San Angelo Panels $1 \text{ lb/yd}^3 = 27 \text{ lb/ft}^3 = 0.5933 \text{ kg/m}^3$ $1 \text{ ksi} = 1,000 \text{ psi} = 6.895 \text{ MPa}$ $1 \text{ oz/yd}^3 = 0.03868 \text{ L/m}^3$ $1 \text{ inch} = 25.4 \text{ mm}$			

Table 6.16: Mix Properties for Louetta CIP Decks and Precast Columns

	Louetta SB HS/HPC CIP Deck	Louetta NB HPC CIP Deck	Louetta NB/SB HS/HPC Column Elements
Casting Date	11-8-96	10-31-96	Multiple
Mix Proportions			
Coarse Aggregate, Type	Crushed Limestone, 1" max.	Crushed Limestone, 1-1/2" max.	Crushed River Gravel, 3/4" max.
Quantity	ASTM Grade 5 1811 pcy	ASTM Grade 4 1856 pcy	ASTM Grade 6 1983 pcy
Fine Aggregate, Type	Natural River Sand	Natural River Sand	Natural River Sand
Quantity	FM = 2.54 1304 pcy	FM = 2.54 1243 pcy	FM = 2.60 1110 pcy
Water, Type	City of Victoria Potable Water	City of Victoria Potable Water	City of Houston Potable Water
Quantity	245 pcy	230 pcy	227 pcy
Cement, Type	Type C-I	Type C-I	Type A-III
Quantity	673 pcy	383 pcy	564 pcy
Fly Ash, Type	ASTM Class C	ASTM Class C	ASTM Class C
Quantity	221 pcy	148 pcy	163 pcy
% Replacement by Weight	32%	28%	22%
Retarder, Type	ASTM Type B	ASTM Type B	ASTM Type B
Quantity	22 oz/cy	45 oz/cy	23.2 oz/cy
HRWR, Type	ASTM Type F	ASTM Type F	ASTM Type F
Quantity	122 oz/cy	None	170.1 oz/cy
Air Entrainment, Type	ASTM C260	ASTM C260	ASTM C260
Quantity	None	2 oz/cy	None
Fresh Concrete Properties			
W/Cm (C+FA), by weight	0.27	0.43	0.31
Slump	7 to 9 inches	3 to 4 inches	9 to 10 inches
Total Air ⊗	1.4 %	5.0 %	2.0 %
Unit Weight	150.2 pcf	143.2 pcf	149.9 pcf
Required Release and Design Compressive Strengths			
Release Strength	NA	NA	6,000 psi
Design Strength ⊕	8,000 psi	4,000 psi	10,000 psi
⊗ No entrained air ⊕ 28-day strengths for HS/HPC and HPC mixes 1 lb/yd ³ = 27 lb/ft ³ = 0.5933 kg/m ³ 1 ksi = 1,000 psi = 6.895 MPa 1 oz/yd ³ = 0.03868 L/m ³ 1 inch = 25.4 mm			

Table 6.17: Mix Properties for N.C.R.O. CIP Class C NSC Piers

	San Angelo Class C - NSC Pier	San Angelo Class C - NSC Pier	San Angelo Class C - NSC Pier
Mix Designation	CP 4-23	CP 4-25	CP 5-23
Casting Date	4-23-96	4-25-96	5-23-96
Mix Proportions			
Coarse Aggregate, Type	Crushed River Gravel, 1-1/4" max.	Crushed River Gravel, 1-1/4" max.	Crushed River Gravel, 1-1/4" max.
Quantity	ASTM Grade 5 2055 pcy	ASTM Grade 5 2069 pcy	ASTM Grade 5 2063 pcy
Fine Aggregate, Type	Natural River Sand FM = 2.70	Natural River Sand FM = 2.70	Natural River Sand FM = 2.70
Quantity	1091 pcy	1082 pcy	1190 pcy
Water, Type	City of San Angelo Potable Water	City of San Angelo Potable Water	City of San Angelo Potable Water
Quantity	253.5 pcy	256.6 pcy	221.3 pcy
Cement, Type	Type LS-II	Type LS-II	Type LS-II
Quantity	564.2 pcy	564.2 pcy	567.9 pcy
Fly Ash, Type	None	None	None
Quantity			
% Replacement by Weight			
Retarder, Type	ASTM Type B	ASTM Type B	ASTM Type B
Quantity	24 oz/cy	24 oz/cy	24 oz/cy
HRWR, Type	None	None	ASTM Type F
Quantity			45 oz/cy
Air Entrainment, Type	ASTM C260	ASTM C260	ASTM C260
Quantity	4.42 oz/cy	6.00 oz/cy	6.00 oz/cy
Fresh Concrete Properties			
W/Cm (C+FA), by weight	0.45	0.45	0.39
Slump	3.75 inches	3.75 inches	9 inches
Total Air	5.5%	5.8%	7.0%
Unit Weight	144.6 pcf	146.8 pcf	142.6 pcf
Concrete Temperature, Deg. F	78	80	94
Ambient Temperature, Deg. F	68	72	94
Required Release and Design Compressive Strengths			
Design Strength $\oplus$	3,600 psi	3,600 psi	3,600 psi
$\oplus$ 28-day strengths for NSC mixes $1 \text{ lb/yd}^3 = 27 \text{ lb/ft}^3 = 0.5933 \text{ kg/m}^3$ $1 \text{ ksi} = 1,000 \text{ psi} = 6.895 \text{ MPa}$ $\text{deg. F} = (9/5)(\text{deg. C}) + 32$ $1 \text{ oz/yd}^3 = 0.03868 \text{ L/m}^3$ $1 \text{ inch} = 25.4 \text{ mm}$			

Table 6.18: Mix Properties for N.C.R.O. CIP Class C NSC Piers (continued)

	San Angelo Class C - NSC Pier	San Angelo Class C - NSC Pier	San Angelo Class C - NSC Pier
Mix Designation	CP 6-5	CP 6-21	CP 6-24
Casting Date	6-5-96	6-21-96	6-24-96
Mix Proportions			
Coarse Aggregate, Type Quantity	Crushed River Gravel, 1-1/4" max. ASTM Grade 5 2061 pcy	Crushed River Gravel, 1-1/4" max. ASTM Grade 5 2043 pcy	Crushed River Gravel, 1-1/4" max. ASTM Grade 5 2054 pcy
Fine Aggregate, Type Quantity	Natural River Sand FM = 2.70 1211 pcy	Natural River Sand FM = 2.70 1170 pcy	Natural River Sand FM = 2.70 1211 pcy
Water, Type Quantity	City of San Angelo Potable Water 229.9 pcy	City of San Angelo Potable Water 205.3 pcy	City of San Angelo Potable Water 214.8 pcy
Cement, Type Quantity	Type LS-II 568.7 pcy	Type LS-II 562.5 pcy	Type LS-II 561.4 pcy
Fly Ash, Type Quantity % Replacement by Weight	None	None	None
Retarder, Type Quantity	ASTM Type B 24 oz/cy	ASTM Type B 24 oz/cy	ASTM Type B 24 oz/cy
HRWR, Type Quantity	ASTM Type F 45 oz/cy	ASTM Type F 45 oz/cy	ASTM Type F 45 oz/cy
Air Entrainment, Type Quantity	ASTM C260 2.93 oz/cy	ASTM C260 4.50 oz/cy	ASTM C260 2.85 oz/cy
Fresh Concrete Properties			
W/Cm (C+FA), by weight	0.40	0.36	0.38
Slump	7 inches	8 inches	7 inches
Total Air	5.5%	7.2%	7.6%
Unit Weight	146.2 pcf	145.6 pcf	144.8 pcf
Concrete Temperature, deg. F	92	102	99
Ambient Temperature, deg. F	93	94	94
Required Release and Design Compressive Strengths			
Design Strength $\oplus$	3,600 psi	3,600 psi	3,600 psi
$\oplus$ 28-day strengths for NSC mixes $1 \text{ lb/yd}^3 = 27 \text{ lb/ft}^3 = 0.5933 \text{ kg/m}^3$ $1 \text{ ksi} = 1,000 \text{ psi} = 6.895 \text{ MPa}$ $\text{deg. F} = (9/5)(\text{deg. C}) + 32$ $1 \text{ oz/yd}^3 = 0.03868 \text{ L/m}^3$ $1 \text{ inch} = 25.4 \text{ mm}$			

Table 6.19: Mix Properties for N.C.R.O. CIP Class C NSC Piers (continued)

	San Angelo Class C - NSC Pier	San Angelo Class C - NSC Pier	
Mix Designation	CP 8-13	CP 6-8-23	
Casting Date	6-5-96	6-21-96	
Mix Proportions			
Coarse Aggregate, Type Quantity	Crushed River Gravel, 1-1/4" max. ASTM Grade 5 2065 pcy	Crushed River Gravel, 1-1/4" max. ASTM Grade 5 2063 pcy	
Fine Aggregate, Type Quantity	Natural River Sand FM = 2.70 1167 pcy	Natural River Sand FM = 2.70 1167 pcy	
Water, Type Quantity	City of San Angelo Potable Water 257.4 pcy	City of San Angelo Potable Water 219.6 pcy	
Cement, Type Quantity	Type LS-II 562.7 pcy	Type LS-II 560.8 pcy	
Fly Ash, Type Quantity % Replacement by Weight	None	None	
Retarder, Type Quantity	ASTM Type B 24 oz/cy	ASTM Type B 24 oz/cy	
HRWR, Type Quantity	ASTM Type F 45 oz/cy	ASTM Type F 45 oz/cy	
Air Entrainment, Type Quantity	ASTM C260 2.73 oz/cy	ASTM C260 3.00 oz/cy	
Fresh Concrete Properties			
W/Cm (C+FA), by weight	0.46	0.39	
Slump	6 inches	6 inches	
Total Air	7.2%	6.4%	
Unit Weight	145.4 pcf	144.6 pcf	
Concrete Temperature, deg. F	85	84	
Ambient Temperature, deg. F	66	74	
Required Release and Design Compressive Strengths			
Design Strength $\oplus$	3,600 psi	3,600 psi	
$\oplus$ 28-day strengths for NSC mixes $1 \text{ lb/yd}^3 = 27 \text{ lb/ft}^3 = 0.5933 \text{ kg/m}^3$ $1 \text{ ksi} = 1,000 \text{ psi} = 6.895 \text{ MPa}$ $\text{deg. F} = (9/5)(\text{deg. C}) + 32$ $1 \text{ oz/yd}^3 = 0.03868 \text{ L/m}^3$ $1 \text{ inch} = 25.4 \text{ mm}$			

Table 6.20: Mix Properties for N.C.R.O. CIP Class F HPC Pier Caps

	San Angelo Class F - NSC Pier Cap	San Angelo Class F - NSC Pier Cap	San Angelo Class F - NSC Pier Cap
Mix Designation	FC 7-31	FC 8-9	FC 9-5
Casting Date	7-31-96	8-9-96	9-5-96
Mix Proportions			
Coarse Aggregate, Type Quantity	Crushed River Gravel, 1-1/4" max. ASTM Grade 5 1988 pcy	Crushed River Gravel, 1-1/4" max. ASTM Grade 5 1998 pcy	Crushed River Gravel, 1-1/4" max. ASTM Grade 5 1993 pcy
Fine Aggregate, Type Quantity	Natural River Sand FM = 2.70 884.4 pcy	Natural River Sand FM = 2.70 884.4 pcy	Natural River Sand FM = 2.70 984.4 pcy
Water, Type Quantity	City of San Angelo Potable Water 267.8 pcy	City of San Angelo Potable Water 288.7 pcy	City of San Angelo Potable Water 243.5 pcy
Cement, Type Quantity	Type LS-II 530.6 pcy	Type LS-II 530.6 pcy	Type LS-II 526.1 pcy
Fly Ash, Type Quantity % Replacement by Weight	ASTM Class C 227 pcy 30%	ASTM Class C 227 pcy 30%	ASTM Class C 227 pcy 30%
Retarder, Type Quantity	ASTM Type B 30 oz/cy	ASTM Type B 30 oz/cy	ASTM Type B 20 oz/cy
HRWR, Type Quantity	None	None	ASTM Type F 114 oz/cy
Air Entrainment, Type Quantity	ASTM C260 8.0 oz/cy	ASTM C260 9.0 oz/cy	ASTM C260 6.0 oz/cy
Fresh Concrete Properties			
W/Cm (C+FA), by weight	0.35	0.38	0.32
Slump	3.5 inches	3.5 inches	6.25 inches
Total Air	4.6%	5.0%	7.0%
Unit Weight	148.7 pcf	148.7 pcf	143.3 pcf
Concrete Temperature, deg. F	89	90	89
Ambient Temperature, deg. F	77	72	75
Required Release and Design Compressive Strengths			
Design Strength $\oplus$	6,000 psi	6,000 psi	6,000 psi
$\oplus$ 28-day strengths for NSC mixes $1 \text{ lb/yd}^3 = 27 \text{ lb/ft}^3 = 0.5933 \text{ kg/m}^3$ $1 \text{ ksi} = 1,000 \text{ psi} = 6.895 \text{ MPa}$ $\text{deg. F} = (9/5)(\text{deg. C}) + 32$ $1 \text{ oz/yd}^3 = 0.03868 \text{ L/m}^3$ $1 \text{ inch} = 25.4 \text{ mm}$			

Table 6.21: Mix Properties for N.C.R.O. CIP Class F HPC Pier Caps (continued)

	San Angelo Class F - NSC Pier Cap	San Angelo Class F - NSC Pier Cap	San Angelo Class F - NSC Pier Cap
Mix Designation	FC 9-25	FC 10-17	FC 10-30
Casting Date	9-25-96	10-17-96	10-30-96
Mix Proportions			
Coarse Aggregate, Type Quantity	Crushed River Gravel, 1-1/4" max. ASTM Grade 5 2004 pcy	Crushed River Gravel, 1-1/4" max. ASTM Grade 5 2000 pcy	Crushed River Gravel, 1-1/4" max. ASTM Grade 5 1991 pcy
Fine Aggregate, Type Quantity	Natural River Sand FM = 2.70 973.3 pcy	Natural River Sand FM = 2.70 980 pcy	Natural River Sand FM = 2.70 986.7 pcy
Water, Type Quantity	City of San Angelo Potable Water 245.8 pcy	City of San Angelo Potable Water 253.9 pcy	City of San Angelo Potable Water 243.5 pcy
Cement, Type Quantity	Type LS-II 523.3 pcy	Type LS-II 525.6 pcy	Type LS-II 529.8 pcy
Fly Ash, Type Quantity % Replacement by Weight	ASTM Class C 227 pcy 30%	ASTM Class C 226 pcy 30%	ASTM Class C 227 pcy 30%
Retarder, Type Quantity	ASTM Type B 20 oz/cy	ASTM Type B 20 oz/cy	ASTM Type B 20 oz/cy
HRWR, Type Quantity	ASTM Type F 83 oz/cy	ASTM Type F NR	ASTM Type F NR
Air Entrainment, Type Quantity	ASTM C260 8.0 oz/cy	ASTM C260 12.0 oz/cy	ASTM C260 6.0 oz/cy
Fresh Concrete Properties			
W/Cm (C+FA), by weight	0.33	0.34	0.32
Slump	8.5 inches	8.5 inches	7.5 inches
Total Air	5.6%	5.8%	6.4%
Unit Weight	142.8 pcf	144.1 pcf	144.1 pcf
Concrete Temperature, deg. F	90	81	70
Ambient Temperature, deg. F	60	70	48
Required Release and Design Compressive Strengths			
Design Strength $\oplus$	6,000 psi	6,000 psi	6,000 psi
$\oplus$ 28-day strengths for NSC mixes 1 lb/yd ³ = 27 lb/ft ³ = 0.5933 kg/m ³ 1 ksi = 1,000 psi = 6.895 MPa deg. F = (9/5)(deg. C) + 32 1 oz/yd ³ = 0.03868 L/m ³ 1 inch = 25.4 mm			

Table 6.22: Mix Properties for N.C.R.O. CIP Class F HPC Pier Caps (continued)

	San Angelo Class F - NSC Pier Cap		
Mix Designation	FC 12-9		
Casting Date	12-9-96		
Mix Proportions			
Coarse Aggregate, Type Quantity	Crushed River Gravel, 1-1/4" max. ASTM Grade 5 2013 pcy		
Fine Aggregate, Type Quantity	Natural River Sand FM = 2.70 995.6 pcy		
Water, Type Quantity	City of San Angelo Potable Water 229.3 pcy		
Cement, Type Quantity	Type LS-II 531.7 pcy		
Fly Ash, Type Quantity % Replacement by Weight	ASTM Class C 226 pcy 30%		
Retarder, Type Quantity	ASTM Type B 20 oz/cy		
HRWR, Type Quantity	ASTM Type F 83 oz/cy		
Air Entrainment, Type Quantity	ASTM C260 6.0 oz/cy		
Fresh Concrete Properties			
W/Cm (C+FA), by weight	0.30		
Slump	6.5 inches		
Total Air	6.2%		
Unit Weight	144.5 pcf		
Concrete Temperature, deg. F	65		
Ambient Temperature, deg. F	59		
Required Release and Design Compressive Strengths			
Design Strength $\oplus$	6,000 psi		
$\oplus$ 28-day strengths for NSC mixes $1 \text{ lb/yd}^3 = 27 \text{ lb/ft}^3 = 0.5933 \text{ kg/m}^3$ $1 \text{ ksi} = 1,000 \text{ psi} = 6.895 \text{ MPa}$ $\text{deg. F} = (9/5)(\text{deg. C}) + 32$ $1 \text{ oz/yd}^3 = 0.03868 \text{ L/m}^3$ $1 \text{ inch} = 25.4 \text{ mm}$			

Table 6.23: Mix Properties for N.C.R.O. CIP Class K HS/HPC Piers

	San Angelo Class K - HPC Pier	San Angelo Class K - HPC Pier	San Angelo Class K - HPC Pier
Mix Designation	KP 4-29	KP 5-2	KP 5-17
Casting Date	4-29-96	5-2-96	5-17-96
Mix Proportions			
Coarse Aggregate, Type	Crushed River Gravel, 1-1/4" max. ASTM Grade 5 2000 pcy	Crushed River Gravel, 1-1/4" max. ASTM Grade 5 1895 pcy	Crushed River Gravel, 1-1/4" max. ASTM Grade 5 1903 pcy
Fine Aggregate, Type	Natural River Sand FM = 2.70 1308 pcy	Natural River Sand FM = 2.70 1437 pcy	Natural River Sand FM = 2.70 1447 pcy
Water, Type	City of San Angelo Potable Water 253.6 pcy	City of San Angelo Potable Water 240.8 pcy	City of San Angelo Potable Water 237.8 pcy
Cement, Type	Type LS-II 492 pcy	Type LS-II 491 pcy	Type LS-II 490.3 pcy
Fly Ash, Type	ASTM Class C 212 pcy 30%	ASTM Class C 214 pcy 30%	ASTM Class C 215.2 pcy 30%
Retarder, Type	ASTM Type B 23 oz/cy	ASTM Type B 23 oz/cy	ASTM Type B 23 oz/cy
HRWR, Type	ASTM Type F 98 oz/cy	ASTM Type F 98 oz/cy	ASTM Type F 98 oz/cy
Air Entrainment, Type	None	None	None
Fresh Concrete Properties			
W/Cm (C+FA), by weight	0.36	0.34	0.34
Slump	8 inches	9 inches	9 inches
Total Air ⊗	NR	NR	NR
Unit Weight	155.6 pcf	155.2 pcf	154.1 pcf
Concrete Temperature, deg. F	70	94	86
Ambient Temperature, deg. F	63	90	72
Required Release and Design Compressive Strengths			
Design Strength ⊕	6,000 psi	6,000 psi	6,000 psi
⊗ No entrained air for HS/HPC mix ⊕ 28-day strengths for NSC mixes $1 \text{ lb/yd}^3 = 27 \text{ lb/ft}^3 = 0.5933 \text{ kg/m}^3$ $1 \text{ ksi} = 1,000 \text{ psi} = 6.895 \text{ MPa}$ $\text{deg. F} = (9/5)(\text{deg. C}) + 32$ $1 \text{ oz/yd}^3 = 0.03868 \text{ L/m}^3$ $1 \text{ inch} = 25.4 \text{ mm}$			

Table 6.24: Mix Properties for N.C.R.O. CIP Class K HS/HPC Piers (continued)

	San Angelo Class K - HPC Pier	San Angelo Class K - HPC Pier	San Angelo Class K - HPC Pier
Mix Designation	KP 5-31	KP 6-17	KP 6-20
Casting Date	5-31-96	6-17-96	6-20-96
Mix Proportions			
Coarse Aggregate, Type Quantity	Crushed River Gravel, 1-1/4" max. ASTM Grade 5 1897 pcy	Crushed River Gravel, 1-1/4" max. ASTM Grade 5 1884 pcy	Crushed River Gravel, 1-1/4" max. ASTM Grade 5 1893 pcy
Fine Aggregate, Type Quantity	Natural River Sand FM = 2.70 1431 pcy	Natural River Sand FM = 2.70 1415 pcy	Natural River Sand FM = 2.70 1432 pcy
Water, Type Quantity	City of San Angelo Potable Water 228.7 pcy	City of San Angelo Potable Water 253.7 pcy	City of San Angelo Potable Water 234.2 pcy
Cement, Type Quantity	Type LS-II 489 pcy	Type LS-II 491 pcy	Type LS-II 487 pcy
Fly Ash, Type Quantity % Replacement by Weight	ASTM Class C 211 pcy 30%	ASTM Class C 214 pcy 30%	ASTM Class C 212 pcy 30%
Retarder, Type Quantity	ASTM Type B 23 oz/cy	ASTM Type B 23 oz/cy	ASTM Type B 23 oz/cy
HRWR, Type Quantity	ASTM Type F 98 oz/cy	ASTM Type F 100 oz/cy	ASTM Type F 98 oz/cy
Air Entrainment, Type Quantity	None	None	None
Fresh Concrete Properties			
W/Cm (C+FA), by weight	0.33	0.36	0.34
Slump	8.75 inches	9 inches	10 inches
Total Air ⊗	NR	NR	NR
Unit Weight	154.8 pcf	152.6 pcf	157.3 pcf
Concrete Temperature, deg. F	90	95	100
Ambient Temperature, deg. F	79	83	86
Required Release and Design Compressive Strengths			
Design Strength ⊕	6,000 psi	6,000 psi	6,000 psi
⊗ No entrained air for HS/HPC mix ⊕ 28-day strengths for NSC mixes $1 \text{ lb/yd}^3 = 27 \text{ lb/ft}^3 = 0.5933 \text{ kg/m}^3$ $1 \text{ ksi} = 1,000 \text{ psi} = 6.895 \text{ MPa}$ $\text{deg. F} = (9/5)(\text{deg. C}) + 32$ $1 \text{ oz/yd}^3 = 0.03868 \text{ L/m}^3$ $1 \text{ inch} = 25.4 \text{ mm}$			

Table 6.25: Mix Properties for N.C.R.O. CIP Class K HS/HPC Piers (continued)

	San Angelo Class K - HPC Pier	San Angelo Class K - HPC Pier	San Angelo Class K - HPC Pier
Mix Designation	KP 6-28	KP 7-8	KP 7-16
Casting Date	6-28-96	7-8-96	7-16-96
Mix Proportions			
Coarse Aggregate, Type Quantity	Crushed River Gravel, 1-1/4" max. ASTM Grade 5 1900 pcy	Crushed River Gravel, 1-1/4" max. ASTM Grade 5 1891 pcy	Crushed River Gravel, 1-1/4" max. ASTM Grade 5 1902 pcy
Fine Aggregate, Type Quantity	Natural River Sand FM = 2.70 1403 pcy	Natural River Sand FM = 2.70 1415 pcy	Natural River Sand FM = 2.70 1440 pcy
Water, Type Quantity	City of San Angelo Potable Water 229.5 pcy	City of San Angelo Potable Water 246.8 pcy	City of San Angelo Potable Water 262 pcy
Cement, Type Quantity	Type LS-II 493 pcy	Type LS-II 488 pcy	Type LS-II 490 pcy
Fly Ash, Type Quantity % Replacement by Weight	ASTM Class C 209 pcy 30%	ASTM Class C 211 pcy 30%	ASTM Class C 212 pcy 30%
Retarder, Type Quantity	ASTM Type B 23 oz/cy	ASTM Type B 23 oz/cy	ASTM Type B 23 oz/cy
HRWR, Type Quantity	ASTM Type F 98 oz/cy	ASTM Type F 98 oz/cy	ASTM Type F 97 oz/cy
Air Entrainment, Type Quantity	None	None	None
Fresh Concrete Properties			
W/Cm (C+FA), by weight	0.33	0.35	0.37
Slump	9.5 inches	10.25 inches	9 inches
Total Air ⊗	NR	NR	NR
Unit Weight	156.4 pcf	154.8 pcf	155.6 pcf
Concrete Temperature, deg. F	99	100	87
Ambient Temperature, deg. F	75	96	88
Required Release and Design Compressive Strengths			
Design Strength ⊕	6,000 psi	6,000 psi	6,000 psi
⊗ No entrained air for HS/HPC mix ⊕ 28-day strengths for NSC mixes $1 \text{ lb/yd}^3 = 27 \text{ lb/ft}^3 = 0.5933 \text{ kg/m}^3$ $1 \text{ ksi} = 1,000 \text{ psi} = 6.895 \text{ MPa}$ $\text{deg. F} = (9/5)(\text{deg. C}) + 32$ $1 \text{ oz/yd}^3 = 0.03868 \text{ L/m}^3$ $1 \text{ inch} = 25.4 \text{ mm}$			

Table 6.26: Mix Properties for N.C.R.O. CIP Class K HS/HPC Pier Caps

	San Angelo Class K - HPC Pier Cap	San Angelo Class K - HPC Pier Cap	San Angelo Class K - HPC Pier Cap
Mix Designation	KC 7-18	KC 7-25	KC 8-22
Casting Date	7-18-96	7-25-96	8-22-96
Mix Proportions			
Coarse Aggregate, Type	Crushed River Gravel, 1-1/4" max.	Crushed River Gravel, 1-1/4" max.	Crushed River Gravel, 1-1/4" max.
Quantity	ASTM Grade 5 1898 pcy	ASTM Grade 5 1898 pcy	ASTM Grade 5 1900 pcy
Fine Aggregate, Type	Natural River Sand FM = 2.70	Natural River Sand FM = 2.70	Natural River Sand FM = 2.70
Quantity	1427 pcy	1462 pcy	1427 pcy
Water, Type	City of San Angelo Potable Water	City of San Angelo Potable Water	City of San Angelo Potable Water
Quantity	250.1 pcy	246.6 pcy	243.1 pcy
Cement, Type	Type LS-II	Type LS-II	Type LS-II
Quantity	490 pcy	490 pcy	491 pcy
Fly Ash, Type	ASTM Class C	ASTM Class C	ASTM Class C
Quantity	209 pcy	216 pcy	209 pcy
% Replacement by Weight	30%	30%	30%
Retarder, Type	ASTM Type B	ASTM Type B	ASTM Type B
Quantity	23 oz/cy	21 oz/cy	21 oz/cy
HRWR, Type	ASTM Type F	ASTM Type F	ASTM Type F
Quantity	98 oz/cy	97 oz/cy	98 oz/cy
Air Entrainment, Type			
Quantity	None	None	None
Fresh Concrete Properties			
W/Cm (C+FA), by weight	0.36	0.35	0.35
Slump	9.5 inches	9.75 inches	9.5 inches
Total Air ⊗	NR	NR	NR
Unit Weight	153.6 pcf	154.2 pcf	154.4 pcf
Concrete Temperature, deg. F	90	93	90
Ambient Temperature, deg. F	82	94	83
Required Release and Design Compressive Strengths			
Design Strength ⊕	8,000 psi	8,000 psi	8,000 psi
⊗ No entrained air for HS/HPC mix ⊕ 28-day strengths for NSC mixes $1 \text{ lb/yd}^3 = 27 \text{ lb/ft}^3 = 0.5933 \text{ kg/m}^3$ $1 \text{ ksi} = 1,000 \text{ psi} = 6.895 \text{ MPa}$ $\text{deg. F} = (9/5)(\text{deg. C}) + 32$ $1 \text{ oz/yd}^3 = 0.03868 \text{ L/m}^3$ $1 \text{ inch} = 25.4 \text{ mm}$			

Table 6.27: Mix Properties for N.C.R.O. CIP Class K HS/HPC Pier Caps (continued)

	San Angelo Class K - HPC Pier Cap	San Angelo Class K - HPC Pier Cap	San Angelo Class K - HPC Pier Cap
Mix Designation	KC 9-12	KC 10-2	KC 10-9
Casting Date	9-12-96	10-2-96	10-9-96
Mix Proportions			
Coarse Aggregate, Type Quantity	Crushed River Gravel, 1-1/4" max. ASTM Grade 5 1900 pcy	Crushed River Gravel, 1-1/4" max. ASTM Grade 5 1900 pcy	Crushed River Gravel, 1-1/4" max. ASTM Grade 5 1911 pcy
Fine Aggregate, Type Quantity	Natural River Sand FM = 2.70 1422 pcy	Natural River Sand FM = 2.70 1449 pcy	Natural River Sand FM = 2.70 1418 pcy
Water, Type Quantity	City of San Angelo Potable Water 263 pcy	City of San Angelo Potable Water 245 pcy	City of San Angelo Potable Water 262 pcy
Cement, Type Quantity	Type LS-II 489 pcy	Type LS-II 488 pcy	Type LS-II 491 pcy
Fly Ash, Type Quantity % Replacement by Weight	ASTM Class C 210 pcy 30%	ASTM Class C 210 pcy 30%	ASTM Class C 210 pcy 30%
Retarder, Type Quantity	ASTM Type B 21 oz/cy	ASTM Type B 21 oz/cy	ASTM Type B 21 oz/cy
HRWR, Type Quantity	ASTM Type F 84 oz/cy	ASTM Type F NR	ASTM Type F 98 oz/cy
Air Entrainment, Type Quantity	None	None	None
Fresh Concrete Properties			
W/Cm (C+FA), by weight	0.38	0.35	0.37
Slump	8.5 inches	6 inches	8.5 inches
Total Air ⊗	NR	NR	NR
Unit Weight	152.8 pcf	153.9 pcf	152.8 pcf
Concrete Temperature, deg. F	84	80	84
Ambient Temperature, deg. F	68	75	81
Required Release and Design Compressive Strengths			
Design Strength ⊕	8,000 psi	8,000 psi	8,000 psi
⊗ No entrained air for HS/HPC mix ⊕ 28-day strengths for NSC mixes $1 \text{ lb/yd}^3 = 27 \text{ lb/ft}^3 = 0.5933 \text{ kg/m}^3$ $1 \text{ ksi} = 1,000 \text{ psi} = 6.895 \text{ MPa}$ $\text{deg. F} = (9/5)(\text{deg. C}) + 32$ $1 \text{ oz/yd}^3 = 0.03868 \text{ L/m}^3$ $1 \text{ inch} = 25.4 \text{ mm}$			

Table 6.28: Mix Properties for N.C.R.O. CIP Class K HPC Pier Caps (continued)

	San Angelo Class K - HPC Pier Cap		
Mix Designation	KC 11-20		
Casting Date	11-20-96		
Mix Proportions			
Coarse Aggregate, Type	Crushed River Gravel, 1-1/4" max. ASTM Grade 5		
Quantity	1907 pcy		
Fine Aggregate, Type	Natural River Sand FM = 2.70		
Quantity	1433 pcy		
Water, Type	City of San Angelo Potable Water		
Quantity	231.2 pcy		
Cement, Type	Type LS-II		
Quantity	491.1 pcy		
Fly Ash, Type	ASTM Class C		
Quantity	211 pcy		
% Replacement by Weight	30%		
Retarder, Type	ASTM Type B		
Quantity	21 oz/cy		
HRWR, Type	ASTM Type F		
Quantity	52 oz/cy		
Air Entrainment, Type			
Quantity	None		
Fresh Concrete Properties			
W/Cm (C+FA), by weight	0.33		
Slump	5.75 inches		
Total Air ⊗	NR		
Unit Weight	151.3 pcf		
Concrete Temperature, deg. F	72		
Ambient Temperature, deg. F	58		
Required Release and Design Compressive Strengths			
Design Strength ⊕	8,000 psi		
⊗ No entrained air for HS/HPC mix ⊕ 28-day strengths for NSC mixes $1 \text{ lb/yd}^3 = 27 \text{ lb/ft}^3 = 0.5933 \text{ kg/m}^3$ $1 \text{ ksi} = 1,000 \text{ psi} = 6.895 \text{ MPa}$ deg. F = (9/5)(deg. C) + 32 $1 \text{ oz/yd}^3 = 0.03868 \text{ L/m}^3$ $1 \text{ inch} = 25.4 \text{ mm}$			

Table 6.29: Mix Properties for N.C.R.O. CIP Deck -Westbound Bridge

	San Angelo Class S Mod - HPC Span 1 - WB	San Angelo Class S Mod - HPC Span 2 - WB	San Angelo Class S Mod - HPC Span 3 -WB
Mix Designation	S2 12-3	S2 12-10	S2 1-21
Casting Date	12-3-96	12-10-96	1-21-97
Mix Proportions			
Coarse Aggregate, Type	Crushed River Gravel, 1-1/4" max.	Crushed River Gravel, 1-1/4" max.	Crushed River Gravel, 1-1/4" max.
Quantity	ASTM Grade 5 1858 pcy	ASTM Grade 5 1863 pcy	ASTM Grade 5 1849 pcy
Fine Aggregate, Type	Natural River Sand FM = 2.70	Natural River Sand FM = 2.70	Natural River Sand FM = 2.70
Quantity	1229 pcy	1233 pcy	1222 pcy
Water, Type	City of San Angelo Potable Water	City of San Angelo Potable Water	City of San Angelo Potable Water
Quantity	252.1 pcy	261.8 pcy	260.5 pcy
Cement, Type	Type LS-II	Type LS-II	Type LS-II
Quantity	426.7 pcy	424.4 pcy	429 pcy
Fly Ash, Type	ASTM Class C	ASTM Class C	ASTM Class C
Quantity	191 pcy	184.4 pcy	184 pcy
% Replacement by Weight	30%	30%	30%
Retarder, Type	None	None	None
Quantity	None	None	None
HRWR, Type	None	None	None
Quantity	None	None	None
Air Entrainment, Type	ASTM C260	ASTM C260	ASTM C260
Quantity	5.0 oz/cy	4.4 oz/cy	4.4 oz/cy
Fresh Concrete Properties			
W/Cm (C+FA), by weight	0.41	0.43	0.43
Slump	3.75 inches	3 inches	4 inches
Total Air	5.0%	4.7%	6.2%
Unit Weight	145.3 pcf	145.5 pcf	154.1 pcf
Concrete Temperature, deg. F	59	66	58
Ambient Temperature, deg. F	43	58	50
Required Release and Design Compressive Strengths			
Design Strength $\oplus$	4,000 psi	4,000 psi	4,000 psi
$\oplus$ 28-day strengths for HPC mixes $1 \text{ lb/yd}^3 = 27 \text{ lb/ft}^3 = 0.5933 \text{ kg/m}^3$ $1 \text{ ksi} = 1,000 \text{ psi} = 6.895 \text{ MPa}$ $\text{deg. F} = (9/5)(\text{deg. C}) + 32$ $1 \text{ oz/yd}^3 = 0.03868 \text{ L/m}^3$ $1 \text{ inch} = 25.4 \text{ mm}$			

Table 6.30: Mix Properties for N.C.R.O. CIP Deck — Westbound Bridge

	San Angelo Class S Mod - HPC Span 4 - WB	San Angelo Class S Mod - HPC Span 5 - WB	San Angelo Class S - NSC Span 6 & 7 - WB
Mix Designation	S2 1-23	S2 1-29	S1 2-15
Casting Date	1-23-97	1-29-97	2-15-97
Mix Proportions			
Coarse Aggregate, Type	Crushed River Gravel, 1-1/4" max.	Crushed River Gravel, 1-1/4" max.	Crushed River Gravel, 1-1/4" max.
Quantity	ASTM Grade 5 1849 pcy	ASTM Grade 5 1856 pcy	ASTM Grade 5 1858 pcy
Fine Aggregate, Type	Natural River Sand FM = 2.70	Natural River Sand FM = 2.70	Natural River Sand FM = 2.70
Quantity	1236 pcy	1240 pcy	1307 pcy
Water, Type	City of San Angelo Potable Water	City of San Angelo Potable Water	City of San Angelo Potable Water
Quantity	260.8 pcy	271.2 pcy	272.6 pcy
Cement, Type	Type LS-II	Type LS-II	Type LS-II
Quantity	428 pcy	428 pcy	609.4 pcy
Fly Ash, Type	ASTM Class C	ASTM Class C	
Quantity	184 pcy	185.6 pcy	None
% Replacement by Weight	30%	30%	
Retarder, Type			
Quantity	None	None	None
HRWR, Type			
Quantity	None	None	None
Air Entrainment, Type	ASTM C260	ASTM C260	ASTM C260
Quantity	5.6 oz/cy	6.0 oz/cy	6.0 oz/cy
Fresh Concrete Properties			
W/Cm (C+FA), by weight	0.43	0.44	0.45
Slump	4 inches	3.5 inches	3 inches
Total Air	5.3%	5.8%	6.0%
Unit Weight	145.6 pcf	145.5 pcf	145.6 pcf
Concrete Temperature, deg. F	61	52	58
Ambient Temperature, deg. F	50	37	44
Required Release and Design Compressive Strengths			
Design Strength $\oplus$	4,000 psi	4,000 psi	4,000 psi
$\oplus$ 28-day strengths for HPC and NSC mixes $1 \text{ lb/yd}^3 = 27 \text{ lb/ft}^3 = 0.5933 \text{ kg/m}^3$ $1 \text{ ksi} = 1,000 \text{ psi} = 6.895 \text{ MPa}$ $\text{deg. F} = (9/5)(\text{deg. C}) + 32$ $1 \text{ oz/yd}^3 = 0.03868 \text{ L/m}^3$ $1 \text{ inch} = 25.4 \text{ mm}$			

Table 6.31: Mix Properties for N.C.R.O. CIP Deck — Westbound Bridge

	San Angelo Class S - NSC E Ramp - WB	San Angelo Class S - NSC Span 8 & 9 - WB	San Angelo Class S Mod- HPC I Ramp - WB
Mix Designation	S1 2-18	S1 2-28	S2 3-4
Casting Date	2-18-97	2-28-97	3-4-97
Mix Proportions			
Coarse Aggregate, Type Quantity	Crushed River Gravel, 1-1/4" max. ASTM Grade 5 1864 pcy	Crushed River Gravel, 1-1/4" max. ASTM Grade 5 1856 pcy	Crushed River Gravel, 1-1/4" max. ASTM Grade 5 1858 pcy
Fine Aggregate, Type Quantity	Natural River Sand FM = 2.70 1313 pcy	Natural River Sand FM = 2.70 1322 pcy	Natural River Sand FM = 2.70 1249 pcy
Water, Type Quantity	City of San Angelo Potable Water 276.2 pcy	City of San Angelo Potable Water 273.7 pcy	City of San Angelo Potable Water 263.3 pcy
Cement, Type Quantity	Type LS-II 611 pcy	Type LS-II 611 pcy	Type LS-II 425.6 pcy
Fly Ash, Type Quantity % Replacement by Weight	None	None	ASTM Class C 186.1 pcy 30%
Retarder, Type Quantity	None	None	None
HRWR, Type Quantity	None	None	None
Air Entrainment, Type Quantity	ASTM C260 4.7 oz/cy	ASTM C260 4.7 oz/cy	ASTM C260 6.0 oz/cy
Fresh Concrete Properties			
W/Cm (C+FA), by weight	0.45	0.45	0.45
Slump	3.5 inches	3.5 inches	4 inches
Total Air	5.6%	5.5%	5.3%
Unit Weight	145.3 pcf	145.3 pcf	146.3 pcf
Concrete Temperature, deg. F	63	62	66
Ambient Temperature, deg. F	52	50	56
Required Release and Design Compressive Strengths			
Design Strength $\oplus$	4,000 psi	4,000 psi	4,000 psi
$\oplus$ 28-day strengths for NSC and HPC mixes $1 \text{ lb/yd}^3 = 27 \text{ lb/ft}^3 = 0.5933 \text{ kg/m}^3$ $1 \text{ ksi} = 1,000 \text{ psi} = 6.895 \text{ MPa}$ $\text{deg. F} = (9/5)(\text{deg. C}) + 32$ $1 \text{ oz/yd}^3 = 0.03868 \text{ L/m}^3$ $1 \text{ inch} = 25.4 \text{ mm}$			

Table 6.32: Mix Properties for N.C.R.O. CIP Deck — Eastbound Bridge

	San Angelo Class K - HS/HPC Span 1 - EB	San Angelo Class K - HS/HPC Span 2 - EB	San Angelo Class K - HS/HPC Span 3 - EB
Mix Designation	SAD 6-12	SAD 6-25	SAD 7-9
Casting Date	6-12-97	6-25-97	7-9-97
Mix Proportions			
Coarse Aggregate, Type	Crushed River Gravel, 1-1/4" max.	Crushed River Gravel, 1-1/4" max.	Crushed River Gravel, 1-1/4" max.
Quantity	ASTM Grade 5 1978 pcy	ASTM Grade 5 1980 pcy	ASTM Grade 5 1898 pcy
Fine Aggregate, Type	Natural River Sand FM = 2.70	Natural River Sand FM = 2.70	Natural River Sand FM = 2.70
Quantity	1127 pcy	1122 pcy	1220 pcy
Water, Type	City of San Angelo Potable Water	City of San Angelo Potable Water	City of San Angelo Potable Water
Quantity	177.1 pcy	179 pcy	187 pcy
Cement, Type	Type LS-II	Type LS-II	Type LS-II
Quantity	489.4 pcy	490 pcy	491 pcy
Fly Ash, Type	ASTM Class C	ASTM Class C	ASTM Class C
Quantity	211 pcy	210 pcy	209.4 pcy
% Replacement by Weight	30%	30%	30%
Retarder, Type	ASTM Type B	ASTM Type B	ASTM Type B
Quantity	28 oz/cy	28 oz/cy	28 oz/cy
HRWR, Type	ASTM Type F NR	ASTM Type F NR	ASTM Type F NR
Quantity			
Air Entrainment, Type	ASTM C260	ASTM C260	ASTM C260
Quantity	3.0 oz/cy	2.0 oz/cy	3.0 oz/cy
Fresh Concrete Properties			
W/Cm (C+FA), by weight	0.25	0.25	0.27
Slump	8 inches	7.5 inches	8.5 inches
Total Air	4.6%	4.5%	5.5%
Unit Weight	149.3 pcf	149.4 pcf	146.7 pcf
Concrete Temperature, deg. F	77	77	81
Ambient Temperature, deg. F	65	66	72
Required Release and Design Compressive Strengths			
Design Strength $\oplus$	6,000 psi	6,000 psi	6,000 psi
$\oplus$ 28-day strengths for HS/HPC mixes $1 \text{ lb/yd}^3 = 27 \text{ lb/ft}^3 = 0.5933 \text{ kg/m}^3$ $1 \text{ ksi} = 1,000 \text{ psi} = 6.895 \text{ MPa}$ $\text{deg. F} = (9/5)(\text{deg. C}) + 32$ $1 \text{ oz/yd}^3 = 0.03868 \text{ L/m}^3$ $1 \text{ inch} = 25.4 \text{ mm}$			

Table 6.33: Mix Properties for N.C.R.O. CIP Deck — Eastbound Bridge

	San Angelo Class K - HS/HPC Span 4 - EB	San Angelo Class K - HS/HPC Span 5 - EB	San Angelo Class K - HS/HPC Span 6 - EB
Mix Designation	SAD 7-23	SAD 7-26	SAD 8-19
Casting Date	7-23-97	7-26-97	8-19-97
Mix Proportions			
Coarse Aggregate, Type	Crushed River Gravel, 1-1/4" max.	Crushed River Gravel, 1-1/4" max.	Crushed River Gravel, 1-1/4" max.
Quantity	ASTM Grade 5 1904 pcy	ASTM Grade 5 1887 pcy	ASTM Grade 5 1896 pcy
Fine Aggregate, Type	Natural River Sand FM = 2.70	Natural River Sand FM = 2.70	Natural River Sand FM = 2.70
Quantity	1213 pcy	1216 pcy	1193 pcy
Water, Type	City of San Angelo Potable Water	City of San Angelo Potable Water	City of San Angelo Potable Water
Quantity	187 pcy	174.8 pcy	189.8 pcy
Cement, Type	Type LS-II	Type LS-II	Type LS-II
Quantity	489.4 pcy	488.3 pcy	491.7 pcy
Fly Ash, Type	ASTM Class C	ASTM Class C	ASTM Class C
Quantity	211 pcy	210.6 pcy	212.2 pcy
% Replacement by Weight	30%	30%	30%
Retarder, Type	ASTM Type B	ASTM Type B	ASTM Type B
Quantity	28 oz/cy	28 oz/cy	28 oz/cy
HRWR, Type	ASTM Type F NR	ASTM Type F NR	ASTM Type F NR
Air Entrainment, Type	ASTM C260	ASTM C260	ASTM C260
Quantity	2.6 oz/cy	2.6 oz/cy	2.7 oz/cy
Fresh Concrete Properties			
W/Cm (C+FA), by weight	0.27	0.25	0.27
Slump	8 inches	9.5 inches	9 inches
Total Air	5.6%	6.6%	6.4%
Unit Weight	145.4 pcf	145.6 pcf	147.7 pcf
Concrete Temperature, deg. F	81	81	75
Ambient Temperature, deg. F	73	80	78
Required Release and Design Compressive Strengths			
Design Strength $\oplus$	6,000 psi	6,000 psi	6,000 psi
$\oplus$ 28-day strengths for HS/HPC mixes $1 \text{ lb/yd}^3 = 27 \text{ lb/ft}^3 = 0.5933 \text{ kg/m}^3$ $1 \text{ ksi} = 1,000 \text{ psi} = 6.895 \text{ MPa}$ $\text{deg. F} = (9/5)(\text{deg. C}) + 32$ $1 \text{ oz/yd}^3 = 0.03868 \text{ L/m}^3$ $1 \text{ inch} = 25.4 \text{ mm}$			

Table 6.34: Mix Properties for N.C.R.O. CIP Deck — Eastbound Bridge

	San Angelo Class K - HS/HPC Span 7 - EB	San Angelo Class K - HS/HPC Span 8 - EB	
Mix Designation	SAD 8-28	SAD 9-4	
Casting Date	8-28-97	9-4-97	
Mix Proportions			
Coarse Aggregate, Type	Crushed River Gravel, 1-1/4" max.	Crushed River Gravel, 1-1/4" max.	
Quantity	ASTM Grade 5 1896 pcy	ASTM Grade 5 1902 pcy	
Fine Aggregate, Type	Natural River Sand FM = 2.70	Natural River Sand FM = 2.70	
Quantity	1193 pcy	1180 pcy	
Water, Type	City of San Angelo Potable Water	City of San Angelo Potable Water	
Quantity	187 pcy	188 pcy	
Cement, Type	Type LS-II	Type LS-II	
Quantity	489.4 pcy	489.4 pcy	
Fly Ash, Type	ASTM Class C	ASTM Class C	
Quantity	210.6 pcy	209.4 pcy	
% Replacement by Weight	30%	30%	
Retarder, Type	ASTM Type B	ASTM Type B	
Quantity	28 oz/cy	28 oz/cy	
HRWR, Type	ASTM Type F NR	ASTM Type F NR	
Quantity			
Air Entrainment, Type	ASTM C260	ASTM C260	
Quantity	2.7 oz/cy	2.7 oz/cy	
Fresh Concrete Properties			
W/Cm (C+FA), by weight	0.27	0.27	
Slump	9.25 inches	9 inches	
Total Air	6.2%	6.4%	
Unit Weight	146.8 pcf	145.2 pcf	
Concrete Temperature, deg. F	80	84	
Ambient Temperature, deg. F	74	72	
Required Release and Design Compressive Strengths			
Design Strength $\oplus$	6,000 psi	6,000 psi	
$\oplus$ 28-day strengths for HS/HPC mixes $1 \text{ lb/yd}^3 = 27 \text{ lb/ft}^3 = 0.5933 \text{ kg/m}^3$ $1 \text{ ksi} = 1,000 \text{ psi} = 6.895 \text{ MPa}$ $\text{deg. F} = (9/5)(\text{deg. C}) + 32$ $1 \text{ oz/yd}^3 = 0.03868 \text{ L/m}^3$ $1 \text{ inch} = 25.4 \text{ mm}$			

6.4 Current Code Provisions Related to NSC and HSC Mechanical Properties

The following are the current recommended code provisions regarding mechanical properties of normal-strength and high-strength concrete. Furthermore, proposed recommendations are presented based on recent research studies of high-strength concrete.

6.4.1 Static Elastic Modulus

The elastic modulus or modulus of elasticity of concrete is one of the most important mechanical properties of concrete because it impacts the serviceability and structural performance of reinforced concrete structures. The elastic modulus of concrete is closely related to the property of the cement paste, the stiffness of the selected aggregates, and also, the method of determining the modulus. A significant amount of research has been conducted on both conventional concrete and high-strength concrete in order to predict the elastic modulus as a function of the compressive strength of the concrete. The American Concrete Institute (ACI), similarly to most national codes throughout the world, has suggested empirical equations to predict the elastic modulus based on the compressive strength. Equation 6.1 presents the recommended empirical code equation for conventional concretes (normal weight concrete) by ACI Committee 318 [101].

$$E_c = 57,000 (f'_c)^{0.5} \quad (6.1)$$

ACI Committee 363 [77] stated that Equation 6.1 overestimates the elastic modulus for compressive strengths over 41.4 MPa (6,000 psi). ACI Committee 363 proposed Equation 6.2 for concretes with compressive strengths in the range of 41.4 MPa (6,000 psi) to 82.7 MPa (12,000 psi).

$$E_c = 40,000 (f'_c)^{0.5} + 10^6 \quad (6.2)$$

Several other recommendations for high-strength concrete have been proposed. Cook [114] reported that both Equations 6.1 and 6.2 presented by ACI underestimated the elastic modulus for high-strength concretes primarily consisting of crushed limestones, granites, and gravels. He proposed Equation 6.3.

$$E_c = w^{2.5} (f'_c)^{0.315} \quad (6.3)$$

where: $w = 151$ pcf.

Shah et al. [115] suggested that Equation 6.1 overestimates the elastic modulus and suggested that Equation 6.4 be used for both conventional and high-strength concretes.

$$E_c = w^{2.5} (f'_c)^{0.325} \quad (6.4)$$

where: w = unit weight of concrete.

ACI Committee 363 has stated that deviations from the predicted values of Equation 6.2 are highly dependent on the properties of the coarse aggregate [77]. This has been substantiated by research conducted within this study and by Iravani [116]. It is generally accepted that it is easier to correlate the compressive strength and the elastic modulus of conventional concretes because the paste matrix is always the weakest link. The paste zones may be stronger than the aggregates in the case of high-strength concrete due to its enhanced paste and transition zone characteristics.

In 1996, Iravani [116] recommended a coarse aggregate coefficient, C_{ca} , for high-strength concretes as presented in Equation 6.5.

$$E_c = 57,000 C_{ca} (f'_c) \quad (6.5)$$

where:

$C_{ca} = 0.71$ for sandstone gravel

$C_{ca} = 0.76$ for siliceous gravel

$C_{ca} = 0.92$ for limestone

$C_{ca} = 0.92$ for dolomite

$C_{ca} = 0.97$ for quartzites

$C_{ca} = 0.82$ for granite

$C_{ca} = 0.97$ for trap rock

$C_{ca} = 0.90$ for diabase

$C_{ca} = 0.61$ for sandstone

While these aggregate coefficients for high-strength concretes presented by Iravani provide a more accurate representation of elastic modulus than Equations 6.1 or 6.2 presented by ACI, it is still preferable to perform trial mixes to develop said properties or to use an elastic modulus based on a specific mix design's historical performance records. This is recommended because of the wide variability for an aggregate within a grouping as presented by Iravani. This is to say that a limestone in south Texas may or may not be representative of a limestone from central Texas in terms of its chemical and physical properties. The impact of coarse aggregate type and content on elastic modulus is discussed in more detail in Section 6.5.1 of this report.

The current *Standard Specifications for Highway Bridges* [117] adopted by AASHTO uses Equation 6.1 by ACI for the elastic modulus for concrete. At this time, AASHTO has not adopted any special recommendations for elastic modulus of high-strength concrete or other “exotic” concretes.

6.4.2 Flexural Strength

The flexural strength is often considered to be a more reliable indicator of the direct tensile strength of concrete. The flexural-to-compressive strength ratio ranges from about 0.23 for a compressive strength of 6.9 MPa (1,000 psi) to 0.11 for a compressive strength of 62.1 MPa (9,000 psi) [4]. Strength in tension is less sensitive to changes in proportioning such as w/cm ratio. A reduction in the w/cm ratio may have a large effect on compressive strength, but a much smaller impact on tensile strength. Failure in tension requires the propagation of only one small flaw or crack to cause failure, while compressive failures can endure quite a bit of microcracking prior to ultimate failure. Flexural strength seems to be less affected by the enhanced paste properties due to hydration. Therefore, the flexural-to-compressive strength ratio decreases with time. Mehta [4] and Neville [19] suggest that mineralogy, size, and shape of the aggregate are important factors that influence the flexural strength of concrete.

Equation 6.6 presents the recommended empirical code equation for conventional concretes (normal weight concrete) by ACI Committee 318 [101].

$$f_r = 7.5 (f'_c)^{0.5} \quad (6.6)$$

ACI Committee 363 recommended Equation 6.7 for prediction of the flexural tensile strength or modulus of rupture for concretes with compressive strengths in the range of 20.7 MPa (3,000 psi) to 82.7 MPa (12,000 psi).

$$f_r = 11.7 (f'_c)^{0.5} \quad (6.7)$$

Shah et al. [115] suggested that Equation 6.8 be used for both conventional and high-strength concretes.

$$f_r = 2.3 (f'_c)^{0.67} \quad (6.8)$$

Iravani [116] recommended that Equation 6.9 be used for concretes with compressive strengths in the range of 50 MPa (7,250 psi) to 100 MPa (14,500 psi).

$$f_r = 12.1 (f'_c)^{0.50} \quad (6.9)$$

The current *Standard Specifications for Highway Bridges* [117] adopted by AASHTO provides no recommendations for concrete tensile strength (flexural or splitting tensile strength). Currently, AASHTO provisions address modulus of rupture (flexural tensile strength) in terms of an allowable stress, which is equivalent to Equation 6.6 recommended by ACI 318. No recommendations are provided for the splitting tensile strength of concrete.

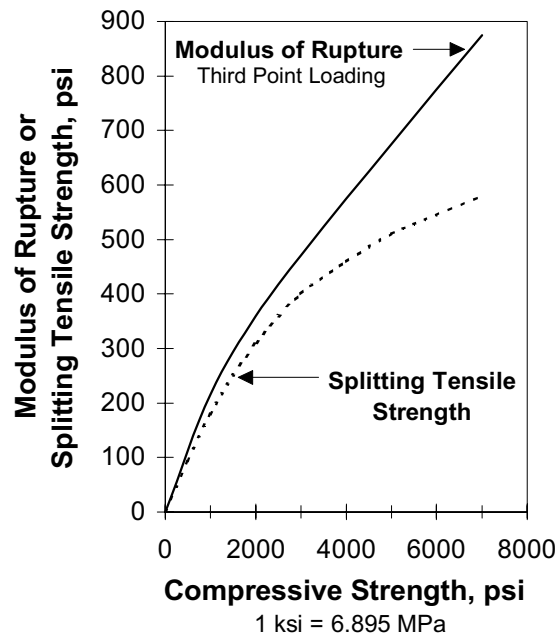


Figure 6.1: Relationship of Flexural and Tensile Strength to Compressive Strength [118]

6.4.3 Splitting Tensile Strength

The tensile strength of concrete is much lower than the compressive strength of concrete, largely because of the ease with which cracks can form and propagate under tensile loads. The tensile strength, while generally assumed to be zero in design, is very important because cracking in

concrete tends to be a tensile failure. The relationship between tensile and compressive strength is not simplistic in nature. It depends on the age and strength of concrete, the type of curing, the type of aggregate, the amount of air entrainment, and the degree of compaction [2]. The method of testing that includes direct tension, splitting tension, or flexure also influences the relationship. In a general sense, for conventional concretes, as the age increases, the ratio of tensile to compressive strength (σ_t/σ_c) decreases as illustrated in Figure 6.1. Crushed and more angular aggregates have also exhibited improved tensile strength.

ACI Committee 318 [101] recommends Equation 6.10 for splitting tensile strength for conventional concretes (normal weight concrete).

$$f_{sp} = 6 (f'_c)^{0.5} \quad (6.10)$$

ACI Committee 363 proposed Equation 6.11 for concretes with compressive strengths in the range of 20.7 MPa (3,000 psi) to 82.7 MPa (12,000 psi).

$$f_{sp} = 7.4 (f'_c)^{0.5} \quad (6.11)$$

A limited amount of data is currently available on the splitting tensile strength of high-strength concrete related to aggregate type or other constituent variables. One of the objectives of this study is to provide the FHWA with a database of results from these two bridge projects.

6.5 Performance-Related Test Results of Laboratory Trial Mixes

The following section presents the performance-related test results from the laboratory trial mixes and discusses the impact that the mix design constituents play on the mechanical and material performance of HPC. These studies included the effect of coarse aggregate type and content, fly ash replacement, temperature development, cement manufacturer and cement type, and the use of HRWRs. Furthermore, a replacement study investigated the impact of fly ash replacement related to current TxDOT specifications for minimum cement content requirements and maximum water-to-cement ratio requirements. Tables 6.35 through 6.43 present the hardened concrete properties for the laboratory trial batches.

Table 6.35: Hardened Concrete Properties for Laboratory Trial Batches

Mix Designation	CG1	CG2	CG3	CG4	CG5	CG6	CG7	CG8
Hardened Concrete Properties								
Compressive Strength (psi)								
1 day	5150	6600	4150	2950	2450	3310	1920	1780
3 days	6850	8700	5590	4960	5750	5000	420	4330
7 days	8120	9110	5720	5950	8850	5820	5780	6090
28 days	8980	10200	7360	7260	9940	7140	7740	8730
56 days	NR	11000	7800	8410	11340	7940	8740	9480
RCPT - Rating	NR	Low	Low	Very Low	Very Low	Moderate	Low	Very Low
56 days moist cured		1990	1740	830	500	2210	1090	660
Water-to-Binder Ratio								
W/Cm (C+ FA), by weight	0.30	0.29	0.36	0.34	0.29	0.39	0.28	0.33
Test Methods: AASHTO T2 (ASTM C39), AASHTO T2771 ksi = 1,000 psi = 6.895 MPa								

Table 6.36: Hardened Concrete Properties for Laboratory Trial Batches (continued)

Mix Designation	CG9	CG10	CG11	CG12	CG13	CG14	CG15	CG16	CG17
Hardened Concrete Properties									
Compressive Strength (psi)									
1 day	7790	5060	4350	6330	4480	4080	4140	3410	2970
3 days	9600	8060	6840	8030	7850	6430	4300	4570	4460
7 days	1450	9920	9970	8240	8360	6640	4790	5560	5660
28 days	11860	10820	11350	8880	9670	8350	5130	6250	6730
56 days	13400	12140	12600	9960	10360	9607	5720	6500	7870
RCPT – Rating	Low	Very Low	Very Low	Low	Very Low	Very Low	Moderate	Low	Very Low
56 days moist cured	1170	820	640	1120	590	360	2650	1170	560
Water-to-Binder Ratio									
W/Cm (C+ FA), by weight	0.29	0.29	0.29	0.33	0.29	0.29	0.39	0.27	0.30
Test Methods: AASHTO T2 (ASTM C39), AASHTO T277									
1 ksi = 1,000 psi = 6.895 MPa									

Table 6.37: Hardened Concrete Properties for Laboratory Trial Batches (continued)

Mix Designation	LSC1	LSC2	LSC3	LSC4	LSC5	RGC1	RGC2	RGC3	RGC4	RGC5
Hardened Concrete Properties										
Compressive Strength (psi)										
7 days	6340	6140	6050	5320	3850	5090	5020	5750	5050	4330
28 days	7920	7740	8900	8070	5990	6180	5970	8060	7280	6130
56 days	8570	8320	9870	9160	6920	6780	6820	8590	7880	7070
84 days	8830	8560	10720	9800	7700	7390	7670	9110	8490	8000
140 days	9080	8800	11570	10240	8260	7870	7880	9410	8860	8180
Modulus of Elasticity (ksi)										
56 days	6330	5600	6660	6500	5900	5690	5400	6120	5650	5560
RCPT - Rating	Low	Moderate	Low	Low	Low	Moderate	Moderate	Low	Low	Moderate
56 days moist cured	1470	2630	1100	1160	1630	2420	3200	1740	1920	2260
Water-to-Binder Ratio										
W/Cm (C+ FA), by weight	0.44	0.53	0.44	0.48	0.51	0.41	0.44	0.44	0.48	0.51
Test Methods: AASHTO T2 (ASTM C39), ASTM C469, AASHTO T277 1 ksi = 1,000 psi = 6.895 MPa										

Table 6.38: Hardened Concrete Properties for Laboratory Trial Batches (continued)

Mix Designation	LS1	LS2	LS3	LS4
Hardened Concrete Properties				
Compressive Strength (psi)				
18 hours	8850	9040	10180	10760
3 days	9510	10210	11380	11530
7 days	9760	10520	11830	11810
28 days	11000	11540	12820	12900
56 days	NR	12140	13770	14520
RCPT - Rating	Low	Low	Very Low	Very Low
56 days moist cured	1230	1180	700	420
Water-to-Binder Ratio				
W/Cm (C+ FA), by weight	0.27	0.27	0.27	0.27
Test Methods: AASHTO T2 (ASTM C39), AASHTO T277 1 ksi = 1,000 psi = 6.895 MPa				

Table 6.39: Hardened Concrete Properties for Laboratory Trial Batches (continued)

Mix Designation	MLS1	MLS2	MLS3	MLS4	MLS5	MLS6	MLS7	MLS8
Hardened Concrete Properties								
Compressive Strength (psi)								
1 day	9030	8170	7250	6390	6620	6720	5650	3300
4 days	10870	11110	10410	8920	9310	9920	9770	8670
7 days	11160	11140	10840	9520	10160	11150	10300	9170
28 days	11630	11880	12120	10480	11340	11690	11570	10400
56 days	12290	12720	12240	10500	11610	11700	NR	NR
Modulus of Elasticity (ksi)								
28 days	4820	4990	4710	5010	5000	5110	4970	4940
56 days	4940	5010	5180	5350	5340	5280	NR	NR
RCPT - Rating	Low	Low	NR	Low	Low	Moderate	Moderate	Moderate
56 days moist cured	1880	1440		1710	1970	3120	2970	3600
Water-to-Binder Ratio								
W/Cm (C+ FA), by weight	0.29	0.28	0.28	0.31	0.29	0.29	0.33	0.38
Test Methods: AASHTO T2 (ASTM C39), ASTM C469, AASHTO T277 1 ksi = 1,000 psi = 6.895 MPa								

Table 6.40: Hardened Concrete Properties for Laboratory Trial Batches (continued)

Mix Designation	MFA1	MFA2	MFA3	RG1	RG2	RG3	SSP-3	SSP-32	HP-3	HP-32
Hardened Concrete Properties										
Compressive Strength (psi)										
1 day	NR	NR	NR	3800	3730	4910	NR	NR	NR	NR
3 days	4590	4050	3530	NR	NR	NR	NR	NR	NR	NR
7 days	5900	5250	4900	8880	9050	8220	8340	8700	6620	6150
28 days	6580	6580	6540	9890	9690	8950	10080	10370	8270	8370
56 days	7280	7280	7710	10680	10990	9910	11840	11450	9520	8820
Modulus of Elasticity (ksi)										
28 days	4210	4230	4520	NR	NR	NR	5970	5980	5560	5830
56 days	4400	4700	5260	NR	NR	NR	5980	6560	6670	6150
RCPT - Rating	NR	NR	NR	NR	NR	NR				
56 days moist cured							323	313	326	302
14 days moist cured & 42 days air dried							443	384	646	NR
Water-to-Binder Ratio										
W/Cm (C+ FA), by weight	0.57	0.51	0.51	0.28	0.28	0.28	0.25	0.20	0.25	0.20
Test Methods: AASHTO T2 (ASTM C39), ASTM C469, AASHTO T277										
1 ksi = 1,000 psi = 6.895 MPa										

Table 6.41: Hardened Concrete Properties for Laboratory Trial Batches (continued)

Mix Designation	AELS1	AELS2	AELS3	AELS4	AELS5	AELS6	AELS7	AELS8	AELS9
Hardened Concrete Properties									
Compressive Strength (psi)									
7 days	10330	10690	6320	9990	8740	8300	10970	10960	9410
28 days	11940	10450	6590	12180	10530	10500	13120	13170	12200
56 days	12520	10880	6520	12490	11770	11530	14860	14820	13440
Modulus of Elasticity (ksi)									
28 days	6500	6460	4070	6570	7000	6850	7030	6810	6250
56 days	6960	6820	5350	7930	7100	6950	7260	6840	6320
Splitting Tensile Strength (psi)									
28 days	990	710	510	860	780	710	960	820	840
56 days	1045	930	790	1070	960	960	1110	940	930
RCPT - Rating									
56 days moist cured	1083	1178	NR	444	307	359	461	523	512
14 days moist cured & 42 days air dried	1163	1227	1426	892	1003	837	655	624	827
Water-to-Binder Ratio									
W/Cm (C+ FA), by weight	0.30	0.30	0.30	0.30	0.30	0.30	0.26	0.27	0.27
Test Methods: AASHTO T2 (ASTM C39), ASTM C469, ASTM C496, AASHTO T277 1 ksi = 1,000 psi = 6.895 MPa deg. F = (9/5)(deg. C) + 32 1 inch = 25.4 mm									

Table 6.42: Hardened Concrete Properties for Laboratory Trial Batches (continued)

Mix Designation	AETR1	AETR2	AETR3	AETR4	AETR5	AETR6	AETR7	AETR8	AETR9
Hardened Concrete Properties									
Compressive Strength (psi)									
7 days	9360	9260	8680	10260	9000	9100	9110	9130	8050
28 days	11110	10480	10080	11420	10870	10630	10630	10890	9490
56 days	11970	10980	10780	12280	11370	11390	11410	11500	10270
Modulus of Elasticity (ksi)									
28 days	6350	5950	5920	7300	6730	5970	5740	6020	5560
56 days	6780	6120	6540	9890	7050	6390	6470	6850	5930
Splitting Tensile Strength (psi)									
28 days	860	610	590	820	840	840	780	830	700
56 days	900	690	610	980	930	930	980	890	920
RCPT - Rating									
56 days moist cured	1343	1484	1614	366	491	448	937	770	773
14 days moist cured & 42 days air dried	1559	1638	1904	453	867	762	1306	938	1063
Water-to-Binder Ratio									
W/Cm (C+ FA), by weight	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30
Test Methods: AASHTO T2 (ASTM C39), ASTM C469, ASTM C496, AASHTO T277 1 ksi = 1,000 psi = 6.895 MPa deg. F = (9/5)(deg. C) + 32 1 inch = 25.4 mm									

Table 6.43: Hardened Concrete Properties for Laboratory Trial Batches (continued)

<i>Mix Designation</i>	AERG1	AERG2	AERG3	AERG4	AERG5	AERG6	AERG7	AERG8	AERG9
<i>Hardened Concrete Properties</i>									
Compressive Strength (psi)									
7 days	8030	7950	7510	7070	8800	6440	9160	8540	8250
28 days	8810	9650	9840	9070	10230	8700	11710	10990	10440
56 days	9790	10450	9910	9230	11510	9540	12390	11590	11320
Modulus of Elasticity (ksi)									
28 days	5570	5430	6290	6210	6490	5610	6290	6260	5920
56 days	5930	5800	6450	6600	6430	5660	6700	6640	6310
Splitting Tensile Strength (psi)									
28 days	NR	NR	800	800	820	790	890	880	850
56 days	810	830	890	810	900	870	930	940	880
RCPT - Rating									
56 days moist cured	1956	1061	1282	1417	574	803	607	576	604
14 days moist cured & 42 days air dried	NR	NR	NR	NR	559	927	772	780	614
<i>Water-to-Binder Ratio</i>									
W/Cm (C+ FA), by weight	0.33	0.32	0.28	0.29	0.29	0.29	0.27	0.27	0.27
Test Methods: AASHTO T2 (ASTM C39), ASTM C469, ASTM C496, AASHTO T277 1 ksi = 1,000 psi = 6.895 MPa deg. F = (9/5)(deg. C) + 32 1 inch = 25.4 mm									

6.5.1 Coarse Aggregate Effects—Aggregate Replacement Study

Three different aggregate contents of 36, 40, and 44 percent and five aggregate types were investigated within this study. The coarse aggregate type and gradation played an important role in the compressive strength development of the concrete mix designs investigated. The aggregate types included trap rock, dolomitic limestone, calcitic limestone, and crushed river gravel. Table 6.44 presents the approximate fracture stresses of these aggregates when tested independent of a concrete mix design.

Table 6.44: Aggregate Fracture Stress by Aggregate Type

Aggregate Type	Fracture Stress
Trap Rock	26 to 28 ksi
Dolomitic Limestone	17 to 18 ksi
Calcitic Limestone	12 to 15 ksi
Crushed River Gravel	10 to 12 ksi
1 ksi = 1,000 psi = 6.895 MPa	

The trap rock (TR), which is the hardest and densest aggregate selected for the coarse aggregate replacement study, resulted in a lower compressive strength when compared to the dolomitic limestone (DL) for the same mix proportions and materials. The difference between the stiffnesses of the mortar and trap rock resulted in stress concentrations at higher stress levels. This resulted in failure of the cylinders at a lower compressive strength value. Cracks generally formed in the cement paste around the aggregate rather than through it. The dense trap rock acted as a stress riser under high stress levels. However, the trap rock did result in the highest elastic modulus for a given aggregate content due to its dense characteristics.

A good analogy for the behavior of the trap rock in these mix designs may be a high rise structure or bridge subjected to high stress levels. Under a lateral load for example, the stiffest elements or components of the structure attract the load and carry it based on their relative stiffnesses throughout the overall structure. Generally, the critical locations for failure are the connections between the collector elements, which transfer the load to the load-resisting elements supporting the load. This is a similar behavior to what occurred with the trap rock. The aggregate, which is much stiffer than the mortar, attracts the load based on its relative stiffness within the concrete; as a result, high local stress levels develop, particularly at angular locations where the shape of the aggregate

varies. In terms of optimizing strength, it was concluded that the key objective was to closely match the moduli of the mortar matrix with the moduli of the aggregate to create a more compatible or homogeneous mix design. Of the aggregates investigated within the replacement study, the Burnet limestone was the most appropriate in satisfying both the compatibility issue in terms of optimizing strength development and also satisfying the modulus of elasticity requirements. As noted in Table 6.44, the aggregate typically fractured in the 117.2 to 124.1 MPa (17 to 18 ksi) range.

The crushed river gravel (CG) mix designs that were investigated tended to develop lower compressive strengths. The crushed river gravel tended to fail in the range of 75 to 83 MPa (10,800 to 12,000 psi). Fracture of the aggregate was clearly visible during compression testing with no strength gain noted beyond 83 MPa (12,000 psi) at long-term testing ages. The crushed river gravel also exhibited poor bond characteristics due to its partially smooth surfaces.

The calcitic limestone (CL) provided good compressive strength development in the 82.7 to 103.4 MPa (12 to 15 ksi) range. However, the elastic modulus development was not suitable for the precast beam requirements. The calcitic limestone could be recommended for HS/HPC components in which elastic modulus was not critical.

Concretes with smaller maximum size aggregate gradation of 13 mm (0.5 in.) yielded slightly higher compressive strength test results than the larger maximum size aggregate gradation of 19 mm (0.75 in.). The smaller aggregate size and, consequently, higher surface area for a given aggregate content, results in a lower bond stress at a given load level and, consequently, higher strength. The concrete tends to act more like a homogenous material.

For all aggregate types, increasing the coarse aggregate content beyond 40 percent did not appear to benefit the compressive strength development. However, the elastic modulus appeared to be a function of the coarse aggregate content and type. The elastic modulus development for different aggregates is illustrated in Figures 6.2, 6.3, and 6.4 for 36, 40, and 44 percent aggregate contents from the laboratory trial mixes. The only coarse aggregate that exhibited an elastic modulus below the empirical equation recommended by ACI Committee 318 was the calcitic limestone. The elastic modulus of the calcitic limestone exceeded the empirical equation recommended by ACI Committee 363 for high-strength concrete. The lower elastic modulus values of the calcitic limestone may be attributed to the abundance of calcite, which is a soft mineral.

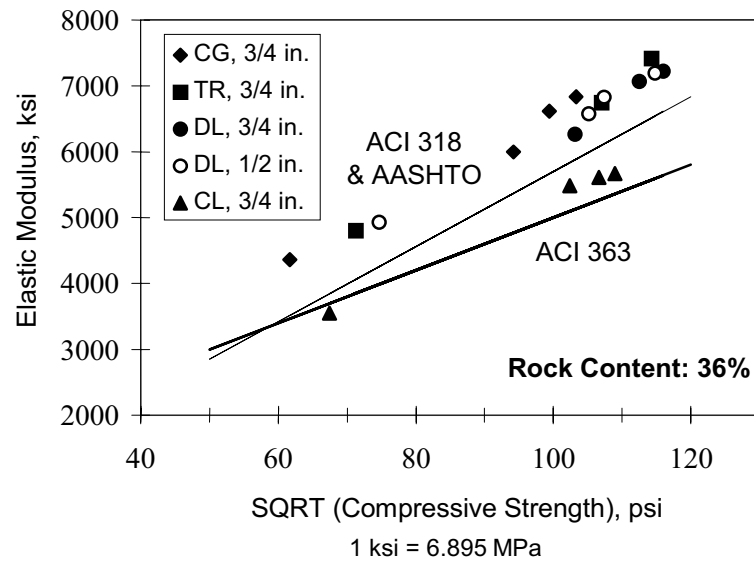


Figure 6.2: Elastic Modulus Development for Concretes with 36% Aggregate Content [22]

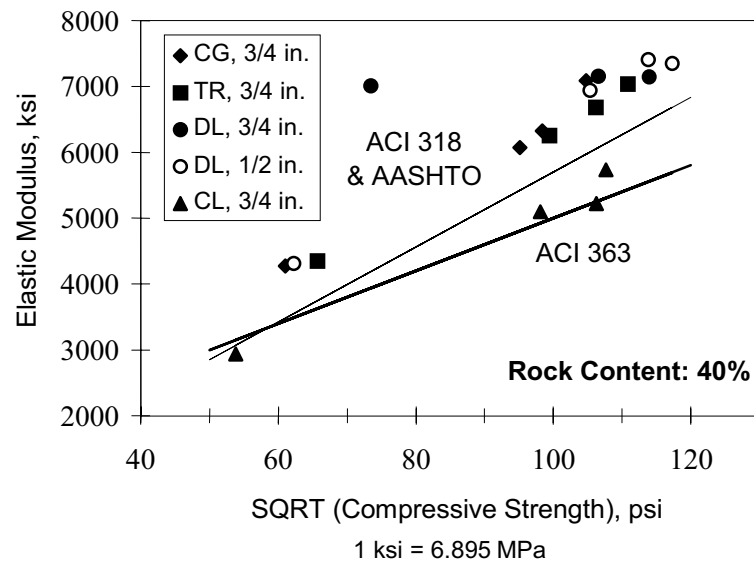


Figure 6.3: Elastic Modulus Development for Concretes with 40% Aggregate Content [22]

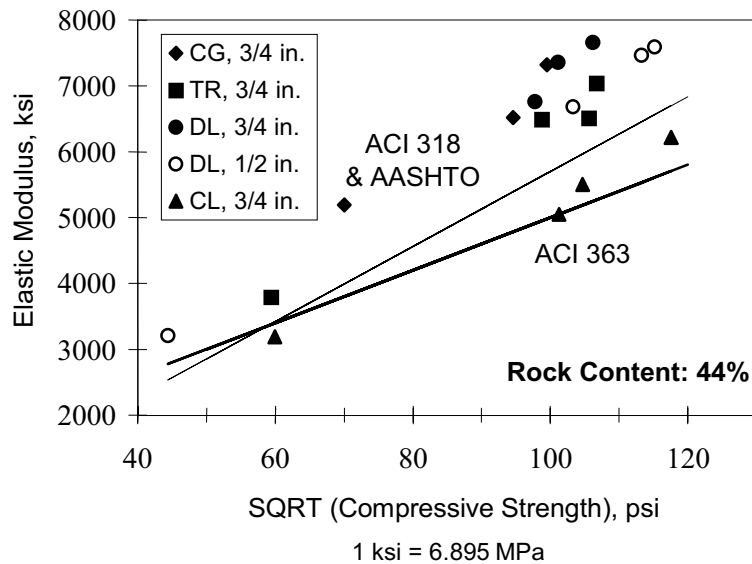


Figure 6.4: Elastic Modulus Development for Concretes with 44% Aggregate Content [22]

In addition to the elastic modulus, the flexural strengths of the trial mix designs were monitored. The flexural strengths for different aggregates are illustrated in Figures 6.5, 6.6, and 6.7 for 36, 40, and 44 percent aggregate contents from the laboratory trial mixes. Increasing the coarse aggregate content appeared to result in a slight reduction in the flexural strength. Increasing the aggregate content without a change in the aggregate size results in an increased interface area. This results in a potentially weaker interface zone than either mortar or aggregate, thus resulting in a reduction in the flexural strength.

Based on the results of the tests on the trial mixes, the 13-mm (0.5-in.) dolomitic limestone from central Texas proved to be the most suitable to meet the performance-based specifications for the precast/prestressed U-beams (Louetta Road Overpass) and AASHTO Type IV Beams (North Concho River Overpass). The dolomitic limestone with its angular shape and elastic moduli compatible to the mortar provided an aggregate with excellent bond characteristics and homogeneous performance. The aggregate was also locally available for the precast fabricator, subsequently resulting in an economical mix design. The smaller aggregate gradation size was selected for the precast/prestressed beams due to the overall congestion of prestress reinforcing and the improved compressive strength development of the smaller maximum aggregate gradation.

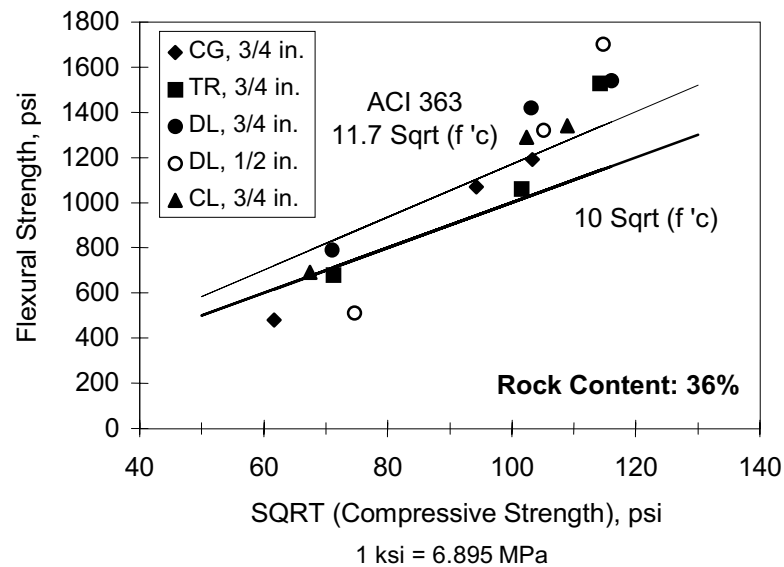


Figure 6.5: Flexural Strength versus Compressive Strength Development for Concretes with 36% Aggregate Content [22]

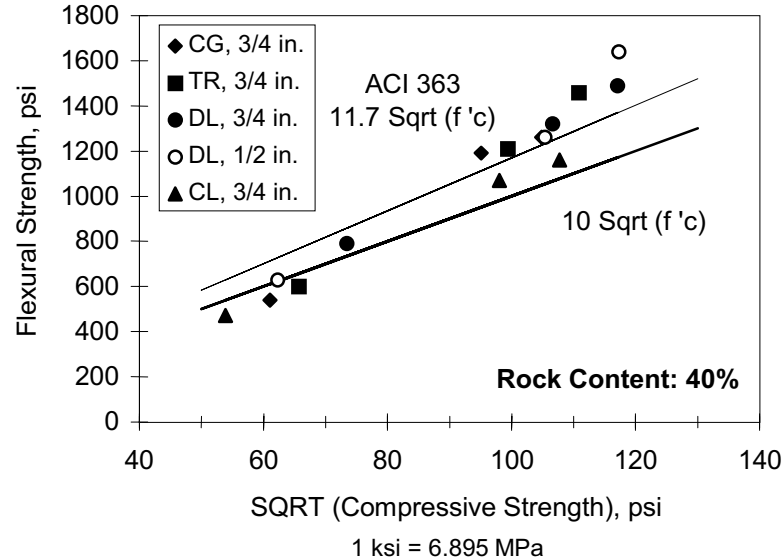


Figure 6.6: Flexural Strength versus Compressive Strength Development for Concretes with 40% Aggregate Content [22]

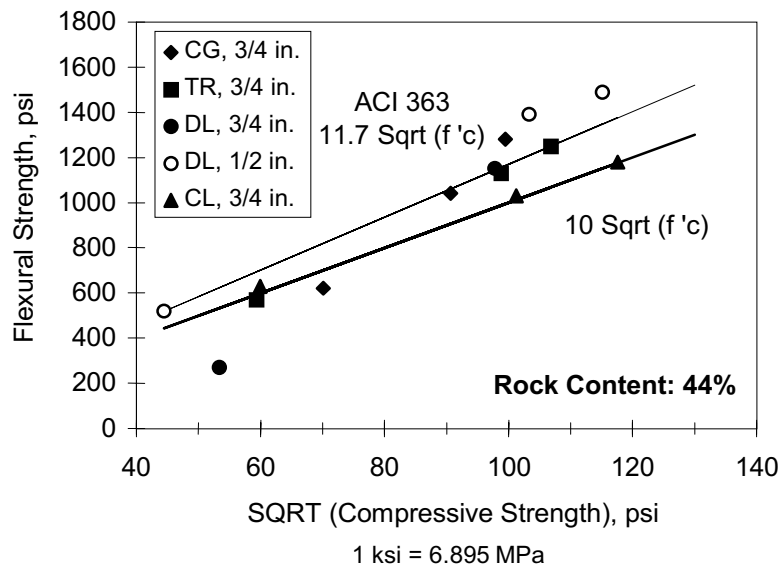


Figure 6.7: Flexural Strength versus Compressive Strength Development for Concretes with 44% Aggregate Content [22]

6.5.2 Temperature Effects—Temperature-Related Study

Two laboratory trial mix designs were subjected to accelerated heat curing to investigate any variation in compressive strength and elastic modulus due to temperature effects. This study was initiated because it was anticipated that the precast/prestressed beams fabricated for both projects would experience higher than normal hydration temperatures due to the high volume of cementitious materials and mass of concrete. Five accelerated curing temperatures were selected: 43, 66, and 88 °C (110, 150, 190 °F). Furthermore, cylinders were also standard-cured at 22.8°C (73 °F—no accelerated heat curing) and cured under a hydration chamber condition as described in Section 5.3.4.2 of this report. The hydration chamber involved the curing condition of cylinders with no external ambient influences. One of the two mix designs incorporated 30 percent fly ash replacement. Figures 6.8 and 6.9 illustrate the elastic modulus development for lab trial mixes with and without fly ash replacement. The fly ash replacement clearly enhanced the later-age strength gain and elastic modulus development. It was evident that the higher temperatures generated higher early strengths, but generally lower long-term strengths. Therefore, the specimens subjected to the lower initial temperature (standard-cured specimen) resulted in the highest long-term strength. Both cases satisfied empirical equations presented by ACI Committees 318 and 363.

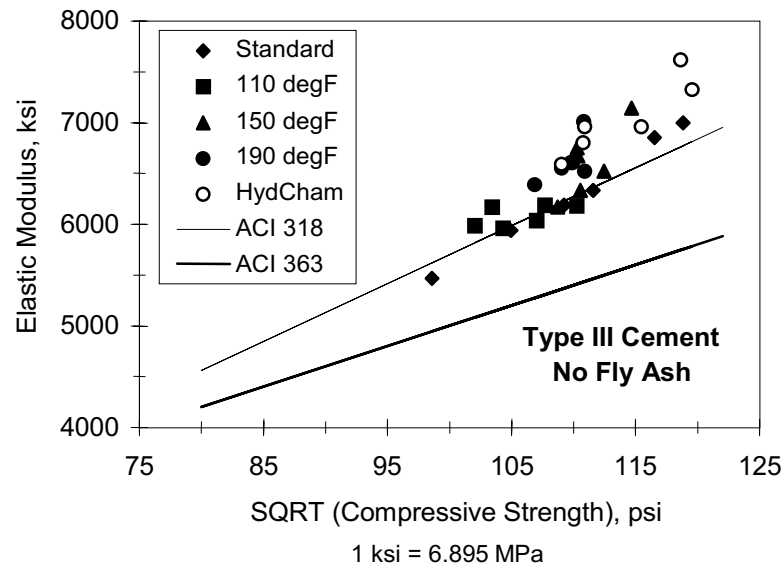


Figure 6.8: Elastic Modulus Development for Concretes without Fly Ash Replacement

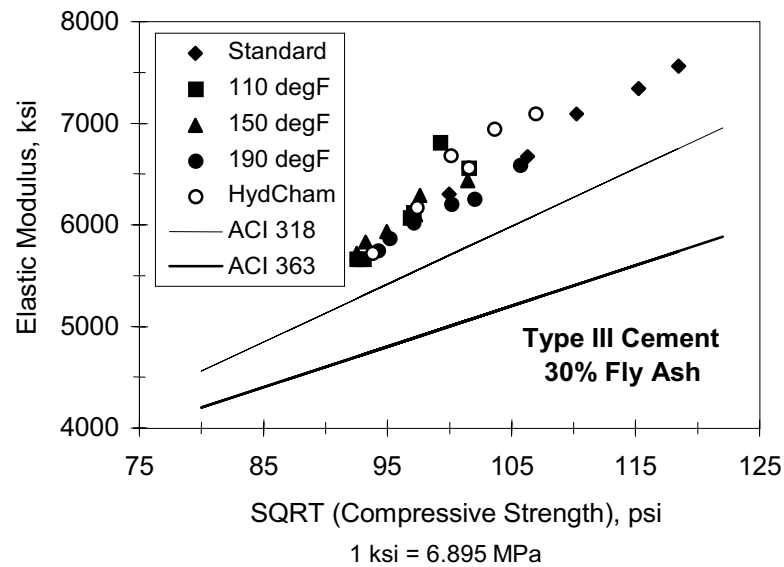


Figure 6.9: Elastic Modulus Development for Concretes with 30% Fly Ash Replacement

A second temperature-related study was undertaken to investigate three soak temperatures of 60.0, 71.1, and 87.8 °C (140, 160, and 190 °F) after a 4-hour preset time for an identical mix design. A fourth mix design was batched with the same mix proportions but with cement produced by a different manufacturer to investigate its impact on the mix design. A Type III cement was selected in all cases, which would be representative of the cement selected for use in the precast/prestressed concrete. The mix designs were designated as LS1 through LS4 and are illustrated in Table 6.4 along with the accelerated heat-curing regime and preset times. A fifth mix design, designated DLS40 [15], is presented to serve as the control. DLS40 was batched previously with standard moist curing in a 22.8 °C (73 °F) environment with no accelerated heat curing. Figures 6.10 and 6.11 illustrate the compressive strength gain of the five identical mix designs under different curing temperatures. Quite a variation was noted in the compressive strength gain when comparing the Type A-III cement with the Type C-III cement under the identical 60.0 °C (140 °F) curing regimes. This emphasizes how unpredictable the interaction is between fly ash and portland cement and its effect on strength. This interaction can only be determined by conducting laboratory trial mixes.

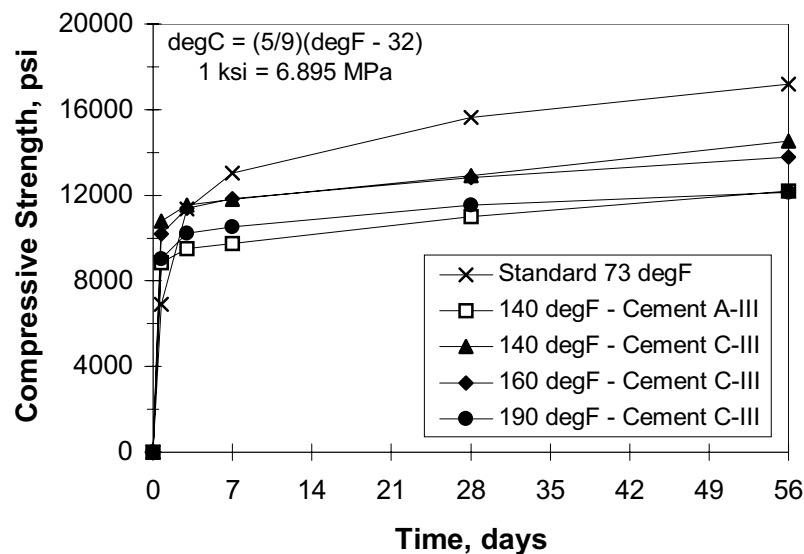


Figure 6.10: Compressive Strength Gain with Time for Accelerated Heat-Cured Laboratory Trial Mixes

The effect of accelerated heat curing is clearly evident in the compressive strength gain development both at early and later-ages as illustrated in Figures 6.10 and 6.11. At early ages (18 to

36 hours), the specimens subjected to accelerated heat curing had a 56 percent higher strength gain (maximum) than the standard cured specimen because of the additional cement hydration that occurred through the accelerated heat curing. At later ages, the accelerated heat curing resulted in a 25 percent lower strength gain (maximum) than the standard-cured specimen. In general, the higher the accelerated heat or steam-curing temperature, the greater the early-age strength, but the lower the long-term strength.

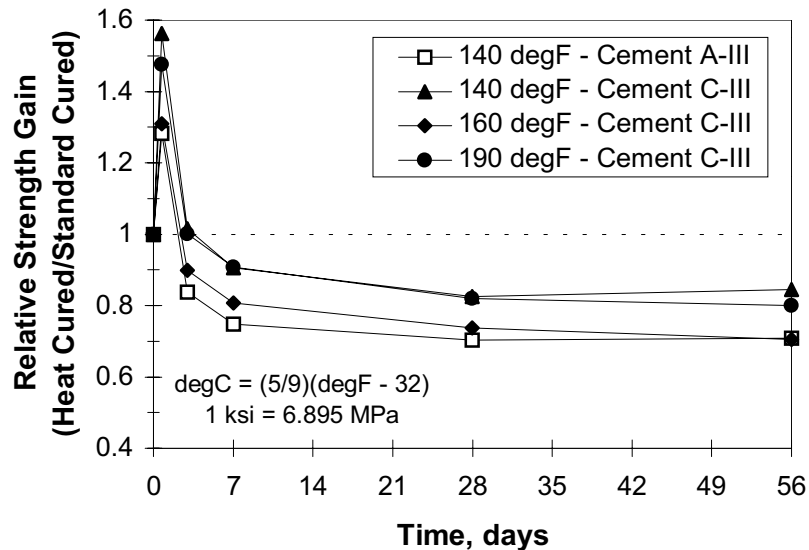


Figure 6.11: Relative Strength Gain with Time for Accelerated Heat-Cured Laboratory Trial Mixes

The effect of accelerated heat curing or steam curing has been well documented for mid-range 41.4 to 55.2 MPa (6,000 to 8,000 psi) concrete strengths for several decades in the precast industry. The use of accelerated heat curing or steam curing has been used to improve plant productivity or to meet release strengths on a daily basis. However, the precast/prestressed beams for the Louetta Road Overpass and North Concho River Overpass presented a unique challenge in which both a high release strength (74.5 MPa) and a high design strength (101.4 MPa) would be required. In fact, these may be the first documented projects in the United States precast industry in which both release and design strengths of this magnitude were required. This laboratory study effectively illustrated the relative strength of concretes at later ages due to temperature effects that could be anticipated when production of the HS/HPC beams was initiated. This topic of early-age and later-age curing aspects of the actual beams that were produced is discussed in greater detail in Chapter 8 of this report.

6.5.3 Fly Ash Effects—Permeability-Related Study

The use of a Class C fly ash is beneficial in several respects. From an economic standpoint, the fly ash replacement reduced the cost of the cementitious material by approximately 20 percent for the HPC mix designs investigated. The beneficial fresh concrete properties of the fly ash replacement included improved workability and finishability while the beneficial hardened concrete properties included improved later-age strength and reduced permeability. Furthermore, the fly ash replacement reduces the initial hydration temperature of the concrete during placement. This reduced temperature also results in a lower thermal cracking potential for the in-place concrete. Figures 6.12 and 6.13 illustrate permeability results of laboratory trial mixes incorporating 0, 25, 30, and 35 percent fly ash replacement without the use of an HRWR. These mix designs may be categorized as HPC. Figures 6.14 and 6.15 illustrate permeability results of laboratory trial mixes incorporating 0, 25, and 35 percent fly ash replacement with the use of an HRWR. These mix designs may be categorized as HS/HPC.

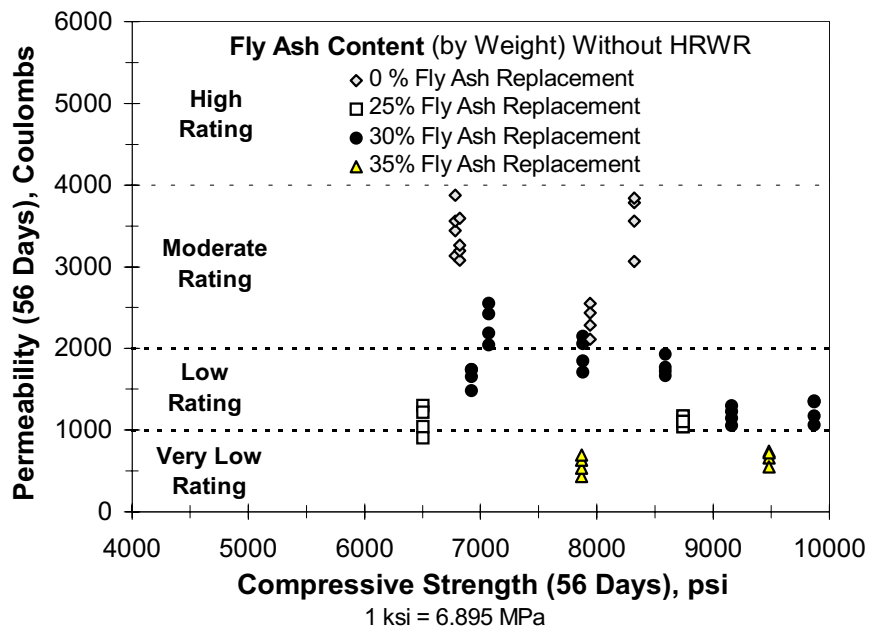


Figure 6.12: Permeability versus Compressive Strength for the Laboratory Trial Mixes without HRWR

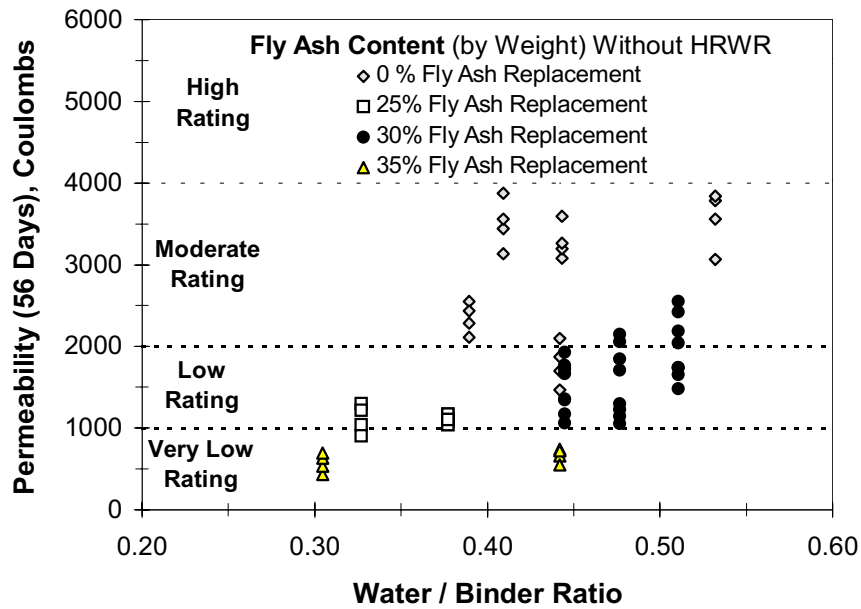


Figure 6.13: Permeability versus Water-to-Binder Ratio for the Laboratory Trial Mixes without HRWR

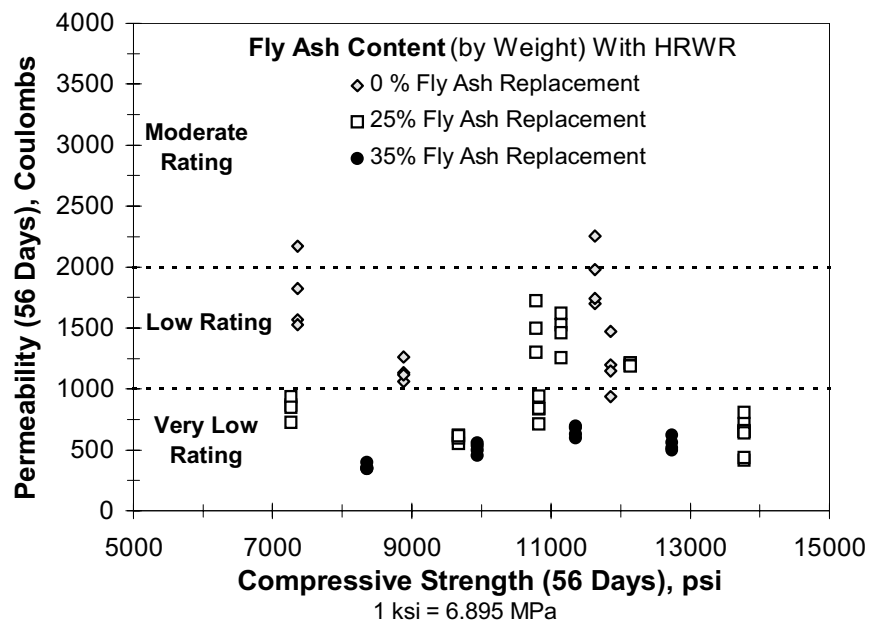


Figure 6.14: Permeability versus Compressive Strength for the Laboratory Trial Mixes with HRWR

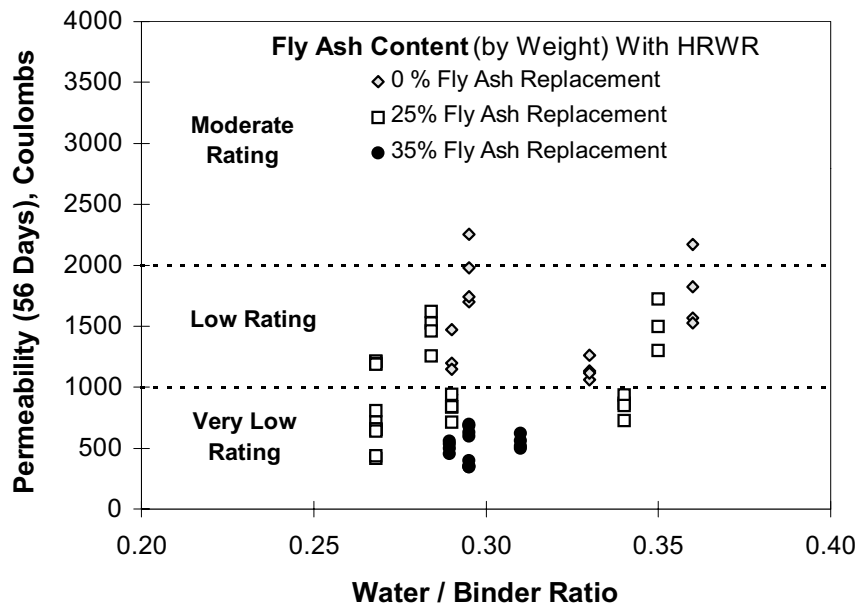


Figure 6.15: Permeability versus Water-to-Binder Ratio for the Laboratory Trial Mixes with HRWR

Figures 6.13 and 6.15 illustrate the permeability versus the water-to-binder ratio or the water-to-cementitious material ratio (w/cm) for the respective trial mixes. The mix designs incorporating 35 percent fly ash replacement exhibited the lowest permeability for a given w/cm ratio (Figures 6.13 and 6.15). Of particular interest is that today, with the use of mineral and chemical admixtures, the w/cm ratio is no longer an acceptable means to ensure a low-permeable concrete. Some construction specifications today dictate a w/cm ratio range as a means to address permeability and durability. This may have been an effective means in the days prior to the widespread use of mineral and chemical admixtures, but is no longer applicable today, particularly for HPCs. It may be noted that in the laboratory trial mixes without an HRWR and with a w/cm ratio of around 0.45 (Figure 6.13), the resulting permeability ranged between 500 and 4,000 coulombs passed. Quite a large variation in the permeability performance of the mix designs may be noted depending on the mix proportioning and constituents.

Similarly, for the laboratory trial mixes with an HRWR and with a water-to-binder ratio of around 0.30 (Figure 6.15), the resulting permeability ranged between 500 and 2,500 coulombs passed. Although this range of permeability performance varies for any given w/cm ratio, this pattern would be consistent at any given w/cm ratio provided enough trial mixes are performed with varying mixture proportions and constituents.

In fact, even with the limited number of lab trial mixes prepared, both Figure 6.13 and Figure 6.15 exhibit the formation of bands or regions based on the mix designs with and without an ASTM Class C fly ash. This is schematically depicted in Figure 6.16. Even though these bands are distinguishable, a conclusive assessment of the bands or regions cannot be drawn based on the limited amount of test data to date. This does, however, support the notion that today's HPCs are clearly engineered concretes designed to address single or multiple performance issues.

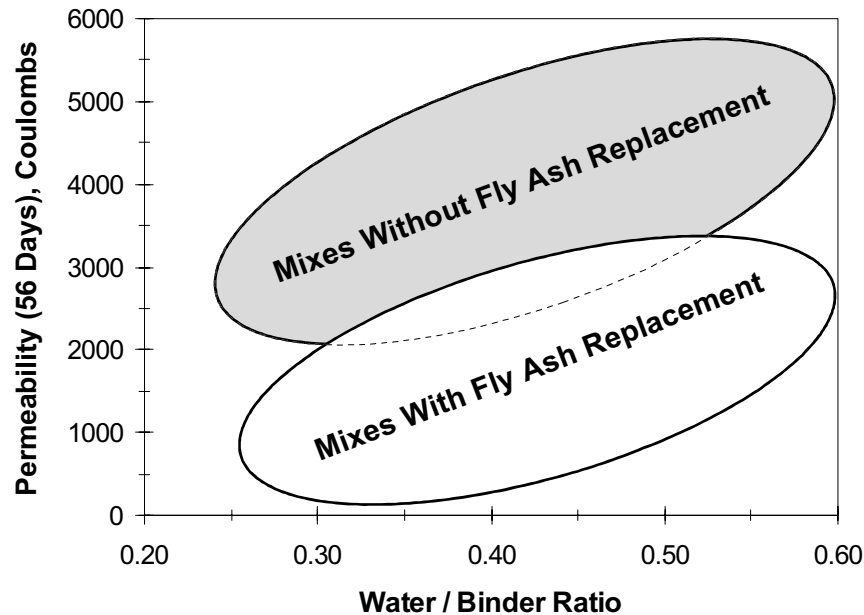


Figure 6.16: Bands of Permeability Performance with and without Fly Ash Replacement based on Laboratory Trial Mixes

Recently, several research studies and publications [123] have questioned the correlation validity of the Rapid Chloride Permeability Test (RCPT) (AASHTO T277) to the Chloride Pondering Test (AASHTO T259) for HPCs and other special concretes. This research topic is part of a 14-month research study conducted at the Construction Materials Research Group at The University of Texas at Austin on the correlation of HPCs. This investigation is referenced in Chapter 10 of this report.

6.5.4 TxDOT Replacement Study for Non HRWR HPC Mixes

Because of placement concerns associated with many HS/HPC mix designs and the increased potential for plastic shrinkage cracking, many design engineers are examining ways to specify durable concrete without requiring an HRWR or a low w/cm ratio. In order to improve the

durability of bridge decks within the infrastructure, research studies such as this replacement study are examining ways to specify mix designs that reduce permeability, but do not adversely affect the workability and finishability of the mix design. In many cases, the high strength that is associated with many mix designs incorporating HRWRs is not required or warranted for bridge decks.

The 1995 TxDOT Standard Specifications for Construction of Highways, Streets and Bridges [72] requires a minimum cement content and maximum w/cm ratio for Class S and Class C concrete. These specification requirements traditionally have been the primary means of addressing durability performance and were developed to provide a high-quality moderate- to low-permeable concrete for use in concrete structures. As discussed in the previous section, with the advent of many chemical admixtures and cementitious replacement materials, the current prescriptive method of minimum cement contents and maximum w/cm ratios to address durability is no longer the most effective approach.

It is suggested that a performance-based system would be a more effective approach for addressing durability in concrete structures. To date, the standard TxDOT specification does not stipulate any performance-based permeability requirements, although several TxDOT projects, including the Louetta Road Overpass and North Concho River Overpass Projects, have had project guidelines for permeability performance. This replacement study presents the current TxDOT specification requirements for Class S and Class C concrete and investigates how a more durable, less expensive concrete may be produced utilizing a lower cementitious content per cubic yard in combination with an ASTM Class C fly ash.

6.5.4.1 TxDOT Specification Requirements. The current TxDOT Standard Specification requirements for Class C and Class S Concretes are presented in Table 6.45. Only Class C and Class S concretes are presented in this discussion because these are the concretes primarily specified for use in bridge structures. Of key interest is the required minimum cement dosage per cubic yard. Six (Class C) and 6 1/2 (Class S) sacks of cement (std. US) are required for the respective classes. With locally available materials in Texas, the compressive strength and the flexural strength requirements are not difficult to meet with the minimum cement content required. This is consistent with the test results presented in this replacement study.

Table 6.45: TxDOT Standard Specifications for Classes of Concrete [72]

Class of Concrete	Cement per Cubic Yard (Cubic Meter)	Minimum Compr. Strength 28 days psi (MPa)	Minimum Flexural Strength 7 days psi (kPa)	Maximum Water-to-Cement Ratio gal/sack (L/kg)	Coarse Aggregate Grade Number	General Use
C	6.0 (335)	3600 (25)	510 (3520)	6.0 (0.53)	1-2-3-4-5	Drilled Shafts, Bridge Sub-structures
S	6.5 (365)	4000 (28)	570 (3930)	5.0 (0.44)	2-3-4-5	Bridge Slab, Wearing Surfaces

The current TxDOT allowance for fly ash is as follows [72]:

When fly ash is used, “cement” shall be defined as “cement plus fly ash.” “Cement plus fly ash” shall be composed of Type I, II, or III portland cement and 20 to 35 percent fly ash by absolute volume, except that for classes of concrete which are specified to have less than 5 sacks of portland cement per cubic yard, the fly ash replacement of cement shall not exceed 25 percent by absolute volume of the specified cement content.

6.5.4.2 Mix Design Description. Two series including five mix designs each were selected for this study. These included five mix designs incorporating a 13-mm (0.5-in.) Burnet limestone, designated LSC1 through LSC5, and five mix designs incorporating a 19-mm (0.75-in.) river gravel, designated RGC1 through RGC5. The mix proportions and fresh concrete properties for these mix designs are presented in Table 6.3. The mix designs were developed to simulate typical design mixes that would be used for normal-strength precast elements (LSC Series) such as precast deck panels and for normal-strength CIP bridge decks or bridge substructures (RGC Series).

A Type III cement was selected for the LSC Series because Type III cements are commonly used in the precast industry to meet release strength requirements. A Type I or II cement is typically used on bridge decks or bridge substructures throughout Texas. The Type II cement would be specified in areas of the state or country where sulfate attack may be a concern. For this reason a cement meeting the requirements of a Type I/II cement was selected for the RGC Series.

The two aggregate sources that are most widely used in Texas are limestone and river gravel. Of these two materials, concrete producers in Texas generally select the aggregate that is most readily available and cost effective. For this reason the aggregate was varied within the study. Mix designs LSC1 and RGC1 are TxDOT Class S mix designs with 6 1/2 sacks of cement per cubic yard

without fly ash replacement. Mix designs LSC2 and RGC2 are TxDOT Class C mix designs with 6 sacks of cement per cubic yard without fly ash replacement. Mix designs LSC3-LSC5 and RGC3-RGC5 incorporated a 30 percent fly ash replacement with a 1/2-sack reduction in cementitious material, respectively (5.5, 5.0, and 4.5 sacks).

The water usage was based on a maximum allowable amount per Table 6.45 or 89-mm (3.5-in.) slump. The 89-mm (3.5-in.) slump was the minimum desirable for workability and placement. It may be noted that some of the mix designs incorporating fly ash replacement actually exceed the recommended water dosage specified by TxDOT in Table 6.45 in order to meet the minimum slump requirements of 89 mm (3.5 in.). This higher water-to-cementitious ratio does emphasize an interesting point. As presented in the test results, durable concrete can be produced with less cement and higher than standard water-to-cementitious ratios than generally presented in most specifications if proper constituents are selected to meet durability concerns.

6.5.4.3 Experimental Results and Discussion. The following test results are presented to compare standard mix designs representing TxDOT Class S and Class C with fly ash replacement mix designs at a lower cementitious content. Each series of mix designs starts with a 6.5-sack mix and incrementally are reduced by 1/2 sack of cementitious material to 4.5 sacks as described in Section 6.5.4.2. Figure 6.17 illustrates the strength gain with time of the LSC Series mix designs. It may be noted that mix designs LSC3, LSC4, and LSC5 continue to gain strength at later ages due to the Class C fly ash replacement. The pozzolanic action of the fly ash is particularly noticeable from 7 to 28 days when there is a much sharper strength gain for the mix designs with the fly ash replacement when compared to mix designs without. The added strength gain may also be attributed to the reduced water demand of the mix due to the particle shape (spherical and small in size) of the fly ash. Mix designations LSC1 and LSC2, which do not incorporate fly ash replacement, level off in strength gain at about 56 to 84 days. This limitation in strength gain is also associated with the characteristics of the Type III cement used in these mix designs. The fineness of the cement particles or high blaine of the cement resulted in additional hydration and strength gain at early ages. This, along with the better bond characteristics of the smaller angular LSC series aggregate, also explains why the LSC series developed slightly higher short-term and long-term (at 140 days) strength when compared to the RGS series. Additionally, Figure 6.18 illustrates a similar pattern for the RGC series mix designs except that mix designations RGC1 and RGC2 have appreciably more strength gain at later ages when compared to LSC1 and LSC2. This may be attributed to the Type I/II cement used in the RGC series mix designs. The Type I/II cement particles had a lower blaine than the Type III cement, which resulted in additional hydration of cement particles at later ages. It is clearly evident as

illustrated in Figures 6.17 and 6.18 that all of the mix designs exceeded the minimum compressive strength requirements at 28 days per Table 6.45.

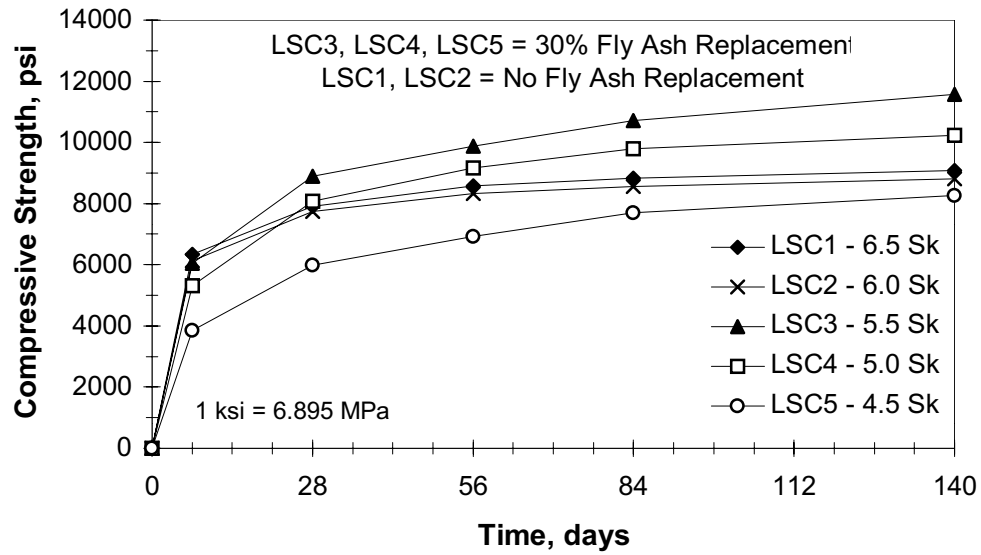


Figure 6.17: Compressive Strength Development with Time for LSC Series

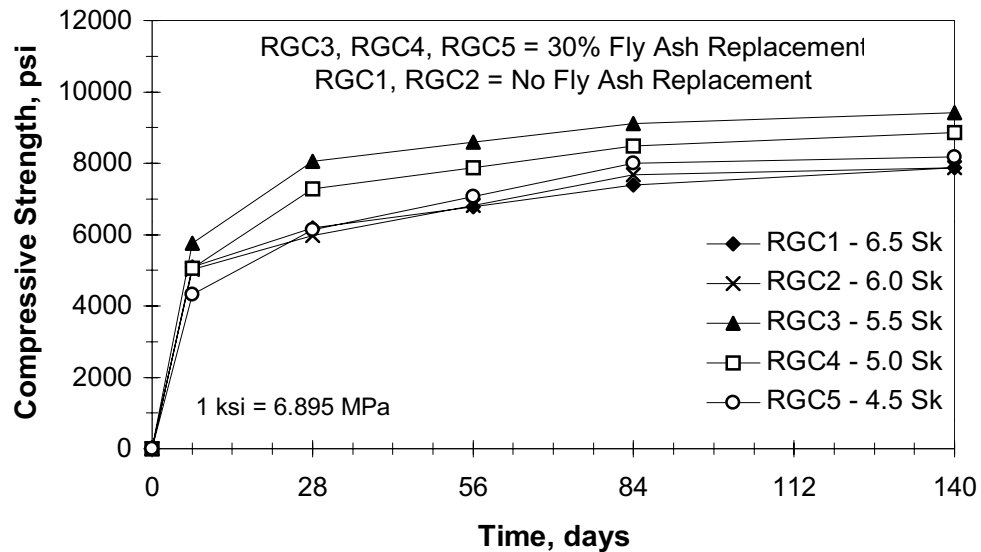


Figure 6.18: Compressive Strength Development with Time for RGC Series

Figure 6.19 illustrates the strength gain of the mix designs at 140 days. This figure gives a comparison between the mix designs with and without fly ash replacement from a strength gain point of view. Clearly the mix designs including the Class C fly ash replacement closely matched or exceeded the TxDOT Class S concrete (6.5 of cement) without fly ash replacement.

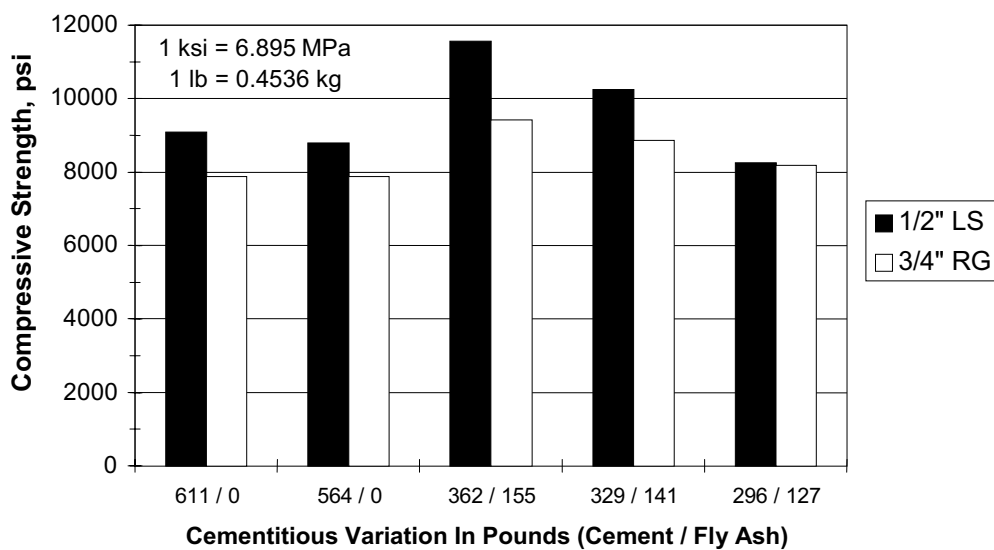


Figure 6.19: Compressive Strength Results at 140 Days for LSC and RGS Series

The 13-mm (0.5-in.) limestone aggregate series generally exhibited higher strength gain than the 19-mm (0.75-in.) river gravel aggregate series (Figure 6.19). This may be attributed to the shape, texture, and size of the two coarse aggregates as well as the cements used. The limestone is angular and rough, whereas the river gravel is generally round and smooth. Furthermore, the smaller gradation of the limestone tends to make the mix act more homogeneously even though the surface area of the smaller rock increases and, therefore, the cement or binder demand is increased. The smaller aggregate has a higher curvature and therefore much better bond characteristics. The bond is typically the “weak link in the chain” for normal-strength concretes as discussed in Section 2.1.

In terms of the elastic modulus, the aggregate type and aggregate content predominantly influence the elastic modulus of the concrete. The coarse aggregate also helps control shrinkage and is responsible for the wear, abrasion, and skid resistance of the hardened concrete. In the case of normal-strength concrete [<41.4 MPa (6,000 psi)], the elastic modulus is usually not influenced significantly by these factors because the failure mechanism is generally a bond failure at a relatively low stress level. For bridge structures, the elastic modulus is particularly critical from a serviceability

standpoint. The mix designs incorporating the 13-mm (0.5-in.) Burnet limestone exhibited a slightly higher elastic modulus when compared to the identical mix designs incorporating the river gravel (Figure 6.20). This may be attributed to the fact that the Burnet limestone is denser than the river gravel and the compressive strengths of the mix designs generally exceeded 55.2 MPa (8,000 psi). For these mix designs, the failure mechanism was not simply a bond failure. From a comparison standpoint, clearly the mix designs including the fly ash replacement closely matched or exceeded the TxDOT Class S concrete elastic modulus even with the reduced content of cementitious material.

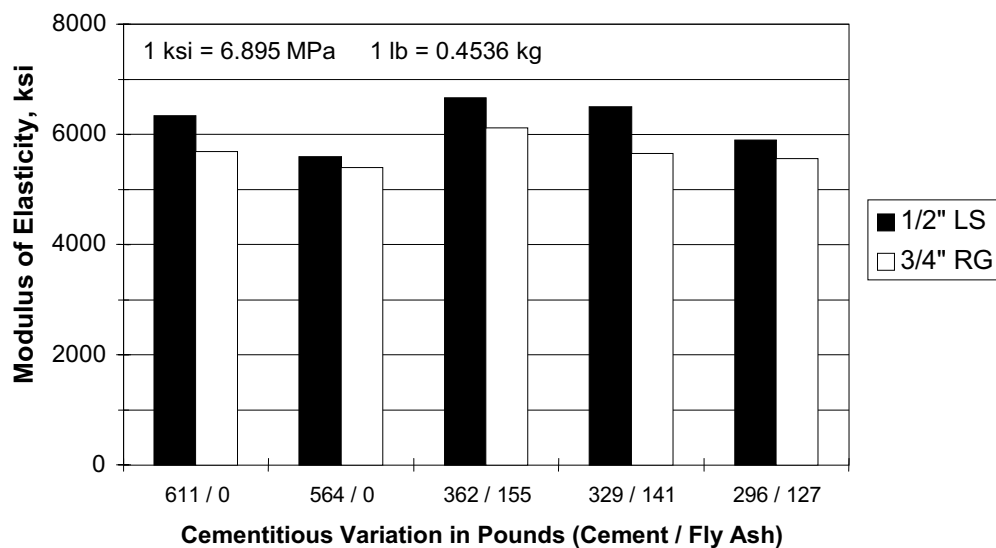


Figure 6.20: Elastic Modulus Development at 56 Days for LSC and RGS Series

Improved durability of concrete in exposed structures is significant from a cost standpoint because durable concrete translates into lower maintenance costs and an extended life expectancy for a structure. Permeability is one measure that can be used to estimate the durability performance of concrete. Permeability controls the rate of entry of moisture that may contain aggressive chemicals and the movement of water during heating or freezing. The two parameters that have the largest influence on durability are the w/cm ratio and the constituents in the mix design. The w/cm ratio plays a dual role in concrete durability. A lower w/cm ratio increases the strength of concrete and, hence, improves the concrete's resistance to cracking from internal stresses generated by adverse reactions. The addition of many mineral admixtures such as a Class C fly ash replacement generally increases the strength gain of concrete, which translates into improved durability. However, some fly ashes may actually decrease resistance to aggressive chemical attack such as sulfate attack if the chemical

composition of the fly ash contains too much sulfate or aluminate [51]. With respect to the Class C fly ash used in these mix designs, designated “J” in Table 5.10, excessive sulfates or aluminates are not a primary concern.

From a comparison standpoint, Figure 6.21 presents the RCPT (AASHTO T277) results for these mixes at 56 days. It may be noted, that in all cases the mix designs incorporating fly ash replacement exhibited lower permeability than the standard Class S and C concretes. The Burnet limestone mix designs exhibited lower permeability than their counterpart river gravel mix designs. Even though the river gravel aggregate is less porous than the Burnet limestone aggregate, the Burnet limestone mixes resulted in higher compressive strength than their counterparts and, thereby, a “tighter” paste matrix (fewer continuous capillaries) with more resistance to moisture penetration.

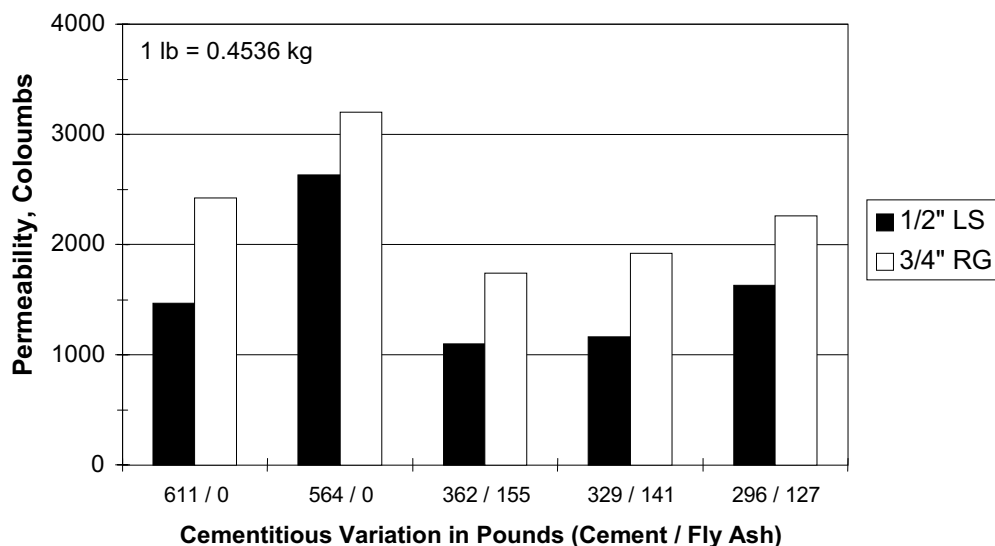


Figure 6.21: Permeability Results at 56 Days for LSC and RGS Series

Naturally, the direct construction costs and long-term maintenance costs are a primary concern for any construction project. Construction costs not only include the raw materials, but also the labor required to construct the project. In terms of the concrete mix designs, the labor costs do not vary in the placement and finishing of the concrete unless a slight savings in labor is realized due to improved pumpability and finishing characteristics of the mix designs including the fly ash replacement. In any case, these savings would not be anticipated to be extensive.

However, this is a different situation from the raw materials standpoint. For the cost comparison of the cementitious material illustrated in Figure 6.22, the following cementitious cost data was assumed based on a conversation with a local cement producer [106]:

Capital Type I/II Cement - \$0.08267 per kg (\$75.00 per ton)

Capital Type III Cement - \$0.08818 per kg (\$80.00 per ton)

JTM Class C Fly Ash - \$0.02425 per kg (\$22.00 per ton)

It is clearly evident that a significant cost saving in raw materials can be attained through the use of fly ash replacement (Figure 6.22). Not only do mix designations LSC3 and RGC3 result in higher strength, higher elastic modulus, decreased permeability, and improved durability, but also a 33 percent cost saving in cementitious materials when compared to the standard TxDOT Class S mix design. This cost saving does not include the life cycle cost savings that would be realized from the improved durability of the mix designs.

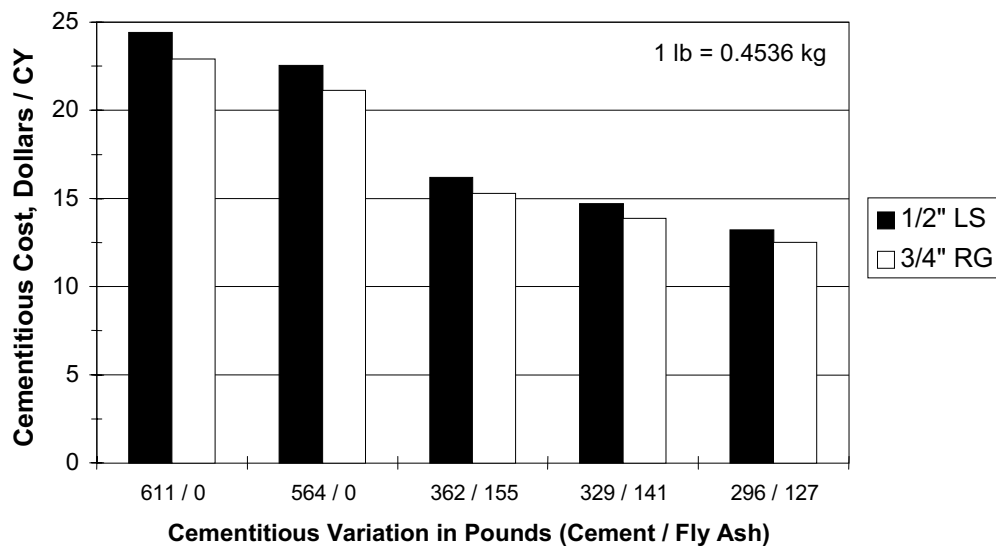


Figure 6.22: Cementitious Cost Comparison for LSC and RGS Series

Perhaps the most appropriate recommendation for a modification to the current TxDOT specifications would be to allow for an alternate performance-based system. Maintain the current minimum cement content and maximum w/cm ratio requirements as presented in Table 6.45 unless a performance-based alternate is satisfied.

The current prescriptive specification does provide an upper threshold of durability performance, although not optimal. The potential cost savings, as presented in this study, would

provide the concrete producer with incentive or motivation to investigate the alternate performance criteria option and also provide the concrete producer the flexibility to achieve optimized mix designs from a durability standpoint. This alternate performance specification could require a maximum permeability rating of 2,000 coulombs at 28 days for Class S concrete and 3,000 coulombs at 28 days for Class C concrete in lieu of the current minimum cement content and maximum water to cementitious ratio requirements based on the AASHTO T277 RCPT method. In this manner, the concrete produced would result in durable concrete that equals or exceeds concrete produced through the traditional methodology.

Other mechanical or material durability factors such as elastic modulus, tensile strength, freeze-thaw resistance, scaling resistance, and abrasion resistance could be addressed in the same alternative performance option. The alternative performance method option could be modified to become a full performance-based specification system after a transitional period when the concrete producers have developed a familiarity with the performance tests and realized the economic benefits from optimizing their mix designs.

6.6 Performance-Related Test Results of Plant-Produced Concrete

Following the verification of concrete property requirements in the plant trial phase, production of concrete for the structures at the respective precast and ready-mix plants was initiated. Tables 6.47 through 6.66 present the performance-related test results for all of the concrete produced for the respective bridge components on both bridge projects. In all, three different types of aggregates were used in production at the various precast and ready-mix plants including crushed river gravel, Burnet limestone, and a south Texas limestone. It may be noted that only in the case of the precast beams was a specific aggregate (Burnet limestone) required to meet the project specifications other than what was currently being used in production at the respective plant. In all other cases, the project specifications were satisfied in the laboratory and field trial mix phases by the plant's primary aggregate source. Only when project specifications were not met were "outside" aggregates considered. This was to maintain the most economical mix design possible. Table 6.46 summarizes the aggregates selected for use in the various HPC components.

Table 6.46: Aggregate Type Selected for HPC Components

HPC Component	High Performance Concrete Bridge Project	
	Louetta	San Angelo
Precast Beams	Burnet Limestone	Burnet Limestone
Precast Deck Panels	Crushed River Gravel	South Texas Limestone
Precast Columns	Crushed River Gravel	<i>Not Applicable</i>
Cast-in-Place Piers	<i>Not Applicable</i>	Crushed River Gravel
Cast-in-Place Pier Caps	<i>Not Applicable</i>	Crushed River Gravel
Cast-in-Place Decks	South Texas Limestone	Crushed River Gravel

Table 6.47: Louetta HPC U-Beam Results—Compressive Strength

Casting Date		9/23/94	9/30/94	10/7/94	10/28/94	11/3/94	11/10/94	2/15/96	2/26/96	3/8/96	As Noted
Release Time		18 hrs	22 hrs	7 days	35 hrs	21 hrs	25 hrs	26 hrs	28 hrs	7 days	
Release Strength (psi) Light Steam Curing 11/3/94	ASTM	5450	7390	11090	7030	NR	7220	7860	7940	10030	12120 ①
	MC	8450	8710	8880	9320	9700	8650	9710	10010	10840	11670 ①
	SB	9090	9590	11460	10780	NR	11090	10500	NR	NR	NR
	SS	9510	10110	11260	11240	10120	11240	NR	10860	NR	NR
	SW	10000	7360	11260	11230	10640	10350	10530	10820	NR	NR
	SF	11070	7180	11100	10740	NR	10320	10470	10990	NR	12130 ①
	HYD	9550	9430	12430	10780	NR	11180	9630	9750	NR	NR
28-Day Strength (psi)	ASTM	13960	13740	14980	NR	NR	15010	13420	14410	12060	12650 ②
	MC	11500	13290	13560	NR	NR	13410	12710	11980	12580	11970 ②
	SB	11730	11940	13370	NR	NR	13270	NR	NR	NR	NR
	SS	12260	12450	13080	NR	NR	12810	NR	NR	NR	NR
	SW	12570	13380	13300	NR	13680	13320	NR	NR	NR	NR
	SF	12700	13340	13100	NR	NR	13480	NR	NR	NR	12390 ②
	HYD	11540	12030	13075	NR	NR	13120	NR	NR	NR	NR
	TxDOT	12420	14430	14010	13990	14320	14550	NR	NR	NR	NR
56-Day Strength (psi)	ASTM	NR	NR	NR	15120	NR	NR	14640	15120	12390	
	MC	NR	NR	NR	14820	NR	NR	13630	14320	13050	
	SB	NR	NR	NR	13480	NR	NR	12500	NR	NR	
	SS	NR	NR	NR	12760	13120	NR	NR	12920	NR	
	SW	NR	NR	NR	NR	14480	NR	12620	13010	NR	
	SF	NR	NR	NR	13640	NR	NR	12560	12270	NR	
	HYD	NR	NR	NR	13120	NR	NR	NR	NR	NR	
	TxDOT	NR	NR	NR	15290	15980	NR	NR	NR	NR	
① 7-day results 2/15/96; ② 7-day results 2/26/96 ASTM: ASTM Moist Cured MC: Member-Cured Cylinders SB: Skewed End Block Match-Cured Cylinders (36" from bottom of form) SS: Nonskewed End Block Match Cured (36" from bottom of form) SW: Web Match-Cured Cylinders (28" from bottom of form) SF Flange Match-Cured Cylinders (46" from bottom of form) HYD: Match-Cured Cylinders based on Hydration Chamber TxDOT: Texas DOT-Cured Cylinders NR: Testing not planned, therefore, results not reported. 1 ksi = 1,000 psi = 6.895 MPa 1 inch = 25.4 mm											

Table 6.48: Louetta HPC U-Beam Results—Elastic Modulus

Casting Date		9/23/94	9/30/94	10/7/94	10/28/94	11/3/94	11/10/94	2/15/96	2/26/96	3/8/96	As Noted
Release Time		18 hrs	22 hrs	7 days	35 hrs	21 hrs	25 hrs	26 hrs	28 hrs	7 days	
Release MOE (ksi) Light Steam Curing 11/3/94	ASTM	5180	5140	6410	5340	NR	5650	5860	5730	5590	6270 ①
	MC	NR	NR	NR	NR	NR	NR	6190	5720	5880	6380 ①
	SB	5870	6380	6320	5840	NR	6290	6190	NR	NR	NR
	SS	NR	NR	NR	NR	NR	NR	NR	5900	NR	NR
	SW	NR	NR	NR	5850	NR	NR	6530	5920	NR	NR
	SF	6420	5110	6380	5300	NR	5480	6290	6020	NR	6390 ①
28-Day MOE (ksi)	ASTM	7080	7030	7760	NR	NR	6890	6300	6220	6370	6350 ②
	MC	5730	6930	7550	NR	NR	6460	6540	5580	6620	6210 ②
	SB	6370	6320	6960	NR	NR	6520	NR	NR	NR	NR
	SS	NR	NR	NR	NR	NR	NR	NR	NR	NR	NR
	SW	NR	NR	NR	NR	NR	NR	NR	NR	NR	NR
	SF	6360	6290	6710	NR	NR	6420	NR	NR	NR	6290 ②
56-Day MOE (ksi)	ASTM	NR	NR	NR	6760	NR	NR	6720	6730	6670	
	MC	NR	NR	NR	6710	NR	NR	6760	6680	7050	
	SB	NR	NR	NR	6200	NR	NR	6420	NR	NR	
	SS	NR	NR	NR	NR	NR	NR	NR	6030	NR	
	SW	NR	NR	NR	NR	NR	NR	5730	6150	NR	
	SF	NR	NR	NR	6840	NR	NR	5660	6260	NR	
① 7-day results 2/15/96; ② 7-day results 2/26/96 ASTM: ASTM Moist Cured MC: Member-Cured Cylinders SB: Skewed End Block Match-Cured Cylinders (36" from bottom of form) SS: Nonskewed End Block Match Cured (36" from bottom of form) SW: Web Match-Cured Cylinders (28" from bottom of form) SF Flange Match-Cured Cylinders (46" from bottom of form) HYD: Match-Cured Cylinders based on Hydration Chamber TxDOT: Texas DOT-Cured Cylinders NR: Testing not planned, therefore, results not reported. 1 ksi = 1,000 psi = 6.895 MPa 1 inch = 25.4 mm											

Table 6.49: North Concho River Overpass Type IV HPC Beam Results—Compressive Strength

Casting Date		4/1/96	2/19/97	2/25/97	3/3/97	3/8/97	3/15/97	3/22/97	3/29/97	4/7/97	4/12/97	4/18/97	4/28/97
Release Time		27 hrs	28 hrs	24 hrs	23 hrs	50 hrs	48 hrs	46 hrs	46 hrs	24 hrs	48 hrs	26 hrs	28 hrs
Release Strength (psi) Lt. Steam Curing 2/25/97	ASTM	6310	7560	7460	6240	9390	8760	9100	9410	7090	8710	7530	7740
	MC	9790	9940	8590	9950	11730	9960	9690	9350	10010	9890	9960	8500
	SB	NR	12070	10840	11970	12290	12120	12410	13460	11540	10560	12730	12330
	SW	NR	10780	9490	NR	12090	NR	11970	11340	11850	9390	11830	11790
	SF	NR	11060	NR	11290	13070	11640	12390	NR	11180	10500	12470	12280
7-Day Strength (psi)	ASTM	11760	12090	12400	13130	12060	11260	11510①	NR	11960	10670①	12050	12120
	MC	12350	10880	12420	12070	13200	12120	12020①	11600①	11130	11580①	11590	12270
	SB	NR	NR	NR	12960	NR	NR	12710①	NR	13630	NR	NR	12950
	SW	NR	11630	12540	NR	NR	12540	NR	12500①	NR	11270①	NR	NR
	SF	NR	NR	NR	13070	14240	NR	NR	NR	NR	NR	13040	NR
28-Day Strength (psi)	ASTM	13570②	14470	14920	15310	13920	13280	14260	15180	13630	14130	13900	14440
	MC	13450②	12980	13950	13180	13640	13670	13750	14100	12400	13730	12930	12920
	SB	NR	13750	NR	14070	13190	13140	13090	14530	12650	12930	13310	12560
	SW	NR	13280	NR	NR	12890	12690	13190	14130	12980	13590	13300	12940
	SF	NR	13370	NR	13960	13730	13190	13140	NR	13050	NR	13440	12610
	TxDOT	NR	12720	14710	13610	13300	13380	13640	NR	12970	14450	13220	14260
56-Day Strength (psi)	ASTM	14790	15240	15930	16040	15010	14690	15360	15360	15000	14590	14350	15320
	MC	14830	13700	14300	13830	14240	14550	14530	14490	12570	14550	13830	13590
	SB	NR	13850	14830	14530	13870	13210	13330	NR	12940	13370	13910	13100
	SW	NR	13980	15220	NR	13750	13880	13550	14270	13460	13640	13570	13170
	SF	NR	13860	NR	14570	14810	13910	13150	NR	13410	NR	13640	13170
	TxDOT	NR	13590	15630	14740	14880	14530	14790	15280	14370	15200	14830	15110
	SC-MC	NR	14650	14530	14570	14790	14590	13670	15530	13460	14700	14860	13980
① tested at 5-days ② tested at 25-days (at Post Tensioning)		SF Flange Match-Cured Cylinders (46" from bottom of form)											
ASTM: ASTM Moist Cured		TxDOT: Texas DOT-Cured Cylinders											
MC: Member-Cured Cylinders		NR: Testing not planned, therefore, results not reported.											
SB: End Block Match-Cured Cylinders (36" from bottom of form)		1 ksi = 1,000 psi = 6.895 MPa											
SW: Web Match-Cured Cylinders (28" from bottom of form)		1 inch = 25.4 mm											

Table 6.50: North Concho River Overpass Type IV HPC Beam Results—Elastic Modulus

Casting Date		4/1/96	2/19/97	2/25/97	3/3/97	3/8/97	3/15/97	3/22/97	3/29/97	4/7/97	4/12/97	4/18/97	4/28/97
Release Time		27 hrs	28 hrs	24 hrs	23 hrs	50 hrs	48 hrs	46 hrs	46 hrs	24 hrs	48 hrs	26 hrs	28 hrs
Release MOE (ksi) Lt. Steam Curing 2/25/97	ASTM	5220	5250	5710	5150	5850	NR	5910	6150	5150	5530	4960	5100
	MC	5950	5640	5540	5970	6080	NR	6010	6140	5440	5720	5220	5000
	SB	NR	6320	6800	6590	6640	NR	6660	6920	6100	5790	6040	6210
	SW	NR	6320	6140	NR	6600	NR	6310	6690	6040	5580	6040	5880
	SF	NR	6520	NR	6500	6970	NR	6310	NC	6310	5720	5970	5940
7-Day MOE (ksi)	ASTM	5780	6480	6350	6570	6300	6110	5980 ①	NR	6460	5730 ①	6180	5820
	MC	6050	5730	6040	6230	6350	6260	6040 ①	5800 ①	5900	5840 ①	5840	5460
	SB	NR	NR	NR	6020	NR	NR	6120 ①	NR	6470	NR	NR	5760
	SW	NR	6110	6100	NR	NR	6220	NR	6400 ①	NR	5250 ①	NR	NR
	SF	NR	NR	NR	6160	6900	NR	NR	NR	NR	NR	5760	NR
28-Day MOE (ksi)	ASTM	7020 ②	6520	6640	6730	6490	6600	6070	6660	5980	6760	6450	6130
	MC	6550 ②	6640	6400	6250	6530	6560	6100	6430	6000	6640	5980	5570
	SB	NR	6230	NR	6430	6190	6430	5590	6360	5380	6720	6130	5500
	SW	NR	6220	NR	NR	6420	6360	5500	5940	5410	6750	6030	5930
	SF	NR	6150	NR	6420	6170	6300	6150	NR	5410	NR	6040	5960
	TxDOT	NR	6330	6690	6800	6140	6320	6160	NR	6140	6340	6460	6250
56-Day MOE (ksi)	ASTM	7080	6760	6700	7070	6630	6900	6370	6810	6750	7110	6500	6950
	MC	6580	6460	6620	6260	6760	6630	6140	6490	6100	6850	6090	5910
	SB	NR	6070	6530	5830	6410	5560	6040	NR	6180	6110	6260	6420
	SW	NR	6230	7050	NR	6360	6000	5950	6320	6640	NR	6110	6020
	SF	NR	6010	NR	5730	6550	5770	5990	NR	5950	6670	6170	6040
	TxDOT	NR	6470	6830	7370	6550	6750	6830	6750	7080	7290	6630	6560
		SC-MC	NR	6850	6300	6610	6790	6700	6490	6970	6650	6770	6480
① tested at 5 days ② tested at 25 days (at Post Tensioning)		SF Flange Match-Cured Cylinders (46" from bottom of form)											
ASTM: ASTM Moist Cured		TxDOT: Texas DOT-Cured Cylinders											
MC: Member-Cured Cylinders		NR: Testing not planned, therefore, results not reported.											
SB: End Block Match-Cured Cylinders (36" from bottom of form)		1 ksi = 1,000 psi = 6.895 MPa											
SW: Web Match-Cured Cylinders (28" from bottom of form)		1 inch = 25.4 mm											

Table 6.51: Louetta U-Beam and N.C.R.O. Bridge Type IV HPC Beam Results—Splitting Tensile Strength (STS)

Casting Date		2/15/96	2/26/96	3/8/96	2/19/97	2/25/97	3/3/97	3/15/97	3/22/97	3/29/97	4/7/97	4/12/97	4/18/97	4/28/97
Release Time		26 hrs	28 hrs	7 days	28 hrs	24 hrs	23 hrs	48 hrs	46 hrs	46 hrs	24 hrs	48 hrs	26 hrs	28 hrs
Member Type		U-Beam	U-Beam	U-Beam	Type IV	Type IV	Type IV	Type IV	Type IV	Type IV	Type IV	Type IV	Type IV	Type IV
28-Day STS (psi)	ASTM	NR	1080	860	NR	NR	1010	1130	1080	NR	930	NR	1050	1120
	MC	NR	920	900	NR	NR	1040	1100	1070	NR	950	NR	930	990
	SB	NR	NR	NR	NR	NR	NR	NR	NR	NR	NR	NR	NR	NR
	SF	NR	NR	NR	NR	NR	NR	NR	NR	NR	NR	NR	NR	NR
56-Day STS (psi)	ASTM	1030	1000	940	1290	990	1050	1060	1050	1080	1050	980	1080	1080
	MC	970	940	980	1060	1000	1190	1020	1000	1060	1040	1040	810	960
	SB	NR	NR	1220	NR	NR	NR	NR	NR	NR	NR	NR	NR	NR
	SF	NR	NR	1100	NR	NR	NR	NR	NR	NR	NR	NR	NR	NR
ASTM: ASTM Moist Cured					TxDOT: Texas DOT-Cured Cylinders									
MC: Member-Cured Cylinders					NR: Testing not planned, therefore, results not reported.									
SB: End Block Match-Cured Cylinders (36" from bottom of form)					1 ksi = 1,000 psi = 6.895 MPa									
SW: Web Match-Cured Cylinders (28" from bottom of form)					1 inch = 25.4 mm									
SF Flange Match-Cured Cylinders (46" from bottom of form)														

Table 6.52: Hardened Concrete Properties for Plant-Produced Concrete

Mix Designation	CP 4-23	CP 4-25	CP 4-25	CP 5-23	CP 6-5	CP 6-5	CP 6-24	CP 6-24
Casting Date	4-23-96	4-25-96	4-25-96	5-23-96	6-5-96	6-5-96	6-24-96	6-24-96
Specimen Curing	<i>Moist</i>	<i>Moist</i>	<i>Site</i>	<i>Moist</i>	<i>Moist</i>	<i>Site</i>	<i>Moist</i>	<i>Site</i>
Hardened Concrete Properties								
Compressive Strength (psi)								
7 days	NR	3480	NR	4460	4640	NR	NR	NR
28 days	4240	4260	NR	NR	5690	5740	5710	5440
56 days	5080	NR	NR	5870	6330	5780	6260	5460
Modulus of Elasticity (ksi)								
28 days	4870	4830	NR	NR	4770	4470	5070	4500
56 days	5020	NR	NR	5330	5320	4840	5590	4510
Splitting Tensile Strength (psi)								
28 days	470	485	NR	NR	510	340	650	550
56 days	490	NR	NR	520	610	575	620	590
RCPT - Rating	Low	Low	Moderate	Moderate	Mod/Low	High	Low	
56 days	1930	1980	3860	2250	2000	4380	1490	NR
Water-to-Binder Ratio								
W/Cm (C+ FA), by weight	0.45	0.45	0.45	0.39	0.40	0.40	0.38	0.38
Test Methods: AASHTO T2 (ASTM C39), ASTM C469, AASHTO T277 1 ksi = 1,000 psi = 6.895 MPa								
NR: Testing not planned, therefore, results not reported.								

Table 6.53: Hardened Concrete Properties for Plant-Produced Concrete

Mix Designation	CP 6-21	CP 8-13	CP 8-13	CP 8-23	F-1	F-1	F-2	F-2
Casting Date	6-21-96	8-3-96	8-13-96	8-23-96	Trial	Trial	Trial	Trial
Specimen Curing	<i>Moist</i>	<i>Moist</i>	<i>Site</i>	<i>Moist</i>	<i>Moist</i>	<i>Site</i>	<i>Moist</i>	<i>Site</i>
Hardened Concrete Properties								
Compressive Strength (psi)								
7 days	5160	NR	NR	NR	NR	NR	NR	NR
28 days	6160	6380	6040	6230	6100	5740	6250	5410
56 days	6660	6990	7360	6500	6460	5990	6400	5620
Modulus of Elasticity (ksi)								
28 days	5480	4980	5120	5890	5100	4850	5600	4770
56 days	5290	5200	5370	5920	5600	4900	5640	4930
Splitting Tensile Strength (psi)								
28 days	540	560	620	600	NR	NR	NR	NR
56 days	770	620	NR	650	NR	NR	NR	NR
RCPT - Rating	Low	Low			Moderate	Moderate		
56 days	1290	1900	NR	NR	2970	3590	NR	NR
Water-to-Binder Ratio								
W/Cm (C+ FA), by weight	0.36	0.46	0.46	0.39	NR	NR	0.30	0.30
Test Methods: AASHTO T2 (ASTM C39), ASTM C469, AASHTO T277 NR: Testing not planned, therefore, results not reported.								

Table 6.54: Hardened Concrete Properties for Plant-Produced Concrete

Mix Designation	FC 7-31	FC 7-31	FC 8-9	FC 9-5	FC 9-5	FC 9-25	FC 10-30	FC 10-30
Casting Date	7-31-96	7-3-96	8-9-96	9-5-96	9-5-96	9-25-96	10-30-96	10-30-96
Specimen Curing	<i>Moist</i>	<i>Site</i>	<i>Moist</i>	<i>Moist</i>	<i>Site</i>	<i>Moist</i>	<i>Moist</i>	<i>Site</i>
Hardened Concrete Properties								
Compressive Strength (psi)								
7 days	4000	NR	4000	NR	NR	NR	NR	NR
28 days	5300	5230	5500	8210	7170	8330	6610	6590
56 days	5960	5350	5880	NR	NR	9100	7590	7310
Modulus of Elasticity (ksi)								
28 days	4980	4630	4890	5560	5370	6040	5910	5400
56 days	5550	4610	5680	NR	NR	6300	5330	5810
Splitting Tensile Strength (psi)								
28 days	500	500	520	630	720	750	640	750
56 days	560	560	560	NR	NR	700	600	760
RCPT - Rating								
56 days	Low 1240	Low 1690	Low 1040	NR	NR	Very Low 920	Very Low 920	Low 1270
Water-to-Binder Ratio								
W/Cm (C+ FA), by weight	0.35	0.35	0.38	0.32	0.32	0.33	0.32	0.32
Test Methods: AASHTO T2 (ASTM C39), ASTM C469, AASHTO T277 1 ksi = 1,000 psi = 6.895 MPa								
NR: Testing not planned, therefore, results not reported.								

Table 6.55: Hardened Concrete Properties for Plant-Produced Concrete

Mix Designation	FC 10-17	FC 12-9	FC 12-9	K-1	K-1	K-2	K-2	K-3
Casting Date	10-17-96	12-9-96	12-9-96	Field Trial	Field Trial	Field Trial	Field Trial	Field Trial
Specimen Curing	<i>Moist</i>	<i>Moist</i>	<i>Site</i>	<i>Moist</i>	<i>Site</i>	<i>Moist</i>	<i>Site</i>	<i>Moist</i>
Hardened Concrete Properties								
Compressive Strength (psi)								
7 days	NR	NR	NR	NR	NR	NR	NR	8520
28 days	8290	7250	6310	11350	10540	11250	10480	10010
56 days	9460	7360	6590	12220	11270	11920	11070	10670
Modulus of Elasticity (ksi)								
28 days	5870	5580	5550	7110	6390	6410	6360	6050
56 days	5600	5530	5450	7150	6410	6600	6400	6130
Splitting Tensile Strength (psi)								
28 days	470	620	570	NR	NR	NR	NR	NR
56 days	820	720	590	NR	NR	NR	NR	NR
RCPT - Rating								
56 days	NR	NR	NR	NR	Very Low 430	NR	Very Low 740	NR
Water-to-Binder Ratio								
W/Cm (C+ FA), by weight	0.32	0.30	0.30	0.29	0.29	0.29	0.29	0.35
Test Methods: AASHTO T2 (ASTM C39), ASTM C469, AASHTO T277								
NR: Testing not planned, therefore, results not reported.								

Table 6.56: Hardened Concrete Properties for Plant-Produced Concrete

Mix Designation	K-3	KP 5-2	KP 4-29	KP 4-29	KP 5-17	KP 5-17	KP 5-31	KP 6-20
Casting Date	Field Trial	5-2-96	4-29-96	4-29-96	5-17-96	5-17-96	5-31-96	6-20-96
Specimen Curing	Site	Moist	Moist	Site	Moist	Site	Moist	Moist
Hardened Concrete Properties								
Compressive Strength (psi)								
7 days	NR	7460	8520	NR	6150	NR	6630	6580
28 days	8090	10270	10310	8800	7790	7700	9700	8990
56 days	8310	10660	11550	9910	NR	NR	NR	10150
Modulus of Elasticity (ksi)								
28 days	5190	5920	6300	5560	5790	5560	6440	6810
56 days	5260	6120	6850	6290	NR	NR	NR	6840
Splitting Tensile Strength (psi)								
28 days	NR	880	920	800	800	820	640	720
56 days	NR	730	900	710	NR	NR	NR	730
RCPT - Rating								
56 days	NR	Very Low 860	Very Low 680	Very Low 720	NR	NR	Very Low 610	NR
Water-to-Binder Ratio								
W/Cm (C+ FA), by weight	0.35	0.34	0.36	0.36	0.34	0.34	0.33	0.34
Test Methods: AASHTO T2 (ASTM C39), ASTM C469, AASHTO T277 1 ksi = 1,000 psi = 6.895 MPa								
NR: Testing not planned, therefore, results not reported.								

Table 6.57: Hardened Concrete Properties for Plant-Produced Concrete

Mix Designation	KP 6-17	KP 6-17	KP 6-28	KP 6-28	KP 7-8	KP 7-16	KP 7-16	KP 7-25
Casting Date	6-17-96	6-17-96	6-28-96	6-28-96	7-8-96	7-16-96	7-16-96	7-25-96
Specimen Curing	<i>Moist</i>	<i>Site</i>	<i>Moist</i>	<i>Site</i>	<i>Moist</i>	<i>Moist</i>	<i>Site</i>	<i>Moist</i>
Hardened Concrete Properties								
Compressive Strength (psi)								
7 days	8520	NR	7500	NR	6420	6600	NR	7340
28 days	9690	8830	9870	9150	9340	9290	7170	9830
56 days	10420	9110	10440	9340	10090	10290	8840	11110
Modulus of Elasticity (ksi)								
28 days	6330	5310	7050	5870	5810	6430	5390	6370
56 days	6730	5530	7200	6100	6260	6710	6030	7080
Splitting Tensile Strength (psi)								
28 days	710	540	450	550	670	610	540	820
56 days	720	680	830	800	770	820	690	880
RCPT - Rating	Very Low	Very Low	Very Low		Very Low	Very Low		
56 days	390	960	280	NR	500	400	NR	NR
Water-to-Binder Ratio								
W/Cm (C+ FA), by weight	0.36	0.36	0.33	0.33	0.35	0.37	0.37	0.35
Test Methods: AASHTO T2 (ASTM C39), ASTM C469, AASHTO T277								
NR: Testing not planned, therefore, results not reported.								

Table 6.58: Hardened Concrete Properties for Plant-Produced Concrete

Mix Designation	KC 7-18	KC 7-18	KC 8-22	KC 9-12	KC 9-12	KC 10-2	KC 10-9	KC 10-9
Casting Date	7-18-96	7-18-96	8-22-96	9-12-96	9-12-96	10-2-96	10-9-96	10-9-96
Specimen Curing	<i>Moist</i>	<i>Site</i>	<i>Moist</i>	<i>Moist</i>	<i>Site</i>	<i>Moist</i>	<i>Moist</i>	<i>Site</i>
Hardened Concrete Properties								
Compressive Strength (psi)								
7 days	6920	NR	8060	7430	NR	7340	7980	NR
28 days	9160	8100	10500	9630	9070	10180	10650	9780
56 days	10350	8770	11390	10460	9150	NR	11990	9930
Modulus of Elasticity (ksi)								
28 days	6510	5680	6550	6670	6080	6540	6200	5420
56 days	6540	6190	6830	6730	6150	NR	6850	6100
Splitting Tensile Strength (psi)								
28 days	550	570	820	670	770	860	830	590
56 days	810	880	860	860	920	NR	890	860
RCPT - Rating	Very Low	Very Low	Very Low	Very Low	Very Low		Low	Low
56 days	490	680	600	820	820	NR	1200	1390
Water-to-Binder Ratio								
W/Cm (C+ FA), by weight	0.36	0.36	0.35	0.38	0.38	0.35	0.37	0.37
Test Methods: AASHTO T2 (ASTM C39), ASTM C469, AASHTO T277 1 ksi = 1,000 psi = 6.895 MPa								
NR: Testing not planned, therefore, results not reported.								

Table 6.59: Hardened Concrete Properties for Plant-Produced Concrete

Mix Designation	KC 11-20	S2 10-18	S1 10-25	S2 12-3	S2 12-3	S2 12-3	S2 12-10	S2 12-10
Casting Date	11-20-96	Field Trial	Field Trial	12-3-96	12-3-96	12-3-96	12-10-96	12-10-96
Specimen Curing	<i>Moist</i>	<i>Moist</i>	<i>Moist</i>	<i>Moist</i>	<i>Site</i>	<i>Maturity</i>	<i>Moist</i>	<i>Site</i>
Hardened Concrete Properties								
Compressive Strength (psi)								
1 day	NR	NR	NR	NR	NR	1440	NR	NR
3 days	NR	NR	NR	2800	NR	3110	2790	NR
7 days	8230	NR	NR	5300	4680	4560	4740	4220
14 days	NR	NR	NR	5690	5480	5580	5310	4580
28 days	9840	6970	5500	7100	6120	6095	5920	4880
56 days	10650	7680	6020	7950	6400	6650	6520	5160
Modulus of Elasticity (ksi)								
1 day	NR	NR	NR	NR	NR	2460	NR	NR
3 days	NR	NR	NR	NR	NR	4120	NR	NR
7 days	NR	NR	NR	5250	4930	4970	5150	3970
14 days	NR	NR	NR	5180	4990	5180	5190	4300
28 days	6650	6050	5570	5250	5100	5190	5300	4520
56 days	6680	6170	5850	5120	5170	5540	5570	4670
Splitting Tensile Strength (psi)								
28 days	780	480	560	660	610	NR	470	430
56 days	820	670	510	710	610	NR	540	510
RCPT - Rating		Low	Moderate	Low	Moderate		Moderate	Moderate
56 days	NR	1380	2490	1870	2970	NR	2200	3200
Water-to-Binder Ratio								
W/Cm (C+ FA), by weight	0.33	0.45	0.44	0.41	0.41	0.41	0.43	0.43
Test Methods: AASHTO T2 (ASTM C39), ASTM C469, AASHTO T277 1 ksi = 1,000 psi = 6.895 MPa								
NR: Testing not planned, therefore, results not reported.								

Table 6.60: Hardened Concrete Properties for Plant-Produced Concrete

Mix Designation	S2 1-21	S2 1-21	S2 1-23	S2 1-23	S2 1-29	S2 1-29	S1 2-15	S1 2-15
Casting Date	1-21-97	1-21-97	1-23-97	1-23-97	1-29-97	1-29-97	2-15-97	2-15-97
Specimen Curing	<i>Moist</i>	<i>Site</i>	<i>Moist</i>	<i>Site</i>	<i>Moist</i>	<i>Site</i>	<i>Moist</i>	<i>Site</i>
Hardened Concrete Properties								
Compressive Strength (psi)								
1 day	NR	NR	NR	NR	NR	NR	2120	NR
3 days	2710	NR	2980	NR	2760	NR	3200	2100
7 days	3150	3210	3460	3330	3490	NR	4120	4010
14 days	3500	3650	4110	3600	3840	3410	4650	4240
28 days	3920	3560	4780	3700	4160	3940	5250	5340
56 days	4390	4450	5350	4700	4760	4560	5900	5740
Modulus of Elasticity (ksi)								
1 day	NR	NR	NR	NR	NR	NR	3690	NR
3 days	NR	NR	NR	NR	NR	NR	4450	3730
7 days	4210	3910	4640	4290	NR	NR	4740	4580
14 days	4590	4040	4620	4080	4840	4470	5160	4860
28 days	4480	4100	5160	4550	4940	4630	5310	4930
56 days	4870	4310	5260	4670	4890	4640	5570	5100
Splitting Tensile Strength (psi)								
28 days	420	430	440	410	370	390	470	NR
56 days	440	470	500	470	550	510	610	520
RCPT - Rating	Moderate		Moderate	High	Moderate	Moderate	Moderate	High
56 days	3680	NR	3790	4300	2790	3970	2880	5610
Water-to-Binder Ratio								
W/Cm (C+ FA), by weight	0.43	0.43	0.43	0.43	0.44	0.44	0.45	0.45
Test Methods: AASHTO T2 (ASTM C39), ASTM C469, AASHTO T277 NR: Testing not planned, therefore, results not reported.								

1 ksi = 1,000 psi = 6.895 MPa

Table 6.61: Hardened Concrete Properties for Plant-Produced Concrete

Mix Designation	S1 2-18	S1 2-18	S1 2-28	S1 2-28	S2 3-4	S2 3-4	K1 4-17	K2 5-12
Casting Date	2-18-97	2-18-97	2-28-97	2-28-97	3-4-97	3-4-97	Field Trial	Field Trial
Specimen Curing	<i>Moist</i>	<i>Site</i>	<i>Moist</i>	<i>Site</i>	<i>Moist</i>	<i>Site</i>	<i>Moist</i>	<i>Moist</i>
Hardened Concrete Properties								
Compressive Strength (psi)								
1 day	NR	NR	NR	NR	NR	NR	NR	NR
3 days	NR	NR	2860	NR	NR	NR	NR	NR
7 days	4580	NR	4260	4180	3530	3260	NR	NR
14 days	NR	NR	NR	NR	NR	NR	5920	NR
28 days	5360	5210	4840	5210	NR	NR	6880	NR
56 days	6010	5780	5640	5290	6450	4350	7940	8560
Modulus of Elasticity (ksi)								
1 day	NR	NR	NR	NR	NR	NR	NR	NR
3 days	NR	NR	NR	NR	NR	NR	NR	NR
7 days	NR	NR	4700	4650	4570	4040	NR	NR
14 days	NR	NR	NR	NR	NR	NR	4880	NR
28 days	4680	4250	4490	4740	NR	NR	5160	NR
56 days	5030	4700	4670	5120	5250	4590	5570	5900
Splitting Tensile Strength (psi)								
28 days	560	530	480	530	NR	NR	600	NR
56 days	620	620	520	560	700	520	NR	860
RCPT - Rating	Low	Moderate	Low	Moderate	Low	Moderate	Low	Very Low
56 days	1940	3580	1460	2420	1110	2620	1080	580
Water-to-Binder Ratio								
W/Cm (C+ FA), by weight	0.45	0.45	0.45	0.45	0.45	0.45	0.34	0.40
Test Methods: AASHTO T2 (ASTM C39), ASTM C469, AASHTO T277 NR: Testing not planned, therefore, results not reported.								

Table 6.62: Hardened Concrete Properties for Plant-Produced Concrete

Mix Designation	K3 5-12	K4 6-2	SAD 6-12	SAD 6-12	SAD 6-25	SAD 6-25	SAD 7-9	SAD 7-9
Casting Date	Field Trial	Field Trial	6-12-97	6-12-97	6-25-97	6-25-97	7-9-97	7-9-97
Specimen Curing	Moist	Moist	Moist	Site	Moist	Site	Moist	Site
Hardened Concrete Properties								
Compressive Strength (psi)								
1 day	NR	NR	4470	NR	NR	NR	NR	NR
3 days	NR	NR	6530	5220	NR	NR	NR	NR
7 days	NR	NR	7170	5600	6680	6960	6720	5250
14 days	NR	NR	8420	5860	8350	8240	7820	5460
28 days	NR	7290	9030	6520	9030	8310	8870	8800
56 days	8560	8150	9800	7290	10240	8420	9620	9060
Modulus of Elasticity (ksi)								
1 day	NR	NR	4400	NR	NR	NR	NR	NR
3 days	NR	NR	5490	5060	NR	NR	NR	NR
7 days	NR	NR	5430	5180	5740	5300	5870	4410
14 days	NR	NR	5640	5260	5880	5480	6040	4690
28 days	NR	5540	5750	5300	6230	5160	6140	4940
56 days	6170	5850	6050	5500	6650	5230	6460	6060
Splitting Tensile Strength (psi)								
28 days	NR	790	830	590	960	770	820	810
56 days	800	760	970	690	1020	890	860	850
RCPT - Rating	Very Low	Very Low	Very Low	Low	Very Low	Very Low	Very Low	Very Low
56 days	470	530	830	1180	480	670	580	630
Water-to-Binder Ratio								
W/Cm (C+ FA), by weight	0.34	0.24	0.25	0.25	0.25	0.25	0.27	0.27
Test Methods: AASHTO T2 (ASTM C39), ASTM C469, AASHTO T277 1 ksi = 1,000 psi = 6.895 MPa								
NR: Testing not planned, therefore, results not reported.								

Table 6.63: Hardened Concrete Properties for Plant-Produced Concrete

<i>Mix Designation</i>	SAD 7-23	SAD 7-23	SAD 7-26	SAD 7-26	SAD 8-19	SAD 8-19	SAD 8-28	SAD 8-28
Casting Date	7-23-97	7-23-97	7-26-97	7-26-97	8-19-97	8-19-97	8-28-97	8-28-97
Specimen Curing	<i>Moist</i>	<i>Site</i>	<i>Moist</i>	<i>Site</i>	<i>Moist</i>	<i>Site</i>	<i>Moist</i>	<i>Site</i>
Hardened Concrete Properties								
Compressive Strength (psi)								
1 day	NR	NR	NR	NR	NR	NR	NR	NR
3 days	NR	NR	NR	NR	NR	NR	NR	NR
7 days	6660	6990	6050	4880	NR	NR	NR	NR
14 days	8060	7270	6850	5200	NR	NR	7230	6860
28 days	8640	7470	8060	8110	8830	8410	8190	7200
56 days	9610	7550	8790	8220	9350	8680	8740	7460
Modulus of Elasticity (ksi)								
1 day	NR	NR	NR	NR	NR	NR	NR	NR
3 days	NR	NR	NR	NR	NR	NR	NR	NR
7 days	5770	4960	5010	4610	NR	NR	NR	NR
14 days	5800	5180	5510	4820	NR	NR	5540	5020
28 days	5900	5700	5700	4950	6650	5460	5780	5410
56 days	6280	5790	5960	5010	6250	5380	5790	4920
Splitting Tensile Strength (psi)								
28 days	780	760	750	730	900	840	730	700
56 days	850	770	800	710	920	910	800	750
RCPT - Rating	Very Low	Very Low	Very Low	Very Low	Very Low	Very Low	Very Low	Very Low
56 days	840	900	750	770	490	630	870	1000
Water-to-Binder Ratio								
W/Cm (C+ FA), by weight	0.27	0.27	0.25	0.25	0.25	0.25	0.27	0.27
Test Methods: AASHTO T2 (ASTM C39), ASTM C469, AASHTO T277 1 ksi = 1,000 psi = 6.895 MPa								
NR: Testing not planned, therefore, results not reported.								

Table 6.64: Hardened Concrete Properties for Plant-Produced Concrete

Mix Designation	SAD 9-4	SAD 9-4	LPC1	LSB1	LSB2	LNB1	LPC 5-29	LPC 5-29
Casting Date	9-4-97	9-4-97	Plant Trial	Plant Trial	Plant Trial	Plant Trial	5-29-96	5-29-96
Specimen Curing	<i>Moist</i>	<i>Site</i>	<i>Moist</i>	<i>Moist</i>	<i>Moist</i>	<i>Moist</i>	<i>Moist</i>	<i>Member</i>
Hardened Concrete Properties								
Compressive Strength (psi)			<i>Col/Panel</i>	<i>K Deck</i>	<i>K Deck</i>	<i>S Deck</i>		
1 day	NR	NR	8350	NR	NR	NR	NR	NR
3 days	NR	NR	9520	NR	NR	NR	NR	NR
7 days	6770	6730	10440	8550	NR	4360	NR	NR
14 days	8060	7070	NR	8800	NR	4630	NR	NR
28 days	8670	7380	12740	10940	10680	5600	12540	12840
56 days	9300	7770	14240	11340	11630	NR	13800	14040
Modulus of Elasticity (ksi)								
1 day	NR	NR	NR	NR	NR	NR	NR	NR
3 days	NR	NR	NR	NR	NR	NR	NR	NR
7 days	5260	4920	NR	NR	NR	NR	NR	NR
14 days	5830	4990	NR	NR	NR	NR	NR	NR
28 days	6160	5170	NR	5520	5070	NR	6730	6710
56 days	6340	5570	7180	5600	5390	NR	7100	7080
Splitting Tensile Strength (psi)								
28 days	790	680	NR	NR	NR	NR	960	930
56 days	810	830	NR	NR	990	NR	1040	960
RCPT - Rating	Very Low	Low	Very Low	Low	Low		Very Low	
56 days	780	1490	540	1500	1970	NR	360	NR
Water- to-Binder Ratio								
W/Cm (C+ FA), by weight	0.27	0.27	0.31	0.35	0.35	0.43	0.31	0.31
Test Methods: AASHTO T2 (ASTM C39), ASTM C469, AASHTO T277 LPC = Louetta Precast Column LD = Louetta Deck 1 ksi = 1,000 psi = 6.895 MPa NR: Testing not planned, therefore, results not reported.								

Table 6.65: Hardened Concrete Properties for Plant-Produced Concrete

Mix Designation	LPP 9-12	LPP 9-12	LPP 9-16	LPP 9-16	LPP 9-18	LPP 9-18	BCW 9-4	BCW 9-4
Casting Date	9-12-96	9-12-96	9-16-96	9-16-96	9-18-96	9-18-96	9-4-96	9-4-96
Specimen Curing	<i>Moist</i>	<i>Member</i>	<i>Moist</i>	<i>Member</i>	<i>Moist</i>	<i>Member</i>	<i>Moist ③</i>	<i>Member ③</i>
Hardened Concrete Properties								
Compressive Strength (psi)								
1 day	6010	NR	NR	NR	NR	NR	NR	NR
3 days	NR	NR	7380 ①	NR	NR	NR	NR	NR
7 days	6640	8260	8620	8870	NR	NR	7500	6660
14 days	NR	NR	NR	NR	NR	NR	NR	NR
28 days	8620	8440	9680	10050	NR	NR	8750	8250
56 days	8810	9040	10330	10280	12370	11930	9420	8610
90 days	NR	NR	NR	NR	12730	12290	NR	NR
Modulus of Elasticity (ksi)								
1 day	5490	NR	NR	NR	NR	NR	NR	NR
3 days	NR	NR	4580 ①	NR	NR	NR	NR	NR
7 days	6000	5280	5530	5910	NR	NR	4640	4280
14 days	NR	NR	NR	NR	NR	NR	NR	NR
28 days	6040	5500	5900	5330	NR	NR	4960	4680
56 days	6450	5640	6390	5650	6210	5890	5090	5000
90 days	NR	NR	NR	NR	6520	6110	NR	NR
Splitting Tensile Strength (psi)								
28 days	680 ②	720 ②	730 ②	750 ②	NR	NR	690	650
56 days	800	780	760	810	NR	NR	730	730
90 days	850	870	820	910	660	810	NR	NR
RCPT - Rating								
56 days	Low	Low	Low	Moderate	Low	Low	Moderate	Moderate
	1420	1860	1460	2580	1260	1550	3250 ④	3700 ④
Water-to-Binder Ratio								
W/Cm (C+ FA), by weight	0.31	0.31	0.31	0.31	0.31	0.31	0.40	0.40
Test Methods: AASHTO T2 (ASTM C39), ASTM C469, AASHTO T277 NR: Testing not planned, therefore, results not reported. ① 2-day results; ② 7-day results; ③ San Angelo WB NSC Precast Panels LPP = Louetta Precast Panel BCW = San Angelo Precast Panel ④ No fly ash replacement by fabricator for NSC or HPC Precast Panels 1 ksi = 1,000 psi = 6.895 MPa								

Table 6.66: Hardened Concrete Properties for Plant-Produced Concrete

Mix Designation	BCW 10-3	BCW 10-3	BCW 2-5	BCW 2-5	BCW 2-11	BCW 2-11		
Casting Date	10-3-96	10-3-96	2-5-97	2-5-97	2-11-97	2-11-97		
Specimen Curing	<i>Moist ❶</i>	<i>Member ❶</i>	<i>Moist ❷</i>	<i>Member ❷</i>	<i>Moist ❷</i>	<i>Member ❷</i>		
Hardened Concrete Properties								
Compressive Strength (psi)								
1 day	5310	NR	3140	NR	NR	NR		
3 days	NR	NR	NR	NR	6060	NR		
7 days	8000	7410	8190	7400	8220	8550		
14 days	NR	NR	NR	NR	NR	NR		
28 days	8970	8580	9940	9680	10150	10200		
56 days	9660	9100	10780	10100	10320	11740		
Modulus of Elasticity (ksi)								
1 day	3990	NR	2990	NR	NR	NR		
3 days	NR	NR	NR	NR	3900	NR		
7 days	4840	4650	4820	5150	4330	4740		
14 days	NR	NR	NR	NR	NR	NR		
28 days	5150	4690	4780	5210	4810	4660		
56 days	5110	4550	4860	4620	5320	4960		
Splitting Tensile Strength (psi)								
28 days	710	650	800	790	780	810		
56 days	740	710	890	730	1020	980		
RCPT - Rating ❸	Moderate		Moderate	Moderate	Moderate	Moderate		
56 days	3050	NR	3430	3060	3130	3440		
Water-to-Binder Ratio								
W/Cm (C+ FA), by weight	0.40	0.40	0.31	0.31	0.31	0.31		
Test Methods: AASHTO T2 (ASTM C39), ASTM C469, AASHTO T277 NR: Testing not planned, therefore, results not reported. ❶ San Ang. WB NSC Precast Panels; ❷ San Ang. EB HPC Precast Panels LPP = Louetta Precast Panel BCW = San Angelo Precast Panel ❸ No fly ash replacement by fabricator for NSC or HPC Precast Panels 1 ksi = 1,000 psi = 6.895 MPa								

Prior to investigating the performance of the individual mix designs used for the selected bridge components, the comparative performance of the aggregates selected for use in the field is presented. Although the constituents and proportions of these HPC mix designs vary to a certain extent, the compatibility of the aggregates is clearly visible.

Figure 6.23 illustrates the elastic modulus development for the three aggregates selected for use in the bridges. The Burnet limestone proved to be the most compatible aggregate in terms of compressive strength gain as discussed in the laboratory trial phase. The crushed river gravel and south Texas limestone aggregates fractured at a maximum compressive strength around 82.7 MPa (12,000 psi) consistent with the maximum fracture stress of the aggregates as presented in Table 6.44. Both the Burnet limestone and the crushed river gravel exhibited very good elastic modulus exceeding ACI 363 guidelines. The composition of the south Texas limestone was primarily calcitic in nature. The abundance of this soft mineral caused a reduced elastic modulus, consistent with the trial laboratory mixes incorporating calcitic limestone. For elements such as a precast deck panel or CIP deck in which elastic modulus was not critical, this aggregate was acceptable in production. The CIP decks were mainly concerned with durability-related issues such as permeability, freeze-thaw resistance, abrasion resistance, and scaling resistance.

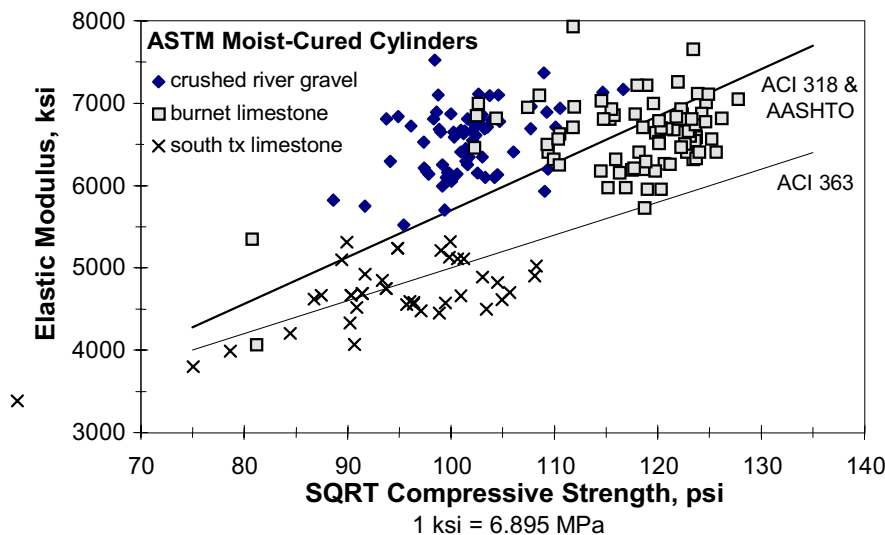


Figure 6.23: Elastic Modulus Development for Aggregates Selected for use in the Bridges

The splitting tensile strength development, illustrated in Figure 6.24, was independent of aggregate type and largely a function of the compressive strength of the concrete. The splitting

tensile strength of the HPC fell between the bands of 6 to 10 $\sqrt{f'_c}$, psi, for the aggregates used in production of the HPC.

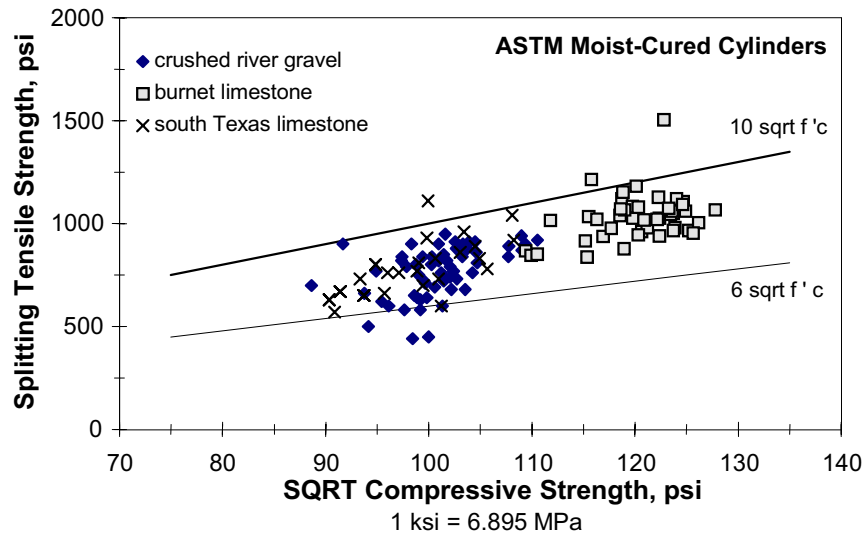


Figure 6.24: Splitting Tensile Strength Development for Aggregates Selected for Use In the Bridges

Cost analysis and comparisons for concrete materials that are presented in subsequent subsections of this chapter and Appendix E of this report are based on the current central Texas market price as of July 1, 1998. The current cost of cement is significantly higher (approximately 25 percent) than in recent years due to the large volume of construction, recent mild winters stimulating year-round construction, and limited stock-pile reserves by cement manufacturers. The material costs are as follows:

Raw Materials

Natural River Sand - \$0.00606 per kg (\$5.50 per ton)

Potable Water - \$0.33 per 1,000 liters (\$1.25 per 1,000 gallons)

ASTM Type I/II Cement - \$0.08267 per kg (\$75.00 per ton)

ASTM Type III Cement - \$0.08818 per kg (\$80.00 per ton)

Aggregate (Limestone/River Gravel) - \$0.00827 per kg (\$7.50 per ton)

Mineral Admixtures

ASTM Class C Fly Ash - \$0.02425 per kg (\$22.00 per ton)

Chemical Admixtures

3rd Generation HRWR Admixture - \$3.96 per liter (\$15 per gallon)

2nd Generation HRWR Admixture - \$2.11 per liter (\$8 per gallon)

Air Entraining Agent Admixture - \$0.46 per liter (\$1.75 per gallon)

Retarding Admixture - \$0.73 per liter (\$2.75 per gallon)

The following subsections present the performance-related test results for the various bridge components monitored during production. These results are presented in the following order:

1. Precast/Prestressed Concrete Beams
2. Substructure Components (Piers and Pier caps)
3. Precast/Prestressed Deck Panels
4. Cast-in-Place Decks

6.6.1 Performance-Related Test Results of Precast/Prestressed Beams

The precast/prestressed beams for both the Louetta Road Overpass and North Concho River Overpass were monitored for mechanical and material property performance both at early and later ages. The U-beams, which were the main supporting element for the Louetta Road Overpass, were fabricated by the Texas Concrete Company in Victoria, Texas, from September 1994 through March 1996. A U-beam located at the precast plant is illustrated in Figure 6.25.



Figure 6.25: Louetta Road Overpass U-Beam



Figure 6.26: North Concho River Overpass AASHTO Type IV Beam

The AASHTO Type IV beams, which were the main supporting element for the North Concho River Overpass, were fabricated by Texas Concrete Company in Victoria, Texas, from February 1997 through April 1997. Because the required prestressing force for the long-span HS/HPC precast members exceeded the capacity of the existing precasting beds, Texas Concrete had a redesign performed to fabricate the members using a two-stage partial prestress (pretensioned) and post-tensioned mechanism. A trial beam, shown in Figure 6.26, was fabricated in April 1996 prior to full-scale production of the members to investigate several structural performance issues. These included stability issues related to transportation and erection of the members as well as camber and serviceability issues associated with the partial prestressed and post-tensioned system. The specifics of this may be referenced in the structural performance-related report by Gross entitled “Field Performance of Prestressed High Performance Concrete Highway Bridges in Texas” [136].

In order to more closely simulate the actual concrete in the members, a commercially available match curing system was selected. Other curing regimes were also investigated including ASTM moist-cured, TxDOT member-cured, and member-cured cylinders as described in Section 5.3.6.1 of this report. Match curing locations that would result in both minimum and maximum peak hydration temperatures were selected throughout the member in order to investigate the impact of temperature on mechanical and material property development. Figures 6.27 and 6.28 illustrate the

locations selected for match curing in both the Louetta U-beam and North Concho River AASHTO Type IV beams.

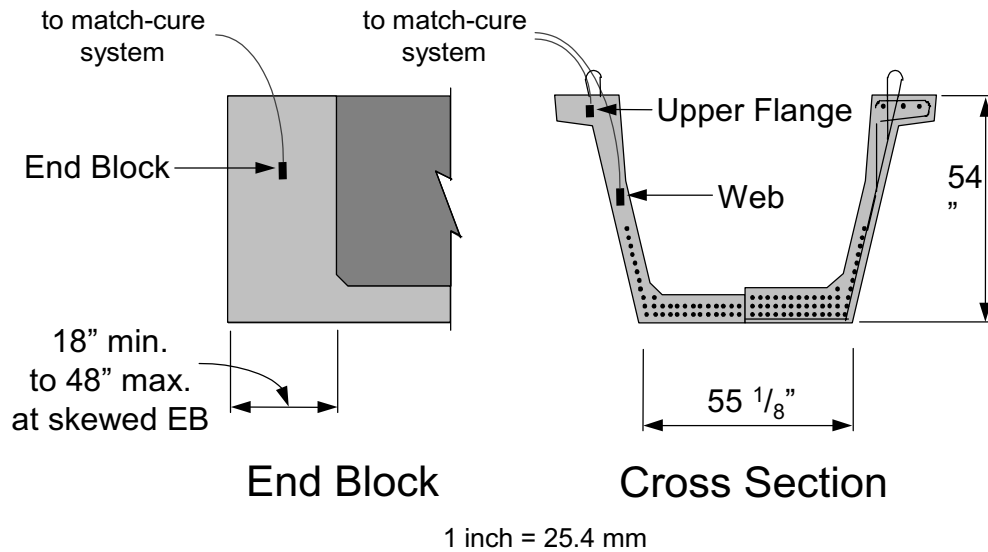


Figure 6.27: Schematic Representation of U-Beam Thermocouple Locations

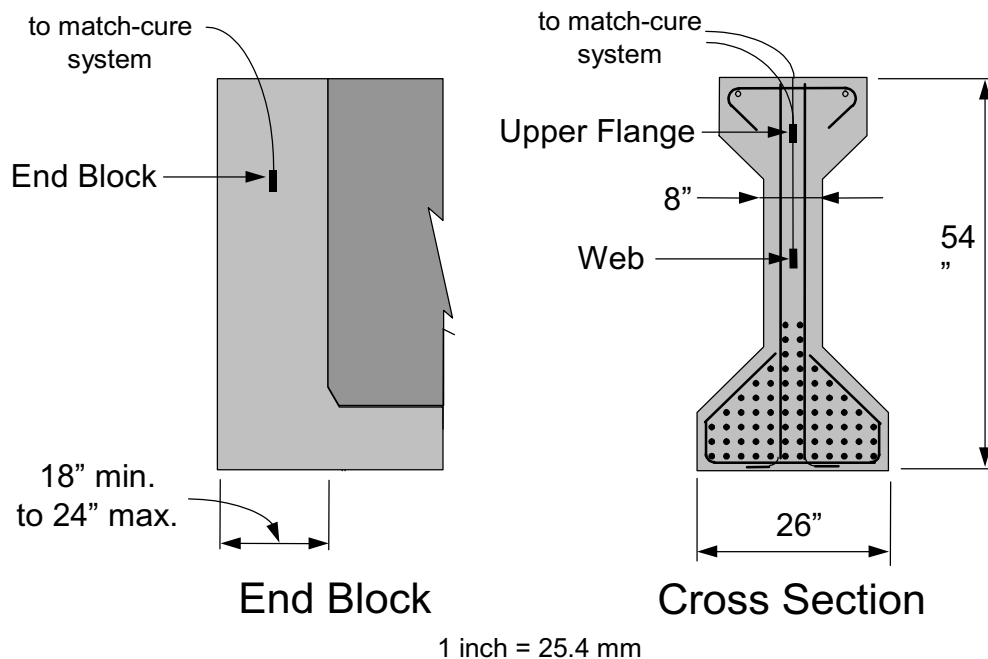


Figure 6.28: Schematic Representation of AASHTO Type IV Thermocouple Locations

The thermocouple indicates the location where temperature development was monitored for match curing. The more massive end block locations generated the maximum peak hydration temperatures for both the U-beam and AASHTO Type IV beams while the selected web locations generated the lowest peak hydration temperatures. Thermocouple locations were stacked at 127-mm (5-in.) intervals vertically in the member during the initial monitoring of both section shapes to determine these critical locations. The following sections present and discuss the performance-related test results of the precast beams including the mechanical and material properties.

The specific impact of hydration temperatures on compressive strength development, elastic modulus development, and permeability is presented and discussed thoroughly in Chapter 8 of this report.

6.6.1.1 Compressive Strength ASTM C39-94. Compressive strength development for the precast beams was monitored both at early and later-age development. Figure 6.29 illustrates the compressive strength gain with time for the ASTM moist-cured cylinders. A logarithmic regression analysis was performed to obtain a best-fit curve for the ASTM moist-cured data. The data collected from the precast beams exhibited an excellent correlation factor ($R^2 = 0.938$). Figure 6.30 illustrates the compressive strength gain with time for the member-cured cylinders. Similarly, a logarithmic regression analysis was performed to obtain a best-fit curve for the member-cured data. The data exhibited a good correlation factor ($R^2 = 0.874$). Figure 6.31 illustrates the compressive strength gain with time for the match-cured cylinders. Finally, a logarithmic regression analysis was performed to obtain a best-fit curve for the match-cured data. The data exhibited a fair correlation factor ($R^2 = 0.626$). The poorer correlation factor for the member-cured and match-cured cylinders may be attributed to the influence of ambient conditions during casting.

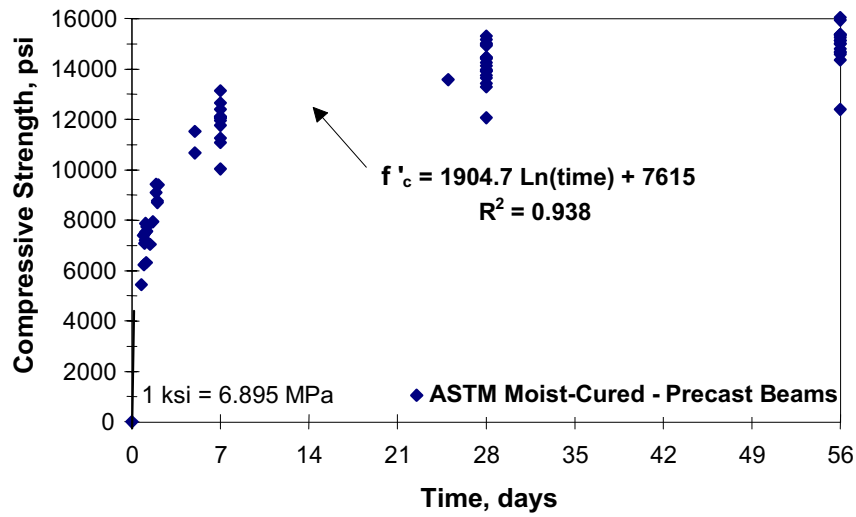


Figure 6.29: Compressive Strength Development for Precast Beam ASTM Moist-Cured Cylinders

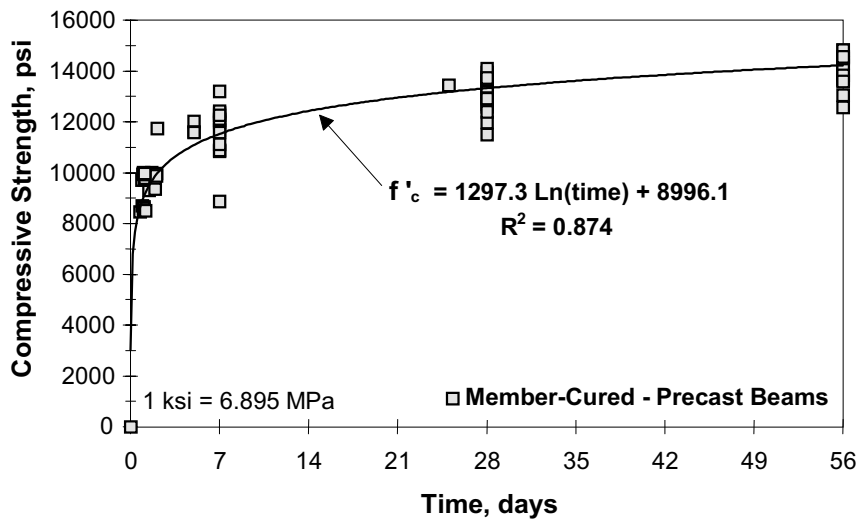


Figure 6.30: Compressive Strength Development for Precast Beam Member-Cured Cylinders

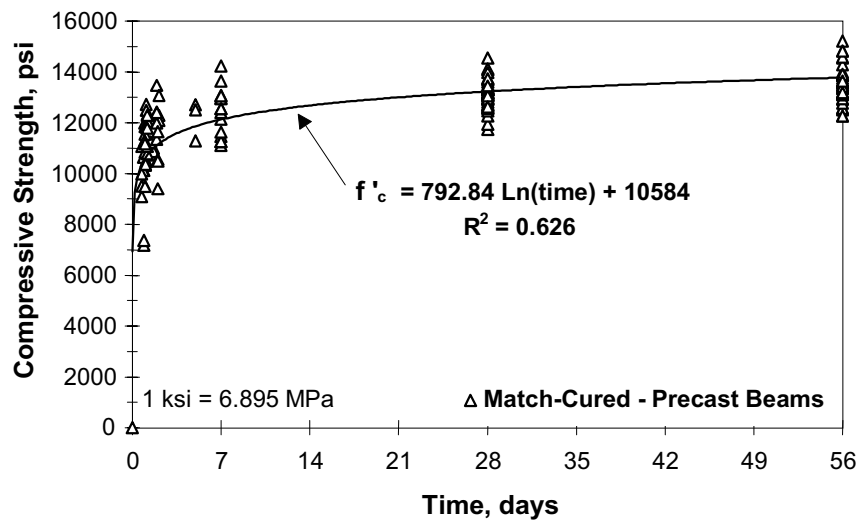


Figure 6.31: Compressive Strength Development for Precast Beam Match-Cured Cylinders

The ASTM moist-cured cylinders were not impacted by ambient conditions because the specimens were cured in a climate-controlled environment. A poorer correlation factor and a noticeably wider scatter at early test ages for the match-cured results were related to the influence of ambient conditions, but also highly impacted by the significant variation in hydration temperatures affected by the match-curing location selected. Hydration temperatures of the concrete between the end blocks and the web varied by as much as 29.4 °C (53 °F) for the U-beam members and 15°C (27 °F) for the AASHTO Type IV members, respectively. Figure 6.31 illustrates this noticeable scatter at early test ages (22 to 48-hours) and accounts for the poorer correlation factor when compared to the other curing regimes.

The concrete temperatures during hydration and the curing condition significantly impacted the compressive strength gain of the concrete. Figure 6.32 illustrates the comparable strength gain development of the curing regimes presented in Figures 6.29, 6.30, and 6.31. In particular, note the higher early strength and lower later-age strength of the match-cured cylinders when compared to the two other standard curing regimes. The hydration temperatures had a significant effect on the early and later-age compressive strength development. This topic is discussed thoroughly in Chapter 8 of this report.

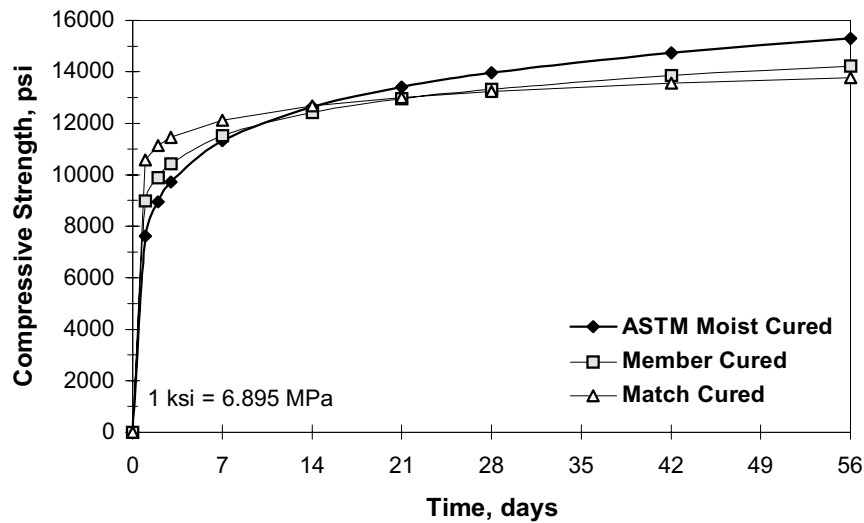


Figure 6.32: Compressive Strength Development for Precast Beam

6.6.1.2 Elastic Modulus ASTM C469-94. Elastic modulus development for the precast beams was monitored both at early and later-age development. Figure 6.33 illustrates the elastic modulus development for the quality control/quality assurance specimens sampled during casting of the precast beams under the curing regimes monitored. The elastic modulus of the precast plant-produced concrete that was sampled from the beam exceeded the empirical equation (Equation 6.2) recommended by ACI Committee 363 for high-strength concrete.

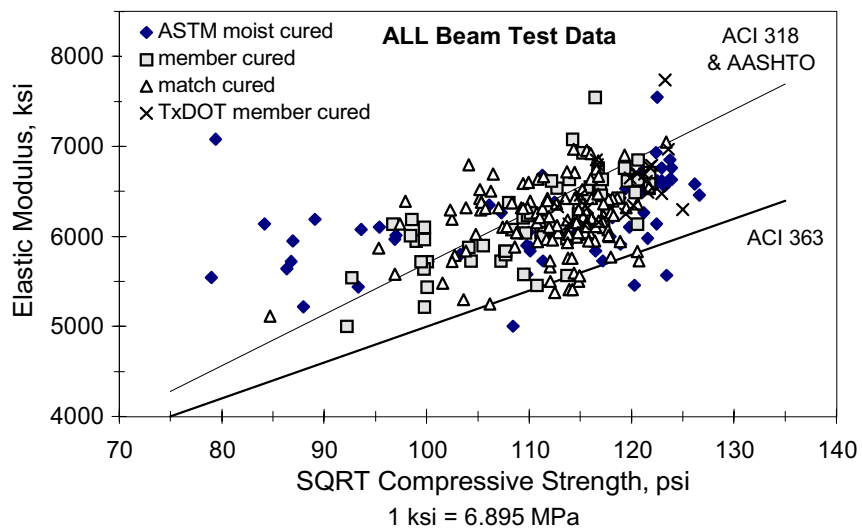


Figure 6.33: Elastic Modulus Development for Precast Beam QC/QA Specimens

A linear regression analysis was performed with and without a zero intercept to obtain the best-fit criteria for the ASTM moist-cured data. The linear regression analysis resulted in an empirical equation of $E_c = 56,300 \sqrt{f'_c}$, psi, with a zero intercept for the data collected for the ASTM moist-cured cylinders. Figure 6.34 illustrates the elastic modulus development for the ASTM moist-cured cylinders. It may be noted that the results for modulus of elasticity for the ASTM moist-cured cylinders closely match the current ACI Committee 318 empirical equation (Equation 6.1) for conventional normal weight concretes.

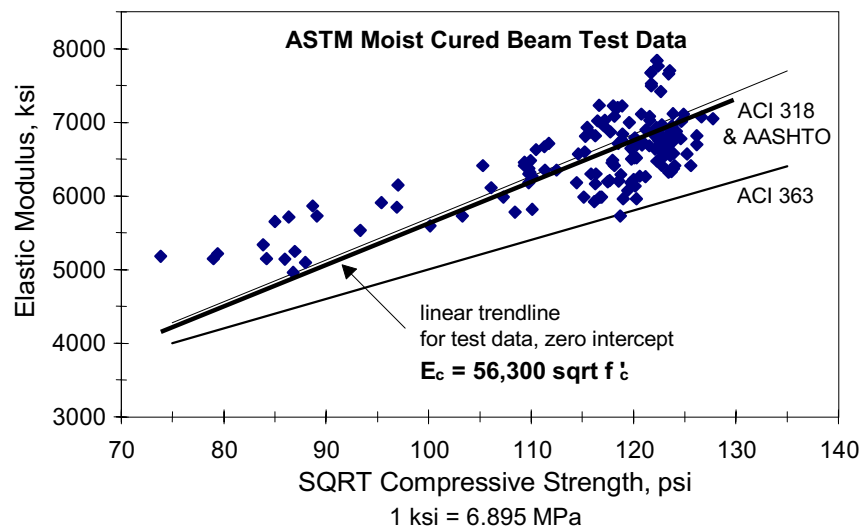


Figure 6.34: Elastic Modulus Development for Precast Beam ASTM Moist-Cured Cylinders

A linear regression analysis was performed with and without a zero intercept to obtain the best-fit criteria for the member-cured data. The linear regression analysis resulted in an empirical equation of $E_c = 55,050 \sqrt{f'_c}$, psi, with a zero intercept for the data collected for the member-cured cylinders. Figure 6.35 illustrates the elastic modulus development for the member-cured cylinders.

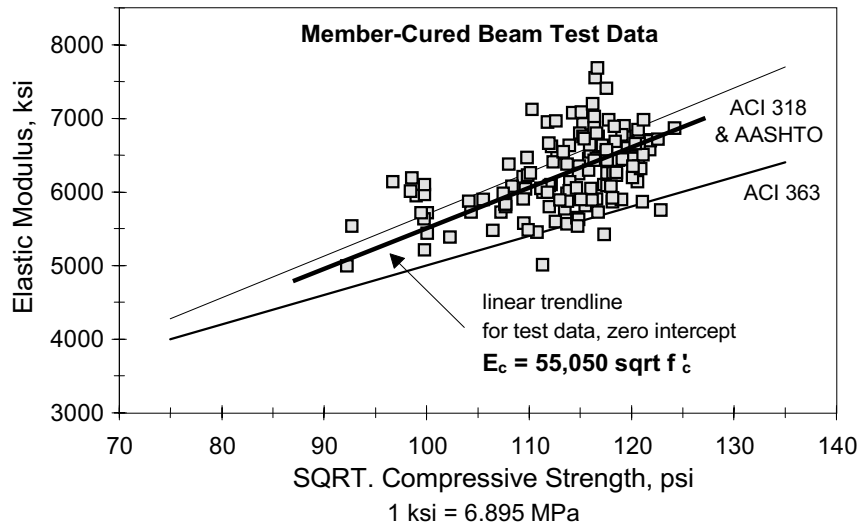


Figure 6.35: Elastic Modulus Development for Precast Beam Member-Cured Cylinders

A linear regression analysis was performed with and without a zero intercept to obtain the best-fit criteria for the match-cured data. The linear regression analysis resulted in an empirical equation of $E_c = 55,000 \sqrt{f'_c}$, psi, with a zero intercept for the data collected for the match-cured cylinders. Figure 6.36 illustrates the elastic modulus development for the match-cured cylinders.

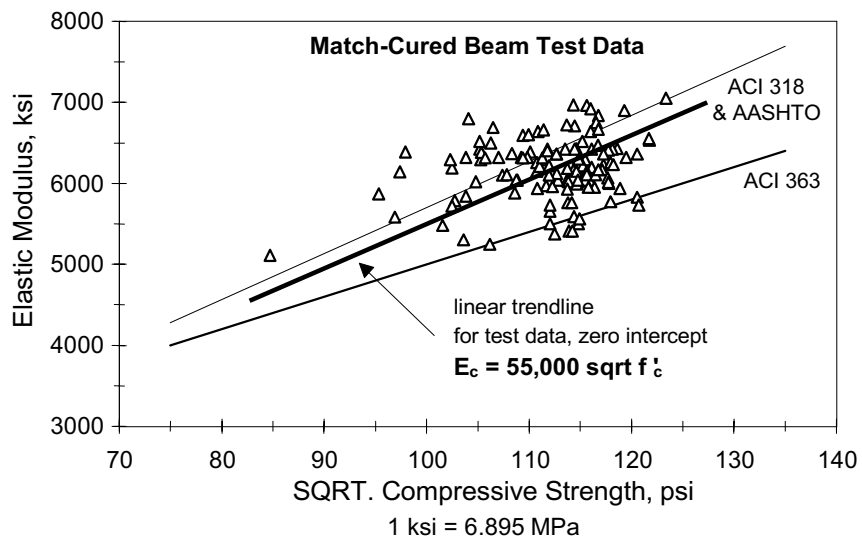


Figure 6.36: Elastic Modulus Development for Precast Beam Match-Cured Cylinders

The match-cured and member-cured cylinders generally exhibited slightly lower elastic modulus than the ASTM moist-cured cylinders as illustrated in Figures 6.34, 6.35, and 6.36. The match-cured and member-cured cylinders developed the majority of their elastic moduli at early ages with a limited amount of gain at later ages. This was due to the high initial curing temperatures. The elastic modulus development of the match-cured cylinders was similar to the compressive strength development where initial values exceeded ASTM moist-cured and member-cured cylinders as discussed in Chapter 8. The ASTM moist-cured cylinders generally exhibited the highest elastic modulus at 56 days when these cylinders attained the highest compressive strength. Table 6.67 summarizes the empirical equations based on the linear regression analysis with and without a zero intercept for the precast beam curing regimes investigated.

Table 6.67: Empirical Equations Based on Linear Regression Analysis for HPC Beam Mix Design

Curing Regime Investigated	Empirical Equation, psi	
	With Zero Intercept	Without Zero Intercept
ASTM Moist-Cured Cylinders	$E_c = 56,300 \sqrt{f'_c}$	$E_c = 38,200 \sqrt{f'_c} + 2,110,000$
Member Cured-Cylinders	$E_c = 55,050 \sqrt{f'_c}$	$E_c = 39,900 \sqrt{f'_c} + 1,730,000$
Match Cured-Cylinders	$E_c = 55,000 \sqrt{f'_c}$	$E_c = 17,200 \sqrt{f'_c} + 4,250,000$
1 ksi = 1,000 psi = 6.895 Mpa		

As documented in the laboratory trial phase of this study, the aggregate and other mix constituents have a dramatic influence on the elastic modulus characteristics of an HS/HPC mix design. It should be emphasized that the empirical equations presented in Table 6.67 are unique for the mix design developed and curing regimes investigated for the precast beams and not a guideline for HS/HPC in general. Any elastic modulus that is assumed based on compressive strength for the design of HS/HPC members by the design engineer should be verified through a trial field batching series on the specific mix design or by the documented historical mix design performance. This is important due to the dramatic impact elastic modulus has on HS/HPC structures, particularly for long-span bridges. For the mix design investigated, the match-cured empirical equation results best estimate the elastic modulus of the concrete in the member.

6.6.1.3 Splitting Tensile Strength ASTM C496-96. Splitting tensile strength development for the precast beams was monitored at later-age development (28 and 56 days). A linear regression analysis was performed with a zero intercept to obtain the best-fit criteria for the ASTM moist-cured data. The linear regression analysis resulted in an empirical equation of $f_{sp} = 8.58 \sqrt{f'_c}$, psi,

distributed between the bands of $7.5 \text{ to } 10 \sqrt{f'_c}$ psi, with a zero intercept for the data collected for the ASTM moist-cured cylinders. Figure 6.37 illustrates the splitting tensile strength for the ASTM moist-cured cylinders.

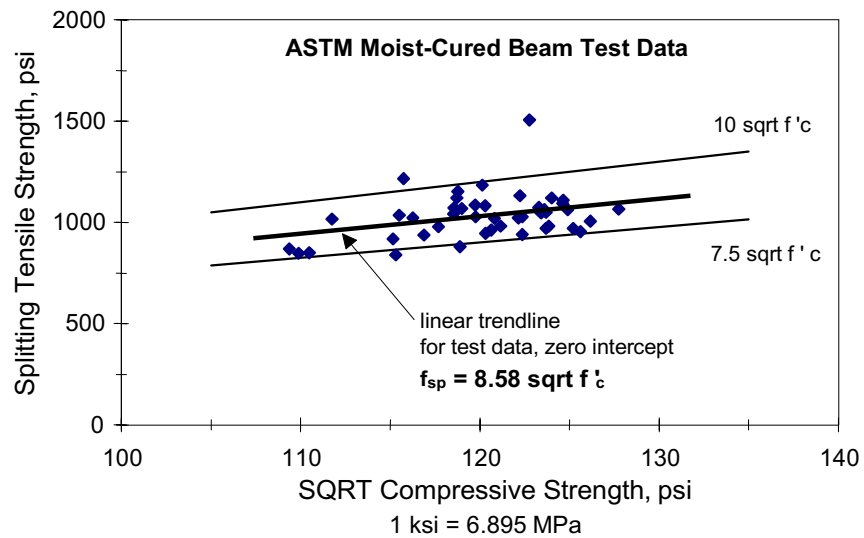


Figure 6.37: Splitting Tensile Strength Development for Precast Beam ASTM Moist-Cured Cylinders

Similarly, a linear regression analysis was performed with a zero intercept to obtain the best-fit criteria for the member-cured data. The linear regression analysis resulted in an empirical equation of $f_{sp} = 8.66 \sqrt{f'_c}$ psi, distributed between the bands of $7.5 \text{ to } 10 \sqrt{f'_c}$ psi, with a zero intercept for the data collected for the member-cured cylinders. Figure 6.38 illustrates the splitting tensile strength for the member-cured cylinders.

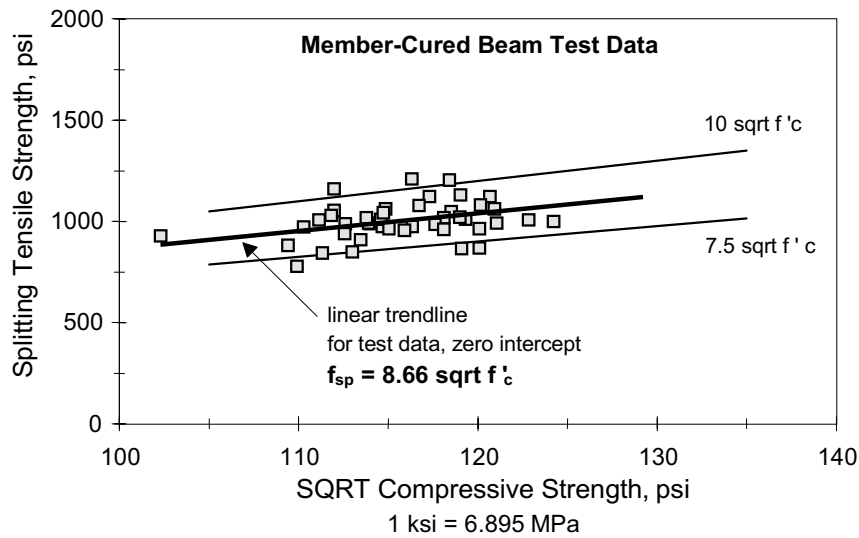


Figure 6.38: Splitting Tensile Strength Development for Precast Beam Member-Cured Cylinders

Finally, a linear regression analysis was performed with a zero intercept to obtain the best-fit criteria for the match-cured data. The linear regression analysis resulted in an empirical equation of $f_{sp} = 10.9 \text{ sqrt } f'_c$, psi, with a zero intercept for the data collected for the match-cured cylinders. Figure 6.39 illustrates the splitting tensile strength for the match-cured cylinders.

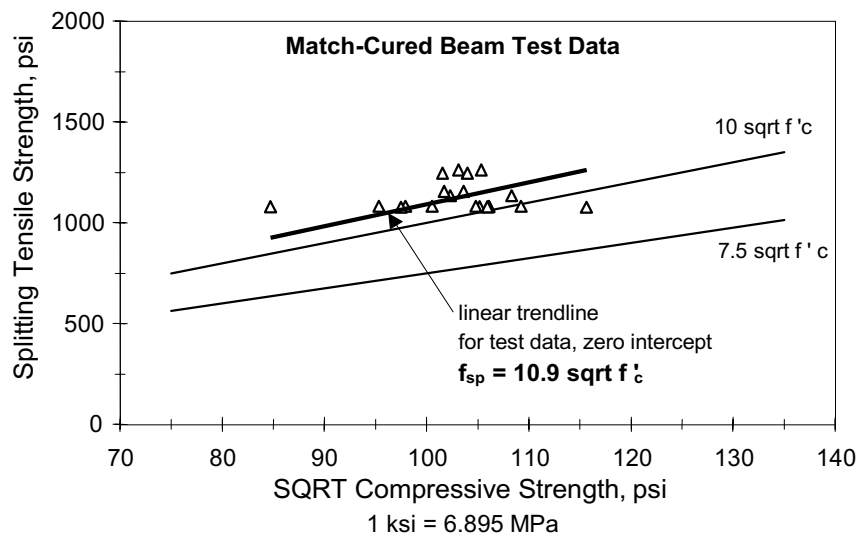


Figure 6.39: Splitting Tensile Strength Development for Precast Beam Match-Cured Cylinders

Higher splitting tensile strength was exhibited for the match-cured cylinders compared to the other curing regimes. It is unclear whether the enhanced splitting tensile strength characteristics of the match-cured cylinders can be attributed to the impact of higher curing temperatures (hydration profile), which resulted in improved transition zone performance (i.e., more resilience to initial crack formation and propagation). Further investigation of this response is warranted. It may be noted that the results for all curing regimes investigated exceeded the current ACI Committee 363 empirical equation of $f_{sp} = 7.4 \sqrt{f'_c}$, psi, (Equation 6.11) for HSC.

6.6.1.4 Rapid Chloride Permeability AASHTO T277. The project objectives provided by TxDOT and the FHWA at the start of this project were to develop a performance concrete mix design with a permeability rating of 1,500 coulombs passed or less. It was determined that this would result in a more durable structure with longer service life and lower maintenance costs. Rapid chloride permeability was monitored for each casting date at 56 days to coincide with the required compressive strength for design. The test date of 56 days is a more appropriate test age for the permeability of concretes with Class C fly ash replacement due to the later-age strength gain characteristics of the fly ash. RCPT testing (AASHTO T277) was performed on precast plant-produced concrete cylinders under various curing conditions as illustrated in Figure 6.40. All of the concrete evaluated under the various curing regimes satisfied the permeability objectives.

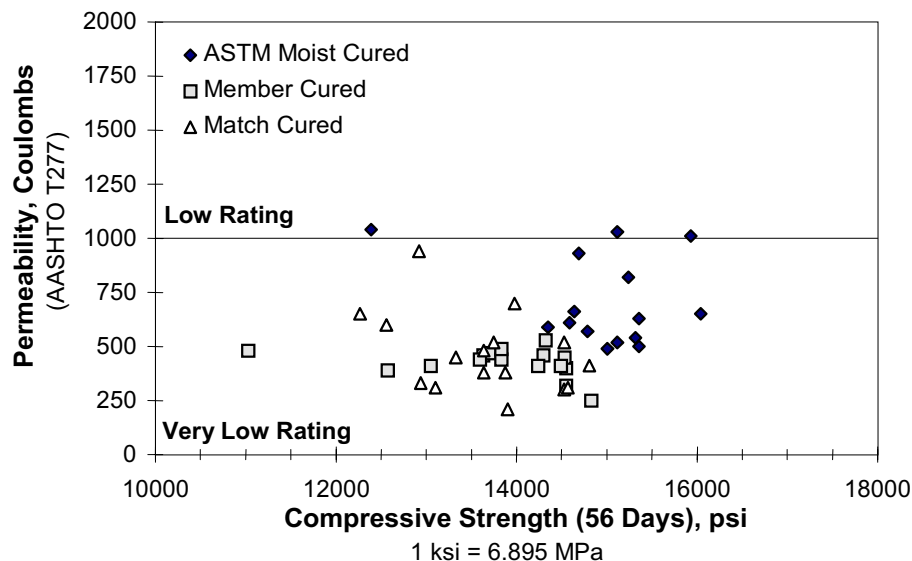


Figure 6.40: Rapid Chloride Permeability (56 Days) for HS/HPC Precast Beams

Over the years, precast plant-produced members have traditionally had a reputation for being very durable, impermeable concrete members. This is illustrated in the test results, particularly for the match-cured cylinders that exhibited the lowest permeability. Figure 6.41 illustrates the relative permeability designated by curing regime for a representative U-beam cast on March 8, 1996. In most cases (see Appendix B), the permeability of the match-cured cylinders was lower than that of their companion member-cured and moist-cured cylinders, as illustrated in Figure 6.41. These results were representative of most of the U-beams cast. However, in cases in which the end blocks reached excessive temperatures ($> 80^{\circ}\text{C}$ [176°F]) during the hydration of cementitious products, the member-cured cylinders exhibited the lowest permeability as illustrated in Figure 6.42 for the U-beam cast on February 26, 1996.

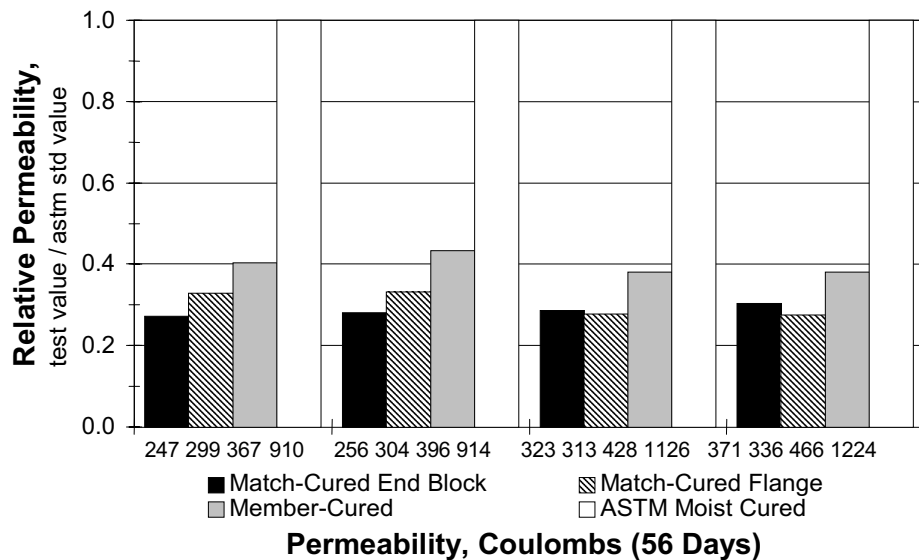


Figure 6.41: Rapid Chloride Permeability (56 Days) for U-Beam Cast on March 8, 1996

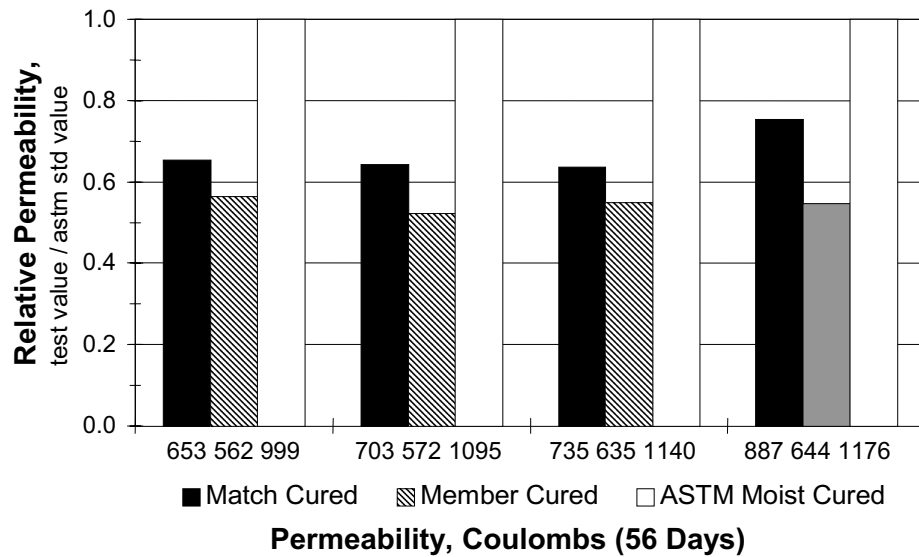


Figure 6.42: Rapid Chloride Permeability (56 Days) for U-Beam Cast on February 26, 1996

On this casting date, the end blocks attained a maximum temperature during hydration of 87.2 °C (189 °F). It is interesting to note the influence of the higher curing temperatures on both the match-cured and member-cured cylinders. Previous research studies have found that ASTM moist-cured cylinders yield lower permeability over site- or field-cured cylinders as a result of the added cement hydration that takes place due to the moist curing. The impact of hydration temperatures on permeability is discussed in greater detail in Chapter 8.

Figure 6.43 illustrates representative permeability results for an AASHTO Type IV precast beam cast on April 7, 1997. These results were consistent with the U-beam results. Permeability test results for all field-produced concrete are presented in Appendix B of this report.

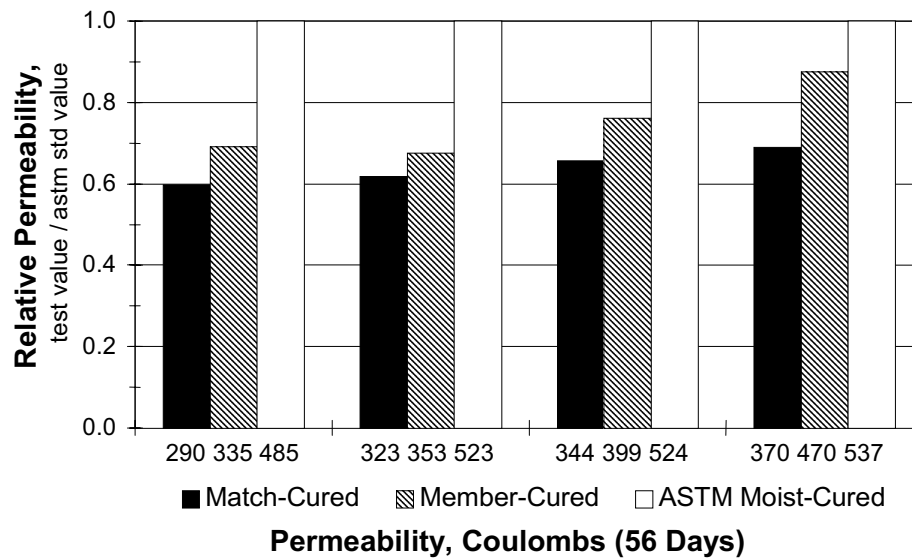


Figure 6.43: Rapid Chloride Permeability (56 Days) for AASHTO Type IV Cast on April 7, 1997

6.6.1.5 Chloride Ponding AASHTO T259. Chloride ion penetration was investigated for the HS/HPC selected for use in the precast beams. Durability-related problems associated with precast/prestressed beams are typically less of a concern than those associated with bridge decks, for example, because of the reduced crack widths (resulting from the prestressing) and the less harsh exposure conditions. Precast beams are typically protected from direct exposure to chemicals such as deicing salts by the bridge decks. For corrosion protection, ACI Committee 222 [139] recommends a maximum acid-soluble Cl^- concentration of 0.08 percent by weight of cement for prestressed concrete and 0.20 percent by weight of cement for reinforced concrete exposed to chlorides in service. For the HS/HPC beam mix design, this limit is equivalent to 0.019 percent by weight of the concrete based on the ACI prestressed concrete limitation.

Results of the 90-day ponding test are presented in Figure 6.44. Chloride percentages reported are the average of six specimen samples from three ponding blocks as described in Section 5.3.5.2.6 of this report. The integral chloride after the 90-day ponding cycle was 1.475. Minor levels of chloride (0.052 percent) were measured at a depth of 19 mm (0.75 in.) and no chloride was measured beyond a depth of 31.75 mm (1.25 in.). The concrete sampled from the precast beams satisfied ACI limitations for acid-soluble Cl^- concentration at a clear cover depth to reinforcing of 46.5-mm (1.83-in.) for the U-beam and 38.1-mm (1.5-in.) for the AASHTO Type IV beam.

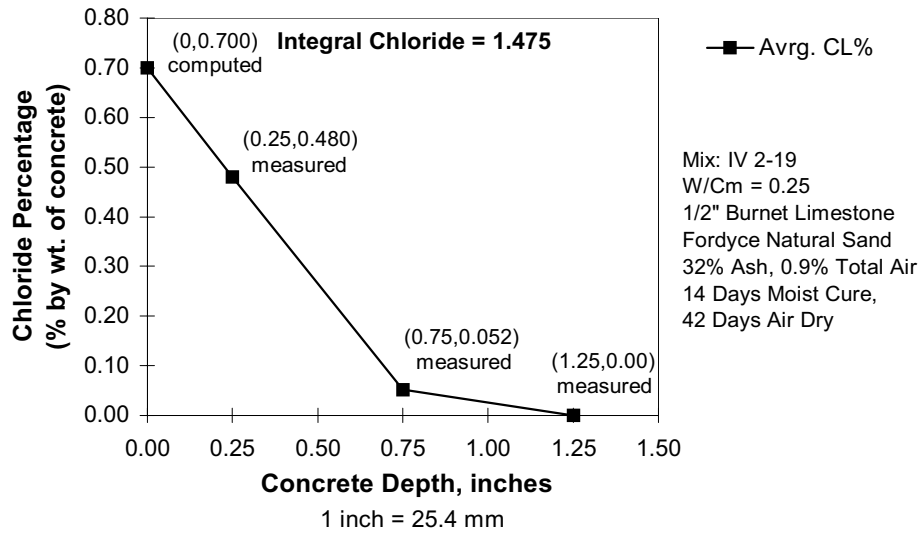


Figure 6.44: Chloride Ion Penetration (90-Day Ponding) for HS/HPC Precast Beams

6.6.1.6 Creep ASTM C512 and Shrinkage ASTM C157. Creep and shrinkage data of the mix design was monitored for 4 months. The specimens were member-cured with the beam until release, followed by ambient curing conditions (similar to the member) to more closely model the actual field creep and shrinkage of the concrete in the members. It should be noted that this process differs from the ASTM C512 specification, which calls for 7 days of moist curing in a 23 ± 1.67 °C (73.4 ± 3.0 °F) environment. The peak curing temperature of these creep and shrinkage specimens was 63 °C (145 °F). The curing temperature of the specimens was similar to temperatures found in the bottom flange and side webs of the U-beams. DEMAC points were placed on unloaded shrinkage specimens at 200-mm (8-in.) gauge lengths. Strains were measured at three levels (top, middle, and bottom) on three sides of the specimen. The nine measurements from one specimen are then averaged with two other specimens for an average shrinkage value. Creep and shrinkage specimens were monitored in a similar matter except that the specimens were loaded to 20.7 MPa (3,000 psi) at 1 day. This value corresponds to approximately 40 percent of f'_c at release. The difference between the loaded and the unloaded specimens is defined as the creep of the concrete. Figure 6.45 presents the measured shrinkage strain and current ACI 209 prediction for shrinkage. The fitted curve quantifies the measured results for the HPC mix design and is defined by Equation 6.12 as follows:

$$\mathcal{E}_{sh} = (0.000510) \frac{t}{35 + t} \quad (6.12)$$

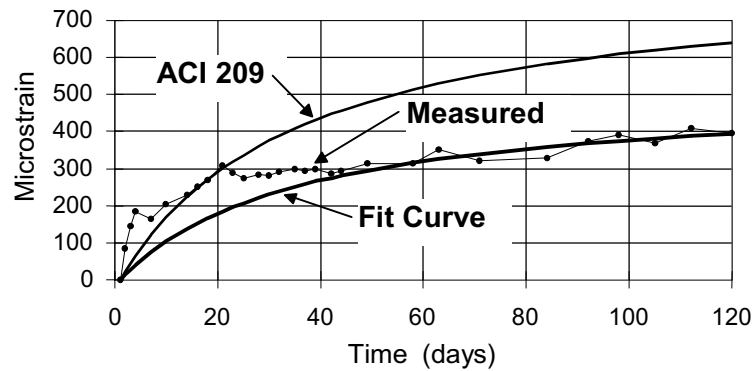


Figure 6.45: Average Shrinkage Strain for Concrete Sampled from the Precast U-Beam Members [22]

Figure 6.46 presents the measured creep coefficient and current ACI 209 prediction for the creep coefficient. The fitted curve quantifies the measured results for the creep coefficient for the HPC mix design and is defined by Equation 6.13 as follows:

$$C_{ct} = (1.95) \frac{t \exp 0.6}{10 + t \exp 0.6} \quad (6.13)$$

The following characteristics were noted for the HPC mix related to creep and shrinkage:

1. Less ultimate creep and shrinkage.
2. 20 percent lower creep rate for loading at 28 days.
3. No excessive shrinkage and creep in spite of the high curing temperatures.

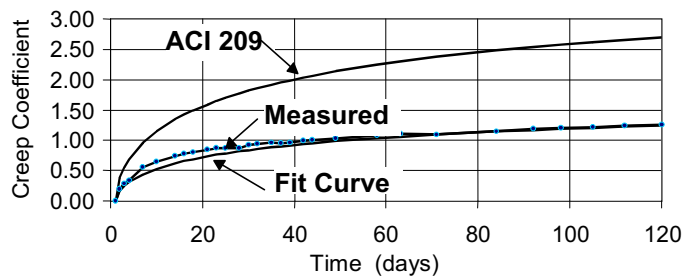


Figure 6.46: Creep Coefficient for Concrete Sampled from the Precast U-Beam Members [22]

The HPC exhibited less creep and shrinkage than comparable normal-strength concrete mixes. The impact that this has on the serviceability and structural performance of the precast beams

and the bridges is discussed extensively by Gross et al. in the comprehensive structural performance-related research report [136].

6.6.1.7 Comparison of Mechanical and Material Properties. Groundwork for this research project was laid during the late 1980s when a research study by Carrasquillo et al. was initiated to investigate the compressive strength development capabilities of precast plants in Texas utilizing readily available materials currently being used in production at the respective plants. The precast plant that was selected for this investigation was the Texas Concrete Company in Victoria, Texas. This is the same precast plant that produced the precast/prestressed beams for the Louetta Road Overpass and North Concho River Overpass projects.

Prior to this research study, the plant had utilized the mix design developed at that time as their primary HS/HPC mix design to meet high early-release strength requirements. Because this “standard HS/HPC” mix design was utilized for the North Concho River Overpass westbound beams, it provides an opportunity to illustrate how the constituent interaction and proportions of an HPC mix design are very important and must be investigated to obtain an optimized mix design with optimal property performance while maintaining an economical mix design. Table 6.68 compares the plant’s standard HS/HPC mix developed in the late 1980s and the “optimized HS/HPC” mix developed approximately 10 years later in the laboratory trial phase of this investigation.

Table 6.68: Comparison of Standard HS/HPC and Optimized HS/HPC Mix Designs Produced at the Texas Concrete Company in Victoria, Texas

Mechanical or Material Property	Westbound Bridge Standard HS/HPC Mix* Casting Date: 3-12-96	Eastbound Bridge Optimized HS/HPC Mix** Casting Date: 4-1-96
Beams at Release	26 Hours to Release	27 Hours to Release
Compressive Strength	8,060 psi	9,790 psi
Elastic Modulus	5,880 ksi	5,950 ksi
Beams at 28 Days		
Compressive Strength	10,260 psi	13,450 psi
Elastic Modulus	6,160 ksi	6,550 ksi
Beams at 56 Days		
Compressive Strength	11,030 psi	14,830 psi
Elastic Modulus	6,250 ksi	6,580 ksi
Rapid Chloride Permeability (at 56 Days)	550 Coulombs (Very Low Rating)	280 Coulombs (Very Low Rating)
* River Gravel Aggregate ** Burnet Limestone Aggregate 1 ksi = 1,000 psi = 6.895 MPa		

By investigating the interaction of all of the constituents including the aggregate type, cement type and manufacturer, fly ash type and manufacturer, and chemical admixtures, all of the mechanical

and material properties were enhanced with no appreciable cost differential between the mixes. Compressive strength at release and 56 days was enhanced by 21.5 percent and 34.5 percent, respectively. In addition, elastic modulus at release and 56 days was enhanced by 1.2 percent and 5.2 percent, respectively. Furthermore, permeability was reduced, which indicates a more durable structure that translates into reduced maintenance costs and a longer life.

6.6.2 Performance-Related Test Results of Substructure Components

Performance-related testing and monitoring of the substructure components were performed as described in the following sections.

6.6.2.1 Louetta Road Overpass Substructure. The substructure elements for the Louetta Road Overpass consisted of 101.6-mm (4-in.) thick precast hollow core post-tensioned elements as illustrated in Figure 6.47. Segments were cast 0.99-m (3.25-ft.) square and either 1.2 or 1.5 m (4.0 or 5.0 ft.) in height. They were fabricated by the Williams Brothers Construction Company, Inc. Precast Plant located in Houston, Texas. The sections were fabricated over a limited number of casting dates in May 1996. Owing to the limited number of casting dates, monitoring for performance-related testing and evaluation by CMRG occurred on only two of the casting dates. The mix design used for the substructure elements incorporated a crushed river gravel and 7-3/4 sacks of cementitious (cement + fly ash) material (standard U.S. units). The compressive strength development of the moist-cured and member-cured cylinders is illustrated in Figure 6.48. Due to the shape and thin walls of the precast member, the substructure element never developed high heat of hydration temperatures that might cause a “burn out” effect (high initial strength gain followed by limited long-term strength gain). The variation between concrete placement temperature and maximum hydration temperature was 13.9 °C (25 °F), significantly lower than the precast beams as discussed in Chapter 8. Consequently, the early and later-age compressive strength development did not vary drastically between curing conditions as illustrated in Figure 6.48. The member-cured cylinders continued to develop very good strength gain beyond the first 7 to 14 days, which was undoubtedly related to the section shape (efficient heat dissipation characteristics of the hollow core section) and the pozzalonic characteristics of the fly ash.



Figure 6.47: Louetta Road Overpass Substructure Element

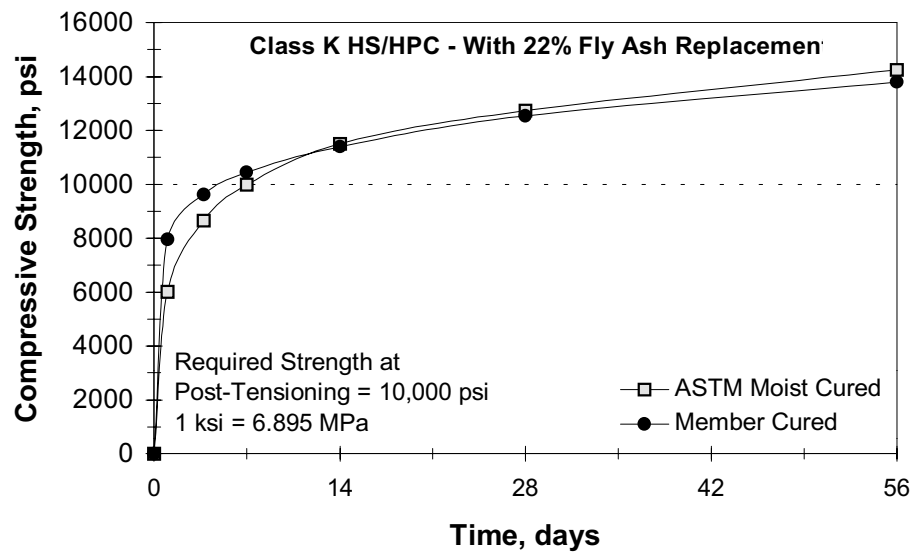


Figure 6.48: Compressive Strength Development for Louetta Road Overpass Precast Column Elements

The elastic modulus development for the Louetta Road Overpass HS/HPC precast column elements is illustrated in Figure 6.49. In terms of compressive strength, these values are equivalent to $62,480 \sqrt{f'_c}$ and $59,750 \sqrt{f'_c}$, psi, for the ASTM moist-cured and member-cured cylinders,

respectively, at a test age of 56 days. Both of these values exceed the current ACI Committee 363 and 318 (AASHTO) empirical equations.

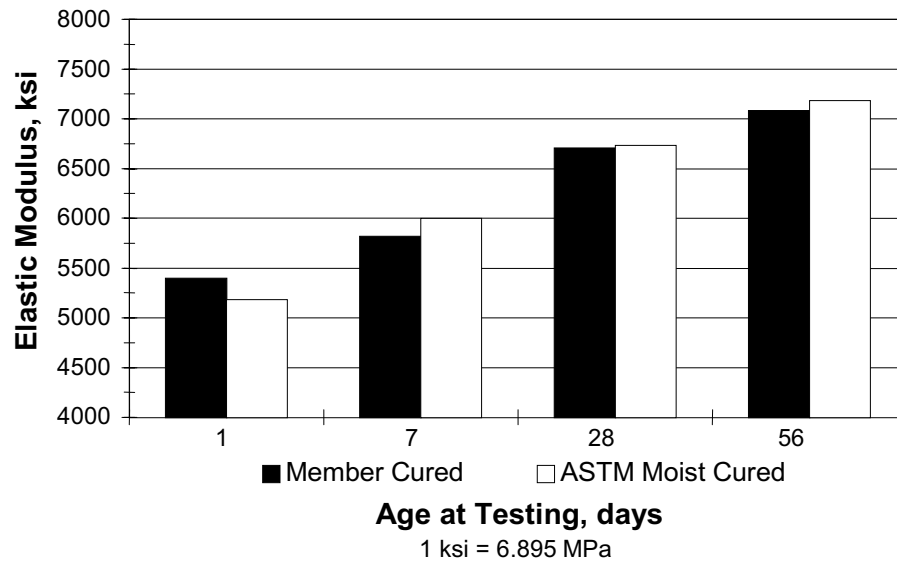


Figure 6.49: Elastic Modulus Development with Time for Louetta Road Overpass Precast Column Elements

The splitting tensile strength development for the Louetta Road Overpass HS/HPC precast column elements is illustrated in Figure 6.50. In terms of compressive strength, these values are equivalent to $8.72 \sqrt{f'_c}$ and $8.06 \sqrt{f'_c}$, psi, for the ASTM moist-cured and member-cured cylinders, respectively, at a test age of 56 days. Both of these values exceed the current ACI Committee 363 and 318 empirical equations of $7.5 \sqrt{f'_c}$ and $6.0 \sqrt{f'_c}$, psi, respectively.

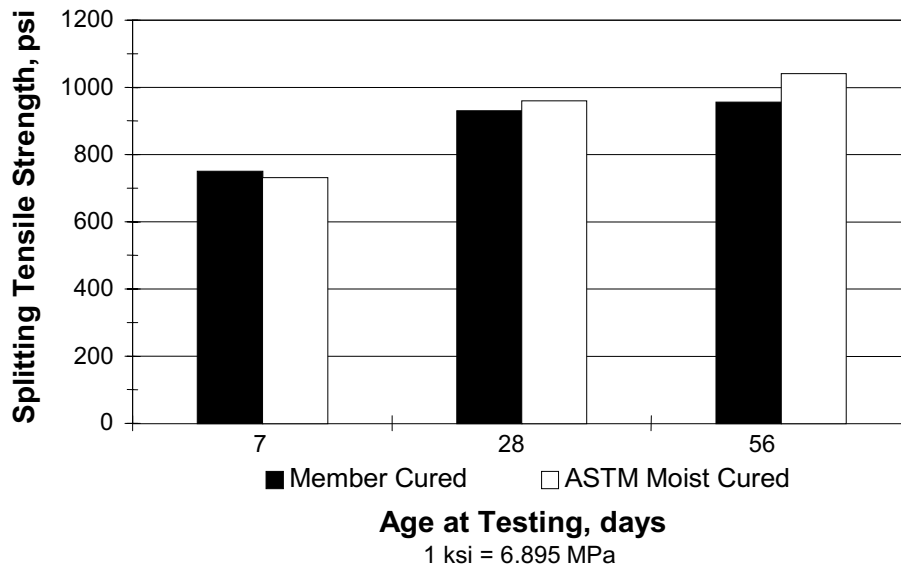


**Figure 6.50: Splitting Tensile Strength Development with Time for Louetta Road Overpass
Precast Column Elements**

Figure 6.51 illustrates the RCPT results for the substructure elements at 56 days. Both curing regimes exhibited permeabilities in the “very low” classification with an average test value of 612 (ASTM moist cured) and 680 (member cured) coulombs. ASTM moist-cured specimens exhibited a slightly lower permeability than did the member-cured specimens as a result of the additional hydration characteristics of the concrete associated with the moist curing of the specimens.

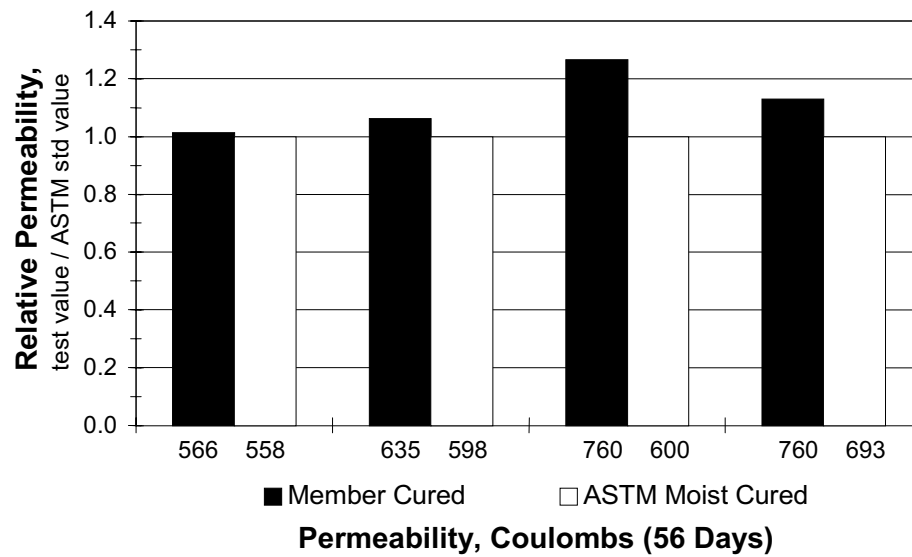


Figure 6.51: Rapid Chloride Permeability (56 Days) for Louetta Road Overpass Precast Column Elements



Figure 6.52: North Concho River Overpass Decorative Pier

6.6.2.2 North Concho River Overpass Substructure. The substructure elements for the North Concho River US 87 and S.O. RR Overpass Bridges encompassed a traditional CIP concrete pier and pier cap with decorative treatment for improved aesthetics as illustrated in Figure 6.52. The

sections were constructed by Jascon, Inc., of Uvalde, Texas. Concrete for the substructure elements was supplied by the Concho Concrete Company of San Angelo, Texas. Construction of the piers began in April 1996 and was completed in December 1996. Three different mix designs were used in casting these members. A Class K HS/HPC was selected for the eastbound mainlane HPC 41.4 MPa (6,000 psi) piers and 55.2 MPa (8,000 psi) pier caps. A Class C NSC was selected for the westbound mainlane 27.6 MPa (4,000 psi) piers and a Class F HPC for the 41.4 MPa (6,000 psi) pier caps. It should be noted that the Class F pier caps were not designated as HPC in the project specifications, but because the mix design incorporated fly ash replacement, it has been designated as HPC for the purposes of this report. Additionally, the same mix design was selected for use in the Class K piers and pier caps. On every casting date, monitoring was done for performance-related testing and evaluation by CMRG for the casting of the piers and pier caps. For the purposes of this report, test results are presented by mix design classification.

6.6.2.2.1 Class C NSC Cast-in-Place Piers. Compressive strength development was monitored for the Class C NSC piers both at early and later-age development. Figure 6.53 illustrates the compressive strength gain with time for the ASTM moist-cured and site/field-cured cylinders. A logarithmic regression analysis was performed to obtain a best-fit curve for the test data. The data exhibited excellent correlation factors ($R^2 = 0.922$ and 0.819 , respectively) for the data collected from the CIP Class C NSC piers.

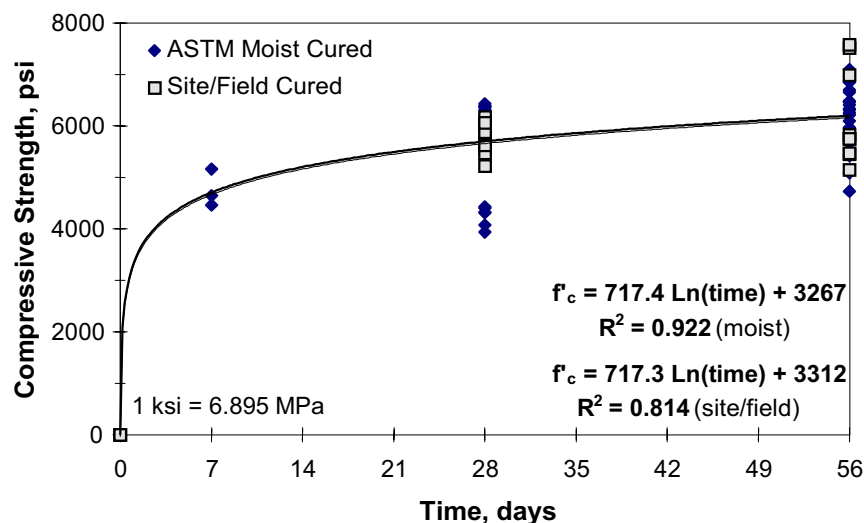


Figure 6.53: Compressive Strength Development for Class C NSC Westbound Bridge Piers

Elastic modulus development was monitored for the Class C NSC piers both at early and later-age development. A linear regression analysis was performed with and without a zero intercept to obtain the best-fit criteria for the ASTM moist-cured and site/field-cured data. The linear regression analysis resulted in an empirical equation of $E_c = 68,250 \sqrt{f'_c}$, psi, for the ASTM moist-cured cylinders and $E_c = 61,850 \sqrt{f'_c}$, psi, for the site/field-cured cylinders with zero intercepts, respectively. Figure 6.54 illustrates the elastic modulus development for the ASTM moist-cured and site/field-cured cylinders. It may be noted that the results for both curing regimes exceeded the current ACI Committee 318 empirical equation (Equation 6.1) for conventional normal weight concretes.

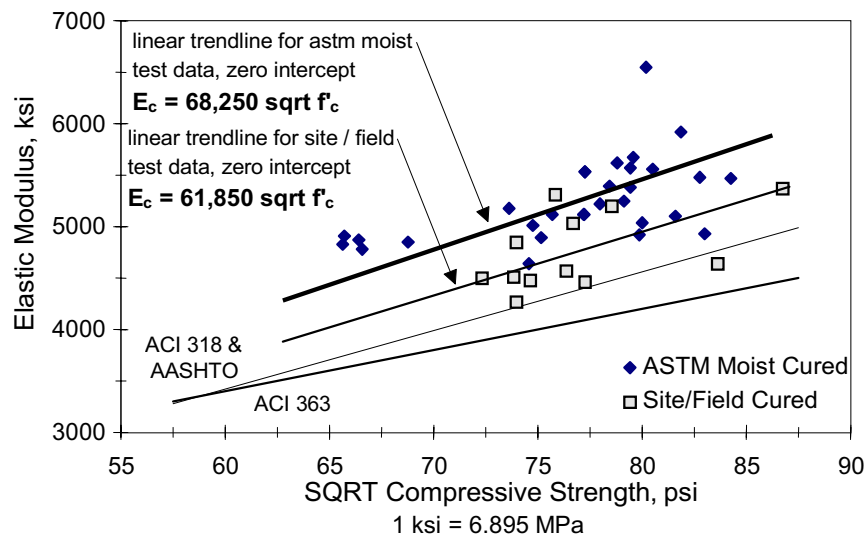


Figure 6.54: Elastic Modulus Development for Class C NSC Westbound Bridge Piers

Splitting tensile strength development was monitored for the Class C NSC piers at later-age development (28 and 56 days). A linear regression analysis was performed with a zero intercept to obtain the best-fit criteria for the ASTM moist-cured and site/field-cured data. The linear regression analysis resulted in an empirical equation of $f_{sp} = 7.46 \sqrt{f'_c}$, psi, for the ASTM moist-cured cylinders and $f_{sp} = 7.09 \sqrt{f'_c}$, psi, for the site/field-cured cylinders distributed between the bands of 5 to 10 sqrt f'_c , psi, with zero intercepts, respectively. Figure 6.55 illustrates the splitting tensile strength for the ASTM moist-cured and site/field-cured cylinders.

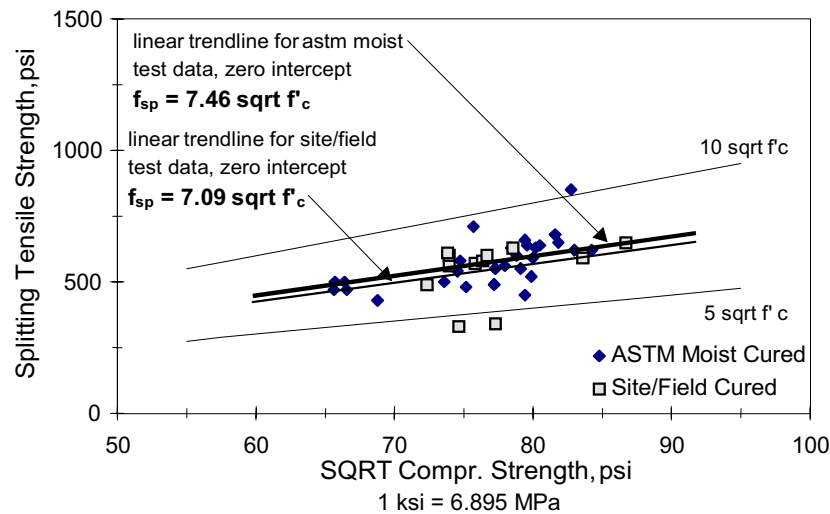


Figure 6.55: Splitting Tensile Strength Development for Class C NSC Westbound Bridge Piers

Rapid chloride permeability was monitored for each casting date at 56 days for the Class C NSC piers. Both ASTM moist-cured and site/field cured-cylinders were investigated as illustrated in Figure 6.56. The Class C mix design did not include any mineral admixtures (namely, fly ash) or water reducers to enhance the impermeability of the concrete. The ASTM moist-cured cylinders were classified as low- to moderate-permeable concrete, while the site/field-cured cylinders were classified as high-permeable concrete (Figure 6.56).

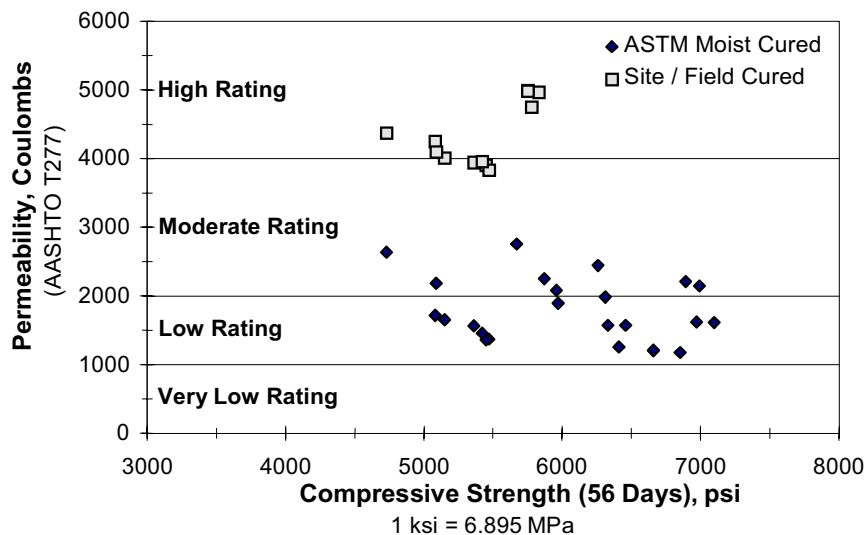


Figure 6.56: Rapid Chloride Permeability (56 Days) for Class C NSC Westbound Bridge Piers

Note the dramatic influence of the moist curing on the permeability performance of the concrete under the varied curing regimes for the NSC without mineral or chemical admixtures. The additional 56 days of moist curing resulted in a much tighter, more impermeable, paste matrix. These ASTM moist-cured specimens exhibited good permeability performance for an NSC. Typical NSC permeabilities using locally available materials in Texas range from about 3,500 to 4,500 coulombs for moist-cured specimens. However, it should be noted that the site/field-cured specimens, which best represent concrete in the structure, did not exhibit very good permeability performance. Figure 6.57 illustrates RCPT test results for a representative casting date. The impact of curing conditions on the permeability performance of NSC without mineral or chemical admixtures is much more dramatic (Figure 6.57) than HPCs with mineral and chemical admixtures as documented in Sections 6.6.2.2.2 and 6.6.2.2.3 of this report.

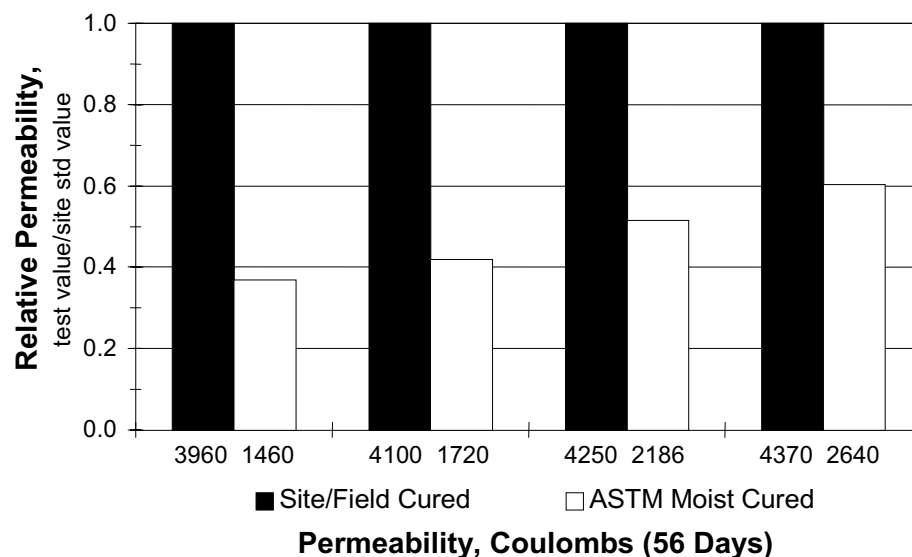


Figure 6.57: Rapid Chloride Permeability (56 Days) for Class C NSC Pier Cast on April 25, 1996

6.6.2.2.2 Class F HPC Cast-in-Place Pier Caps. Compressive strength development was monitored for the Class F HPC pier caps both at early and later-age development. Figure 6.58 illustrates the compressive strength gain with time for the ASTM moist-cured and site/field-cured cylinders. A logarithmic regression analysis was performed to obtain a best-fit curve for the test data. The data exhibited excellent correlation factors ($R^2 = 0.810$ and 0.951 , respectively) for the data collected from the CIP Class F HPC pier caps.

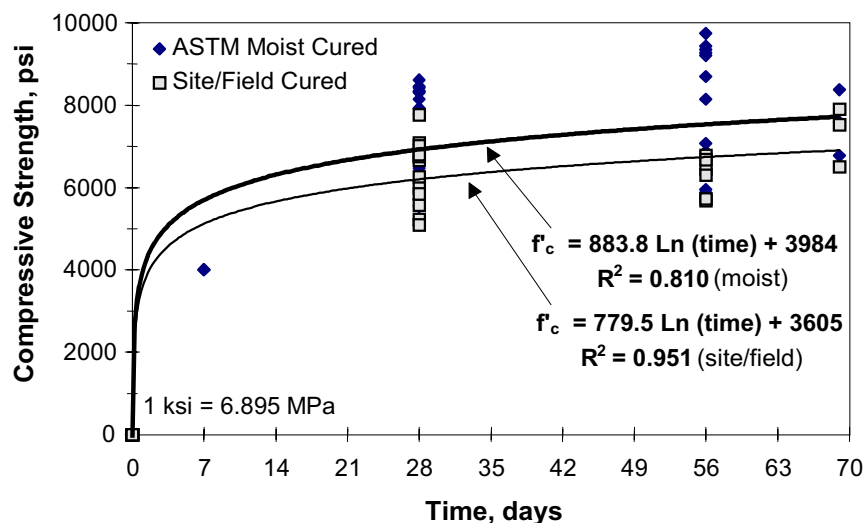


Figure 6.58: Compressive Strength Development for Class F HPC Westbound Bridge Pier Caps

Elastic modulus development was monitored for the Class F HPC pier caps both at early and later-age development. A linear regression analysis was performed with and without a zero intercept to obtain the best-fit criteria for the ASTM moist-cured and site/field-cured data. The linear regression analysis resulted in an empirical equation of $E_c = 67,600 \sqrt{f'_c}$, psi, for the ASTM moist-cured cylinders and $E_c = 67,040 \sqrt{f'_c}$, psi, for the site/field-cured cylinders with zero intercepts, respectively. Figure 6.59 illustrates the elastic modulus development for the ASTM moist-cured and site/field-cured cylinders. It may be noted that the results for both curing regimes exceeded the current ACI Committee 318 and 363 empirical equations (Equations 6.1 and 6.2) for conventional normal weight concretes and HSC, respectively.

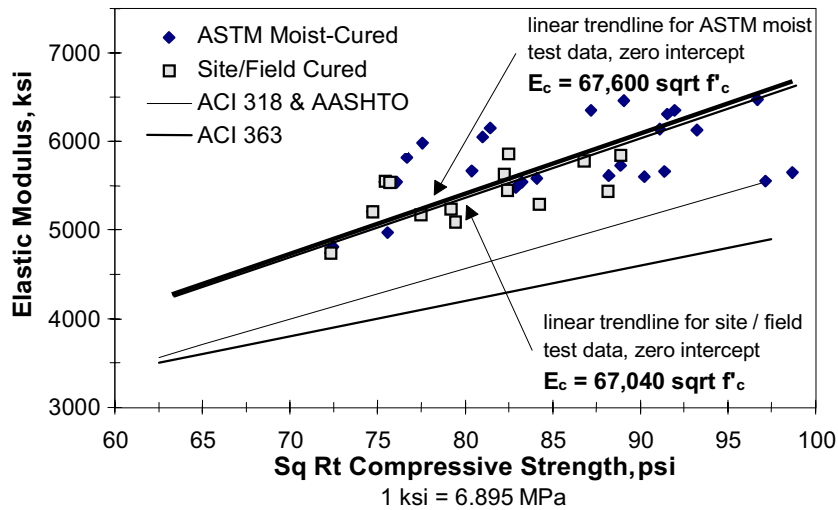


Figure 6.59: Elastic Modulus Development for Class F HPC Westbound Bridge Pier Caps

Splitting tensile strength development was monitored for the Class F HPC pier caps at later-age development (28 and 56 days). A linear regression analysis was performed with a zero intercept to obtain the best-fit criteria for the ASTM moist-cured and site/field-cured data. The linear regression analysis resulted in an empirical equation of $f_{sp} = 7.28 \text{ sqrt } f'_c$, psi, for the ASTM moist-cured cylinders and $f_{sp} = 7.81 \text{ sqrt } f'_c$, psi, for the site/field-cured cylinders distributed between the bands of 6 to 10 sqrt f'_c , psi, with zero intercepts, respectively. Figure 6.60 illustrates the splitting tensile strength for the ASTM moist-cured and site/field-cured cylinders.

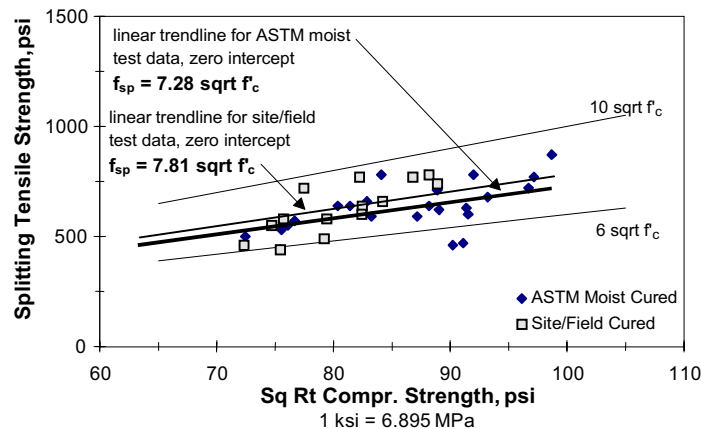


Figure 6.60: Splitting Tensile Strength Development for Class F HPC Westbound Bridge Pier Caps

Rapid chloride permeability was monitored for each casting date at 56 days for the Class F HPC pier caps. Both ASTM moist-cured and site/field cured-cylinders were investigated as illustrated in Figure 6.61. In both cases, the specimens were classified as low-permeable concrete (Figure 6.61).

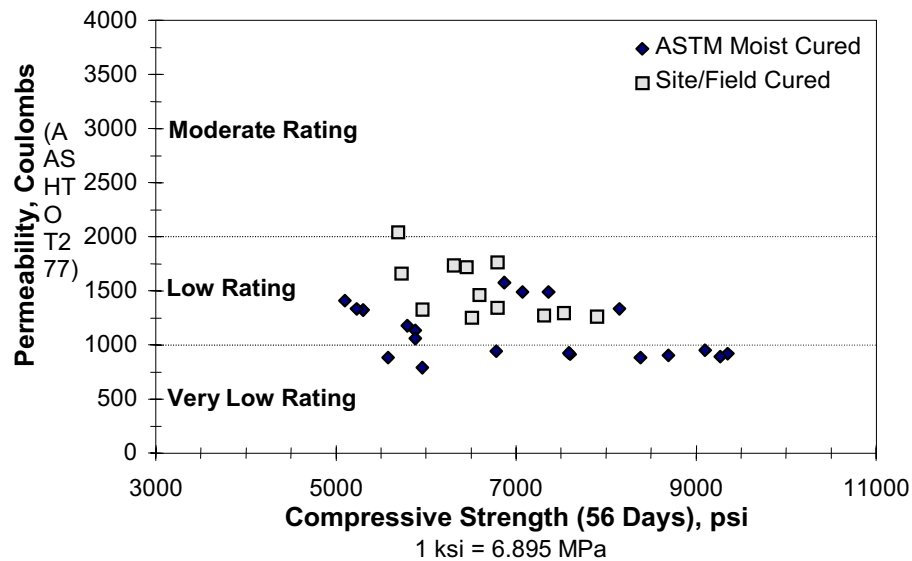


Figure 6.61: Rapid Chloride Permeability (56 Days) for Class F HPC Westbound Bridge Pier Caps

Figure 6.62 illustrates RCPT test results for a representative casting date. Note that the influence of the moist curing on the permeability performance of the concrete was not as dramatic (Figure 6.62) as compared to the Class C NSC (Figure 6.57). This may be attributed to the admixtures and the lower w/cm ratio of the HPC.

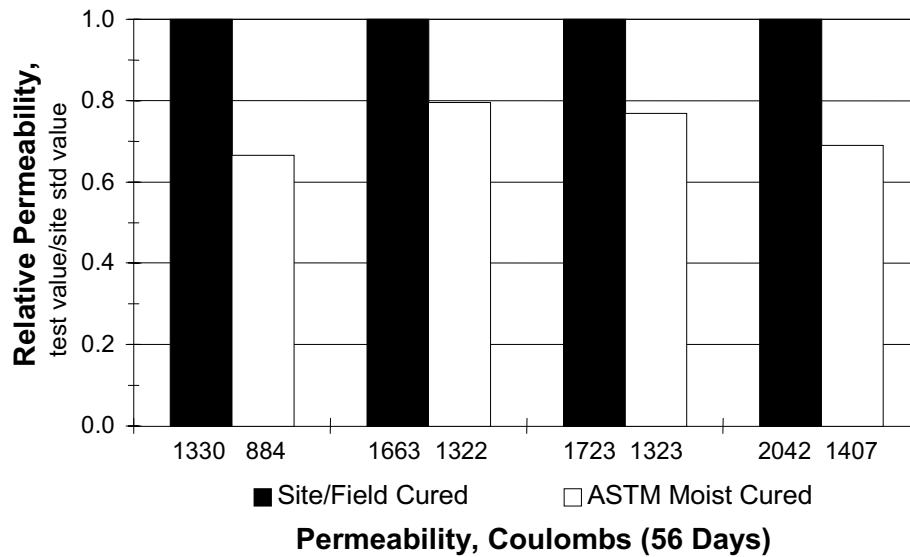


Figure 6.62: Rapid Chloride Permeability (56 Days) for Class F HPC Pier Cap Cast on July 31, 1996

6.6.2.2.3 *Class K HS/HPC Cast-in-Place Piers & Pier Caps.* Compressive strength development was monitored for the Class K HS/HPC piers and pier caps both at early and later-age development. As previously stated, the same mix design was selected for both of these components. Therefore, for reporting purposes, the data has been grouped. Figure 6.63 illustrates the compressive strength gain with time for the ASTM moist-cured and site/field-cured cylinders. A logarithmic regression analysis was performed to obtain a best-fit curve for the test data. The data exhibited excellent correlation factors ($R^2 = 0.950$ and 0.921 , respectively) for the data collected from the CIP Class K HS/HPC piers and pier caps.

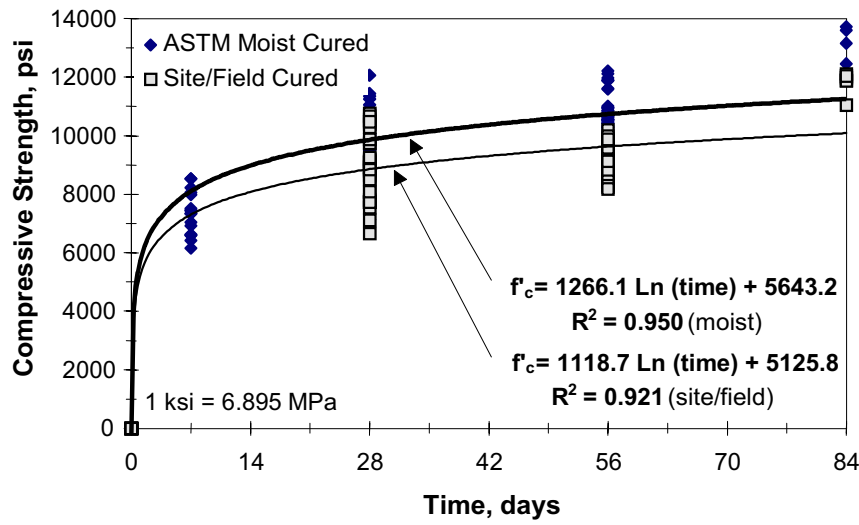


Figure 6.63: Compressive Strength Development for Class K HS/HPC Eastbound Bridge Piers and Pier Caps

Elastic modulus development was monitored for the Class K HS/HPC piers and pier caps both at early and later-age development. A linear regression analysis was performed with and without a zero intercept to obtain the best-fit criteria for the ASTM moist-cured and site/field-cured data. The linear regression analysis resulted in an empirical equation of $E_c = 64,180 \sqrt{f'_c}$, psi, for the ASTM moist-cured cylinders and $E_c = 61,800 \sqrt{f'_c}$, psi, for the site/field-cured cylinders with zero intercepts, respectively. Figure 6.64 illustrates the elastic modulus development for the ASTM moist-cured and site/field-cured cylinders. It may be noted that the results for both curing regimes exceeded the current ACI Committee 318 and 363 empirical equations (Equations 6.1 and 6.2) for conventional normal weight concretes and HSC, respectively.

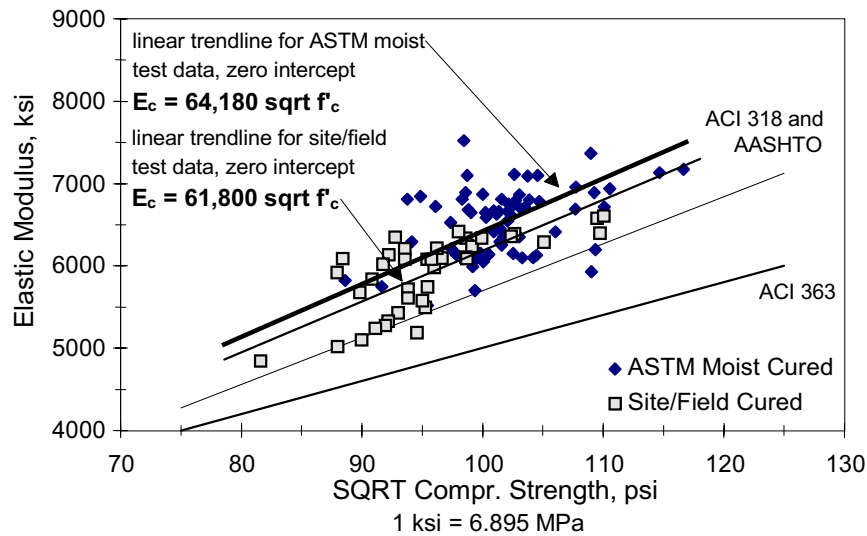


Figure 6.64: Elastic Modulus Development for Class K HS/HPC Eastbound Bridge Piers and Pier Caps

Splitting tensile strength development was monitored for the Class K HS/HPC piers and pier caps at later-age development (28 and 56 days). A linear regression analysis was performed with a zero intercept to obtain the best-fit criteria for the ASTM moist-cured and site/field-cured data. The linear regression analysis resulted in an empirical equation of $f_{sp} = 7.64 \sqrt{f'_c}$, psi, for the ASTM moist-cured cylinders and $f_{sp} = 7.81 \sqrt{f'_c}$, psi, for the site/field-cured cylinders distributed between the bands of 6 to 10 $\sqrt{f'_c}$, psi, with zero intercepts, respectively. Figure 6.65 illustrates the splitting tensile strength for the ASTM moist-cured and site/field-cured cylinders. Both values satisfy the current ACI Committee 363 empirical equation of $f_{sp} = 7.4 \sqrt{f'_c}$, psi, for HSC.

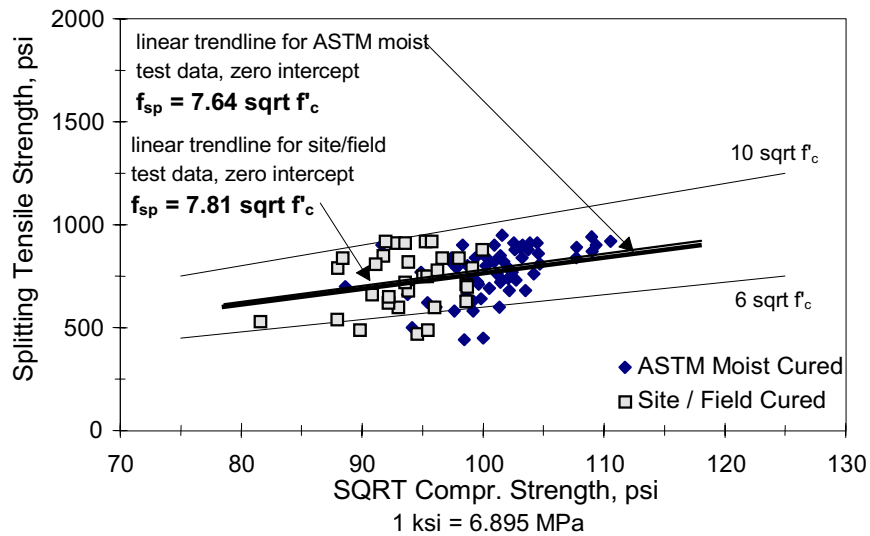


Figure 6.65: Splitting Tensile Strength Development for Class K HS/HPC Eastbound Bridge Piers and Pier Caps

Rapid chloride permeability was monitored for each casting date at 56 days for the Class K HS/HPC piers and pier caps. Both ASTM moist-cured and site/field-cured cylinders were investigated as illustrated in Figure 6.66. In both cases the cylinders were classified as low- to very low-permeable concrete (Figure 6.65). Figure 6.66 illustrates RCPT test results for a representative casting date. Note that the influence of the moist curing on the permeability performance of the concrete was minimal (Figure 6.67) as compared to the Class C NSC (Figure 6.57).

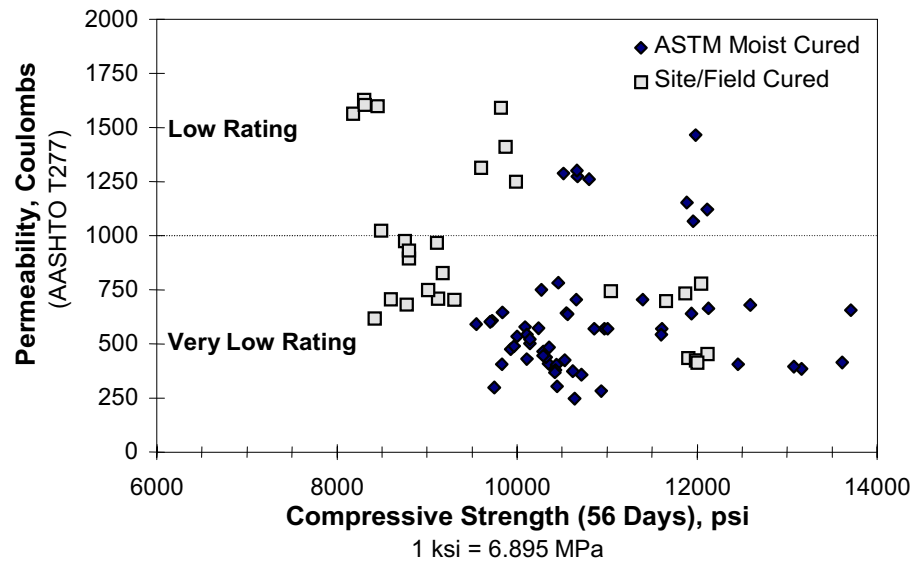


Figure 6.66: Rapid Chloride Permeability (56 Days) for Class K HS/HPC Eastbound Bridge Piers and Pier Caps

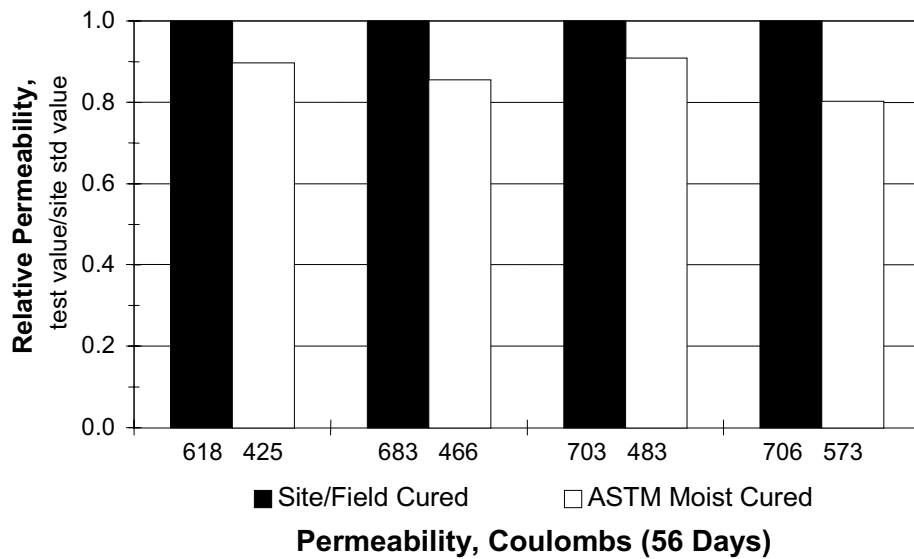


Figure 6.67: Rapid Chloride Permeability (56 Days) for Class K HS/HPC Pier Cap Cast on September 12, 1996

In the case of mix designs with an ASTM Class C fly ash replacement and HRWR, the ASTM moist-cured cylinders provided a reasonable representation (± 20 percent) for the permeability performance of the concrete in the structure (i.e., site/field-cured cylinders) at 56 days.

6.6.2.2.4 Comparison of Mechanical and Material Properties. A comparison of the mechanical and material properties monitored for the mix designs selected for use in the substructure elements is presented in Table 6.69 to illustrate the enhanced performance characteristics of the HS/HPC and HPC over the conventional concretes currently utilized in Texas. The standard TxDOT Class C concrete, which was used for the non-HPC (NSC) piers, required 6 sacks of cement per cubic yard of concrete as per TxDOT Standard Specifications [72] (Table 6.45), while the Class F HPC and Class K HS/HPC used 8 and 7.5 sacks (std. U.S.) of cementitious material per cubic yard of concrete, respectively. The Class F HPC mix utilized slightly more cementitious material than the Class K HS/HPC to compensate for the associated loss of strength due to the use of an air-entraining agent. Air entrainment was required per TxDOT standard specification [72] for Class C and Class F concrete. The Class K HS/HPC did not utilize air entrainment because the piers and pier caps were not subjected to critical saturation as discussed in Section 2.3.1.2. Furthermore, due to the very low w/cm ratio (< 0.35) of the HS/HPC mix design, a discontinuous system of capillaries could be anticipated within 36 to 48 hours after casting. Without a network of continuous capillaries, saturation of the concrete beyond the critical saturation level of 91.7 percent is not physically possible. Therefore, freeze-thaw resistance for the HS/HPC piers and pier caps was not warranted.

From a mechanical property performance standpoint, the Class K HS/HPC mix design exhibited an improvement in compressive strength, elastic modulus, and splitting tensile strength by 77.2, 23.6, and 37.5 percent, respectively, over the Class C NSC (Table 6.69). The Class K HS/HPC also exhibited a very low permeability, which translates into reduced maintenance costs and an extended life expectancy from 40 years to an anticipated service life of 75 years. Of particular interest, is the variation between permeability when comparing the ASTM moist-cured cylinders with the companion site/field-cured cylinders.

Figure 6.68 illustrates the average permeability test results from monitored casting dates in which both ASTM moist-cured and site/field-cured cylinders were investigated for the North Concho River Overpass substructure elements. The illustration has been classified as mix designs with and without fly ash and HRWRs to interpret the impact of moist curing and admixtures on the specimens including data from all three classified mix designs presented above. For the mix designs without fly ash and HRWR, Figure 6.68 illustrates that the permeability of the ASTM moist-cured specimens is a poor representation of the permeability of the concrete in the structure (site/field-cured specimens). In fact, these two conditions result in different permeability classifications, moderate (ASTM moist-cured

specimens) and high (site/field-cured specimens). Mix designs with fly ash had permeabilities with a similar classification, low (w/FA and w/o HRWR) and very low (w/FA and HRWR).

Table 6.69: Comparison of North Concho River Overpass Mix Design Mechanical and Material Property Performance

Mechanical or Material Property	Class C NSC	Class F HPC	Class K HS/HPC
Required Design Strength at 28 Days	4,000 psi	6,000 psi	piers - 6,000 psi caps - 8,000 psi
Substructure Element at 28 Days			
Compressive Strength	5,570 psi	7,070 psi	9,870 psi
Elastic Modulus	5,220 ksi	5,620 ksi	6,450 ksi
Splitting Tensile Strength	560 psi	590 psi	770 psi
Substructure Element at 56 Days			
Compressive Strength	5,740 psi	7,560 psi	10,730 psi
Elastic Modulus	4,750 ksi	5,830 ksi	6,590 ksi
Splitting Tensile Strength	500 psi	650 psi	800 psi
Rapid Chloride Permeability (at 56 Days)	1,960 Coulombs (Low Rating)	1,120 Coulombs (Low Rating)	630 Coulombs (Very Low Rating)
Values presented above are the average of all casting dates monitored for the specified mix design. Results presented above are for ASTM Moist-Cured Cylinders. 1 ksi = 1,000 psi = 6.895 MPa			

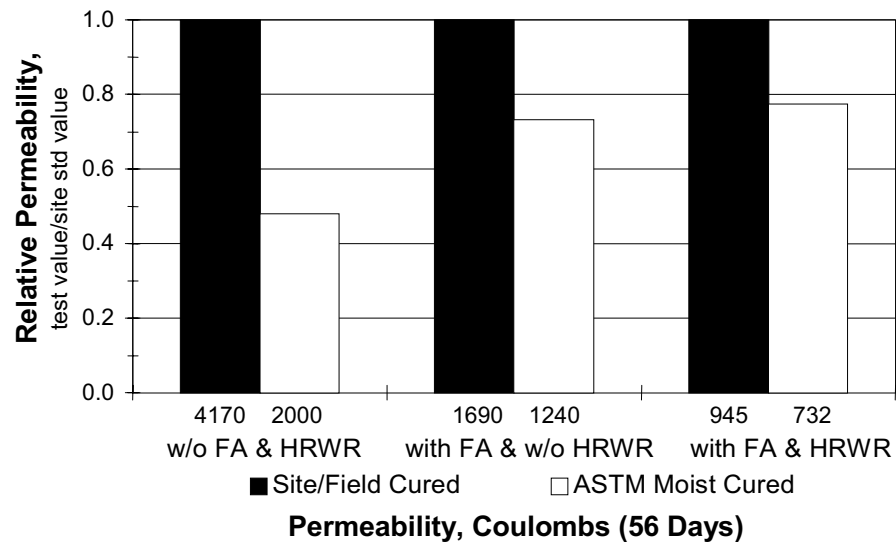


Figure 6.68: Average Rapid Chloride Permeability (56 Days) for North Concho River Substructure Mix Designs with and without Fly Ash and HRWRs

For conventional concretes without mineral or chemical admixtures, it is clear that the permeability of ASTM moist-cured specimens does not result in comparable values for concrete in the field. However, for HPCs using mineral or chemical admixtures, ASTM moist-cured cylinders provide a reasonable representation for expected field performance for CIP concrete.

From a materials cost standpoint, the cementitious costs were actually lower for the Class K HS/HPC when compared to the Class C NSC due to the use of a 30 percent ASTM Class C fly ash replacement. Cementitious cost for the Class K and Class C concretes were \$15.84 and \$16.17 per cubic meter (\$20.72 and \$21.15 per cubic yard), respectively. The mix designs were comparable from an overall material costs perspective. The Class K HS/HPC and Class C NSC mix designs were \$29.50 and \$27.28 per cubic meter (\$38.59 and \$35.68 per cubic yard), respectively, for all materials. The cost represented an increase of 8.2 percent for the Class K HS/HPC when compared to the Class C NSC. This higher initial cost is easily offset by the long-term maintenance cost savings and extended life expectancy of the bridge. This does not include transportation and placement costs that were similar. Comprehensive material costs for the mix designs are illustrated in Appendix E.

6.6.3 Performance-Related Test Results of Precast Deck Panels

Performance-related testing and monitoring of the precast deck panels were performed as follows.

6.6.3.1 Louetta Road Overpass Precast Deck Panels. The precast deck panels for the Louetta Road Overpass consisted of 88.9-mm (3.5-in.) thick precast/transversely prestressed panels with 9.5-mm (0.38-in.) diameter strands spaced at 150-mm (6-in.) o.c. and a layer of welded wire fabric (WWF 12x4xW3.5xW7.5) at mid-depth. The sections were fabricated by the Williams Brothers Construction Company, Inc. Precast Plant located in Houston, Texas. The panels were fabricated over a limited number of casting dates in September 1996. Due to the limited number of casting dates, only four of the plant castings were monitored for performance-related testing and evaluation by CMRG.

Williams Brothers Construction Company, Inc. utilized the same mix design for both the northbound and southbound precast deck panels. The mix design used for the precast panels incorporated a crushed river gravel and 7-3/4 sacks (std. U.S.) of cementitious (cement + fly ash) materials. The compressive strength development of the moist-cured and member-cured cylinders is illustrated in Figure 6.69. A logarithmic regression analysis was performed to obtain a best-fit curve for the test data. The data exhibited very good correlation factors ($R^2 = 0.834$ and 0.819 , respectively) for the data collected from the precast deck panels. The variation between concrete placement

temperature and peak hydration temperature was 16.7 °C (30 °F), and the panels monitored on September 12, 1996 attained a maximum concrete temperature of 53.9 °C (129 °F) during hydration. Consequently, the early and long-term compressive strength development did not vary drastically between curing conditions, as illustrated in Figure 6.69. This result was similar to that observed for the Louetta Road Overpass precast column elements.

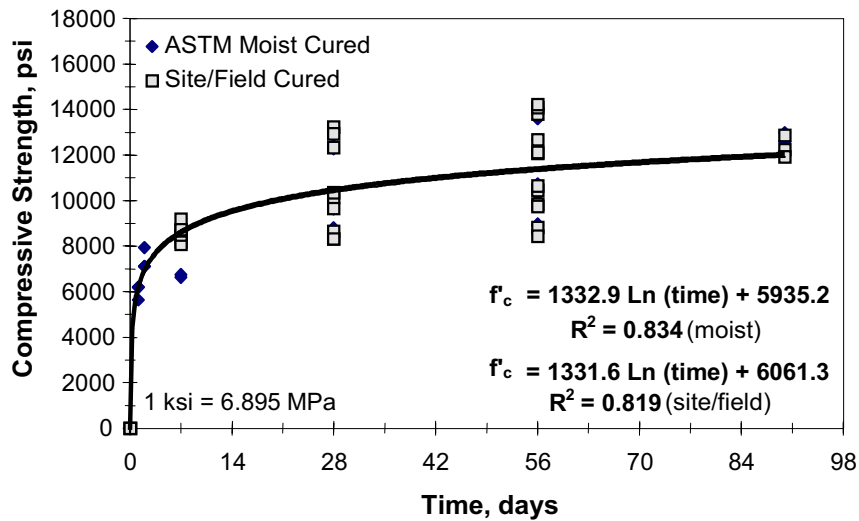


Figure 6.69: Compressive Strength Development for Louetta Road Overpass Precast Deck Panels

The elastic modulus development for the Louetta Road Overpass HS/HPC precast panels is illustrated in Figure 6.70. A linear regression analysis was performed with a zero intercept to obtain the best-fit criteria for the ASTM moist-cured and member-cured data. The linear regression analysis resulted in an empirical equation of $E_c = 61,160 \sqrt{f'_c}$, psi, for the ASTM moist-cured cylinders and $E_c = 57,430 \sqrt{f'_c}$, psi, for the member-cured cylinders with zero intercepts, respectively. Both of these values exceed the current ACI Committee 363 and 318 (AASHTO) empirical equations. Approximately 90 percent of the long-term (56 day) elastic modulus development for the member-cured specimens occurred at early-age (1 day) while only 70 percent occurred for the ASTM moist-cured specimens as illustrated in Figure 6.71. The more pronounced elastic modulus development for the member-cured specimens at early age may be attributed to the influence of the hydration temperature of the panels.

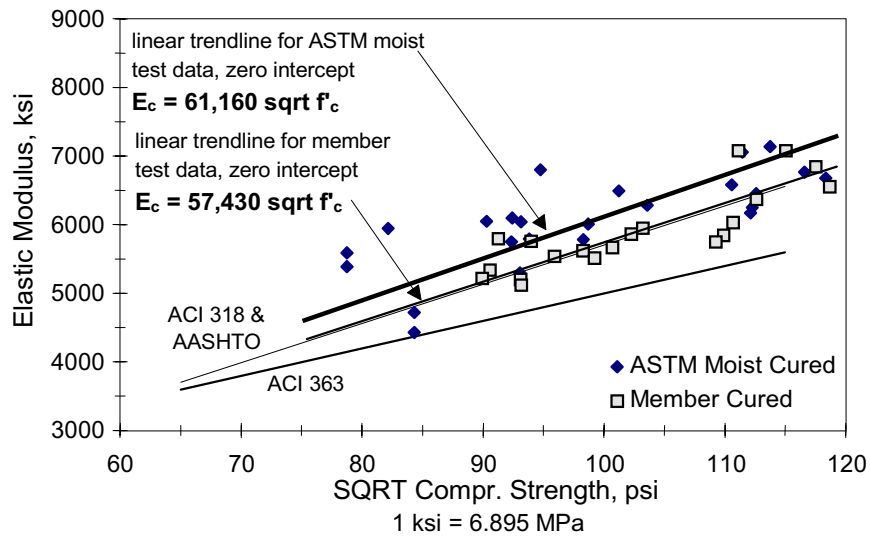


Figure 6.70: Elastic Modulus Development for Louetta Road Overpass Precast Deck Panels

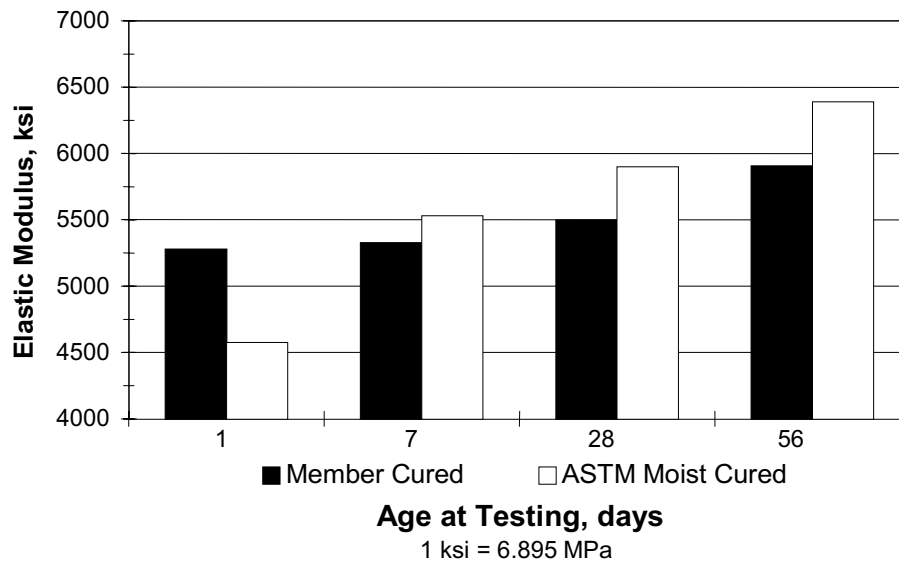


Figure 6.71: Elastic Modulus Development with Time for Louetta Road Overpass Precast Deck Panels

Splitting tensile strength development was monitored for the precast deck panels at later-age development (7, 28, and 56 days). A linear regression analysis was performed with a zero intercept to obtain the best-fit criteria for the ASTM moist-cured and member-cured data.

The linear regression analysis resulted in an empirical equation of $f_{sp} = 7.85 \sqrt{f'_c}$, psi, for the ASTM moist-cured cylinders and $f_{sp} = 8.06 \sqrt{f'_c}$, psi, for the member-cured cylinders distributed between the bands of 6 to 10 $\sqrt{f'_c}$, psi, with zero intercepts, respectively. Figure 6.72 illustrates the splitting tensile strength for the ASTM moist-cured and member-cured cylinders.

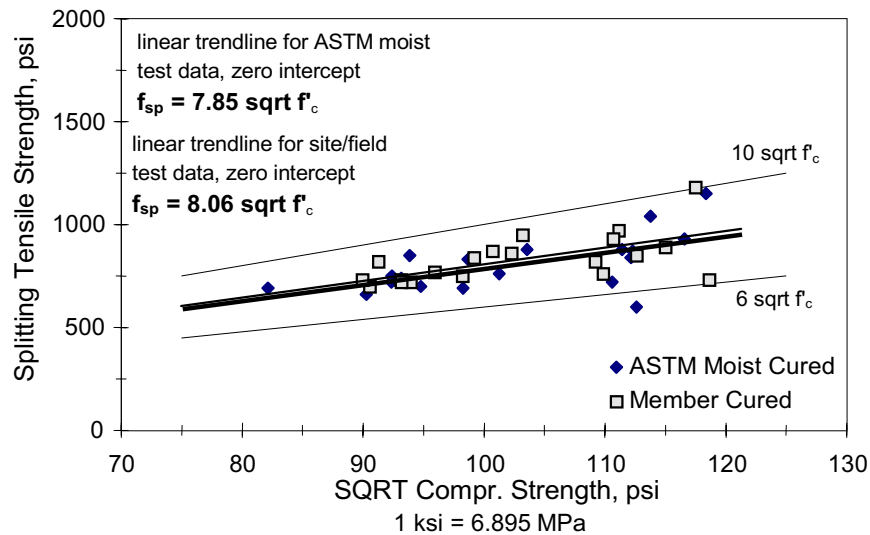


Figure 6.72: Splitting Tensile Strength Development for Louetta Road Overpass Precast Deck Panels

Figure 6.73 illustrates the RCPT results at 56 days for the precast deck panels, while Figure 6.74 illustrates individual test results for a representative casting date. ASTM moist-cured specimens exhibited permeabilities in the low classification with an average test value of 1,380 coulombs satisfying project guidelines for permeability. Member-cured specimens exhibited permeabilities in the low to moderate classification with an average test value of 1,990 coulombs. ASTM moist-cured specimens exhibited a slightly lower permeability than the member-cured specimens due to the additional hydration characteristics of the concrete associated with the moist curing of the specimens.

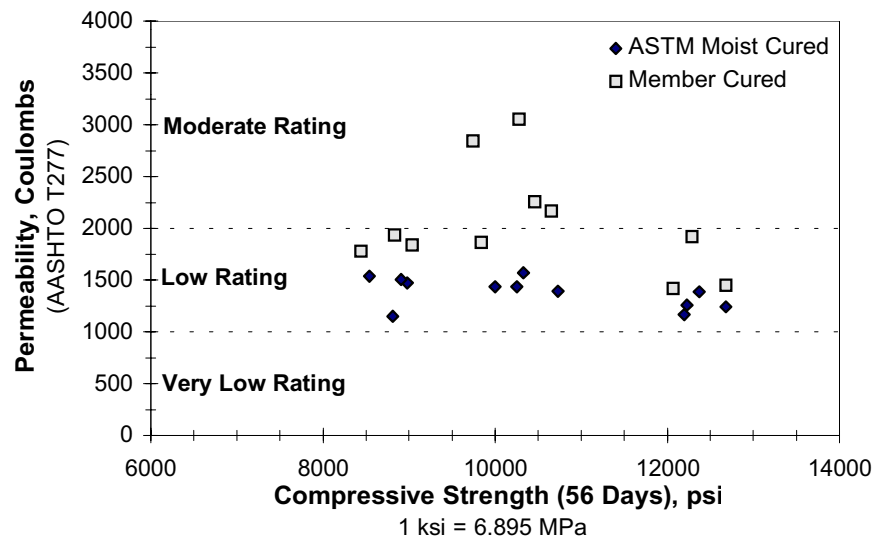


Figure 6.73: Rapid Chloride Permeability (56 Days) for Louetta Road Overpass Precast Deck Panels

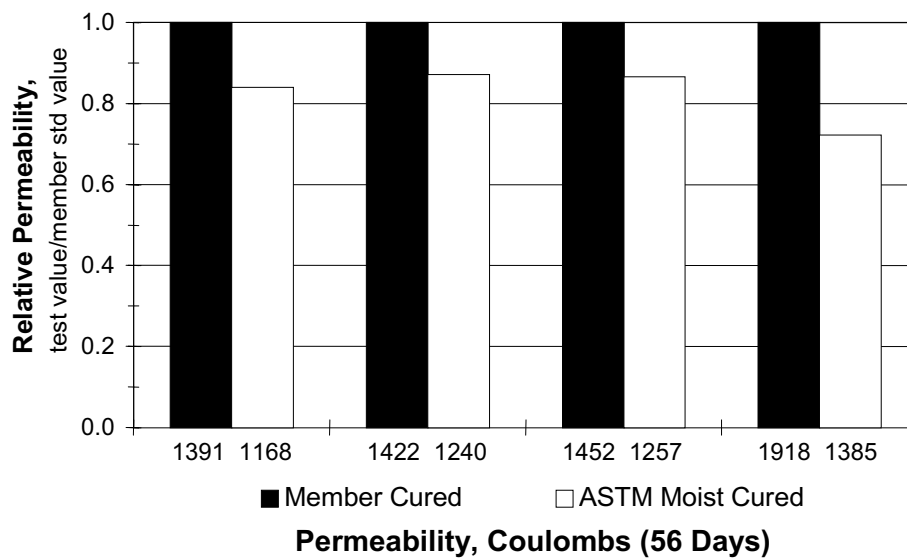


Figure 6.74: Rapid Chloride Permeability (56 Days) for Louetta Road Overpass Precast Deck Panels Cast September 18, 1996

Because of the high surface-area-to-volume ratio (SA/V) of 7.44 for the precast panels, excessive hydration temperatures were not encountered during casting due to efficient heat

dissipation. Therefore, concrete temperatures during hydration did not significantly impact the permeability of the member-cured specimens as was the case for the precast beams as discussed in Chapter 8. With the exception of two data points located in the mid-moderate classification, the ASTM moist-cured specimens provided a reasonable representation for the permeability of the concrete in the panels (member-cured specimens).



Figure 6.75: North Concho River Overpass Bridge Precast Deck Panels

6.6.3.2 North Concho River Overpass Precast Deck Panels. The precast deck panels (Figure 6.75) for the North Concho River Overpass consisted of 101.6-mm (4-in.) thick precast/transversely prestressed panels with 9.5-mm (0.38-in.) diameter strands spaced at 150-mm (6-in.) and a layer of welded wire fabric (WWF 12x4xW3.5xW7.5) at mid-depth. The panels were fabricated by Bexar Concrete Works, Inc. located in San Antonio, Texas. The westbound (WB) overpass panels were fabricated from early September 1996 through early October 1996. The eastbound (EB) overpass panels were fabricated during the months of January and February 1997. Each mix design for the WB and EB precast panels incorporated an HRWR by the fabricator in order to attain release strengths within a 24-hour period, thereby not impeding plant productivity. The WB and EB panel mix designs incorporated 6 and 7 sacks (std. U.S.) of cement, respectively, utilizing the plant's standard south Texas crushed limestone. The EB precast panel mix design was developed in the laboratory by the researchers under Phase I of this research program to incorporate crushed river gravel and a 30 percent fly ash replacement to reduce permeability and improve the long-term durability of the concrete. However, the fabricators elected not to incorporate fly ash replacement because of their production limitations. They anticipated the construction of a new batch plant with a storage/production silo designated for fly ash distribution, but the new plant was not completed prior to the fabrication of the panels, therefore, the fabricators elected not to incorporate fly ash or

nonstandard aggregates (those from an outside source). The use of the soft porous south Texas limestone and lack of fly ash replacement affected the permeability performance of the EB HPC panels as discussed below.

6.6.3.2.1 Westbound Precast Deck Panels. As noted above, similar mix designs were used for both the North Concho River Overpass westbound and eastbound precast panels by the fabricator, with the exception of cement content (Table 6.15). Although the WB mix design has very good strength gain characteristics due to the use of an HRWR, for reporting purposes, the mix design has been designated as NSC because of its modest required design strength of 34.5 MPa (5,000 psi). The compressive strength development of the moist-cured and member-cured cylinders is illustrated in Figure 6.76. A logarithmic regression analysis was performed to obtain a best-fit curve for the test data. The data exhibited excellent correlation factors ($R^2 = 0.969$ and 0.984 , respectively) for the data collected from the precast deck panels. The variation between concrete placement temperature and maximum hydration temperature was $3.3\text{ }^{\circ}\text{C}$ ($6\text{ }^{\circ}\text{F}$), and the panels monitored on September 4, 1996 attained a peak hydration temperature of $40\text{ }^{\circ}\text{C}$ ($104\text{ }^{\circ}\text{F}$). Consequently, the early and long-term compressive strength development did not vary drastically between curing conditions, as illustrated in Figure 6.76.

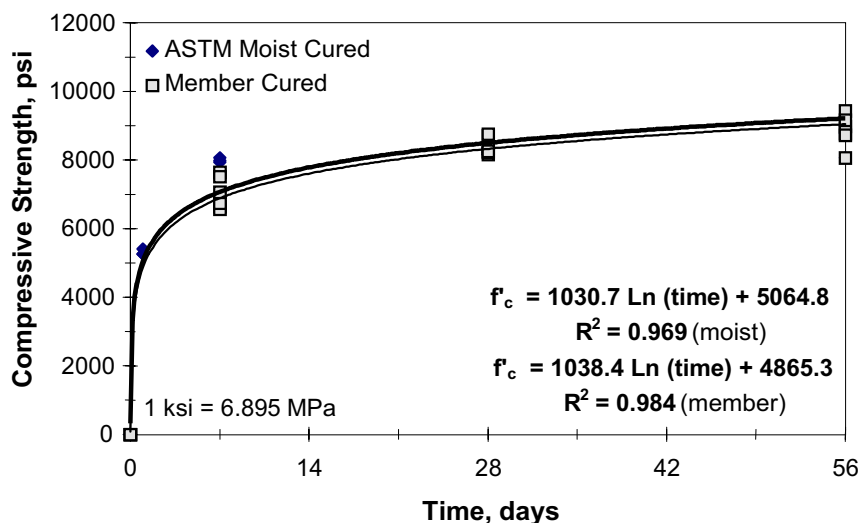


Figure 6.76: Compressive Strength Development for North Concho River Overpass Westbound NSC Precast Deck Panels

The elastic modulus development for the North Concho River Overpass NSC precast panels is illustrated in Figure 6.77. A linear regression analysis was performed with a zero intercept to obtain the best-fit criteria for the ASTM moist-cured and member-cured data. The linear regression analysis resulted in an empirical equation of $E_c = 54,880 \sqrt{f'_c}$, psi, for the ASTM moist-cured cylinders and $E_c = 50,860 \sqrt{f'_c}$, psi, for the member-cured cylinders with zero intercepts, respectively. Neither of these values satisfied the current ACI 318 (AASHTO) empirical equation for conventional concrete, primarily due to the composition of the south Texas limestone (abundance of the mineral calcite).

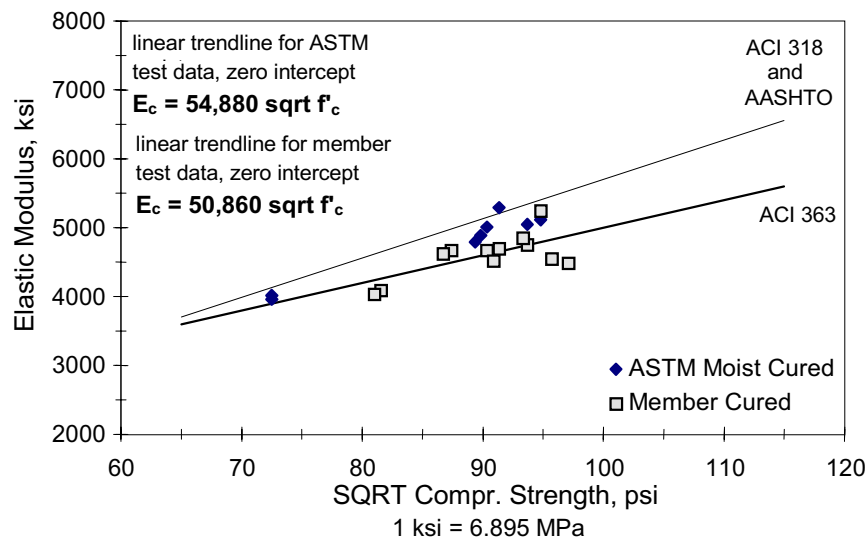


Figure 6.77: Elastic Modulus Development for North Concho River Overpass Westbound NSC Precast Deck Panels

Splitting tensile strength development was monitored for the precast deck panels at later-age development (28 and 56 days). A linear regression analysis was performed with a zero intercept to obtain the best-fit criteria for the ASTM moist-cured and member-cured data. The linear regression analysis resulted in an empirical equation of $f_{sp} = 7.77 \sqrt{f'_c}$, psi, for the ASTM moist-cured cylinders and $f_{sp} = 7.33 \sqrt{f'_c}$, psi, for the member-cured cylinders distributed between the bands of 5 to 10 sqrt

f'_c , psi, with zero intercepts, respectively. Figure 6.78 illustrates the splitting tensile strength for the ASTM moist-cured and member-cured cylinders.

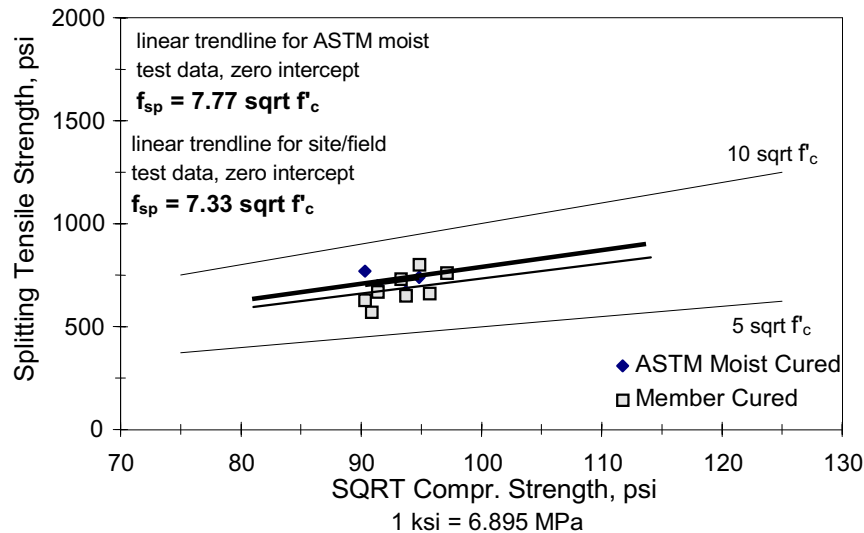


**Figure 6.78: Splitting Tensile Strength Development for North Concho River Overpass
Westbound NSC Precast Deck Panels**

6.6.3.2.2 Eastbound Precast Deck Panels. The eastbound panels were designated as HPC with a required design strength of 41.4 MPa (6,000 psi). The compressive strength development of the moist-cured and member-cured cylinders is illustrated in Figure 6.79. A logarithmic regression analysis was performed to obtain a best-fit curve for the test data. The data exhibited excellent correlation factors ($R^2 = 0.931$ and 0.961 , respectively) for the data collected from the precast deck panels. The variation between concrete placement temperature and peak hydration temperature was 3.9°C (7°F), and the panels monitored on February 5, 1997 attained a maximum concrete temperature during hydration of 21.7°C (71°F) for. Consequently, the early and long-term compressive strength development did not vary drastically between curing conditions, as illustrated in Figure 6.79.

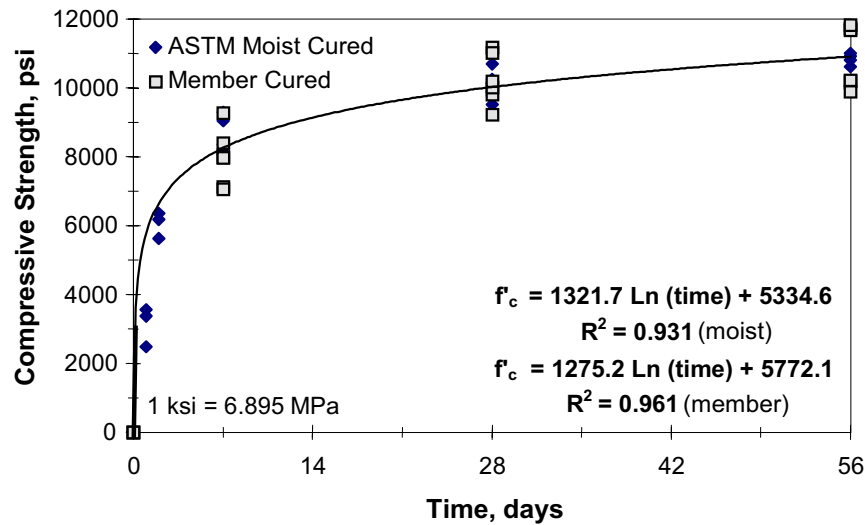


Figure 6.79: Compressive Strength Development for North Concho River Overpass Eastbound HPC Precast Deck Panels

The elastic modulus development for the North Concho River Overpass HPC precast panels is illustrated in Figure 6.80. A linear regression analysis was performed with a zero intercept to obtain the best-fit criteria for the ASTM moist-cured and member-cured data. The linear regression analysis resulted in an empirical equation of $E_c = 49,290 \sqrt{f'_c}$, psi, for the ASTM moist-cured cylinders and $E_c = 47,970 \sqrt{f'_c}$, psi, for the member-cured cylinders with zero intercepts, respectively. Neither of these values satisfied the current ACI Committee 363 and 318 (AASHTO) empirical equations because of the aggregate selected by the fabricator. The effect of a "soft" aggregate in terms of elastic modulus development for an HS/HPC (Figure 6.80) is much more pronounced than for a more conventional mix design (Figure 6.77).

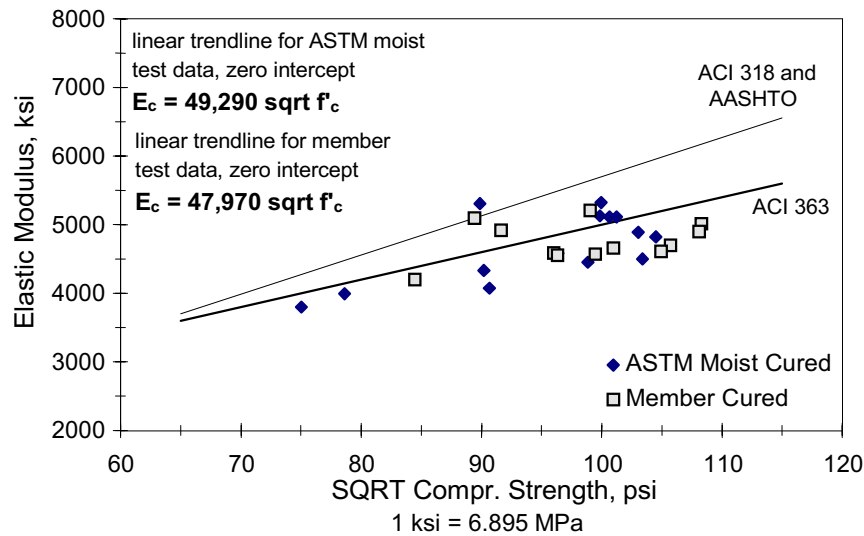


Figure 6.80: Elastic Modulus Development for North Concho River Overpass Eastbound HPC Precast Deck Panels

Splitting tensile strength development was monitored for the precast deck panels at later-age development (28 and 56 days). A linear regression analysis was performed with a zero intercept to obtain the best-fit criteria for the ASTM moist-cured and member-cured data. The linear regression analysis resulted in an empirical equation of $f_{sp} = 8.33 \sqrt{f'_c}$, psi, for the ASTM moist-cured cylinders and $f_{sp} = 7.70 \sqrt{f'_c}$, psi, for the member-cured cylinders distributed between the bands of 6 to 10 $\sqrt{f'_c}$, psi, with zero intercepts, respectively. Figure 6.81 illustrates the splitting tensile strength for the ASTM moist-cured and member-cured cylinders.

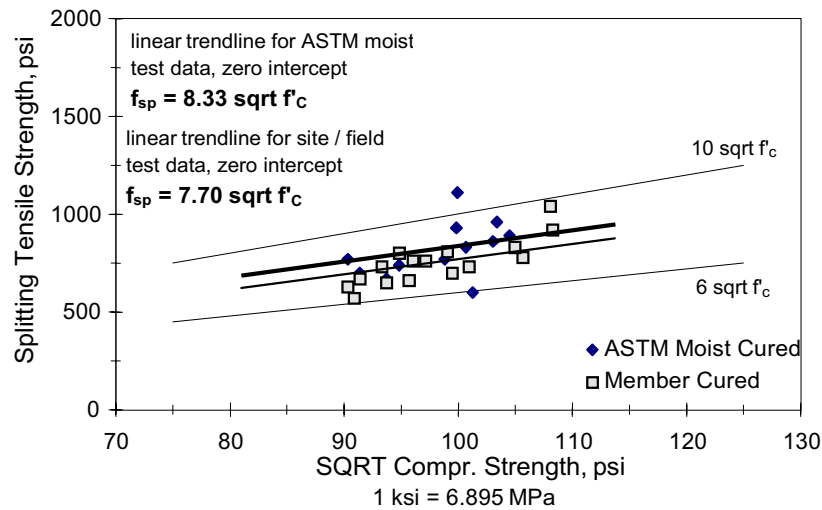


Figure 6.81: Splitting Tensile Strength Development for North Concho River Overpass Eastbound HPC Precast Deck Panels

Figure 6.82 illustrates the RCPT results at 56 days for the EB HPC precast deck panels. While the EB precast panels developed excellent strength gain characteristics at 56 days [68.9 to 82.7 MPa (10,000 to 12,000 psi)] with a low w/cm ratio (0.38), the permeability was merely classified as moderate for both curing regimes investigated. It is clearly evident that the HPC mix design selected by the fabricator does not address permeability.

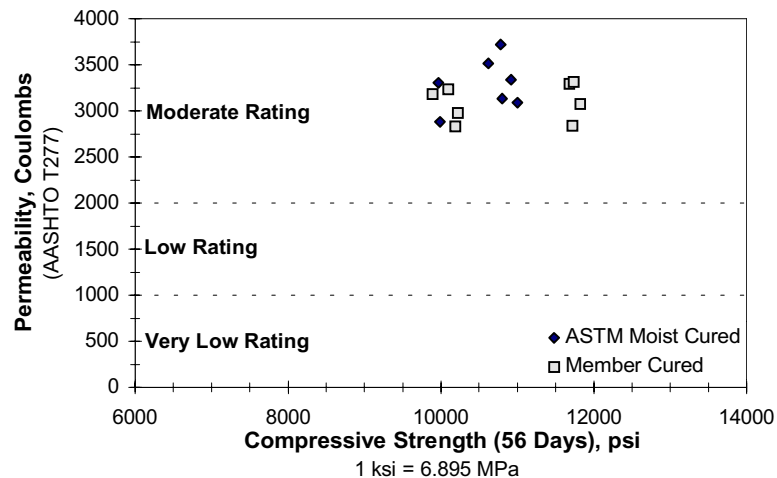


Figure 6.82: Rapid Chloride Permeability (56 Days) for North Concho River Overpass Eastbound HPC Precast Deck Panels

This further validates the concept discussed in Section 6.5.3 that neither compressive strength nor w/cm ratio is the best indicator of permeability or durability performance for a particular mix design. The interaction and selection of materials are extremely important and must be investigated if durability performance is the primary performance issue for the mix design. In this case, the selection of the soft porous aggregate and relatively porous paste matrix by the fabricator resulted in what could be classified as poor permeability performance, particularly for a high-strength mix design. Fortunately, this should not dramatically impact the long-term performance of the HPC eastbound bridge because the precast panels are protected from chemical exposure and weathering by the CIP deck.

6.6.4 Performance-Related Test Results of Cast-in-Place Decks

Durability, rather than strength, is typically the controlling characteristic for bridge decks when it comes to design and long-term performance. High-strength concrete is most often not required for bridge decks from a design standpoint unless unusual beam spacing widths are utilized. Bridge decks are quite often exposed to traffic, wear, chemical exposure, and varying weather conditions, while other bridge elements such as mainspan members and substructure elements are quite often protected from many of these conditions. It is not unusual, therefore, that there was strong interest in the materials performance and long-term durability performance of the HPC selected for use in the bridge decks under Phase III of this research study. The following sections present the mechanical and material property performance of the concrete sampled from the CIP decks.

6.6.4.1 Louetta Road Overpass CIP Decks. The CIP decks for the Louetta Road Overpass, illustrated in Figure 6.83, encompassed an 88.9-mm (3.5-in.) thick CIP deck reinforced with 12.7-mm-dia. rebar at 304.8-mm o.c. (#4 at 12-in. o.c.) transversely and 15.9-mm-dia. rebar 152.4-mm o.c. (#5 at 6-in. o.c.) longitudinally. All three spans for each bridge were cast as a single pour with tooled control joints placed to control cracking at interior supports. Expansion joints for each bridge were located at each abutment at the ends of the bridge. This method of construction eliminated interior expansion joints and will provide better long-term durability performance at the interior bent lines. Williams Brothers Construction Company, Inc., of Houston, Texas, constructed the decks. Lopez-Gloria Ready Mix of Houston, Texas, supplied concrete for the CIP decks.



Figure 6.83: Louetta Road Overpass Cast-in-Place Deck

Construction of the decks occurred on October 31, 1996 (northbound) and November 8, 1996 (southbound). Two different mix designs were used in casting the northbound and southbound decks. A Class K HS/HPC was selected for the southbound mainlane 55.2 MPa (8,000 psi) deck while a Class S Modified HPC was selected for the northbound mainlane 27.6 MPa (4,000 psi) deck. The compressive strength development for the decks is illustrated in Figure 6.84. Both mix designs satisfied the 28-day design strength requirements in approximately 7 days. Table 6.70 summarizes the performance-related test results of the two decks. The curing conditions investigated included ASTM moist-cured and site/field-cured specimens. Compressive strengths of the site/field-cured specimens tested 12.7 percent (NB-HPC) and 4.3 percent (SB-HS/HPC) lower than their counterpart ASTM moist-cured specimens at 28 days.

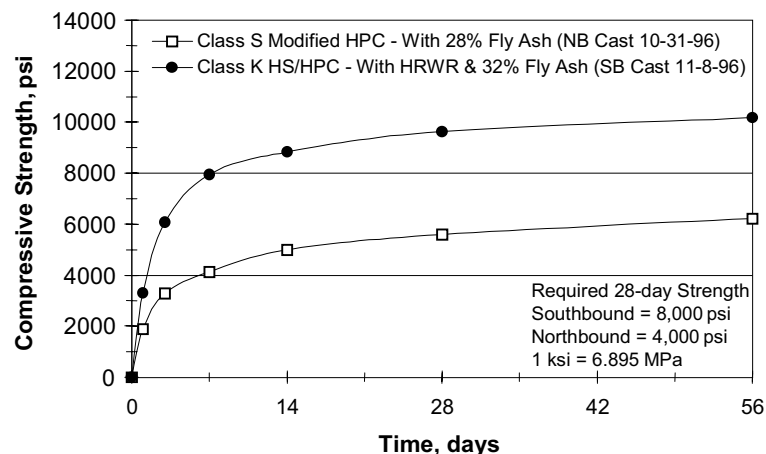


Figure 6.84: Compressive Strength Development for Louetta Road Overpass Cast-in-Place Decks

Table 6.70: Comparison of Louetta Road Overpass Cast-in-Place Deck Mechanical and Material Property Performance

Mechanical or Material Property	Northbound Deck Class S Modified HPC Casting Date: 10-31-96		Southbound Deck Class K HS/HPC Casting Date: 11-8-96	
Required Design Strength at 28 Days	4,000 psi		8,000 psi	
Curing Regime	ASTM Moist	Site/Field	ASTM Moist	Site/Field
Cast-in-Place Deck at 28 Days				
Compressive Strength	5,600 psi	4,890 psi	9,630 psi	9,220 psi
Elastic Modulus	4,520 ksi	4,460 ksi	5,170 ksi	4,730 ksi
Splitting Tensile Strength	460 psi	465 psi	740 psi	725 psi
Cast-in-Place Deck at 56 Days				
Compressive Strength	5,700 psi	5,090 psi	9,740 psi	9,100 psi
Elastic Modulus	4,870 ksi	4,010 ksi	5,750 ksi	4,990 ksi
Splitting Tensile Strength	540 psi	550 psi	820 psi	730 psi
Rapid Chloride Permeability (at 56 Days)	1,730 Coulombs Low Rating	2,120 Coulombs Mod. Rating	900 Coulombs V-Low Rating	1,300 Coulombs Low Rating
1 ksi = 1,000 psi = 6.895 MPa NB = Northbound Deck SB = Southbound Deck				

In addition to the mechanical and material properties presented in Table 6.70, chloride ion penetration was investigated for the HS/HPC and HPC selected for use in the CIP bridge decks. For corrosion protection, ACI Committee 222 [139] recommends a maximum acid-soluble Cl^- concentration of 0.08 percent by weight of cement for prestressed concrete and 0.20 percent by weight of cement for reinforced concrete exposed to chlorides in service. For the Class K HS/HPC deck mix design, this limit is equivalent to 0.035 percent by weight of the concrete based on the ACI reinforced concrete limitation. For the Class S Modified HPC deck mix design, this limit is equivalent to 0.028 percent by weight of the concrete. Results of the 90-day ponding tests are presented in Figure 6.85.

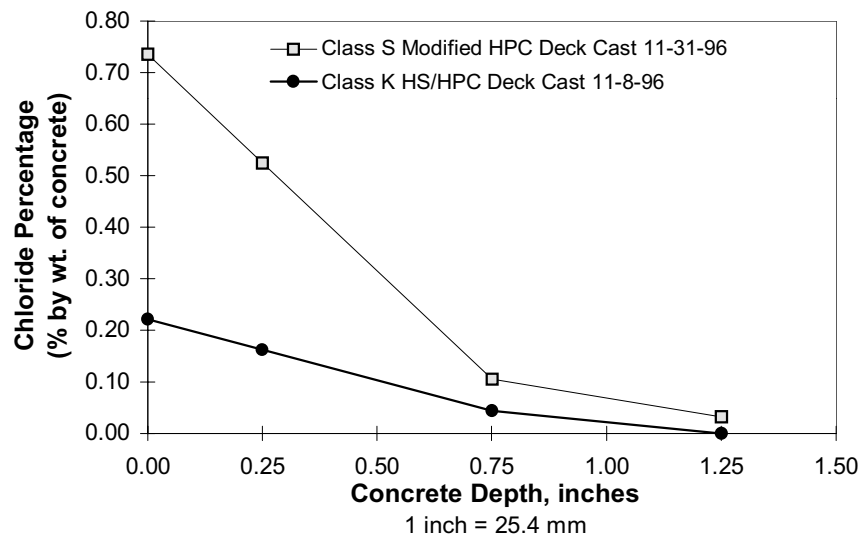


**Figure 6.85: Chloride Ion Penetration (90-Day Ponding) for Louetta Road Overpass
Cast-in-Place Decks**

Chloride percentages reported are the average of six specimen samples from three ponding blocks as described in Section 5.3.5.2.6 of this report. The integral chloride after the 90-day ponding cycle was 0.55 and 1.75, respectively, for the Class K HS/HPC and Class S Modified HPC. These levels of integral chloride may be classified as very low- and high-permeable concrete, respectively, based on Whiting's integral chloride definition [119]. Minor levels of chloride were measured at a depth of 19 mm (0.75 in.) and no chloride was measured beyond a depth of 31.75 mm (1.25 in.) for the Class K HS/HPC. Since the same aggregate type was selected for both of these mix designs, it can be concluded that the paste matrix of the HS/HPC is significantly more impermeable than the HPC consistent with the RCPT test results.

The concrete sampled from the CIP decks satisfied ACI limitations for acid-soluble Cl^- concentration at a clear cover depth to reinforcing of 50.8 mm (2 in.). It may be anticipated that the combination of a low w/cm ratio (0.43 - NB and 0.35 - SB) and sufficient clear cover [50.8 mm (2 in.)] will provide adequate corrosion protection for the deck reinforcing steel on a long-term basis. This conclusion has been reached based on the performance of similar concretes and the findings presented in a previous research study conducted at The University of Texas at Austin by Poston, Carrasquillo, and Breen on the durability performance of bridge decks [53].



Figure 6.86: North Concho River Overpass Cast-in-Place Deck

6.6.4.2 North Concho River Overpass CIP Decks. The CIP decks for the North Concho River Overpass, illustrated in Figure 6.86, encompassed an 88.9-mm (3.5-in.) thick CIP deck reinforced with 15.9-mm-dia. rebar at 304.8-mm o.c. (#5 at 12-in. o.c.) transversely and 12.7-mm-dia. rebar 228.6-mm o.c. (#4 at 9-in. o.c.) longitudinally. Each mainspan for each bridge was cast as a single pour with construction and expansion joints as illustrated in Table 6.71. Continuous construction of the CIP decks to further enhance durability performance at the bent lines was not feasible for the mainspans because of the length and multi-span condition of the bridges. Jascon, Inc. of Uvalde, Texas, constructed the decks. Concho Concrete Company of San Angelo, Texas, supplied concrete for the CIP decks.

Table 6.71: North Concho River Overpass Joint Types

Bent Number	Westbound Bridge (WB)	Eastbound Bridge (EB)
Abutment 1	Expansion	Expansion
Bent 2	Expansion	Expansion
Bent 3	Construction	Construction
Bent 4	Expansion	Expansion
Bent 5	Construction	Construction
Bent 6	Expansion	Expansion
Bent 7	Continuous	Expansion
Bent 8	Expansion	Construction
Bent 9 WB / Abutment 2 EB	Continuous	Expansion
Abutment 2 WB	Expansion	NA
Spans 1 through 6 of the WB and spans 1 through 5 of the EB are considered mainspans.		
Spans 6 & 7 and 8 & 9 of the WB were cast as a single pour.		

Construction of the westbound decks occurred from December 3, 1996 through February 28, 1997. Two different mix designs were used in casting the westbound decks as described in Section 4.4.2 as part of the long-term durability study. A Class S Modified HPC was selected for spans 1 through 5 of WB while a standard TxDOT Class S NSC was selected for spans 6 through 9. The required design strength of the WB decks was 27.6 MPa (4,000 psi).

Construction of the eastbound decks occurred from June 12, 1997 through September 4, 1997. A single mix design (Class K HS/HPC) was used in casting the westbound decks as described in Section 4.4.2 as part of the long-term durability study. The required design strength of the EB decks was 41.4 MPa (6,000 psi).

In addition to the mechanical property performance-related testing that was investigated throughout this study, extensive durability testing was conducted on the North Concho River Overpass decks. These included rapid chloride permeability, chloride ponding, freeze-thaw resistance, abrasion resistance, and deicer scaling resistance. The results of the durability-related testing (material) and mechanical property testing are presented in the following subsections.

6.6.4.2.1 Class S NSC Cast-in-Place Deck. Compressive strength development was monitored for the Class S NSC CIP decks both at early and later-age development. Figure 6.87 illustrates the compressive strength gain with time for the ASTM moist-cured and site/field-cured cylinders.

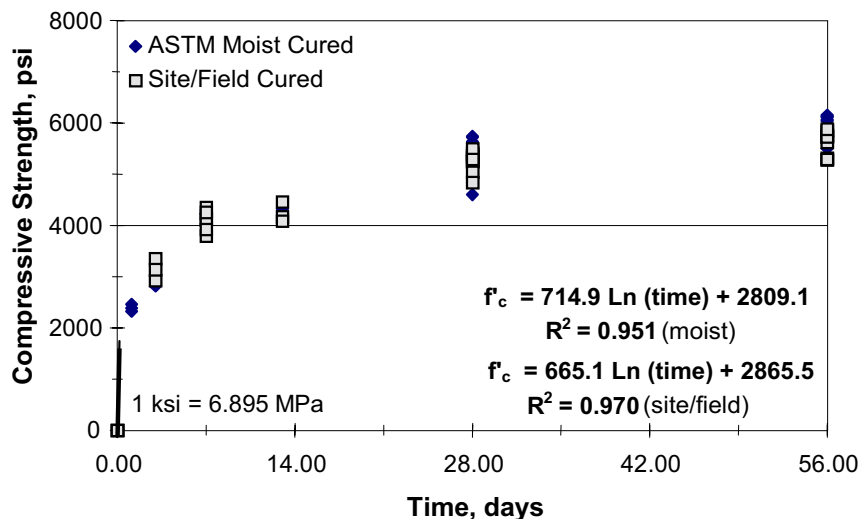


Figure 6.87: Compressive Strength Development for North Concho River Overpass Class S NSC Decks

A logarithmic regression analysis was performed to obtain a best-fit curve for the test data. The data exhibited excellent correlation factors ($R^2 = 0.951$ and 0.970 , respectively) for the data collected from the Class S NSC CIP decks.

Elastic modulus development was monitored for the North Concho River Overpass Class S NSC CIP decks is illustrated in Figure 6.88. A linear regression analysis was performed to obtain the best-fit criteria for the ASTM moist-cured and site/field-cured data. The linear regression analysis resulted in an empirical equation of $E_c = 70,685 \sqrt{f'_c}$, psi, for the ASTM moist-cured cylinders and $E_c = 67,420 \sqrt{f'_c}$, psi, for the site/field-cured cylinders with zero intercepts, respectively. Both of these values satisfied the current ACI 318 (AASHTO) empirical equation for conventional concrete.

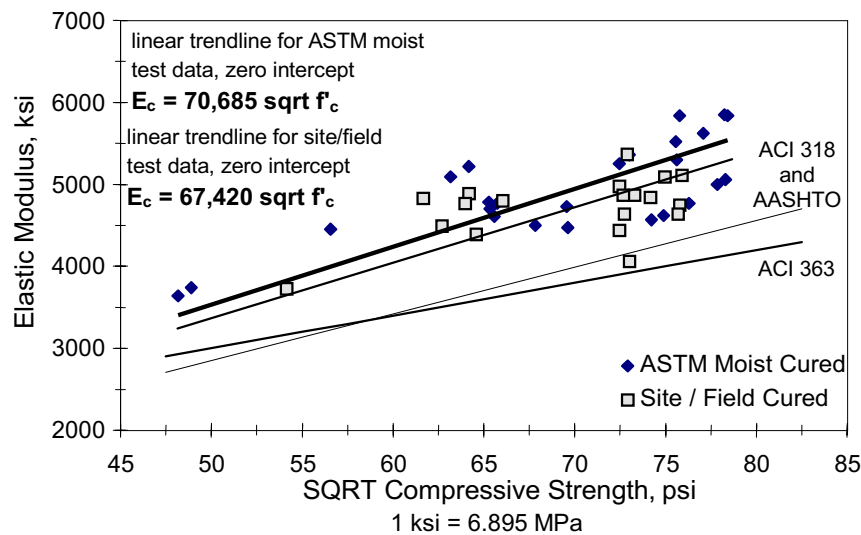


Figure 6.88: Elastic Modulus Development for North Concho River Overpass Class S NSC Decks

Splitting tensile strength development was monitored for the CIP deck at later-age development (28 and 56 days). A linear regression analysis was performed with a zero intercept to obtain the best-fit criteria for the ASTM moist-cured and member-cured data. The linear regression analysis resulted in an empirical equation of $f_{sp} = 7.33 \sqrt{f'_c}$, psi, for the ASTM moist-cured cylinders and $f_{sp} = 7.41 \sqrt{f'_c}$, psi, for the site/field-cured cylinders distributed between the bands of 5 to 10 $\sqrt{f'_c}$, psi, with zero intercepts, respectively. Figure 6.89 illustrates the splitting tensile strength for the ASTM moist-cured and member-cured cylinders.

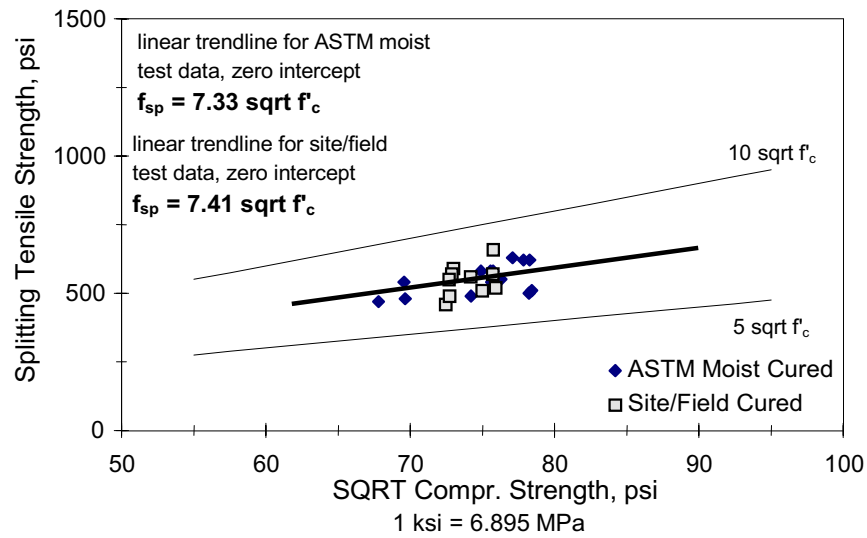


Figure 6.89: Splitting Tensile Strength Development for North Concho River Overpass Class S NSC Decks

Rapid chloride permeability was monitored for each casting date at 56 days for the Class S NSC CIP decks. Both ASTM moist-cured and site/field-cured cylinders were investigated as illustrated in Figure 6.90.

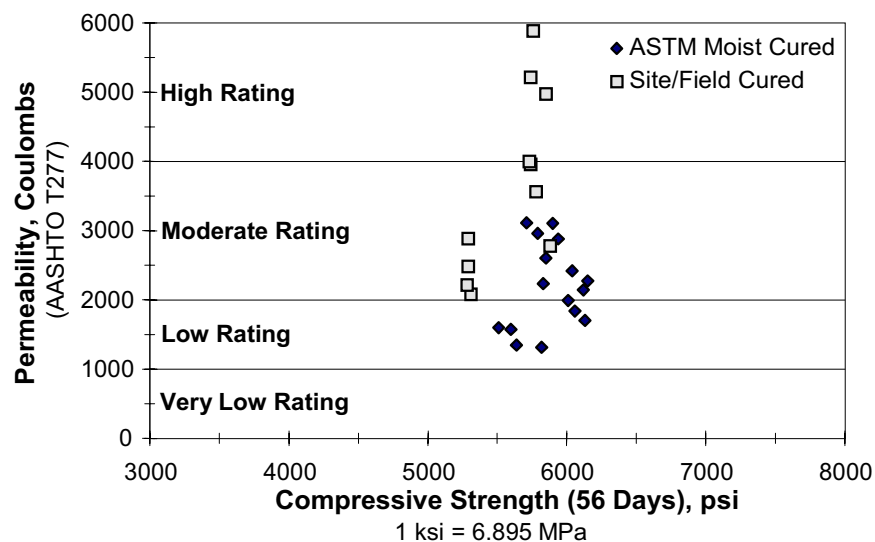


Figure 6.90: Rapid Chloride Permeability (56 Days) for North Concho River Overpass Class S NSC Decks

The Class S mix design was the standard 6.5 sack TxDOT deck mix design and did not include any mineral admixtures (namely, fly ash) or water reducers to enhance the impermeability of the concrete. The ASTM moist-cured cylinders were classified as low- to moderate-permeable concrete, while the site/field-cured cylinders were classified as moderate- to high-permeable concrete (Figure 6.90). The ASTM moist-cured and site/field-cured cylinders resulted in an average RCPT of 2,190 and 3,870 coulombs, respectively, both average values in the moderate classification. On two casting dates, February 18, 1997 and February 28, 1997, the ASTM moist-cured cylinders were subjected to high curing temperatures by field personnel due to a tank heating element malfunction. These casting dates correspond to the permeability results in the low range (Figure 6.90). Excluding these two casting dates, the average RCPT for the ASTM moist-cured cylinders was 2,690 coulombs. The permeability results for the ASTM moist-cured and site/field-cured cylinders are consistent with conventional mix designs used throughout Texas.

Chloride ion penetration was investigated for the Class S NSC selected for use in the CIP bridge decks. For corrosion protection, ACI Committee 222 [139] recommends a maximum acid-soluble Cl^- concentration of 0.20 percent by weight of cement for reinforced concrete exposed to chlorides in service.

For the Class S NSC deck mix design, this limit is equivalent to 0.031 percent by weight of the concrete based on the ACI reinforced concrete limitation. Results of the 90-day ponding tests are presented in Figure 6.91. The integral chloride after the 90-day ponding cycle was 1.171 for the mix design, which may be classified as moderate permeable concrete based on Whiting's integral chloride definition [119]. Minor levels of chloride were measured at a depth of 19 mm (0.75 in.) and no chloride was measured beyond a depth of 31.75 mm (1.25 in.) for the Class S NSC. The concrete sampled from the CIP decks satisfied ACI limitations for acid-soluble Cl^- concentration at a clear cover depth to reinforcing of 50.8 mm (2 in.).

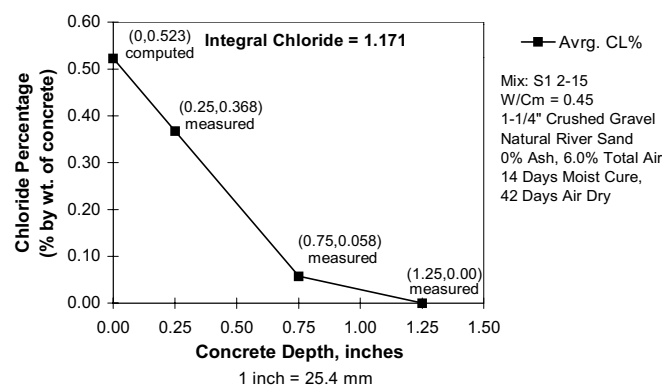


**Figure 6.91: Chloride Ion Penetration (90-Day Ponding) for North Concho River Overpass
Class S NSC Deck Cast 2-15-97**

Freeze-thaw resistance was investigated for the Class S NSC selected for use in the bridge decks. Casting was monitored on three separate dates with a minimum of three specimens subjected to freeze-thaw testing per casting date. Comprehensive freeze-thaw resistance test results for each specimen are presented in Table 6.72.

**Table 6.72: Freeze-Thaw Resistance Test Results for North Concho River Overpass Class S
NSC Decks**

Specimen ID (specimen #)	Casting Date	Total Freeze-Thaw Cycles	Mass Change (percent)	Durability Factor
S1 2-15 ①	2-15-97			
1		314	-1.88	95.7
2		314	-2.36	99.0
3		314	-2.22	94.0
S1 2-18 ①	2-18-97			
1		300	-3.64	91.4
2		300	-3.20	89.4
3		300	-3.21	92.1
S1 2-28 ①	2-28-97			
1		300	-2.16	96.7
2		300	-2.04	93.7
3		300	-2.28	96.9
4		300	-1.54	98.9
5		300	-1.62	96.8
6		300	-3.35	96.8
Average of all Specimens			-2.46	95.1
Freeze-Thaw Resistance Testing performed by FHWA, McLean, Virginia. Durability factors are calculated based on N=300 cycles. A negative number related to mass change represents a loss in mass. ① concrete sample location from truck. 1 inch = 25.4 mm				

The durability factor for the castings on the three dates sampled ranged between 91 and 96 as illustrated in Figure 6.92. The NSC exhibited excellent freeze-thaw resistance. The loss of mass varied between 2 and 3.5 percent as illustrated in Figure 6.93. An average durability factor of 95 and loss in mass of 2.5 percent was indicated for all the specimens sampled from the Class S NSC mix design.

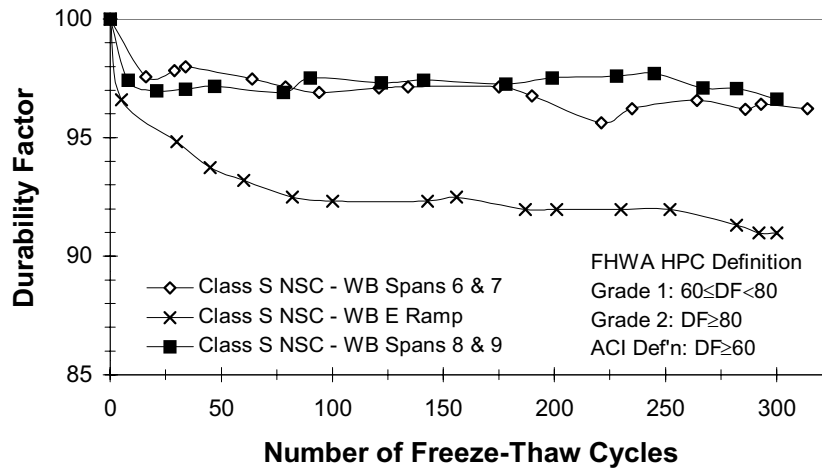


Figure 6.92: Freeze-Thaw Resistance for North Concho River Overpass Class S NSC Decks
Durability Factor versus Number of Freeze-Thaw Cycles

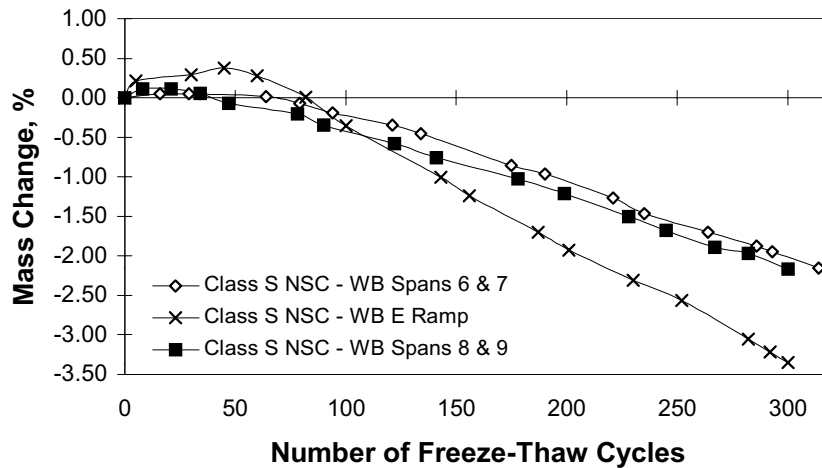
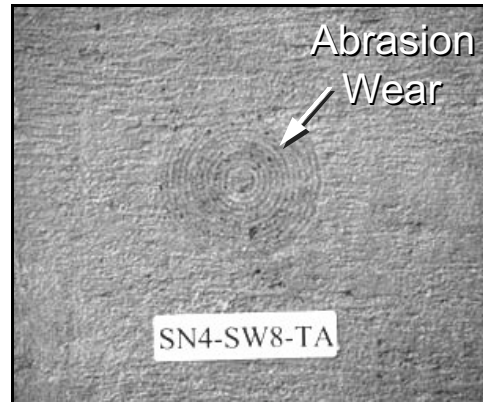


Figure 6.93: Freeze-Thaw Resistance for North Concho River Overpass Class S NSC Decks
Percent Mass Change versus Number of Freeze-Thaw Cycles

Abrasion resistance was investigated for the Class S NSC selected for use in the bridge decks. Monitoring was done on two separate casting dates with a minimum of two specimens subjected to abrasion testing per casting date. A representative specimen after the 6-minute wear cycle is illustrated in Figure 6.94. Abrasion resistance test results for each specimen are presented in

Table 6.73. The average depth of wear was 1.2-mm (0.047-in.) for the specimens sampled from the Class S NSC mix design.



Class S NSC

Wear = 1.47 mm

(Specimen No. 1)

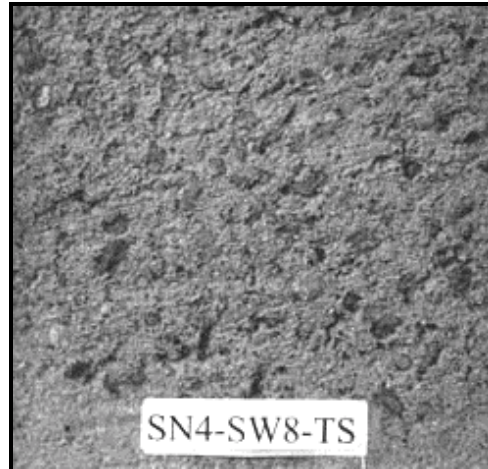
Figure 6.94: Abrasion Resistance for North Concho River Overpass Class S NSC Deck Cast February 28, 1997

Table 6.73: Abrasion Resistance Test Results for North Concho River Overpass Class S NSC Decks

Specimen ID (specimen #)	Casting Date	Initial Height (in.)	6-Minute Height (in.)	Depth of Wear (in.)	Percent Wear After 6 Minutes
S1 2-18 ①	2-18-97				
1		0.415	0.374	0.041	9.9
2		0.393	0.352	0.041	10.4
S1 2-28 ①	2-28-97				
1		0.430	0.372	0.058	13.5
2		0.351	0.305	0.046	13.1
3		0.265	0.217	0.048	18.1
Average of all Specimens	-	-	-	0.047 (1.2 mm)	13.0
Abrasion Resistance Testing performed by TxDOT Materials and Test Division, Austin, Texas. ① concrete sample location from truck. 1 inch = 25.4 mm					

Scaling resistance was investigated for the Class S NSC selected for use in the bridge decks. Monitoring was done on two separate casting dates with a minimum of two specimens subjected to scaling deicing testing per casting date. A representative specimen after fifty cycles is illustrated in

Figure 6.95. Scaling resistance test results for each specimen are presented in Table 6.74. The average visual condition rating was 4.5 for the specimens sampled from the Class S NSC mix design. This may be classified as moderate to severe surface damage.



Class S NSC

Visual Rating = 5

(damage = severe)

**Figure 6.95: Deicer Scaling Resistance for North Concho River Overpass Class S NSC Deck
Cast February 28, 1997**

Table 6.74: Scaling Resistance Test Results for North Concho River Overpass Class S NSC

Decks

Specimen ID (specimen #)	Casting Date	Condition of Surface After 50 Cycles
S1 2-18 ①	2-18-97	
1		4
2		3.5
S1 2-28 ①	2-28-97	
1		5
2		5
Average of all Specimens		4.5 rating = moderate - severe
Scaling Resistance Testing and Visual Damage Rating performed by TxDOT Materials and Test Division, Austin, Texas. ① concrete sample location from truck. 1 inch = 25.4 mm		

6.6.4.2.2 *Class S Modified HPC Cast-in-Place Deck.* Compressive strength development was monitored for the Class S Modified HPC CIP decks both at early and later-age development. Figure 6.96 illustrates the compressive strength gain with time for the ASTM moist-cured and site/field-cured cylinders. A logarithmic regression analysis was performed to obtain a best-fit curve for the test data. The data exhibited fair correlation factors ($R^2 = 0.621$ and 0.612 , respectively) for the data collected from the Class S Modified HPC CIP decks. The reduced correlation factor may be attributed to the wide scatter of data on the various test dates, as illustrated in Figure 6.96. This scatter may be attributed to the varied placement conditions encountered on the dates the decks were cast. Ambient temperatures varied from a minimum of 2.8°C to a maximum of 17.8°C (37 to 64°F) on these dates.

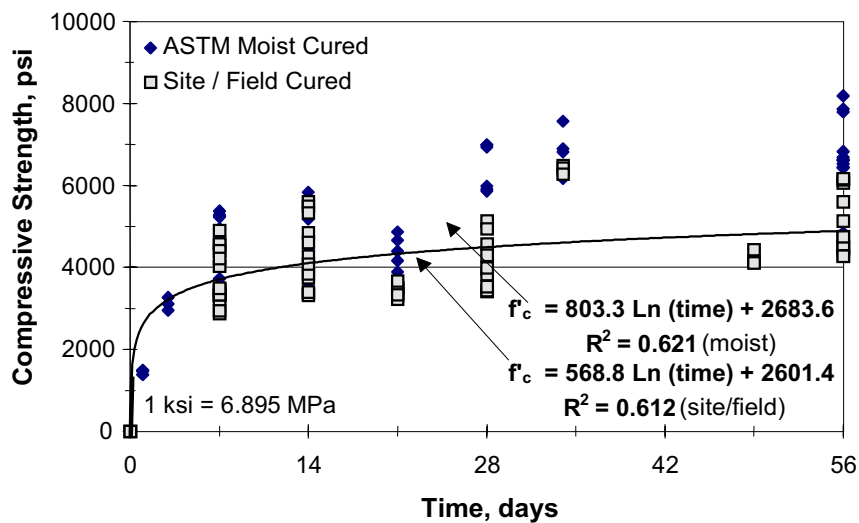


Figure 6.96: Compressive Strength Development for North Concho River Overpass Class S Modified HPC Decks

The contractor did not elect to use any special cold weather concreting practices such as external heaters or batt insulation to prevent heat loss from the mix design during hydration. The contractor also did not attempt to increase the placement temperature of the concrete either by heated aggregates or heated mixing water. On several of the casting dates, the strength gain characteristics of the concrete were impacted by the cold ambient conditions. The cold temperatures slowed the hydration process of the cementitious materials, resulting in a slower strength gain.

Elastic modulus development was monitored for the North Concho River Overpass Class S Modified HPC CIP decks (illustrated in Figure 6.97). A linear regression analysis was performed to obtain the best-fit criteria for the ASTM moist-cured and site/field-cured data. The linear regression analysis resulted in an empirical equation of $E_c = 68,950 \sqrt{f'_c}$, psi, for the ASTM moist-cured cylinders and $E_c = 67,610 \sqrt{f'_c}$, psi, for the site/field-cured cylinders with zero intercepts, respectively. Both of these values satisfied the current ACI Committee 363 and 318 (AASHTO) empirical equations.

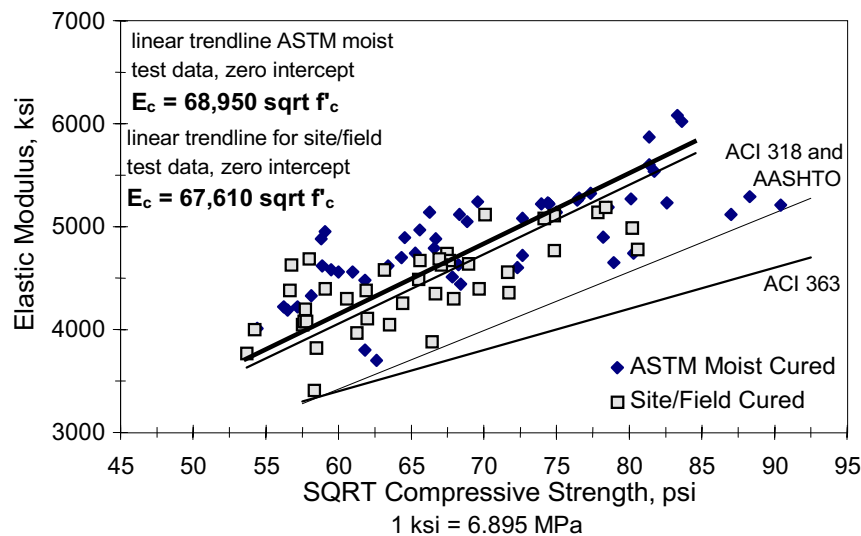


Figure 6.97: Elastic Modulus Development for North Concho River Overpass Class S Modified HPC Decks

Splitting tensile strength development was monitored for the CIP deck at later-age development (28 and 56 days). A linear regression analysis was performed with a zero intercept to obtain the best-fit criteria for the ASTM moist-cured and member-cured data. The linear regression analysis resulted in an empirical equation of $f_{sp} = 6.94 \sqrt{f'_c}$, psi, for the ASTM moist-cured cylinders and $f_{sp} = 7.12 \sqrt{f'_c}$, psi, for the site/field-cured cylinders distributed between the bands of 5 to 10 sqrt f'_c , psi, with zero intercepts, respectively. Figure 6.98 illustrates the splitting tensile strength for the ASTM moist-cured and member-cured cylinders.

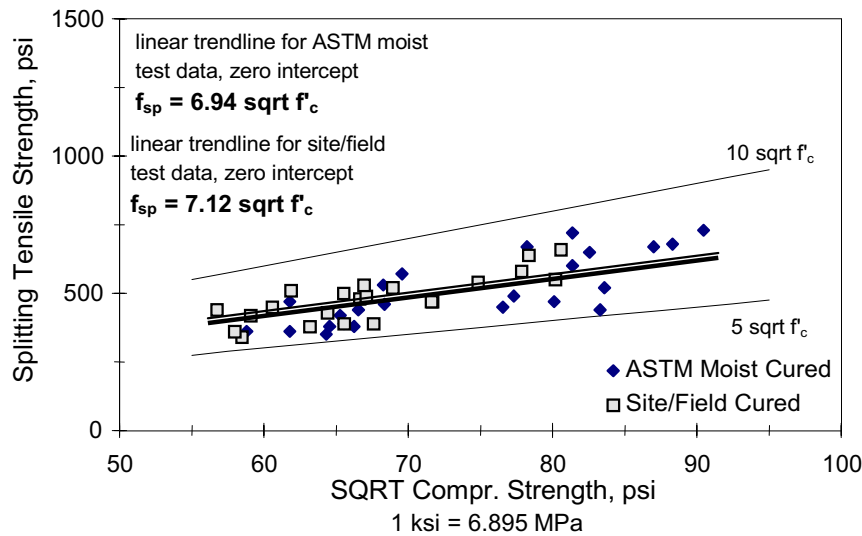


Figure 6.98: Splitting Tensile Strength Development for North Concho River Overpass Class S Modified HPC Decks

Rapid chloride permeability was monitored for each casting date at 56 days for the Class S Modified HPC CIP decks. Both ASTM moist-cured and site/field-cured cylinders were investigated as illustrated in Figure 6.99. The Class S Modified mix design was identical to the standard TxDOT Class S mix design except that it incorporated 30 percent fly ash replacement. The ASTM moist-cured cylinders were classified as low- to moderate-permeable concrete, while the site/field-cured cylinders were classified as moderate- to high-permeable concrete (Figure 6.90). The ASTM moist-cured and site/field-cured cylinders resulted in an average RCPT of 2,580 and 3,410 coulombs, respectively, both average values in the “moderate” classification.

Chloride ion penetration was investigated for the Class S Modified HPC selected for use in the CIP bridge decks. For corrosion protection, ACI Committee 222 [139] recommends a maximum acid-soluble Cl^- concentration of 0.20 percent by weight of cement for reinforced concrete exposed to chlorides in service. For the Class S Modified HPC deck mix design, this limit is equivalent to 0.031 percent by weight of the concrete based on the ACI reinforced concrete limitation. Results of the 90-day ponding tests are presented in Figure 6.100. The integral chloride after the 90-day ponding cycle was 0.851 for the mix design, which may be classified as low/moderate-permeable concrete based on Whiting’s integral chloride definition [119]. Minor levels of chloride were measured at a depth of 19

mm (0.75 in.) and no chloride was measured beyond a depth of 31.75 mm (1.25 in.) for the Class S Modified HPC. The concrete sampled from the CIP decks satisfied ACI limitations for acid-soluble Cl^- concentration at a clear cover depth to reinforcing of 50.8 mm (2 in.).

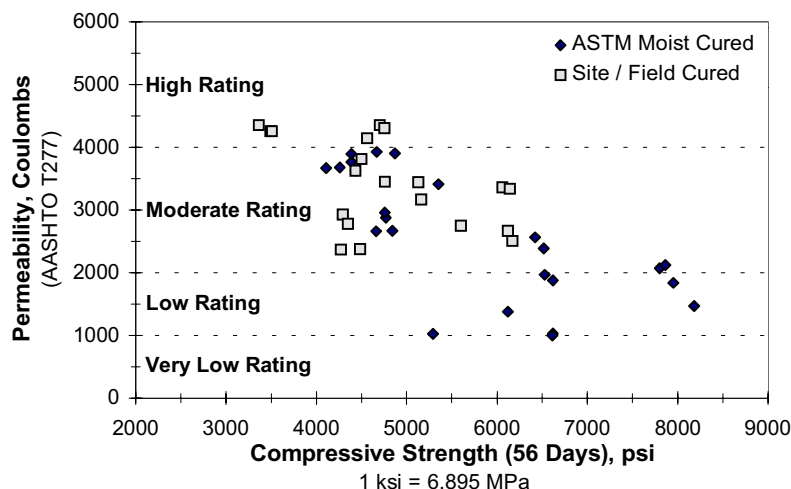


Figure 6.99: Rapid Chloride Permeability (56 Days) for North Concho River Overpass Class S Modified HPC Decks

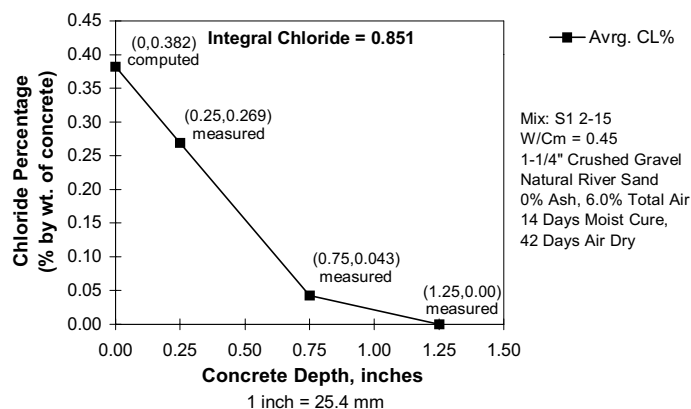


Figure 6.100: Chloride Ion Penetration (90-Day Ponding) for North Concho River Overpass Class S Modified HPC Deck Cast December 3, 1996

Freeze-thaw resistance was investigated for the Class S Modified HPC selected for use in the bridge decks. Monitoring was done on two separate casting dates with a minimum of three specimens

subjected to freeze-thaw testing per casting date. Comprehensive freeze-thaw resistance test results for each specimen are presented in Table 6.75. The durability factor for the specimens sampled on the three casting dates ranged between 96 and 99 as illustrated in Figure 6.101. The HPC exhibited excellent freeze-thaw resistance.

Table 6.75: Freeze-Thaw Resistance Test Results for North Concho River Overpass Class S Modified HPC Decks

Specimen ID (specimen #)	Casting Date	Total Freeze-Thaw Cycles	Mass Change (percent)	Durability Factor
S2 1-29 ①	1-29-97			
1		302	-2.93	98.6
2		302	-2.79	99.0
3		302	-3.70	99.0
S2 3-4 ①	3-4-97			
1		300	-2.28	94.5
2		300	-1.69	97.0
3		300	-1.59	96.9
4		300	-1.65	96.9
5		300	-1.58	96.9
6		300	-1.52	96.9
Average of all Specimens			-2.19	97.3
Freeze-Thaw Resistance Testing performed by FHWA, McLean, Virginia. Durability factors are calculated based on N=300 cycles. A negative number related to mass change represents a loss in mass. ① concrete sample location from truck. 1 inch = 25.4 mm				

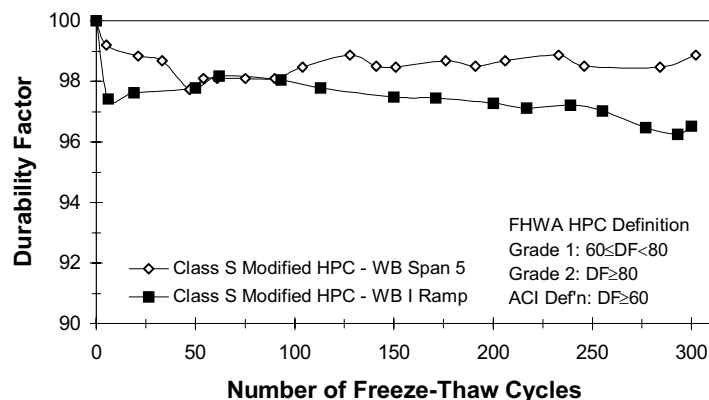


Figure 6.101: Freeze-Thaw Resistance for North Concho River Overpass Class S Modified HPC Decks
Durability Factor versus Number of Freeze-Thaw Cycles

The loss of mass varied between 1.5 and 3 percent as illustrated in Figure 6.102. An average durability factor of 97 and loss in mass of 2.2 percent was indicated for all the specimens sampled from the Class S Modified HPC mix design.

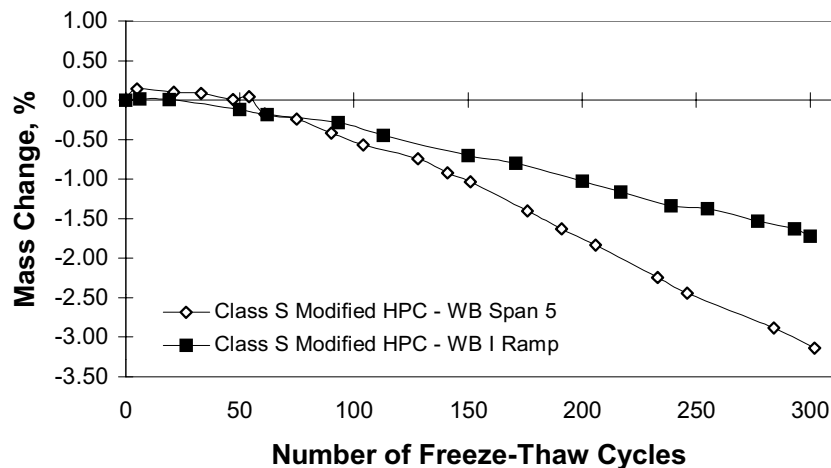


Figure 6.102: Freeze-Thaw Resistance for North Concho River Overpass Class S Modified HPC Decks; Percent Mass Change versus Number of Freeze-Thaw Cycles

Abrasion resistance was investigated for the Class S Modified HPC selected for use in the bridge decks. The scope of durability-related testing of the decks was augmented to include abrasion and scaling resistance testing in February 1997.

At that time, all of the Class S Modified HPC decks had been cast except for the casting conducted on March 3, 1997. For this reason, the casting on only one date was investigated for abrasion resistance of the Class S Modified HPC. Three specimens were subjected to abrasion testing on one casting date. A representative specimen after the 6-minute wear cycle is illustrated in Figure 6.103. Abrasion resistance test results for each specimen are presented in Table 6.76. The average depth of wear was 1.8 mm (0.072 in.) for the specimens sampled from the Class S Modified HPC mix design.



Class S Modified HPC

Wear = 1.09 mm

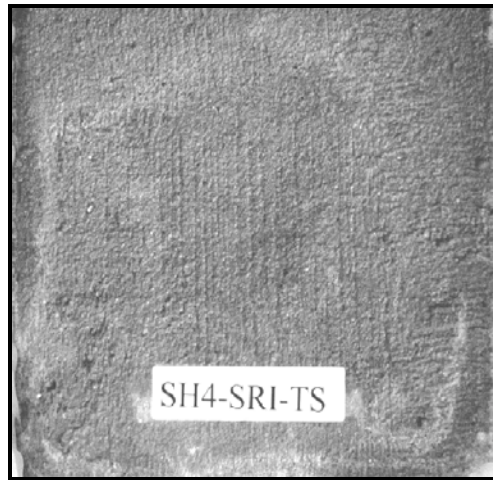
(Specimen No. 3)

**Figure 6.103: Abrasion Resistance for North Concho River Overpass Class S Modified HPC
Deck Cast March 4, 1997**

**Table 6.76: Abrasion Resistance Test Results for North Concho River Overpass Class S
Modified HPC Decks**

Specimen ID (specimen #)	Casting Date	Initial Height (in.)	6 Minute Height (in.)	Depth of Wear (in.)	Percent Wear After 6 Minutes
S2 3-4 ①	3-4-97				
1		0.377	0.300	0.077	20.4
2		0.388	0.291	0.097	25.0
3		0.386	0.343	0.043	11.1
Average of all Specimens	-	-	-	0.072 (1.8 mm)	18.8
Abrasion Resistance Testing performed by TxDOT Materials and Test Division, Austin, Texas. ① concrete sample location from truck. 1 inch = 25.4 mm					

Scaling resistance was investigated for the Class S Modified HPC selected for use in the bridge decks. As with the abrasion resistance testing, monitoring was done on only one casting date with two specimens subjected to scaling deicing testing. A representative specimen after fifty cycles is illustrated in Figure 6.104. Scaling resistance test results for each specimen are presented in Table 6.77.



Class S Modified HPC

Visual Rating = 0

(damage = none)

Figure 6.104: Deicer Scaling Resistance for North Concho River Overpass Bridge Class S Modified HPC Deck Cast March 4, 1997

Table 6.77: Scaling Resistance Test Results for North Concho River Overpass Class S Modified HPC Decks

Specimen ID (specimen #)	Casting Date	Condition of Surface After 50 Cycles
S2 3-4 ①	3-4-97	
1		0
2		0
Average of all Specimens		0 rating = none
Scaling Resistance Testing and Visual Damage Rating performed by TxDOT Materials and Test Division, Austin, Texas. ① concrete sample location from truck. 1 inch = 25.4 mm		

The average visual condition rating was 0 for the specimens sampled from the Class S Modified HPC mix design. This may be classified as no surface damage.

6.6.4.2.3 Class K HS/HPC Cast-in-Place Deck. Compressive strength development was monitored for the Class K HS/HPC CIP decks both at early and later-age development. Figure 6.105 illustrates the compressive strength gain with time for the ASTM moist-cured and site/field-cured cylinders. A logarithmic regression analysis was performed to obtain a best-fit curve for the test data.

The data exhibited fair correlation factors ($R^2 = 0.846$ and 0.665 , respectively) for the data collected from the Class K HS/HPC CIP decks.

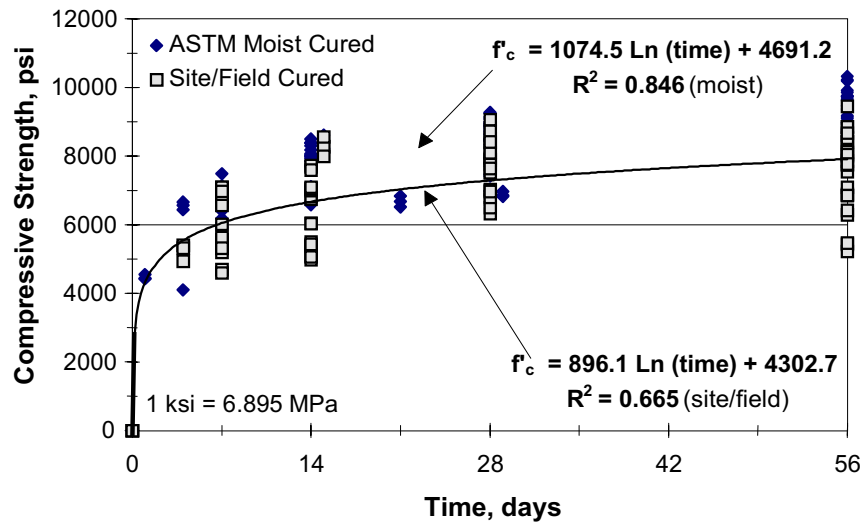


Figure 6.105: Compressive Strength Development for North Concho River Overpass Class K HS/HPC Decks

Elastic modulus development was monitored for the North Concho River Overpass Class K HS/HPC CIP decks is illustrated in Figure 6.106. A linear regression analysis was performed to obtain the best-fit criteria for the ASTM moist-cured and site/field-cured data. The linear regression analysis resulted in an empirical equation of $E_c = 64,190 \sqrt{f'_c}$, psi, for the ASTM moist-cured cylinders and $E_c = 62,050 \sqrt{f'_c}$, psi, for the site/field-cured cylinders with zero intercepts, respectively. Both of these values satisfied the current ACI Committee 363 and 318 (AASHTO) empirical equations.

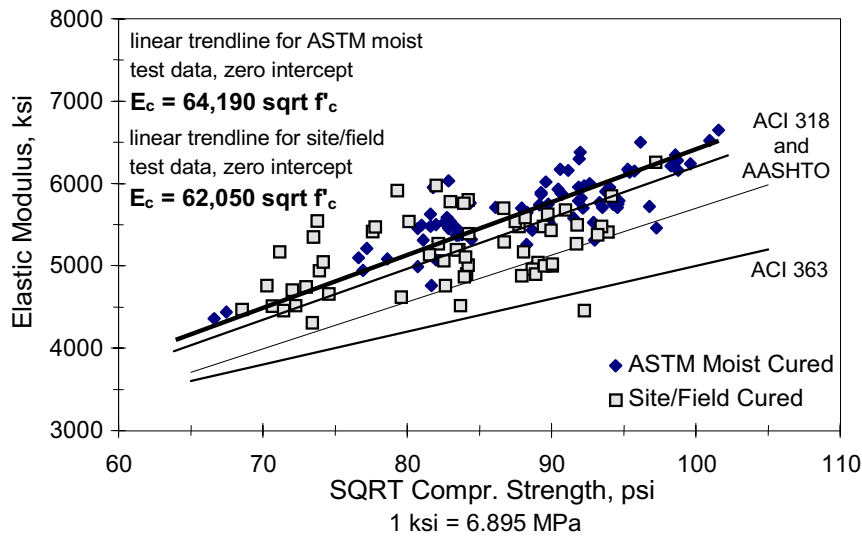


Figure 6.106: Elastic Modulus Development for North Concho River Overpass Bridge Class K HS/HPC Decks

Splitting tensile strength development was monitored for the CIP deck at later-age development (28 and 56 days). A linear regression analysis was performed with a zero intercept to obtain the best-fit criteria for the ASTM moist-cured and member-cured data. The linear regression analysis resulted in an empirical equation of $f_{sp} = 8.80 \sqrt{f'_c}$, psi, for the ASTM moist-cured cylinders and $f_{sp} = 8.79 \sqrt{f'_c}$, psi, for the site/field-cured cylinders distributed between the bands of 6 to 10 $\sqrt{f'_c}$, psi, with zero intercepts, respectively. Figure 6.107 illustrates the splitting tensile strength for the ASTM moist-cured and member-cured cylinders.

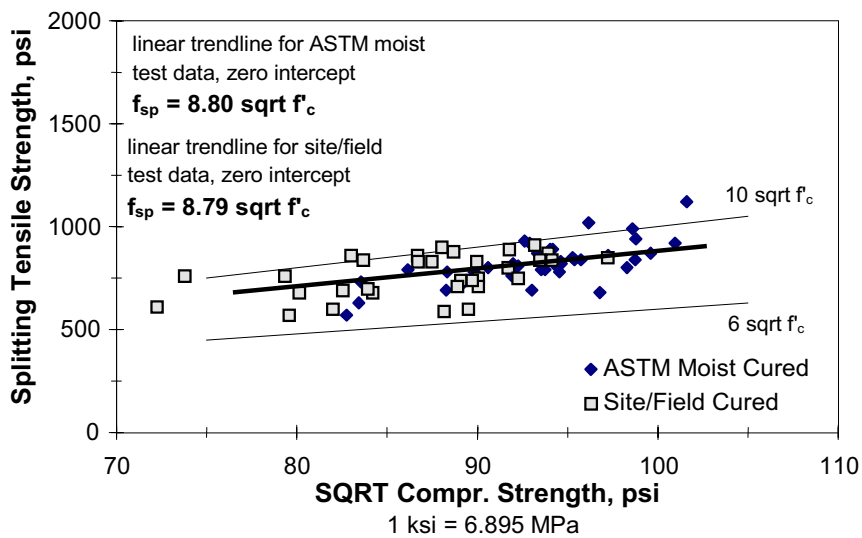


Figure 6.107: Splitting Tensile Strength Development for North Concho River Overpass Class K HS/HPC Decks

Rapid chloride permeability was monitored for each casting date at 56 days for the Class K HS/HPC CIP decks. Both ASTM moist-cured and site/field-cured cylinders were investigated as illustrated in Figure 6.108. The Class K mix design incorporated an HRWR and 30 percent fly ash replacement. The ASTM moist-cured cylinders were classified as very low-permeable concrete, while the site/field-cured cylinders were classified as very low- to low-permeable concrete (Figure 6.108). The ASTM moist-cured and site/field-cured cylinders resulted in an average RCPT of 690 and 910 coulombs, respectively, both average values in the “very low” classification.

Chloride ion penetration was investigated for the Class K HS/HPC selected for use in the CIP bridge decks. For corrosion protection, ACI Committee 222 [139] recommends a maximum acid-soluble Cl^- concentration of 0.20 percent by weight of cement for reinforced concrete exposed to chlorides in service. For the Class K HS/HPC deck mix design, this limit is equivalent to 0.034 percent by weight of the concrete based on the ACI reinforced concrete limitation.

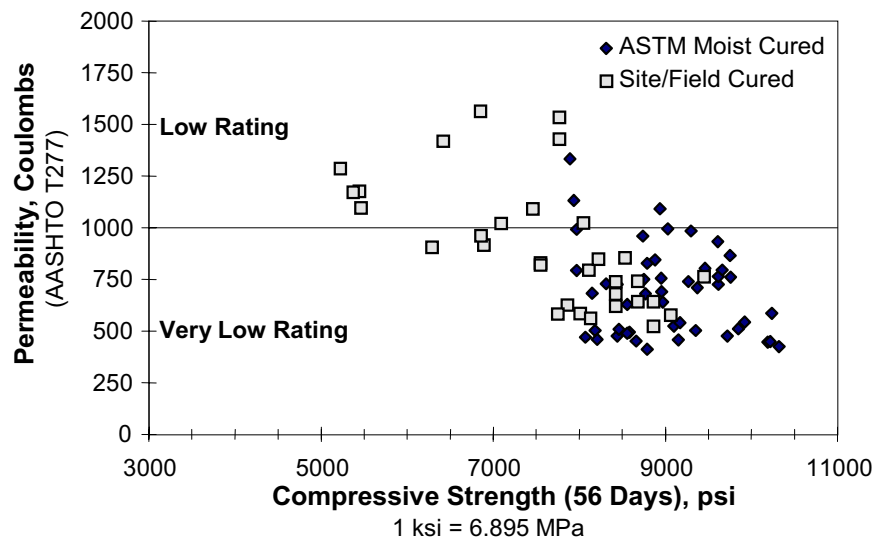


Figure 6.108: Rapid Chloride Permeability (56 Days) for North Concho River Overpass Class K HS/HPC Decks

Results of the 90-day ponding tests are presented in Figures 6.109 and 6.110 for the casting monitored two dates. The integral chloride after the 90-day ponding resulted in an average value of 0.502, which may be classified as very low-permeable concrete based on Whiting's integral chloride definition [119]. Minor levels of chloride were measured at a depth of 19 mm (0.75 in.) and no chloride was measured beyond a depth of 31.75 mm (1.25 in.) for the Class K HS/HPC. The concrete sampled from the CIP decks satisfied ACI limitations for acid-soluble Cl^- concentration at a clear cover depth to reinforcing of 50.8 mm (2 in.).

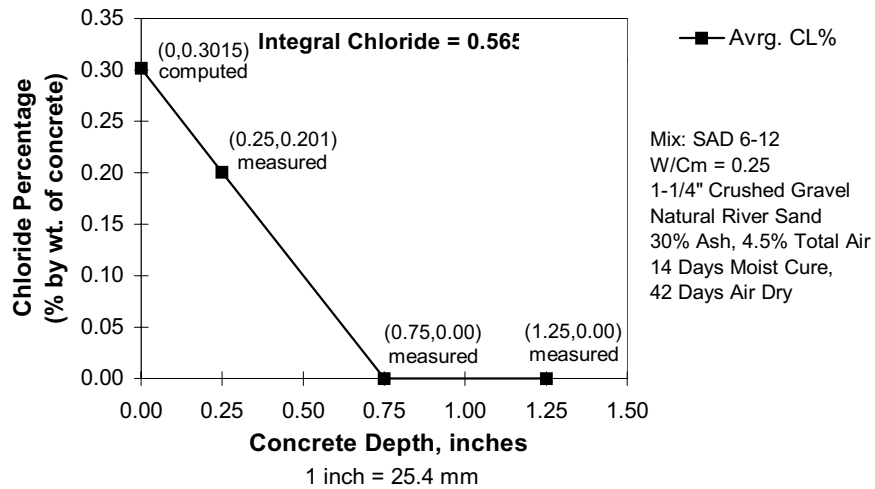


**Figure 6.109: Chloride Ion Penetration (90-Day Ponding) for North Concho River Overpass
Class K HS/HPC Deck Cast June 12, 1997**

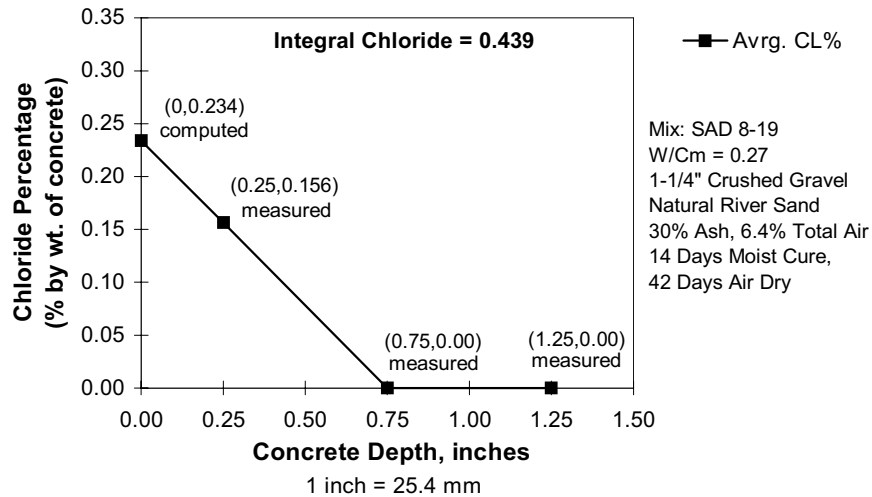


**Figure 6.110: Chloride Ion Penetration (90-Day Ponding) for North Concho River Overpass
Class K HS/HPC Deck Cast August 19, 1997**

Freeze-thaw resistance was investigated for the Class K HS/HPC selected for use in the bridge decks. Monitoring was done on four separate casting dates with a minimum of three specimens subjected to freeze-thaw testing per casting date. Samples were also taken for investigation from two additional locations: 1) directly from the transit mixer prior to pumping and 2)

from the pump. Comprehensive freeze-thaw resistance test results for each specimen are presented in Table 6.78. The durability factor for the specimens sampled on three casting dates ranged between 97 and 99 as illustrated in Figure 6.111. The HS/HPC exhibited excellent freeze-thaw resistance. The loss of mass varied between 0.25 and 1 percent as illustrated in Figure 6.112. An average durability factor of 98 and loss in mass of 0.75 percent was indicated for all the specimens sampled from the Class K HS/HPC mix design. No significant variation was noted between sample locations as illustrated in Table 6.78.

Table 6.78: Freeze-Thaw Resistance Test Results for North Concho River Overpass Class K HS/HPC Decks

Specimen ID (specimen #)	Casting Date	Total Freeze-Thaw Cycles	Mass Change (percent)	Durability Factor
SAD 6-12	6-12-97			
1 ❶		300	-0.86	97.9
2 ❶		300	-1.12	99.0
3 ❶		300	-0.92	96.9
4 ❷		300	-1.15	97.0
5 ❷		300	-1.06	98.0
6 ❷		300	-1.30	98.0
SAD 6-25 ❶	6-25-97			
1		300	-0.35	97.1
2		300	-0.28	97.1
3		300	-0.32	96.0
SAD 7-9	7-9-97			
1 ❶		300	-0.76	97.0
2 ❶		300	-1.17	98.0
3 ❶		300	-0.96	97.0
4 ❷		300	-0.92	96.9
5 ❷		300	-0.65	96.9
6 ❷		300	-0.85	97.5
SAD 7-23	7-23-97			
1 ❶		300	-0.40	99.9
2 ❶		300	-0.64	97.0
3 ❶		300	-0.72	98.4
4 ❷		300	-0.34	101.1
5 ❷		300	-0.46	99.0
6 ❷		300	-0.43	100.0
Average of all Specimens			-0.75	97.9
Freeze-Thaw Resistance Testing performed by FHWA, McLean, Virginia. A negative number related to mass change represents a loss in mass. ❶ concrete sample location from truck. ❷ concrete sample location from pump. 1 inch = 25.4 mm				

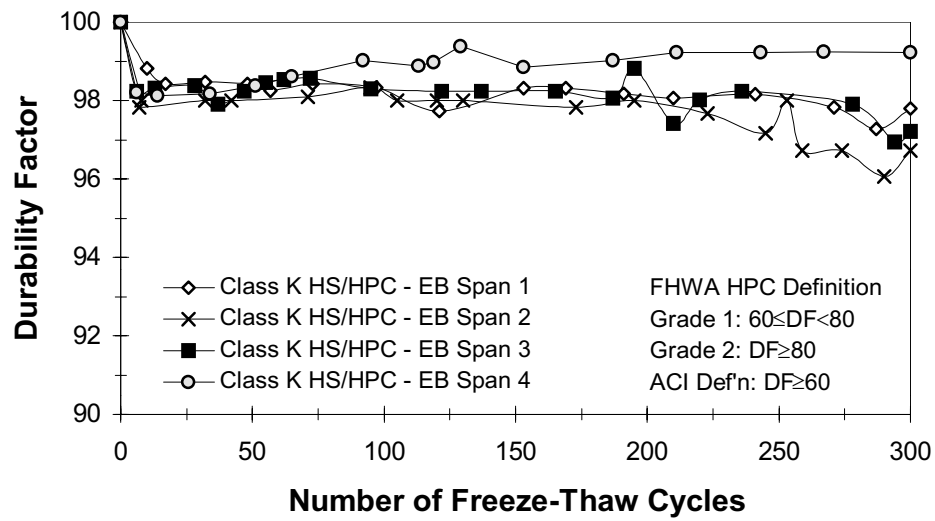


Figure 6.111: Freeze-Thaw Resistance for North Concho River Overpass Class K HS/HPC Decks

Durability Factor versus Number of Freeze-Thaw Cycles

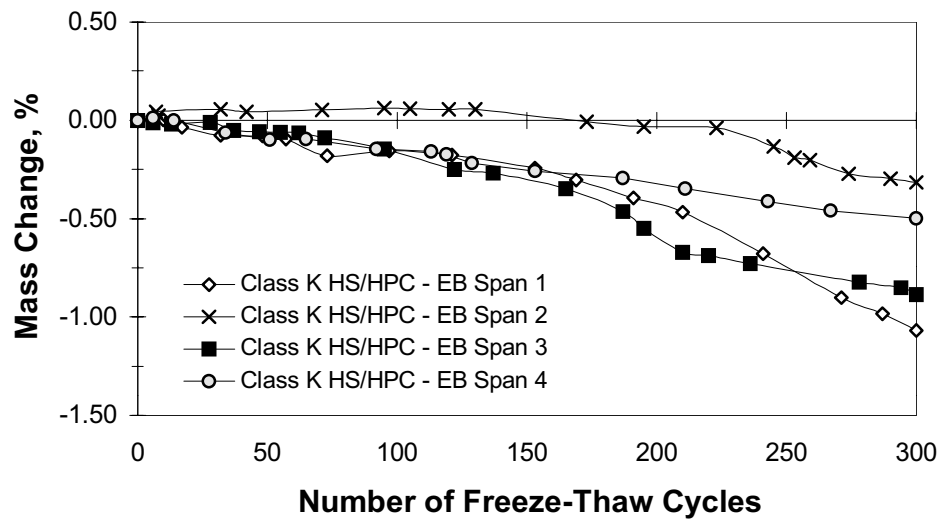


Figure 6.112: Freeze-Thaw Resistance for North Concho River Overpass Class K HS/HPC Decks

Percent Mass Change versus Number of Freeze-Thaw Cycles

Abrasion resistance was investigated for the Class K HS/HPC selected for use in the bridge decks. Monitoring was done on four separate casting dates with a minimum of three specimens subjected to abrasion testing per casting date. Samples were also taken for investigation from two additional locations: 1) directly from the transit mixer prior to pumping and 2) from the pump. A representative specimen after the 6-minute wear cycle is illustrated in Figure 6.113. Comprehensive abrasion resistance test results for each specimen are presented in Table 6.79. The average depth of wear was 1.0 mm (0.039 in.) for the specimens sampled from the Class K HS/HPC mix design. No significant variation was noted between sample locations as illustrated in Table 6.79.



Class K HS/HPC

Wear = 0.84 mm

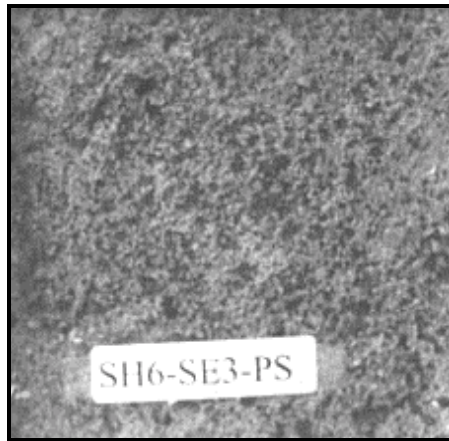
(Specimen No. 2)

**Figure 6.113: Abrasion Resistance for North Concho River Overpass Class K HS/HPC Deck
Cast June 9, 1997**

**Table 6.79: Abrasion Resistance Test Results for North Concho River Overpass Class K
HS/HPC Decks**

Specimen ID (specimen #)	Casting Date	Initial Height (in.)	6-Minute Height (in.)	Depth of Wear (in.)	Percent Wear After 6 Minutes
SAD 6-12 ②	6-12-97				
1		0.343	0.318	0.025	7.3
2		0.311	0.291	0.039	12.5
3		0.325	0.267	0.058	17.8
SAD 6-25 ①	6-25-97				
1		0.345	0.306	0.039	11.3
2		0.335	0.311	0.024	7.2
3		0.447	0.408	0.039	8.7
SAD 7-9	7-9-97				
1 ①		0.299	0.264	0.035	11.7
2 ①		0.275	0.235	0.040	14.5
3 ①		0.265	0.202	0.063	23.8
4 ②		0.391	0.348	0.043	11.0
5 ②		0.254	0.221	0.033	12.9
6 ②		0.329	0.280	0.049	14.9
SAD 7-23 ①	7-23-97				
1		0.338	0.302	0.036	10.7
2		0.347	0.313	0.034	9.8
3		0.339	0.301	0.038	11.2
Average of all Specimens	-	-	-	0.039 ① (1.0 mm) 0.041 ② (1.0 mm)	12.1 ① 12.7 ②
Abrasion Resistance Testing performed by TxDOT Materials and Test Division, Austin, Texas. ① concrete sample location from truck. ② concrete sample location from pump. 1 inch = 25.4 mm					

Scaling resistance was investigated for the Class K HS/HPC selected for use in the bridge decks. Monitoring was done on four separate casting dates with a minimum of two specimens subjected to scaling deicing testing per casting date. Samples were also taken for investigation from two additional locations: 1) directly from the transit mixer prior to pumping and 2) from the pump. A representative specimen after 50 cycles is illustrated in Figure 6.114.



Class K HS/HPC

Visual Rating = 1

(damage = low)

**Figure 6.114: Deicer Scaling Resistance for North Concho River Overpass Class K HS/HPC
Deck Cast July 9, 1997**

Scaling resistance test results for each specimen are presented in Table 6.80. The average visual condition rating was 2 for concrete sampled from the truck and 3 for concrete sampled from the pump. This may be classified as low and moderate surface damage, respectively. No significant variation was noted between sample locations as illustrated in Table 6.80.

6.6.4.3 Comparison of CIP Mechanical and Material Properties. In order to better illustrate the advantages of HPC over conventional concretes, the following comparison study is presented regarding the mechanical and durability test results compiled during this investigation for the North Concho River Overpass bridge decks. Results comparing the concrete mix design performance for the Louetta Road Overpass decks are presented and discussed in Section 6.6.4.1. Compressive strength development for the Louetta Road Overpass decks is illustrated in Figure 6.84 while the comparison of mechanical and material property performance is presented in Table 6.70. The relative material costs associated with the Class S Modified and Class K mix designs were \$21.03 and \$29.79 per cubic meter (\$27.51 and \$38.97 per cubic yard), respectively.

6.6.4.3.1 Compressive Strength ASTM C39-94. Compressive strength development from a representative casting date for the North Concho River Overpass CIP decks is illustrated in Figure 6.115. These casting dates were selected based on similar ambient and placement conditions to serve as a more representative comparison. All three mix designs satisfied the 28-day design

strength requirements in approximately 7 days. The later-age strength gain characteristics of the ASTM Class C fly ash is clearly evident when comparing the Class S and Class S Modified mix designs. These mix designs were identical except for the 30 percent fly ash replacement used for the Class S Modified mix design. The Class K mix design exhibited very good strength gain characteristics through the use of a fly ash replacement and HRWR.

**Table 6.80: Scaling Resistance Test Results for North Concho River Overpass Class K
HS/HPC Decks**

Specimen ID (specimen #)	Casting Date	Condition of Surface After 50 Cycles
SAD 6-12 ② 1 2	6-12-97	3 3
SAD 6-25 ① 1 2	6-25-97	4 4
SAD 7-9 1 ① 2 ① 3 ② 4 ②	7-9-97	3 3 1 1
SAD 7-23 ① 1 2	7-23-97	2 2.5
Average of all Specimens		2 ① 3 ② rating = moderate - severe
Scaling Resistance Testing and Visual Damage Rating performed by TxDOT Materials and Test Division, Austin, Texas. ① concrete sample location from truck. ② concrete sample location from pump. 1 inch = 25.4 mm		

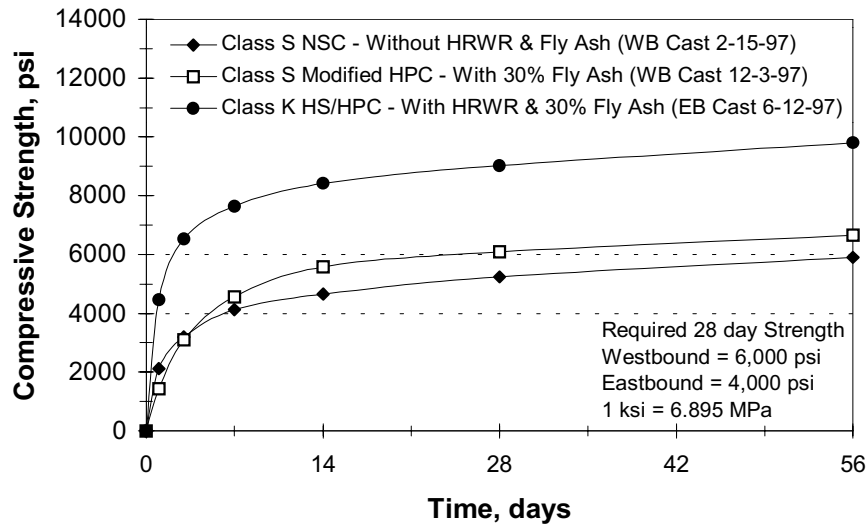


Figure 6.115: Compressive Strength Gain with Time for North Concho River Overpass Cast-in-Place Decks

6.6.4.3.2 Elastic Modulus ASTM C469-94. The linear regression analysis for the Class S NSC, Class S Modified HPC, and Class K HS/HPC resulted in an empirical equation of $E_c = 70,685 \sqrt{f'_c}$, psi, $E_c = 68,950 \sqrt{f'_c}$, psi, and $E_c = 64,190 \sqrt{f'_c}$, psi, respectively, for the ASTM moist-cured specimens. These specimens resulted in an average elastic modulus at 56 days of 36.4 GPa (5,280 ksi), 36.5 GPa (5,300 ksi), and 42.3 GPa (6,130 ksi), respectively, for the casting dates on which monitoring was done. The HPC and HS/HPC resulted in elastic moduli of 0.4 and 16.1 percent higher than the NSC, respectively.

6.6.4.3.3 Splitting Tensile Strength ASTM C496-96. The linear regression analysis for the Class S NSC, Class S Modified HPC, and Class K HS/HPC resulted in an empirical equation of $f_{sp} = 7.33 \sqrt{f'_c}$, psi, $f_{sp} = 6.94 \sqrt{f'_c}$, psi, and $f_{sp} = 8.79 \sqrt{f'_c}$, psi, respectively, for the ASTM moist-cured specimens. These specimens resulted in an average elastic modulus at 56 days of 3.9 MPa (565 psi), 4.1 MPa (590 psi), and 5.9 MPa (860 psi), respectively, for the casting dates on which monitoring was done. The HPC and HS/HPC resulted in splitting tensile strengths of 4.4 and 52.2 percent higher than the NSC, respectively.

6.6.4.3.4 Rapid Chloride Permeability AASHTO T277. The average RCPT results for the three classifications of mix designs used in the bridge decks are illustrated in Figure 6.116. Only the Class K HS/HPC resulted in permeability classifications in the same range for the two curing regimes. Four of the casting dates for the Class S Modified decks were during cold weather, which impacted

the natural hydration curve and the permeability of the concrete. Permeability testing for the Class S Modified mix design, which was conducted during more ideal casting conditions (mild weather), resulted in low permeability (1,110) similar to the trial field mix.

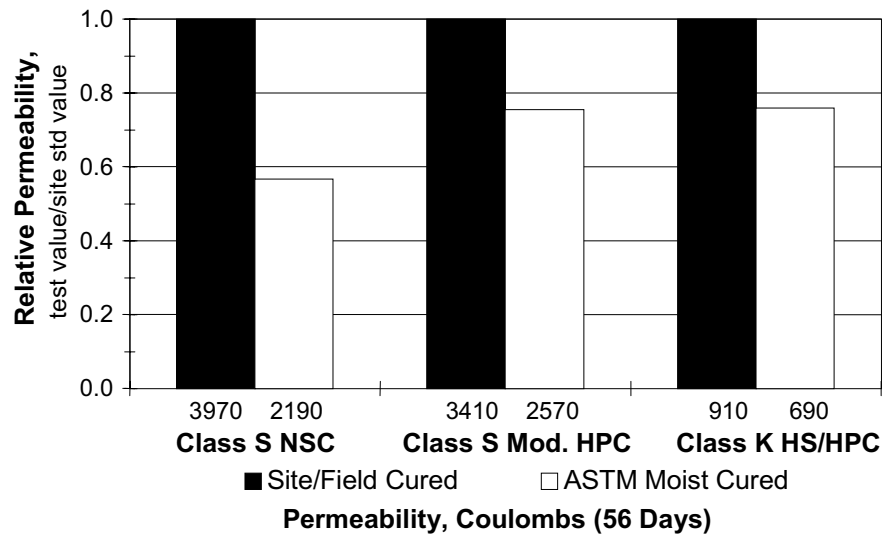


Figure 6.116: Average Rapid Chloride Penetration (56 Days) for North Concho River Overpass Cast-in-Place Decks

The difference between ASTM moist-cured specimens and site/field-cured specimens may also be noted in Figure 6.116. The variability between the ASTM moist-cured and site/field-cured specimens for the Class S NSC was 56.7 percent on average while the HPC and HS/HPC were above 75 percent. When specifying a permeability requirement, the design engineer should be aware that for NSC, the ASTM moist-cured specimens do not provide an accurate indication for the permeability of the concrete in the structure. These findings are consistent with the results from the substructure components (Figure 6.68).

6.6.4.3.5 Chloride Ponding AASHTO T259. Chloride ponding results for the three North Concho River Overpass deck mix designs are illustrated in Figure 6.117. Samples were taken on one casting date for the Class S and Class S Modified mix designs, while samples were taken on two casting dates for the Class K mix design. The Class S NSC exhibited the highest chloride penetration after the 90-day ponding cycle while the Class K HS/HPC exhibited the lowest. As compared to the Class S NSC, the fly ash replacement used for the Class S Modified mix design reduced the chloride ion penetration by 25.8 percent at a depth of 19 mm (0.75 in.). No chloride ion penetration was noted

for the Class K HS/HPC at a depth of 19 mm (0.75 in.). Clearly, the Class K HS/HPC exhibits significantly better resilience to chloride penetration, but to quantitatively suggest a time-to-corrosion is difficult.

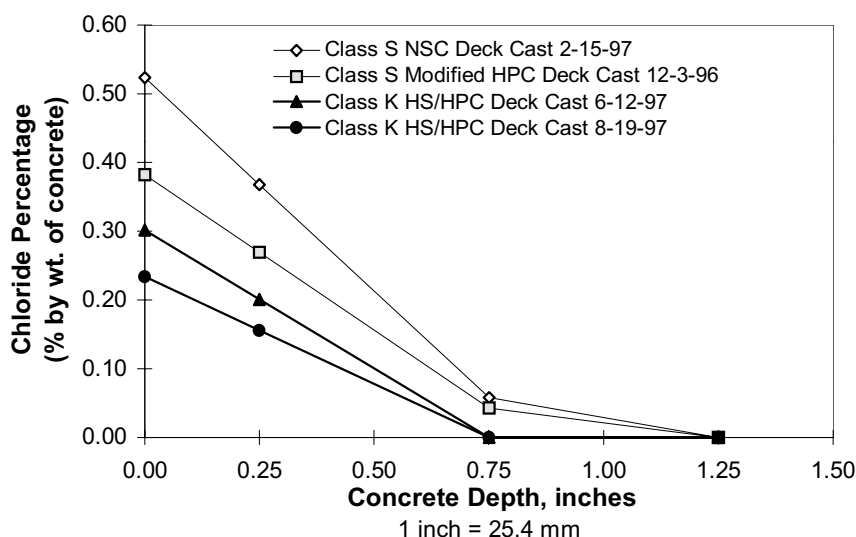


Figure 6.117: Chloride Ion Penetration (90-Day Ponding) for North Concho River Overpass CIP Decks

Many agencies and building codes have proposed chloride threshold values as part of durability models to predict the onset of corrosion. Most performance prediction models are based on Fick's second law of diffusion and the diffusion of chlorides into concrete. These models predict the ingress of chloride ions into concrete and assume that a threat of corrosion exists when the concentration of chloride ions at the level of the steel exceeds established chloride concentration corrosion thresholds. To establish an appropriate diffusion coefficient for such a model is difficult, particularly for a bridge deck, due to the diverse factors that affect corrosion resistance. These diverse factors include permeability, depth of cover, chloride environment, and protection methods (epoxy coated reinforcing, cathodic protection, corrosion inhibitors, etc.).

Because the North Concho River Overpass decks incorporate three different mix designs and spans with and without epoxy-coated reinforcing, it is particularly difficult to establish an appropriate diffusion coefficient. However, a rough prediction of performance may be suggested for the spans with the HS/HPC without epoxy-coated reinforcing if a chloride diffusion coefficient of $0.04 \text{ (in.}^2\text{/yr)}$ is assumed based on the chloride ion penetration and RCPT results attained for the Class K HS/HPC

decks. A conservative environmental chloride concentration of 5 lb/yd³ can also be assumed for San Angelo based on the limited number of freeze-thaw cycles (Figure 2.15) when deicing salts might be used and on the overall mild climate (Figure 2.19). The concentration of chloride ions in seawater is approximately 30 lb/yd³, contrasting salt concentrations for decks where chlorides are frequently washed off the deck surface by rain water. Using Fick's Second Law of Diffusion, illustrated by Equation 6.12, and assuming a clear cover depth of 63.5 mm (2.5 in.) and a chloride concentration threshold of 1.5 lb/yd³ proposed by Clear [140], a duration of exposure can be back-calculated. The 63.5-mm (2.5-in.) clear cover is based on the typical clear cover values measured during casting of the decks.

$$C_{x,t} = C_o \left[1 - \operatorname{erf} \left(\frac{x}{2\sqrt{tD}} \right) \right] \quad (6.12)$$

where:

- $C_{x,t}$ = chloride content at depth x and time t (lb/yd³)
- C_o = environmental chloride concentration (lb/yd³)
- erf = mathematical standard error function
- x = depth into concrete [typically cover depth] (inches)
- t = duration of exposure [required service life] (years)
- D = diffusion coefficient (in.²/years)

The resulting rough prediction for the corrosion service life of the Class K HPC deck without epoxy reinforcing is 75 years. This compares to a rough prediction of 15 years for the NSC without epoxy reinforcing based on a chloride diffusion coefficient of 0.20 (in.²/yr). Even this rough prediction, without including the corrosion-resistant benefits of the epoxy-coated reinforcing, serves to illustrate the long-term benefits of the HS/HPC over conventional concretes.

6.6.4.3.6 Freeze-Thaw Resistance ASTM C666. Freeze-thaw resistance testing was conducted on concrete sampled from the North Concho River Overpass CIP decks. All concrete sampled from both the westbound and eastbound decks satisfied the ACI criteria for durable concrete (DF>60). In addition, all of the concrete sampled satisfied the FHWA Grade 2 criteria for freeze-thaw resistance established under the HPC definition as presented in Chapter 3 (DF>80). No severe or moderate surface damage was exhibited for any of the concrete sampled as illustrated in Figure 6.118.

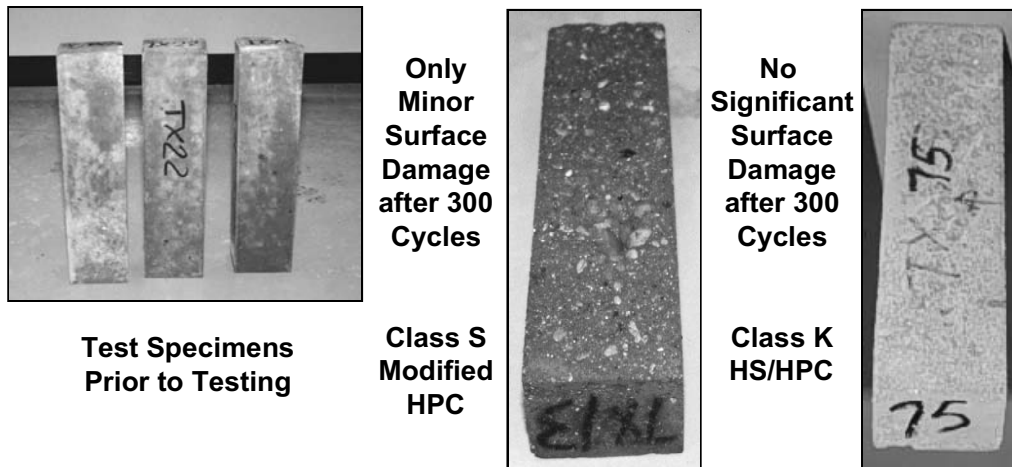


Figure 6.118: Freeze-Thaw Specimens Prior to and after 300 Cycles

When investigating the three different mix designs selected for use in the North Concho River Overpass bridge decks, only a minor variation in freeze-thaw resistance performance was noted. Each mix design incorporated air entrainment in the 5 to 7 percent range. Figure 6.119 illustrates the durability factor versus number of cycles for three representative casting dates. Both the Class K HS/HPC and Class S Modified HPC resulted in durability factors exceeding 95 after 300 cycles while the Class S NSC resulted in a durability factor of approximately 92. The only variation noted between the Class K HS/HPC and the Class S Modified HPC was the relative level of distress exhibited after 300 cycles. Figure 6.120 illustrates the mass change versus the number of cycles for the aforementioned casting dates. A loss of mass for the specimen may be associated with surface scaling and other forms of deterioration and distress. Therefore, a greater loss in mass indicates a higher relative level of freeze-thaw damage.

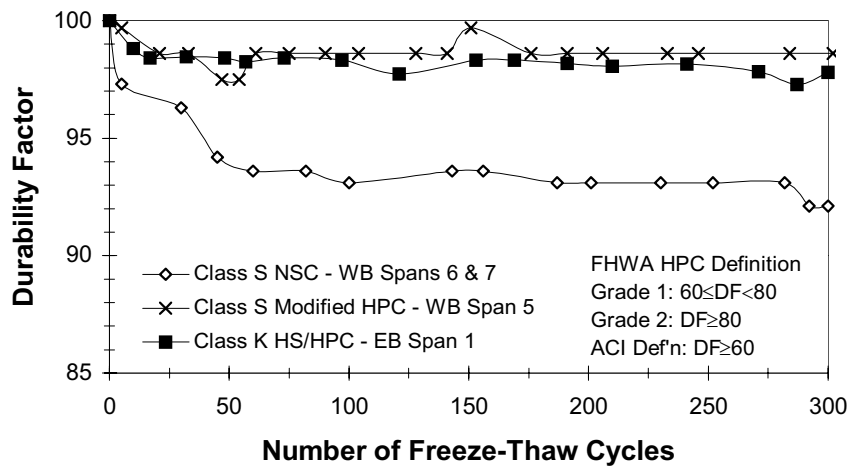


Figure 6.119: Freeze-Thaw Resistance for North Concho River Overpass Decks Durability Factor versus Number of Freeze-Thaw Cycles

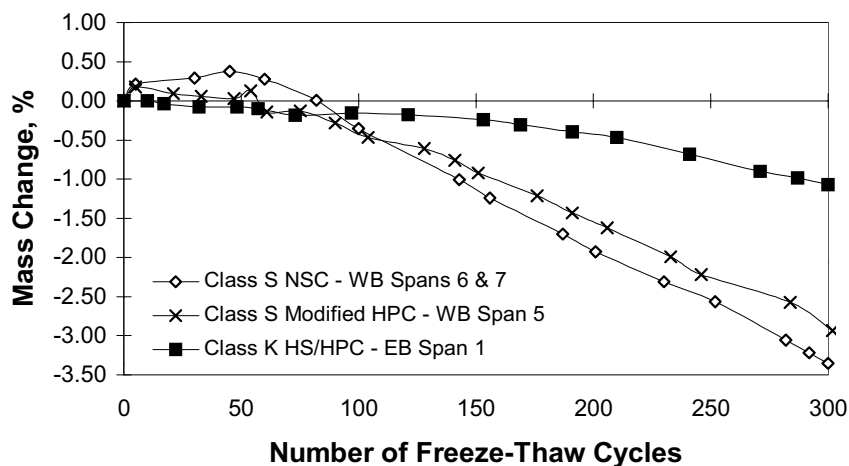


Figure 6.120: Freeze-Thaw Resistance for North Concho River Overpass Decks Percent Mass Change versus Number of Freeze-Thaw Cycles

The Class K HS/HPC exhibited a change in mass of less than 1 percent, while the Class S Modified HPC and Class S NSC exhibited a loss in mass in the 2 to 3.5 percent range. While the HS/HPC resulted in slightly less surface damage, no specimens exhibited significant distress or deterioration. The Class S NSC and Class S Modified HPC primarily exhibited a slightly greater level of surface damage (scaling) compared to the HS/HPC after 300 cycles.

The impact of compressive strength on the freeze-thaw resistance was minimal for concretes incorporating an adequate air void system as illustrated in Figure 6.121. Freeze-thaw resistant concretes can be produced with and without high compressive strengths. For concretes without air entrainment and a high compressive strength (w/cm ratio below 0.30), adequate freeze-thaw resistance may be anticipated due to a discontinuous capillary pore structure and low permeability. HS/HPC without air entrainment was not investigated within the scope of this study.

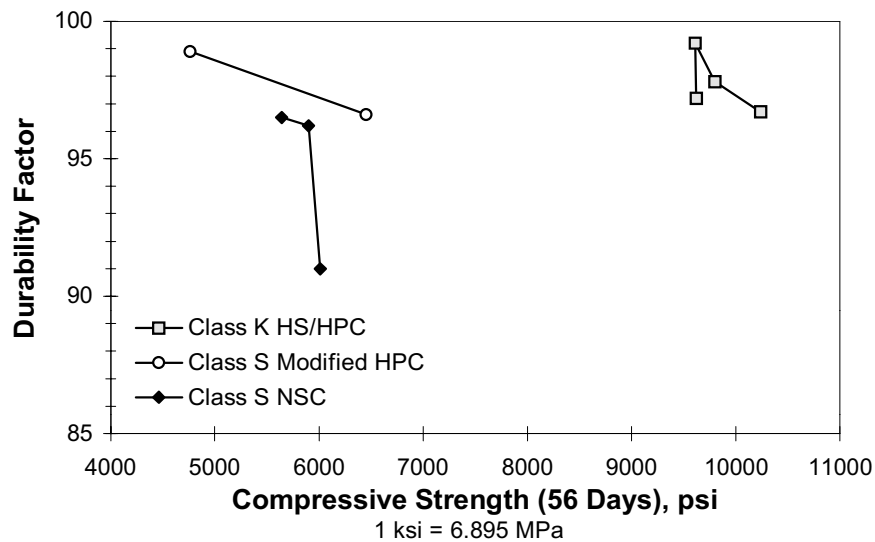


Figure 6.121: Durability Factor versus Compressive Strength for North Concho River Overpass CIP Decks

From this discussion, it may be concluded that the enhanced compression strength characteristics generally associated with many HPCs has little influence on the freeze-thaw resistance of concretes that incorporate sound materials and an adequate well-distributed air void system. Although not investigated within the scope of this study, HPC without air entrainment combined with a low w/cm ratio (<0.30) can also be expected to result in good freeze-thaw resistance due to the discontinuous capillary pore structure and low permeability.

In addition to freeze-thaw testing, internal temperature profiles for Span 4 of the eastbound deck were monitored for a 90-day period. Monitoring was performed to record maximum thermal gradient profiles and to document the standard temperature profile response of the concrete deck to

any freeze-thaw cycles. Monitoring occurred from January 16, through April 16, 1998. It was anticipated that freeze-thaw cycles (if any) and maximum thermal gradients within the deck would occur during this time period. Ambient temperatures dipped below freezing on 19 occasions during the 90-day monitoring period as illustrated in Figure 6.122.

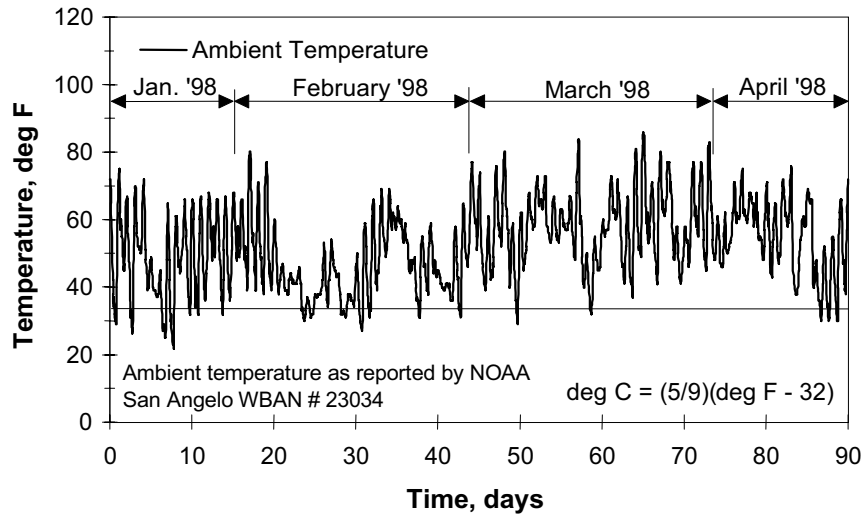


Figure 6.122: Ambient Temperature in San Angelo, Texas, from January 16 through April 16, 1998

Figure 6.123 illustrates the concrete and ambient temperature profiles over a 10-day period (January 16–26, 1998) in which the most severe ambient conditions occurred. Three depths within the concrete deck were monitored. On only two instances did any portion of the concrete attain a temperature below freezing (Figure 6.123). Even if there were significant exposure temperatures to cause numerous freeze-thaw cycles of the deck, it is unlikely that either the WB or EB HPC decks will ever experience any significant level of freeze-thaw damage due to the low permeability and discontinuous capillary structure of the HPC. The maximum thermal gradient temperature recorded during a 24-hour period occurred on March 5, 1998. The maximum gradient between the deck and ambient conditions was 21 °C (37.8 °F).

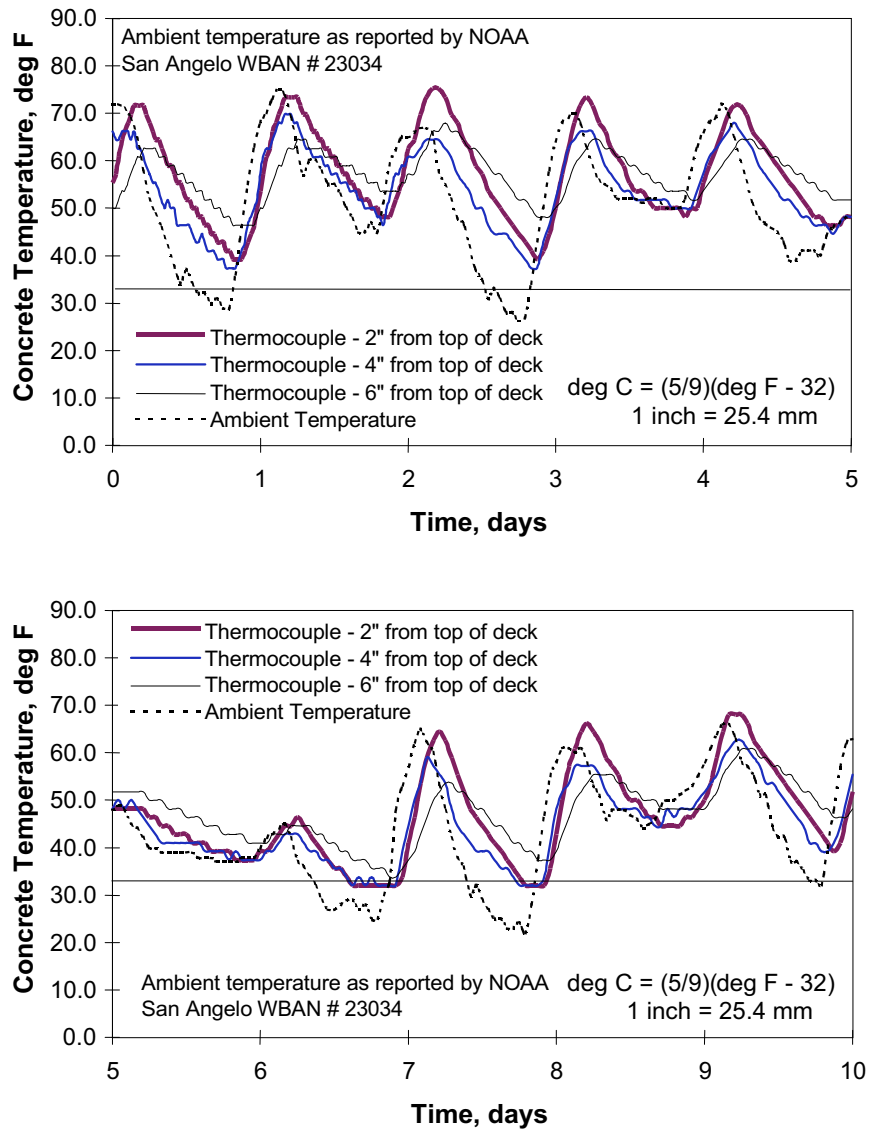


Figure 6.123: Concrete Deck Response to Ambient Conditions for the North Concho River Overpass Deck

6.6.4.3.7 *Abrasion Resistance ASTM C944.* Abrasion resistance testing was conducted on concrete sampled from the North Concho River Overpass CIP decks. The Class S NSC and Class S Modified HPC that was sampled satisfied the FHWA Grade 1 criteria for abrasion resistance while the Class K HS/HPC that was sampled satisfied the FHWA Grade 2 criteria for abrasion resistance established under the HPC definition as presented in Chapter 3 (DF>80). In general, the Class K HS/HPC exhibited the least wear for all of the concrete sampled as illustrated in Figure 6.124.

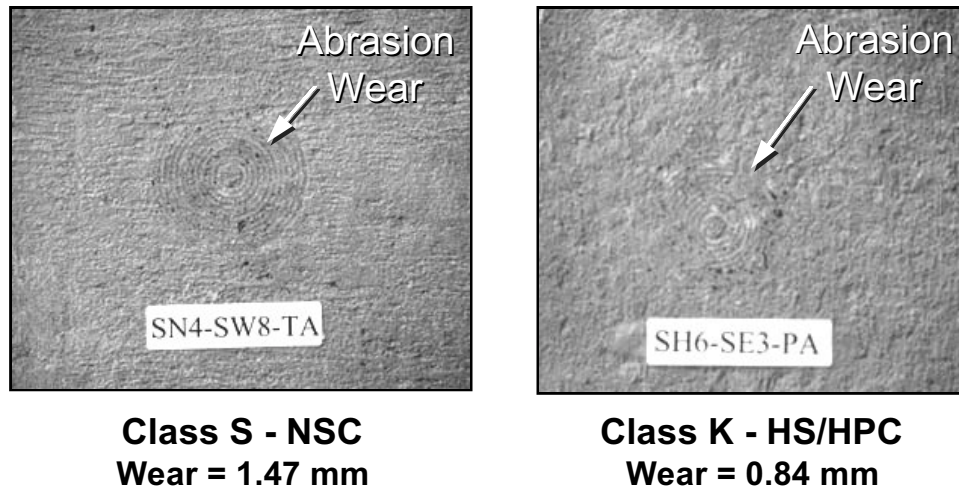


Figure 6.124: Comparison of Abrasion Resistance for Class S NSC and Class K HS/HPC after 6-Minute Wear Cycles

The impact of compressive strength on the wear resistance of the concrete specimens is illustrated in Figure 6.125. It may be concluded that the compressive strength of the concrete was the primary factor that influenced the variation in wear resistance of the concrete because the aggregates, finishing, and curing techniques were similar for all of the concrete specimens. These results are consistent with previous research studies [37] that have resulted in similar findings. HS/HPC can generally be expected to result in less wear resistance. For the concrete specimens investigated, the HS/HPC resulted in 17.0 percent less wear on average than conventional concrete specimens after the initial 6-minute wear cycle.

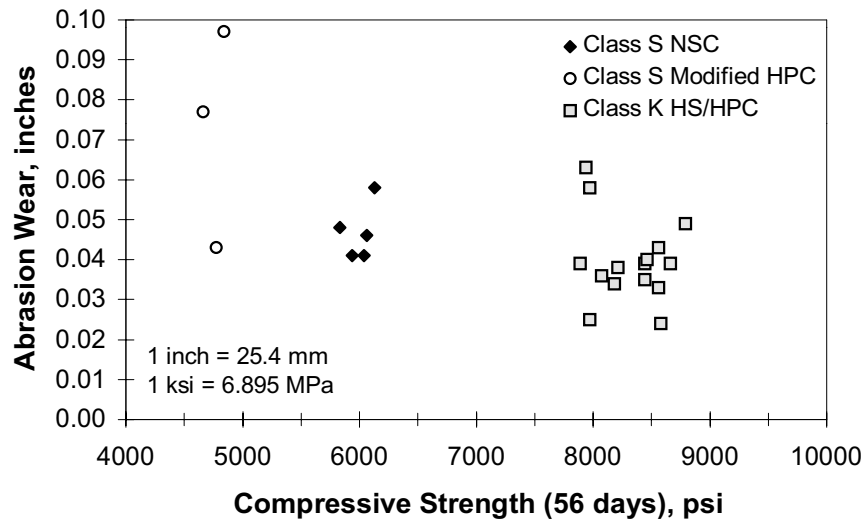


Figure 6.125: Abrasion Wear versus Compressive Strength

6.6.4.3.8 *Scaling Resistance ASTM C672.* Figure 6.126 presents a visual comparison of the scaling resistance for the mix designs selected for use in the bridge decks. The NSC clearly had the most severe scaling damage of the three mixes investigated. The HS/HPC exhibited low to moderate damage, while the HPC resulted in no surface damage.

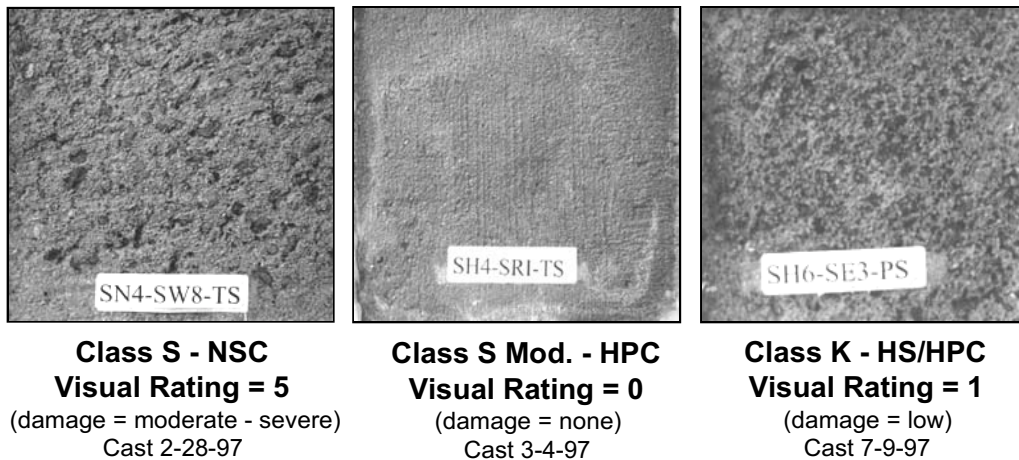


Figure 6.126: Comparison of Scaling Resistance for Class S NSC, Class S Modified HPC, and Class K HS/HPC after Fifty Cycles

This performance level related to the HS/HPC was not anticipated, but may be explained by the fact that the scaling resistance of concrete is greatly influenced by the finishing, placement, and curing conditions (Section 2.3.5). It should be noted that all of the Class S NSC and Class S Modified HPC decks were cast from December 1996 through March 1997 during more ideal placement and curing conditions. The Class K HS/HPC decks were cast from June 1997 through September 1997 during the hottest months in Texas where the placement and curing conditions were less than ideal. Continuous fogging and moist (wet mat) curing were much more critical to avoid surface-related problems during the casting of these decks due to the environmental conditions and the nature of the Class K HS/HPC mix design. The mix design had little or no bleed water rising to the surface of the concrete due to the low w/cm ratio of the mix design. Furthermore, while the specimens were cured alongside the decks, they were not subjected to fogging or moist curing during the first 24 to 48 hours as the decks were. It may be concluded that the Class K HS/HPC specimens were more adversely affected by the environmental and curing conditions at the time they were cast. While the Class K HS/HPC consistently had less visual damage than the Class S NSC, it is believed that the Class K HS/HPC would have resulted in similar visual ratings as the Class S Modified HPC if it had been cast and cured under identical conditions because the material and mixture characteristics of the two mix designs were similar.

The fly ash replacement selected for use in the Class S Modified HPC mix design appeared to significantly improve the scaling resistance when compared to the Class S NSC. The spherical shape of fly ash particles enhanced the finishing characteristics of the concrete resulting in a more dense, durable surface. All other characteristics including materials, curing procedures, air entrainment, and surface sealers were identical for the Class S and Class S Modified decks. Consequently, it may be concluded that the fly ash replacement enhanced the scaling resistance of the concrete for the specimens evaluated.

As anticipated, the compressive strength of the concrete did not dramatically influence the scaling resistance of the concrete as illustrated in Figure 6.127. The concrete that had the lowest compressive strength actually resulted in the highest scaling resistance. This is consistent with previous research studies [41, 47] that conclude that the primary factors affecting scaling resistance are materials, mixture characteristics, curing and curing procedures, air entrainment, aging effects, and sealers. In some cases, the higher strengths associated with many HPCs may actually decrease scaling resistance unless recommended finishing and curing techniques are strictly followed. This is particularly critical for low w/cm ratio concretes (<0.35).

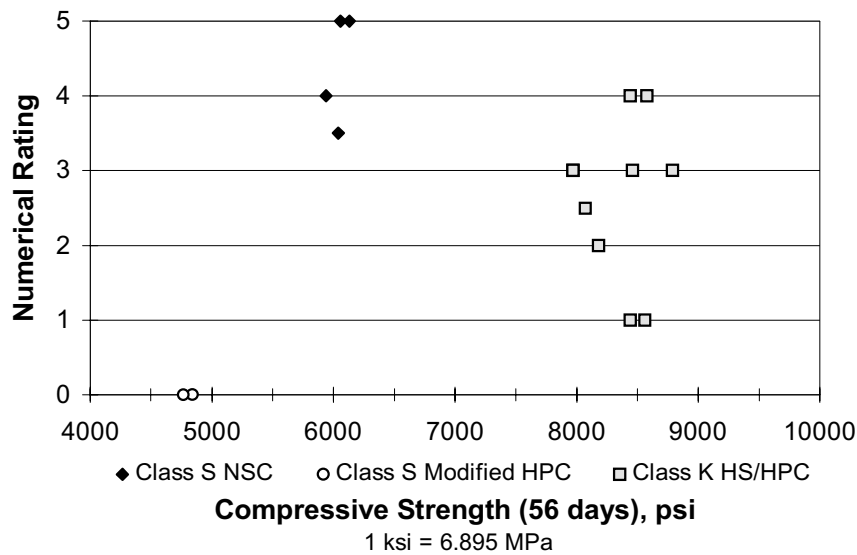


Figure 6.127: Scaling Resistance Numerical Rating versus Compressive Strength

6.6.4.3.9 Material Cost Comparison. The relative material costs associated with the Class S NSC, Class S Modified HPC, and Class K HS/HPC mix designs were \$26.00, \$22.24, and \$32.39 per cubic meter (\$33.98, \$29.09, and \$42.37 per cubic yard), respectively. The Class S Modified HPC not only yielded superior mechanical and material performance as compared with the Class S NSC, but also reduced material cost as a result of the fly ash replacement. For HPC bridge decks where high strength is not required, the Class S Modified HPC mix design may be recommended in lieu of the Class K HS/HPC owing to the placement and finishing concerns, particularly in hot, dry, windy environments.

CHAPTER 7. STATISTICAL ANALYSIS OF PLANT-PRODUCED CONCRETE

7.1 Introduction

The statistical analysis of both the precast plant-produced HPC and ready-mix batch plant-produced HPC was investigated. The results of the analysis have been divided into three primary sections. These sections include the precast/prestressed concrete beams, the substructure piers and pier caps, and the cast-in-place decks for the North Concho River Overpass. Statistical analysis for the Louetta Road Overpass substructure piers and decks as well as the North Concho River Overpass and Louetta Road Overpass precast deck panels were not investigated as part of this study due to the limited number of casting dates. The statistical analysis investigated the variation and performance of the HPC selected for use in the aforementioned bridge components. Personnel from CMRG at The University of Texas at Austin performed testing services for all mechanical concrete properties discussed herein.

7.2 Precast Plant-Produced Precast/Prestressed Beams

A statistical analysis of the precast plant-produced precast/prestressed concrete beams was performed for the twenty casting dates monitored. Three curing regimes were investigated including ASTM moist-cured, member-cured, and match-cured cylinders. Variables investigated as part of the statistical analysis included average moving compressive strength, standard deviation, and coefficient of variation. The mix design performance was compared to current ACI Committee 301 code recommendations for minimum required strength and ACI Committee 214 criteria for concrete performance and laboratory testing proficiency.

Figure 7.1 illustrates the cumulative average compressive strength at 56 days for the precast/prestressed beams for each of the curing regimes investigated. Each curing regime exceeded ACI Committee 301 required average compressive strength criteria of 74.5 MPa (10,800 psi) based on the mix design's standard deviation and design strength. The ASTM moist-cured specimens exhibited the highest average compressive strength. The moving average from three sequential casting dates for the compressive strength at 56 days is illustrated in Figure 7.2.

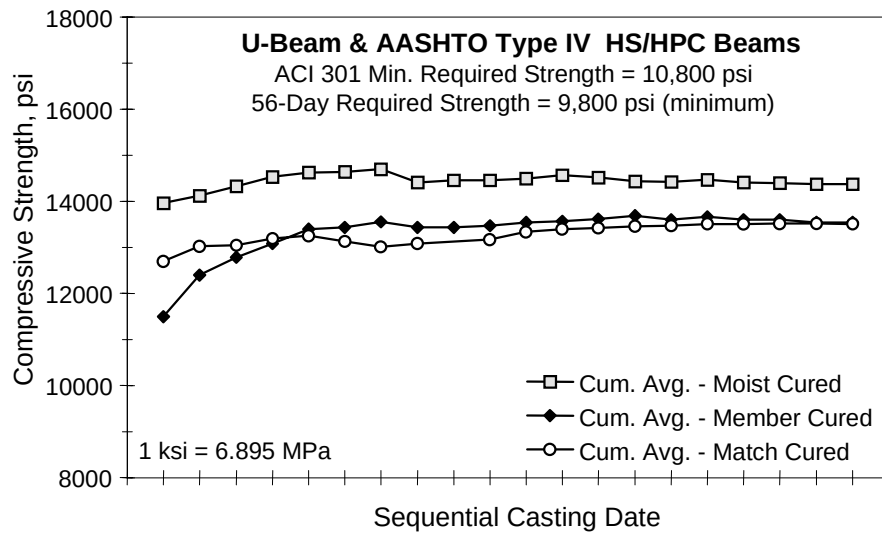


Figure 7.1: Cumulative Average of Compressive Strength (56 Days) for the Precast/Prestressed Beams

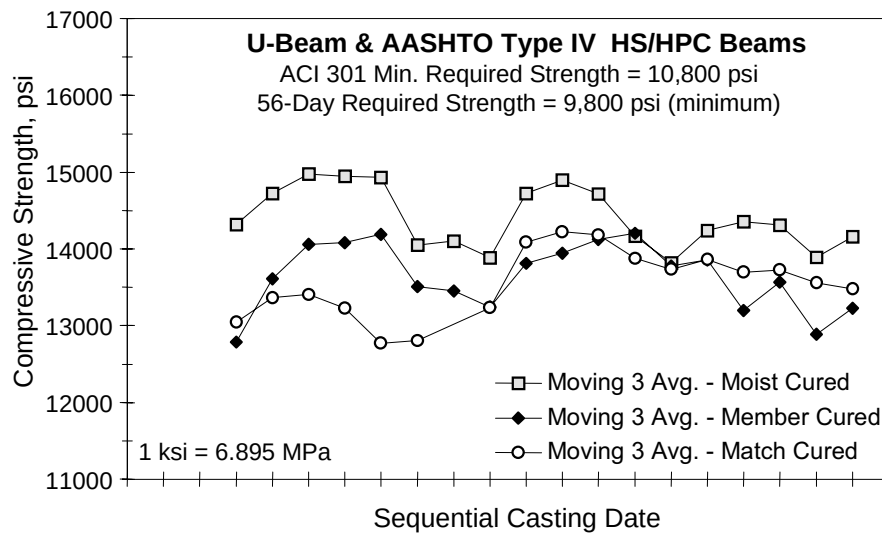


Figure 7.2: Moving Average from Three Sequential Casting Dates for Compressive Strength (56 Days) for the Precast/Prestressed Beams

The moving average is an indicator of the consistency of concrete strength produced by the precast plant. Each curing regime exhibited similar performance trends with peaks and valleys. The within test compressive strength variation for the ASTM moist-cured, member-cured, and match-cured specimens was ± 2.62 MPa (381 psi), ± 3.03 MPa (440 psi), and ± 2.10 MPa (305 psi), respectively. This within test variation is equivalent to approximately 2.5 to 3.7 percent of the 98 percent confidence interval strength for each curing regime. Each curing regime resulted in a zero percent probability of a failing test based on the 98 percent confidence interval. Figures 7.3, 7.4, and 7.5 illustrate the compressive strength distribution and 98 percent confidence interval for each curing regime. The ASTM moist-cured and match-cured specimens exhibited the tightest distribution grouping while the member-cured specimens displayed the widest distribution. The ASTM moist-cured cylinders were cured under controlled conditions at a constant temperature of 21.1 ± 1.7 °C (70 ± 3 °F) and were not impacted by ambient temperatures other than indirectly by the concrete temperature at placement. The ambient conditions at casting undoubtedly impacted the member-cured cylinders the most severely because the specimens were cured alongside the bed location, exterior of the members.

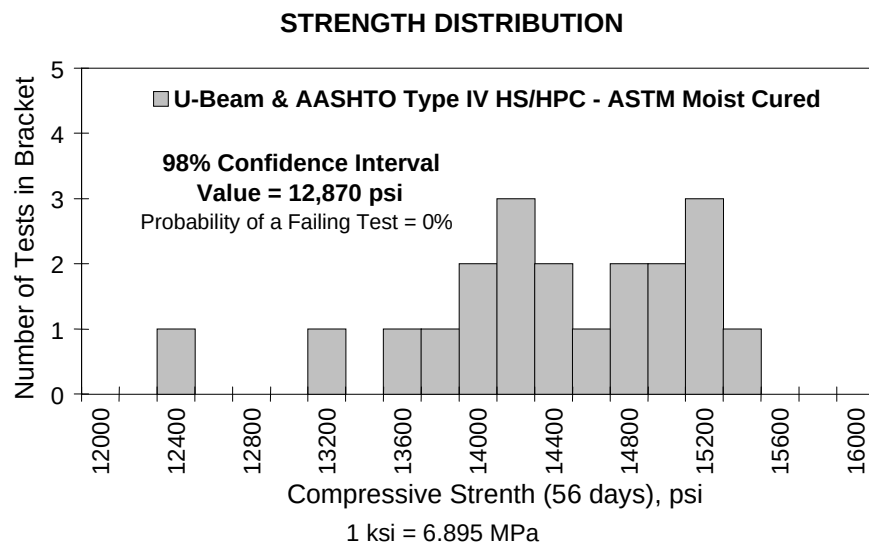


**Figure 7.3: Compressive Strength Distribution and Confidence Interval
for the Precast/Prestressed BeamsASTM Moist-Cured Cylinders**

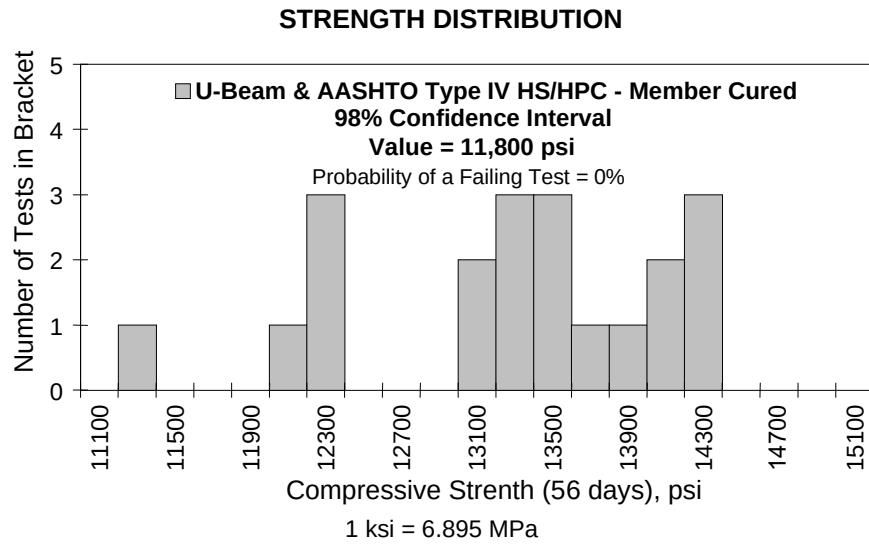


Figure 7.4: Compressive Strength Distribution and Confidence Interval for the Precast/Prestressed Beams—Member-Cured Cylinders

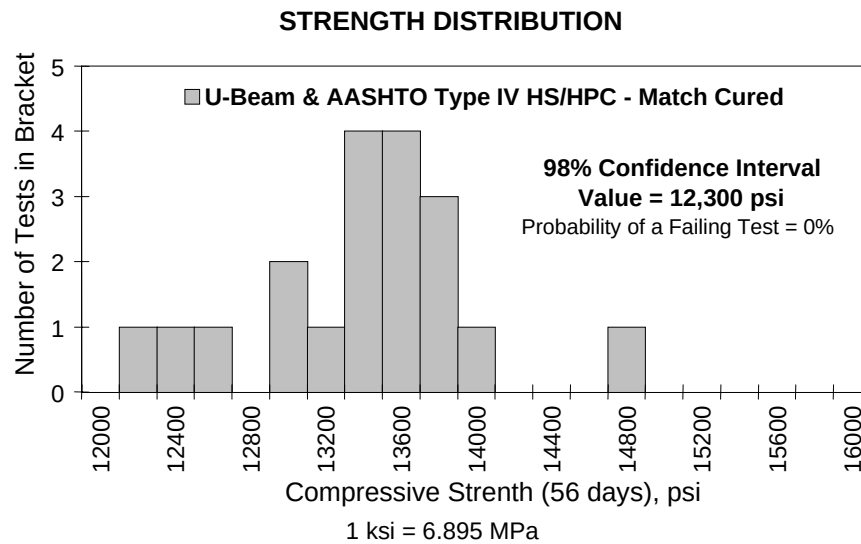


Figure 7.5: Compressive Strength Distribution and Confidence Interval for the Precast/Prestressed Beams—Match-Cured (Upper Flange) Cylinders

Member-cured specimens were not only subjected to the heat of hydration generated by the member, but also a variety of ambient conditions from casting date to casting date. The match-cured

specimens were influenced to a lesser degree by ambient conditions at casting because the match-curing location was centered in a mass of concrete.

In addition to the compressive strength performance trends, the variability of the mix design was investigated. The cumulative standard deviation for the three curing regimes is presented in Figure 7.6. The ACI Committee 214 concrete performance classification for the ASTM moist-cured and member-cured specimens was in the “poor” range based on the standard deviation. The ACI Committee 214 concrete performance classification for the match-cured specimens was in the “good” range. It should be noted that HS/HPCs have exhibited higher variability compared to conventional concretes due to their high strength and the fact that they are more dramatically influenced by curing, sampling, and testing. Therefore, in general, a higher standard deviation compared to conventional concretes is anticipated for HS/HPC. The general downward trend in the standard deviation demonstrates that the precast plant produces the concrete more consistently as they continue to gain experience with the mix design.

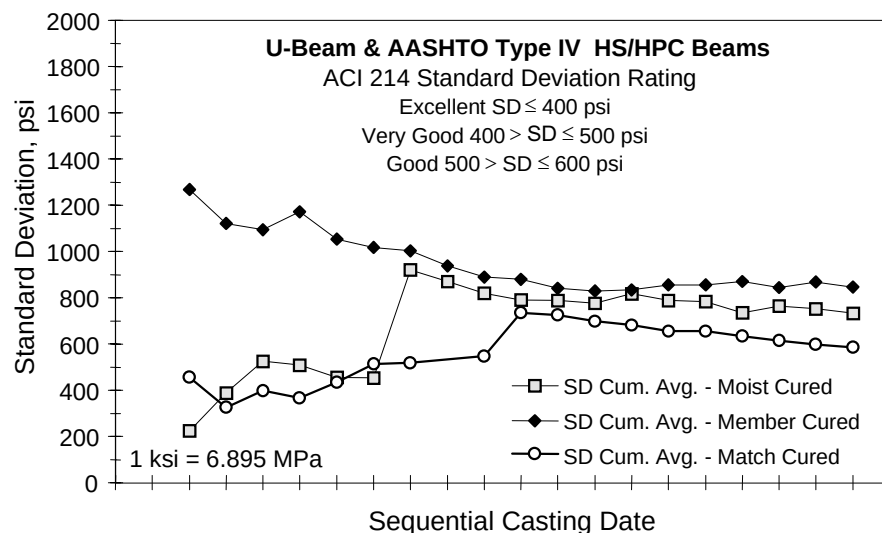


Figure 7.6: Cumulative Standard Deviation for the Precast/Prestressed Beams

The testing proficiency during production was also investigated. The within test coefficient of variation and testing proficiency for the three curing regimes is presented in Figure 7.7. All three curing regimes resulted in an ACI Committee 214 laboratory testing proficiency in the “excellent” range based on the coefficient of variation. The match-cured specimens had the lowest coefficient of variation while the member-cured specimens had the highest coefficient of variation. The ASTM

moist-cured and match-cured specimens were very consistent throughout the entire testing phase. The member-cured specimens exhibited a slightly higher within test variability. Table 7.1 summarizes the results of the statistical analysis for the precast/prestressed beams.

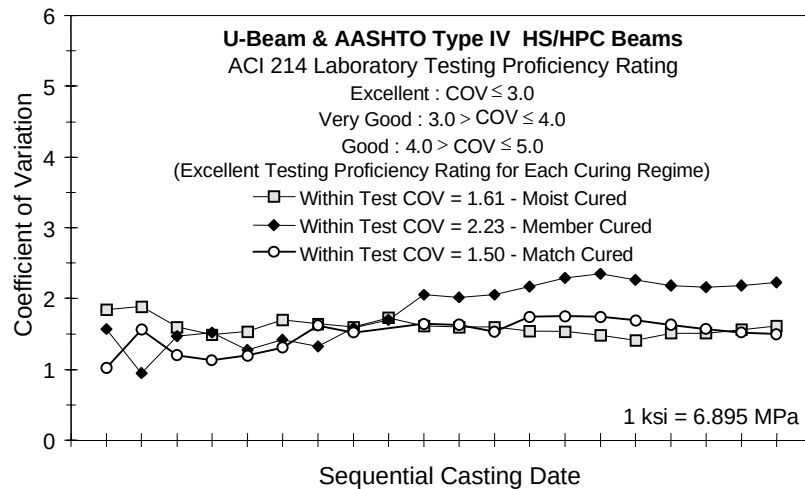


Figure 7.7: Within Test Coefficient of Variation and Testing Proficiency for Precast/Prestressed Beams

Table 7.1: Summary of Statistical Analysis for Precast/Prestressed Beams (56-Day)

Statistical Criteria	Moist-cured	Member-cured	Match-cured
Required 56-Day Design Strength (psi)	9,800-14,700 ❶	9,800-14,700 ❶	9,800-14,700 ❶
Casting Dates Sampled	20	20	19
Total Test Specimens	60	60	57
Average Compressive Strength (psi)	14,376	13,542	13,502
Minimum Required Avg. Strength (psi)	11,150	11,430	10,800
Margin of Extra Performance (psi)	+3,226	+2,112	+2,702
Cumulative Avg. Standard Deviation (psi)	732	845	586
Concrete Performance (ACI 214)	Poor	Poor	Good
Within Test Coefficient of Variation	1.61	2.23	1.50
Average Range of Test (psi)	392	511	342
Laboratory Testing Proficiency (ACI 214)	Excellent	Excellent	Excellent
98 % Confidence Interval Value ❷ (psi)	12,870	11,800	12,300
Confidence Interval Range ❸ (psi)	13,995-14,760	13,102-13,982	13,197-13,807
Probability of a Failing Test	0%	0%	0%

❶ Required Compressive Strength at 56 days varied from 9,800 to 14,700 psi based on the span location and the design requirements. Probability of a failing test based on lowest strength criteria.
❷ Includes ACI 301 correction factor.
❸ Confidence Interval Range is for raw data without correction factor.
Match-cured Data is based on Upper Flange Location, which is most representative for the average strength of the member.
1 ksi = 1,000 psi = 6.895 MPa

7.3 North Concho River Overpass Batch Plant-produced Substructure Components

A statistical analysis of the ready-mix batch plant-produced concrete piers and pier caps was performed for the three classifications of concretes monitored during casting of the substructure components. These included the Class C NSC, Class F HPC, and Class K HS/HPC piers and pier caps. Two curing regimes were investigated including ASTM moist-cured and site/field-cured cylinders. Variables investigated as part of the statistical analysis included average moving compressive strength, standard deviation, and coefficient of variation. The performances of the three mix designs were compared to current ACI Committee 301 code recommendations for minimum required strength and ACI Committee 214 criteria for concrete performance and laboratory testing proficiency. The following sub-sections present and discuss the statistical analysis for the high performance concretes.

7.3.1 North Concho River Overpass Class K HS/HPC Piers and Pier Caps

Figure 7.8 illustrates the cumulative average and moving average compressive strength at 28-days for the Class K HS/HPC piers and pier caps. The ASTM moist-cured cylinders exceeded ACI Committee 301 required average compressive strength criteria of 52.5 MPa (7,610 psi) based on the mix design standard deviation and design strength.

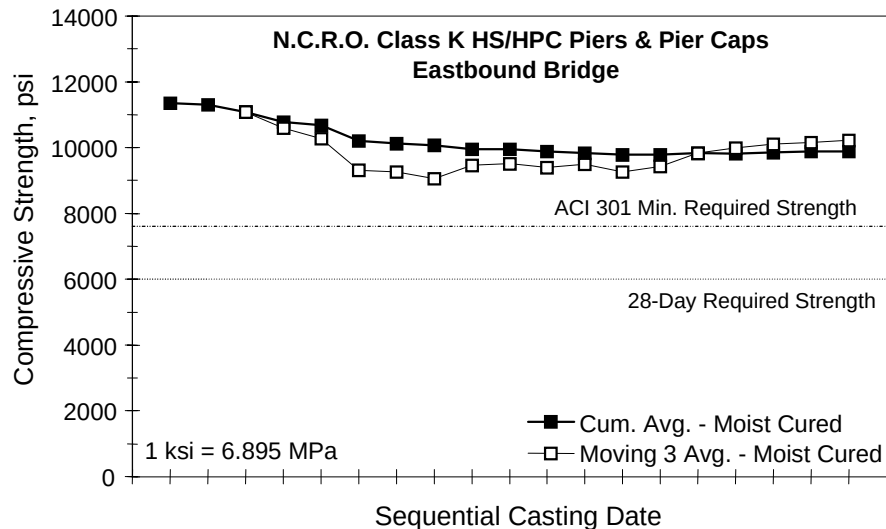


Figure 7.8: Compressive Strength (28-days) Cumulative Average and Moving Average for the Class K HS/HPC Piers and Pier Caps

Figure 7.9 illustrates the compressive strength distribution and 98 percent confidence interval for the ASTM moist-cured specimens. The 98 percent confidence interval value was 56.5 MPa (8,190 psi) with a zero percent probability of a failing test.

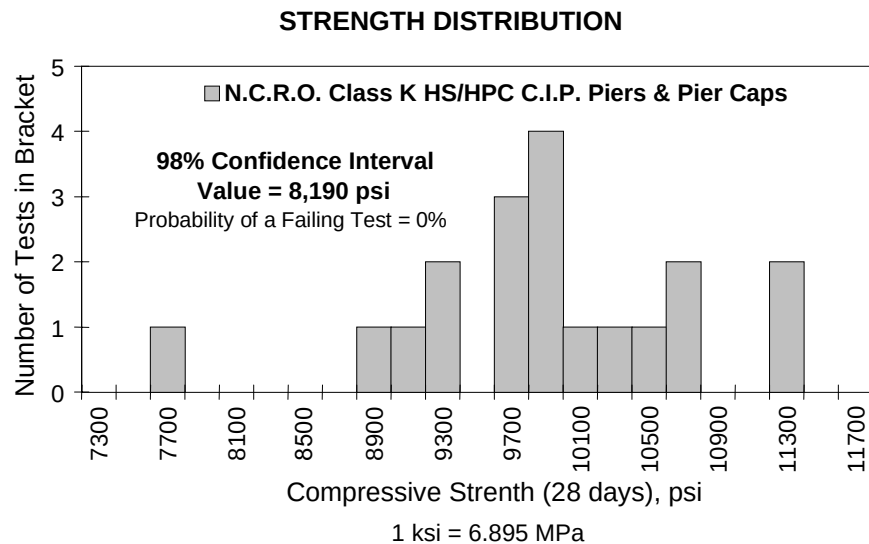


**Figure 7.9: Compressive Strength Distribution and Confidence Interval
for the Class K HS/HPC Piers and Pier Caps**

The cumulative standard deviation and moving standard deviation for the substructure elements are presented in Figure 7.10. The ACI Committee 214 concrete performance classification for the ASTM moist-cured specimens was in the “poor” range based on the standard deviation. After the initial eight casting dates, the standard deviation significantly improved and ranged between 0.7 to 4.8 MPa (100 and 700 psi), which is classified in the “good to excellent” ranges. As the batch plant became more familiar with producing the HPC, the improved consistency in production was noted by the downward trend in standard deviation. This was the first HPC mix design (incorporating an HRWR and fly ash replacement) that the ready-mix batch plant in San Angelo ever produced. While the nature of HPCs is much different than conventional concretes, this demonstrates that even batch plants without any previous experience in producing HPCs can do so consistently in a relatively short period of time.

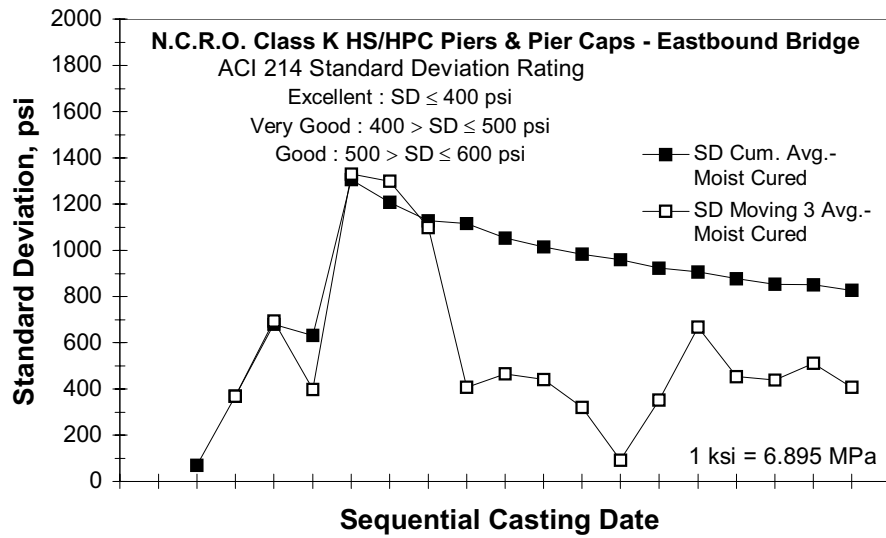


Figure 7.10: Cumulative and Moving Standard Deviation for the Class K HS/HPC Piers and Pier Caps

The testing proficiency and the within test coefficient of variation for the mix design is illustrated in Figure 7.11. The ASTM moist-cured specimens had an ACI Committee 214 laboratory testing proficiency in the “very good” range based on the coefficient of variation.

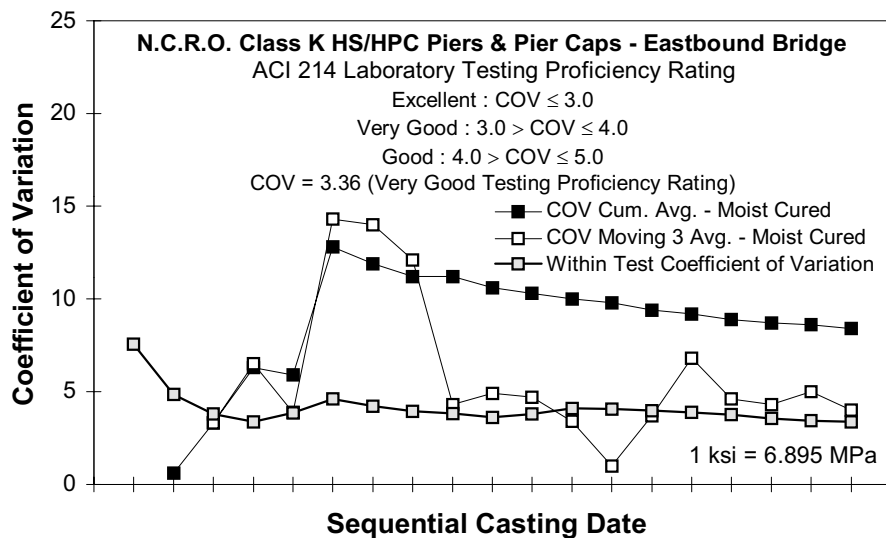


Figure 7.11: Within Test Coefficient of Variation and Testing Proficiency for the Class K HS/HPC Piers and Pier Caps

7.3.2 North Concho River Overpass Class F HPC Pier Caps

Figure 7.12 illustrates the cumulative average and moving average compressive strength at 28-days for the Class F HPC pier caps. Due to the poor strength gain characteristics of the mix design after two casting dates, the Class F HPC mix design was modified to improve the strength gain characteristics and overdesign strength of the mix. This included incorporating an HRWR. Although this mix design was not classified as an HPC in the project specifications, because of the HRWR the mix design has been classified as HPC for reporting purposes. The ASTM moist-cured cylinders exceeded the 28-day design strength requirements, but did not satisfy ACI Committee 301 required average compressive strength criteria of 52.5 MPa (7,610 psi) based on the mix design standard deviation and design strength. The 98 percent confidence interval value was 42.4 MPa (6,148 psi) with a 1.3 percent probability of a failing test after the modification in the mix design occurred.

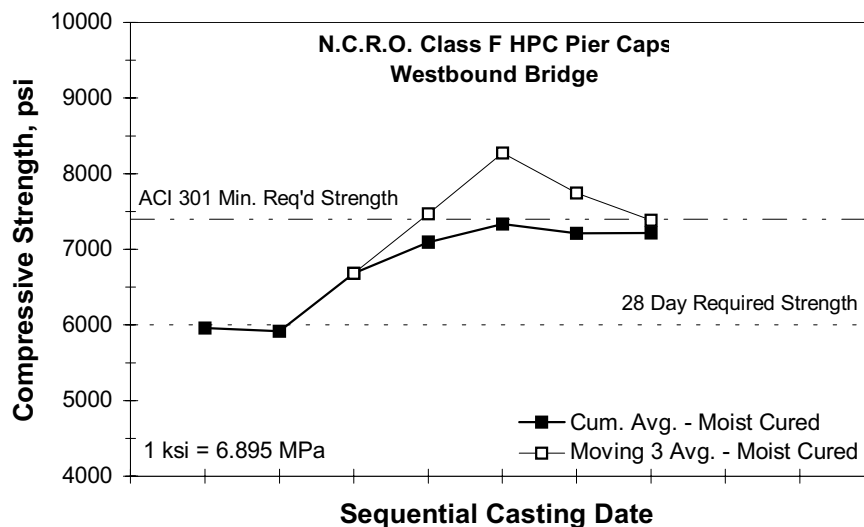


Figure 7.12: Compressive Strength (28-days) Cumulative Average and Moving Average for the Class F HPC Pier Caps

The cumulative standard deviation and moving standard deviation for the pier caps are presented in Figure 7.13. The ASTM moist-cured specimens resulted in an ACI Committee 214 concrete performance classification in the “failure” range based on the standard deviation. This is primarily due to the modification in the mix design, which dramatically affected the strength gain

characteristics of the mix design and, thereby, the standard deviation. The ASTM moist-cured specimens had a “poor” concrete performance classification for the modified mix design alone.

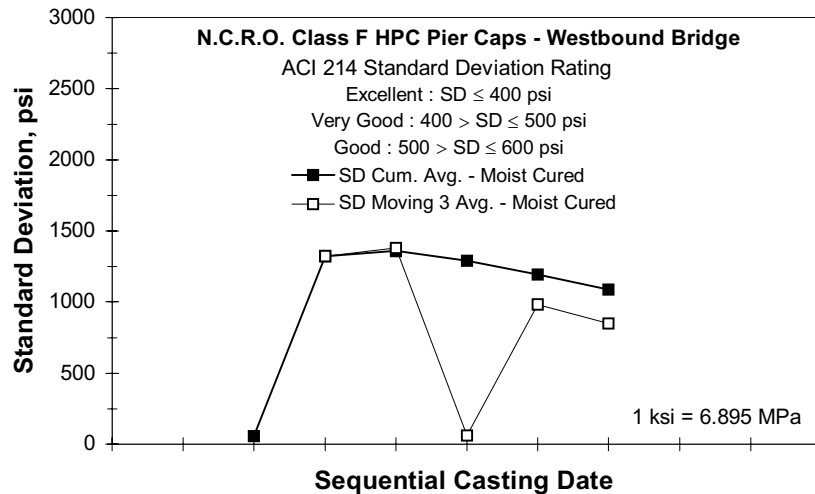


Figure 7.13: Cumulative and Moving Standard Deviation for the Class F HPC Pier Caps

The testing proficiency and the within test coefficient of variation for the mix design are illustrated in Figure 7.14. The ASTM moist-cured had an ACI Committee 214 laboratory testing proficiency in the “good” range based on the coefficient of variation. The ASTM moist-cured specimens had a “very good” laboratory testing proficiency classification for the modified mix design alone.

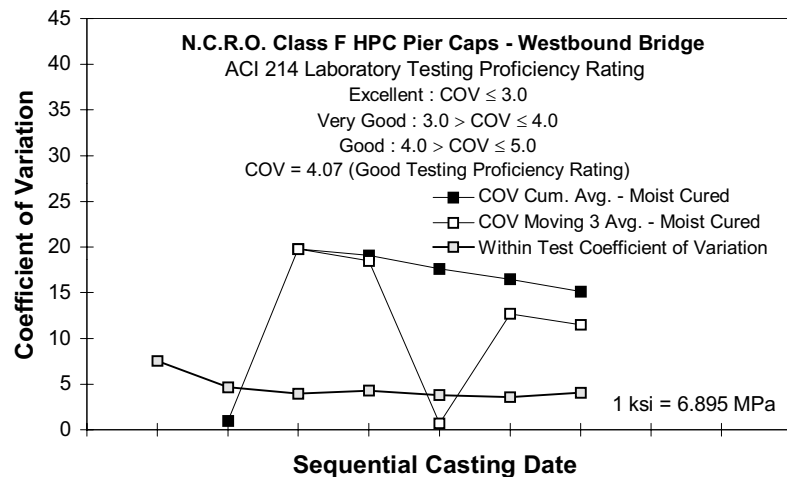


Figure 7.14: Within Test Coefficient of Variation and Testing Proficiency for the Class F HPC Pier Caps

7.3.3 Comparison of North Concho River Overpass Substructure Components

For comparison purposes, the statistical analyses for the three mix designs are summarized in Table 7.2. The mix design for the Class F HPC pier caps was modified after the first two casting dates as previously documented. The bracketed values for this mix design indicate the mix design performance after the modification in the mix design occurred. It may be noted that the HPCs had performance criteria similar to the NSC produced by the ready-mix batch plant. Only the Class K HS/HPC did not have the probability of a failing test.

Table 7.2: Summary of Statistical Analysis for North Concho River Overpass Substructure Components (28-Day)

Statistical Criteria	Class K HS/HPC	Class F HPC ④	Class C NSC
Required 28-Day Design Strength (psi)	6,000-8,000 ①	6,000	4,000
Casting Dates Sampled	19	7 [5]	8
Total Test Specimens	57	21 [15]	24
Average Compressive Strength (psi)	9,887	7,216 [7,736]	5,567
Minimum Required Avg. Strength (psi)	7,610	7,400	5,200
Margin of Extra Performance (psi)	+2,277	-184 [+336]	+367
Cumulative Avg. Standard Deviation (psi)	826	1089 [773]	851
Concrete Performance (ACI 214)	Poor	Fails [Poor]	Poor
Within Test Coefficient of Variation	3.36	4.07 [3.89]	2.92
Average Range of Test (psi)	563	497 [510]	375
Laboratory Testing Proficiency (ACI 214)	Very Good	Good [V-Good]	Excellent
98% Confidence Interval Value ② (psi)	8,190	4,977 [6,148]	3,820
Confidence Interval Range ③ (psi)	8,361-9,521	6,649-7,783	5,124-6,010
Probability of a Failing Test	0%	13.67% [1.25%]	3.29%
① Required Compressive Strength at 28-days for the piers and pier caps was 6,000 and 8,000 psi, respectively. Probability of a failing test based on lowest strength criteria. ② Includes ACI 301 correction factor. ③ Confidence Interval Range is for raw data without correction factor. ④ Mix design modified after first two casting dates, which accounts for the overall poor performance. Bracketed value indicates the performance of the mix design after the modification in the mix design occurred. Results are presented for ASTM moist-cured Specimens. 1 ksi = 1,000 psi = 6.895 MPa			

7.4 North Concho River Overpass Batch Plant-Produced Cast-in-Place Decks

A statistical analysis was performed on the batch plant-produced concrete decks for the two classifications of high performance concretes monitored during casting of the components. These were the Class S Modified HPC and Class K HS/HPC decks. A statistical analysis of the Class S NSC CIP decks was not performed due to the limited number of casting dates (four). The curing regimes for the ASTM moist-cured and site/field-cured cylinders were investigated. Variables investigated as part of the statistical analysis included average moving compressive strength, standard deviation, and coefficient of variation. The performances of the two mix designs were compared to current ACI Committee 301 code recommendations for minimum required strength and ACI Committee 214 criteria for concrete performance and laboratory testing proficiency.

7.4.1 North Concho River Overpass Class K HS/HPC CIP Decks

Figure 7.15 illustrates the cumulative average and moving average compressive strength at 28-days for the Class K HS/HPC eastbound decks. The ASTM moist-cured cylinders exceeded ACI Committee 301 required average compressive strength criteria of 51.0 MPa (7,400 psi) based on the mix design standard deviation and design strength. All of the concrete exceed the 28-day required strength.

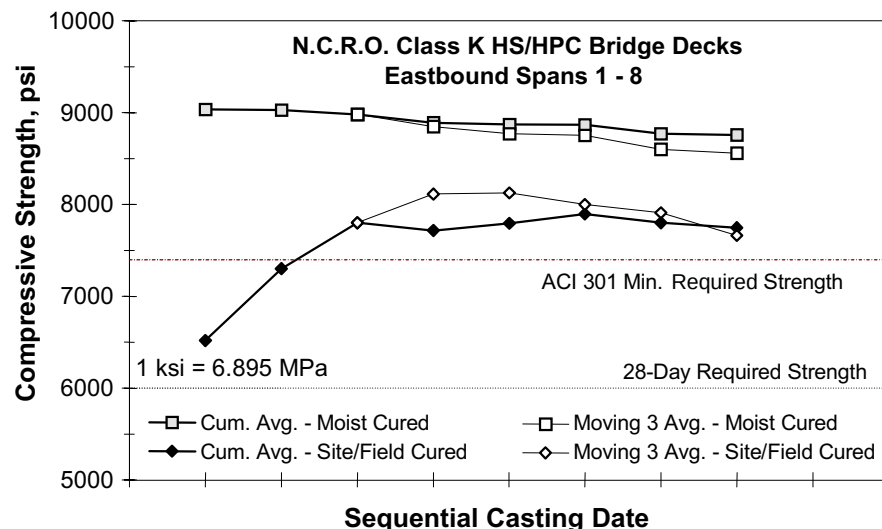


Figure 7.15: Compressive Strength (28-days) Cumulative Average and Moving Average for the Class K HS/HPC CIP Decks

The variations between site/field-cured and ASTM moist-cured specimens were quite dramatic. The site/field-cured specimens tested from 5.5 to 17.2 MPa (800 to 2,500 psi) less than the ASTM moist-cured cylinders on average. The 98 percent confidence interval value was 56.5 MPa (8,196 psi) with a zero percent probability of a failing test. The cumulative standard deviation and moving standard deviation for the substructure elements are presented in Figure 7.16. The ACI Committee 214 concrete performance classification for the ASTM moist-cured specimens was in the “excellent” range based on the standard deviation.

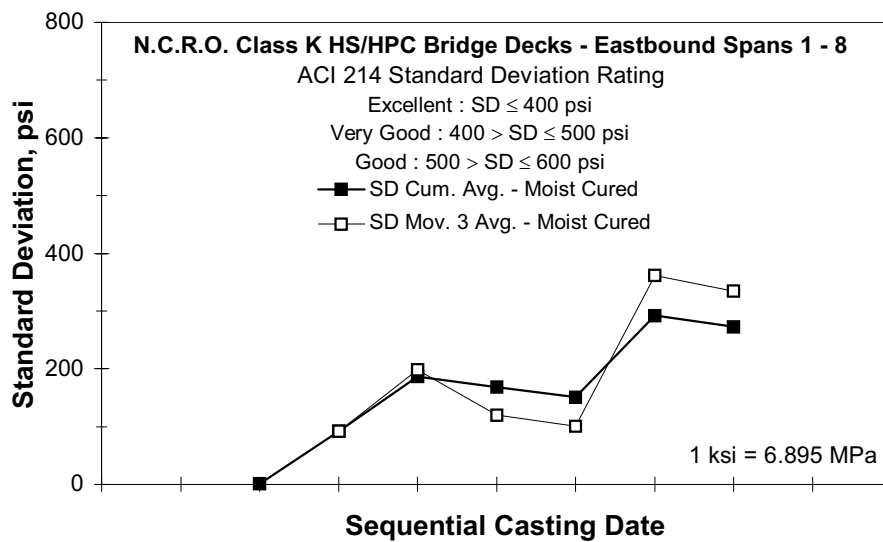


Figure 7.16: Cumulative and Moving Standard Deviation for the Class K HS/HPC CIP Decks

The testing proficiency and the within test coefficient of variation for the mix design is illustrated in Figure 7.17. The ASTM moist-cured specimens had an ACI Committee 214 laboratory testing proficiency in the “excellent” range based on the coefficient of variation. The within test coefficient of variation was very consistent for this mix design.

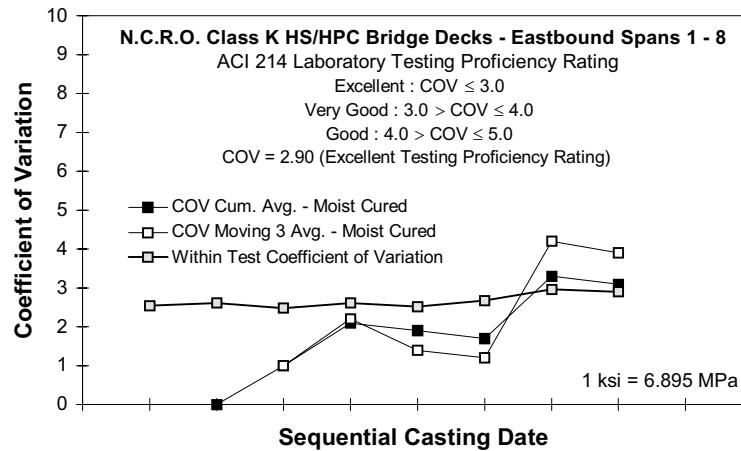


Figure 7.17: Within Test Coefficient of Variation and Testing Proficiency for the Class K HS/HPC CIP Decks

7.4.2 North Concho River Overpass Class S Modified HPC CIP Decks

Figure 7.18 illustrates the cumulative average and moving average compressive strength at 28-days for the Class S Modified HPC westbound decks. The ASTM moist-cured cylinders exceeded ACI Committee 301 required average compressive strength criteria of 35.9 MPa (5,200 psi) based on the mix design standard deviation and design strength. The site/field-cured specimens tested approximately 5.5 MPa (800 psi) less than the ASTM moist-cured cylinders.

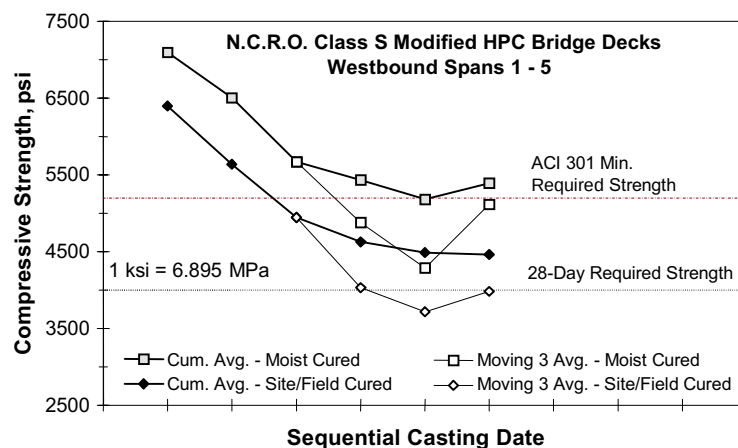


Figure 7.18: Compressive Strength (28-days) Cumulative Average and Moving Average for the Class S Modified HPC CIP Decks

The 98 percent confidence interval value was 37.2 MPa (5,390 psi) with a 14.0 percent probability of a failing test. The cumulative standard deviation and moving standard deviation for the substructure elements are presented in Figure 7.19.

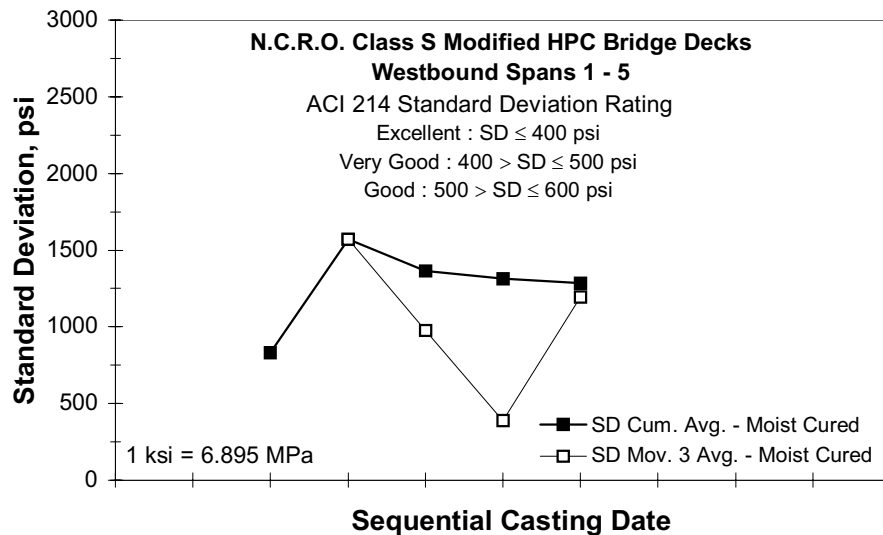


Figure 7.19: Cumulative and Moving Standard Deviation for the Class S Modified HPC CIP Decks

The ASTM moist-cured specimens had an ACI Committee 214 concrete performance classification in the “poor” range based on the standard deviation. The testing proficiency and the within test coefficient of variation for the mix design is illustrated in Figure 7.20. The ASTM moist-cured specimens had an ACI Committee 214 laboratory testing proficiency in the “very good” range based on the coefficient of variation. The within test coefficient of variation was very consistent for this mix design.

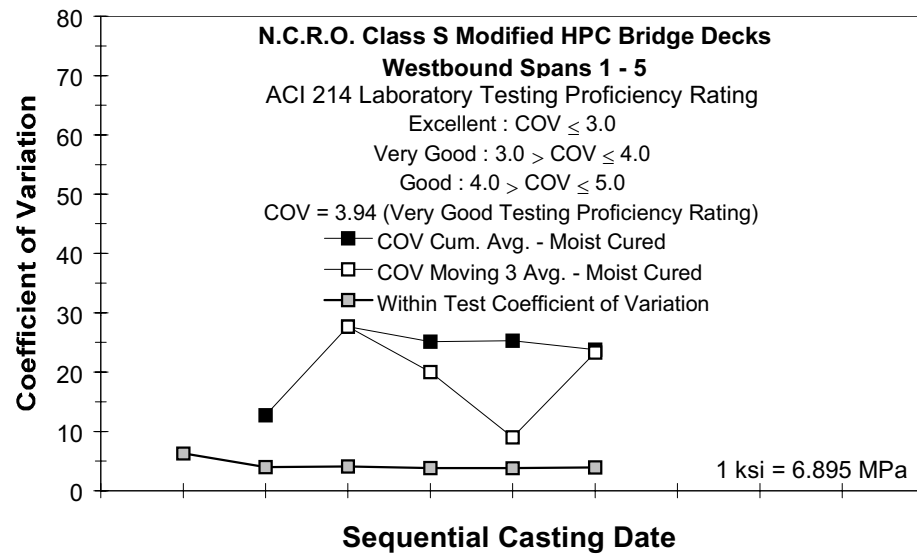


Figure 7.20: Within Test Coefficient of Variation and Testing Proficiency for the Class S Modified HPC CIP Decks

7.3.3 Comparison of North Concho River Overpass CIP Decks

For comparison purposes, the statistical analyses for the two HPC mix designs are summarized in Table 7.3. The HS/HPC had excellent overall performance while the Class S Modified HPC had what may be categorized as fair overall performance. The Class S Modified decks were cast during cold weather conditions that impacted the natural hydration curve of the mix design and affected the strength gain characteristics of the concrete. This would account for the low margin of extra performance and high probability of a failing test.

**Table 7.3: Summary of Statistical Analysis for North Concho River
Overpass Cast-in-Place HPC and HS/HPC Decks (28-Day)**

Statistical Criteria	Class K HS/HPC	Class S Modified HPC
Required 28-day Design Strength (psi)	6,000	4,000
Casting Dates Sampled	8	6
Total Test Specimens	24	18
Average Compressive Strength (psi)	8,757	5,390
Minimum Required Avg. Strength (psi)	7,400	5,200
Margin of Extra Performance (psi)	+1,357	+190
Cumulative Avg. Standard Deviation (psi)	273	1283
Concrete Performance (ACI 214)	Excellent	Poor
Within Test Coefficient of Variation	2.90	3.94
Average Range of Test (psi)	431	360
Laboratory Testing Proficiency (ACI 214)	Excellent	Very Good
98% Confidence Interval Value ❶ (psi)	8,196	2,753
Confidence Interval Range ❷ (psi)	8,615-8,890	4,722-6,058
Probability of a Failing Test	0%	14.01%
❶ Includes ACI 301 correction factor. ❷ Confidence Interval Range is for raw data without correction factor. Results are presented for ASTM moist-cured Specimens. 1 ksi = 1,000 psi = 6.895 MPa		

CHAPTER 8. EARLY-AGE CURING ASPECTS OF PRESTRESSED/PRECAST BEAMS

8.1 Introduction

A commercially available match-curing system was utilized during the production of the precast/prestressed beams. One reason for using this system was to more closely monitor the concrete properties within the members. This was required not only as a secondary quality control measure to verify specification requirements, but also to investigate the time-dependent performance of the HPC precast beams as part of the structural performance-related investigation. A secondary purpose of the system was to investigate the use of match-curing technology as a QC/QA tool in the precast industry. The use of match-curing technology is a direct application of the concrete maturity theory. Concrete maturity theory uses an empirical base to relate the time-temperature profile of a particular mix design to its strength at early ages and has been shown to be an excellent indication of the actual concrete strength development both at early and later ages. An in-depth perspective and description of the maturity method is presented in Chapter Nine as part of the investigation of its use in cast-in-place HPC bridge decks.

The goal of match-curing technology is to produce quality control specimens that closely represent actual concrete in the member by monitoring and matching the temperature-time profile of the concrete in the member to the quality control specimens. Thermocouples were placed at selected locations in the precast member in which monitoring and match curing were desired. Access of the thermocouple from the precast bed line to the data acquisition system (DAS) was provided through an underground duct system installed by the fabricator. The duct system was installed for easier access, improved system reliability, and quicker thermocouple installation. The thermocouple access at the bed line is illustrated in Figure 8.1. Through the use of a data acquisition system, the temperature-time profile for the actual member was monitored at 6-minute intervals. This system was located in an enclosed building located adjacent to the precasting beds. The data acquisition system and interface box contained eight controllable channels and six uncontrollable channels to monitor temperature-time profiles. Each controllable channel was capable of controlling the temperatures of six match-curing molds. Match-curing molds, as illustrated in Figure 8.2, were used to produce the quality control specimens.

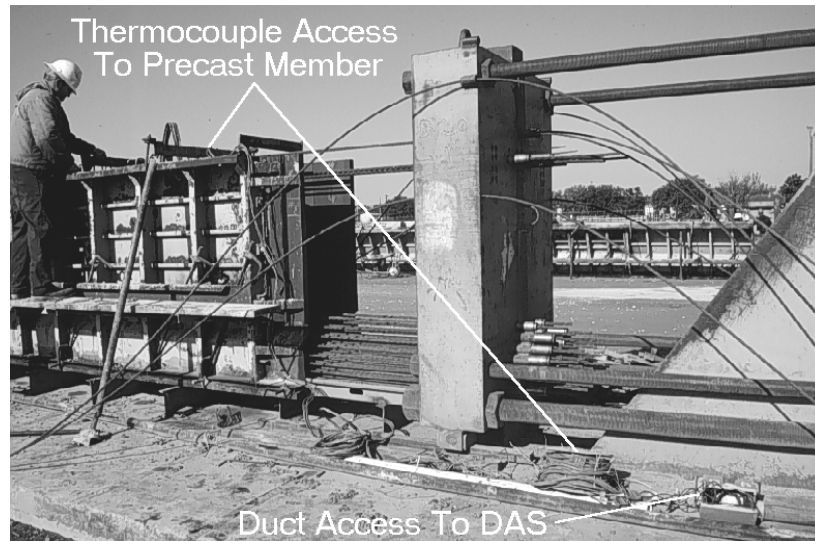


Figure 8.1: Shielded Thermocouple Wire Access from the Precast Member to the Data Acquisition System

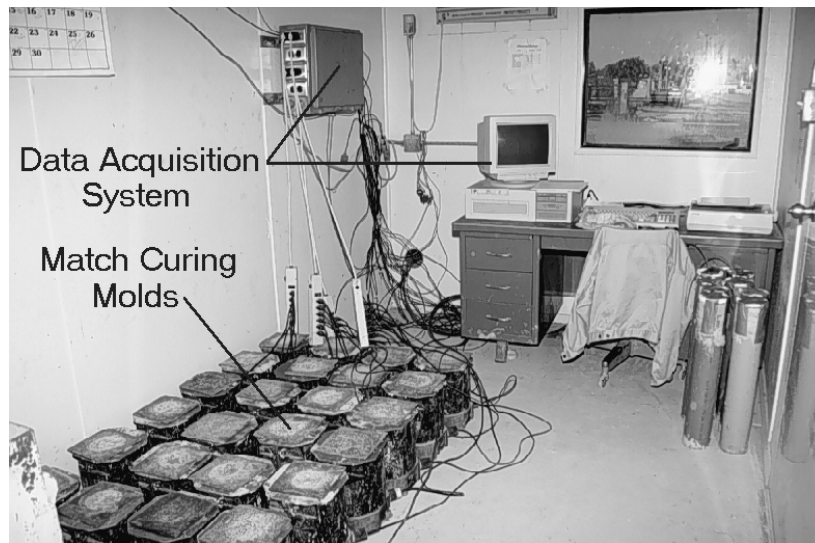


Figure 8.2: Climate-Controlled Room Incorporating Match-Curing Data Acquisition System and Match-Curing Molds

The steel molds were standard 100-mm dia. x 200-mm long (4-in. dia. x 8-in. long) cylinders and were surrounded by an internal heating coil that was insulated by a protective material. The match-curing steel molds were located in a room where the temperature of the environment could be controlled (Figure 8.2) because the match-curing system could not cool the specimens, only heat them. The room was maintained at a constant temperature of 21.1 ± 1.7 °C (70 ± 3 °F), so that the

temperature profile of the match-cured specimens would not *lag* behind the actual temperature-time profile of the concrete in the precast member. The concept of the researchers was to use an improved method of quality control to better estimate/verify concrete properties in the members without decreasing plant productivity. Due to the high design-strength requirements and performance-based specification system, it was determined that match-curing technology would provide the best indication of the mechanical and material properties of the concrete in the precast members. The method was successfully used from a QC/QA standpoint without decreasing plant productivity and is discussed in further detail in Section 8.4 of this report.

8.1.1 Louetta Road Overpass U-beam Instrumentation

The temperature-time development of the concrete was monitored at four critical locations on the Louetta Road Overpass U-beams for each casting date monitored by the researchers. To determine these critical locations, thermocouples were stacked at 127-mm (5-in.) intervals vertically in the member for the first beam monitored.

The maximum peak hydration temperatures occurred in the two end block locations (square and skewed) 914.4 mm (36 in.) vertically from the bottom of the member. End blocks were required for the U-beams to satisfy support requirements on the skewed abutments and precast pier caps. The maximum peak hydration temperature, non-end block location, occurred at mid-depth of the upper flange location while the minimum peak hydration temperature occurred in the web at mid-depth. These minimum and maximum peak hydration temperature locations for the U-beams are illustrated in Figure 8.3. A fully instrumented U-beam with thermocouples after form removal is illustrated in Figure 8.4. It may be noted that the thermocouple for the upper flange and web locations, was located a minimum of 6.1 m (20 ft.) from the end block locations. This location was selected to avoid any influence on the flange and web hydration curves induced by the end block.

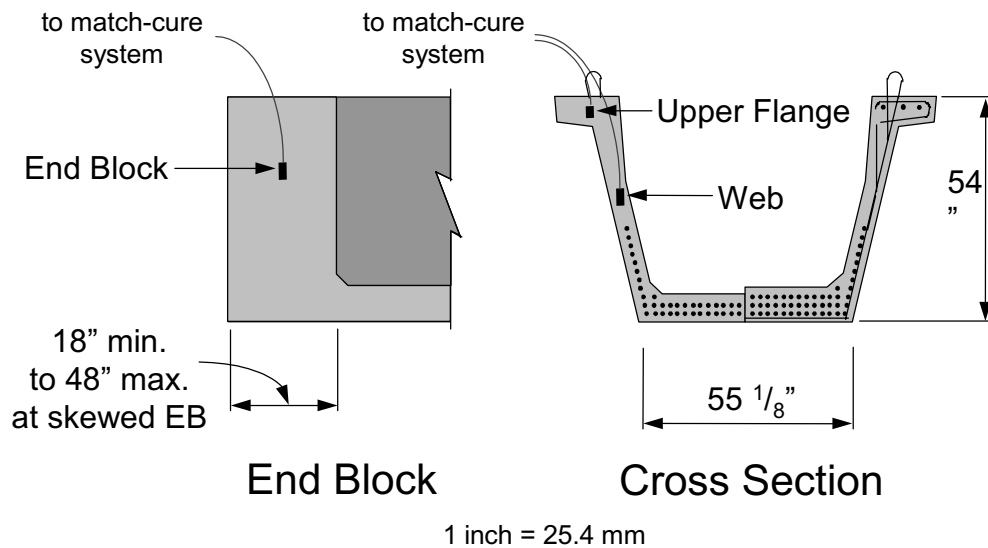


Figure 8.3: Schematic Representation of U-beam Thermocouple Locations

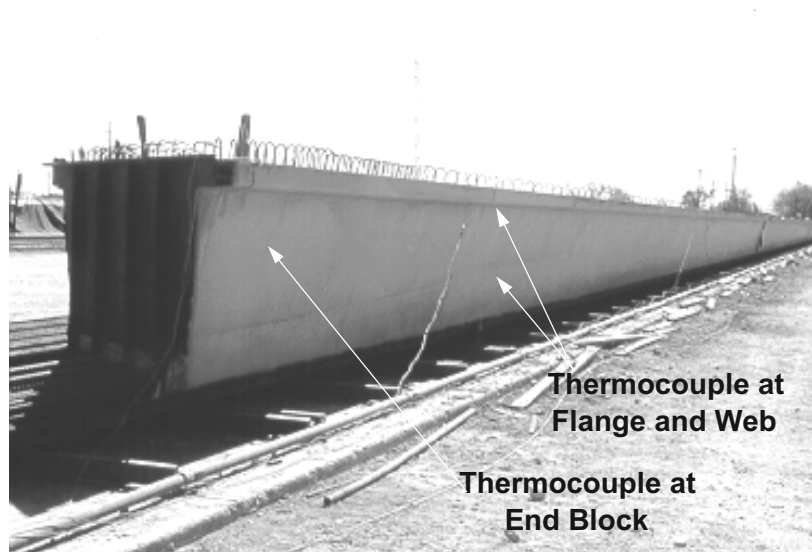


Figure 8.4: Instrumented U-beam after Removal of Formwork

8.1.2 North Concho River Overpass AASHTO Type IV Beam Instrumentation

The temperature-time development of the concrete was monitored at three critical locations on the North Concho River Overpass AASHTO Type IV precast beams for each casting date when monitoring was done by the researchers. To determine these critical locations, thermocouples were stacked at 127-mm (5-in.) intervals vertically in the member for the first casting date. The maximum

peak hydration temperatures occurred in the square end block location 914.4-mm (36-in.) vertically from the base of the member.

The end blocks for the AASHTO Type IV beams were required as part of the two-stage stressing operation selected by the fabricator as a result of the member redesign. The end block, nondraped prestressing strands, and post-tensioning ducts are illustrated in Figure 8.5. The maximum peak hydration temperature, non-end block location, occurred in the upper flange location 1168.4-mm (46-in.) vertically from the base of the member. The minimum peak hydration temperature for the locations monitored occurred in the web at mid-depth. These minimum and maximum peak hydration temperature locations for the AASHTO Type IV beams are illustrated in Figure 8.6.



Figure 8.5: North Concho River Overpass AASHTO Type IV Beam

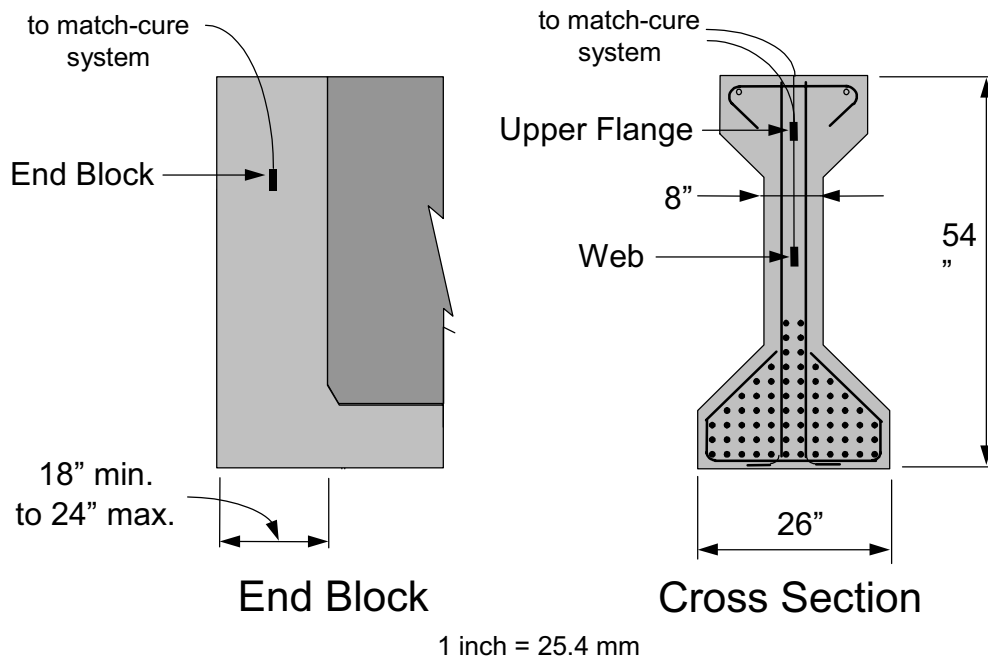


Figure 8.6: Schematic Representation of AASHTO Type IV Thermocouple Locations

8.2 Influence of Hydration Temperature and Temperature Rise on Mechanical Property Development of HPC Precast/Prestressed Beams

The impact of hydration temperature on the compressive strength, elastic modulus, and splitting tensile strength was investigated for both the Louetta Road Overpass U-beams and North Concho River Overpass AASHTO Type IV Beams. The following subsections discuss the influence of concrete temperature during hydration on both the early and long-term property development.

8.2.1 Compressive Strength Development

The compressive strength development was monitored both at early and later ages. Figure 8.7 illustrates the early-age compressive strength versus the maximum hydration temperature for all of the casting dates and curing locations monitored.

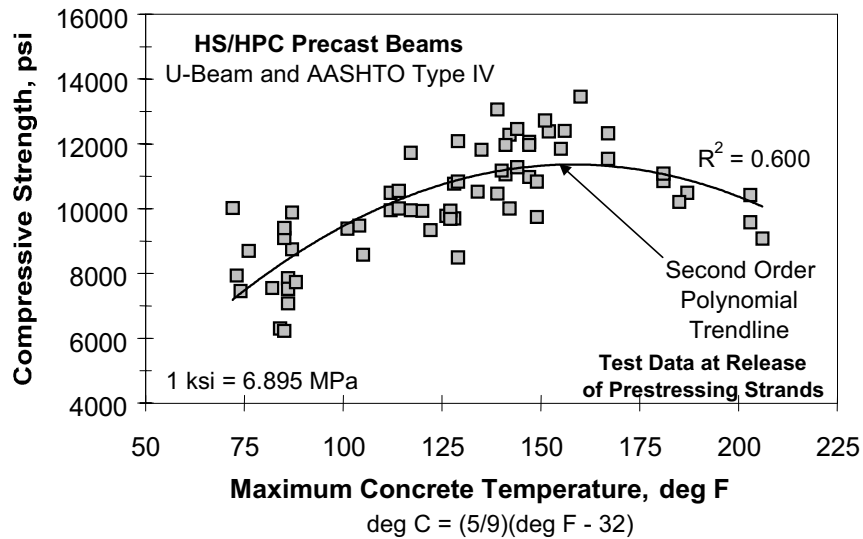


Figure 8.7: Early-Age (at Release) Compressive Strength versus Maximum Concrete Temperature during Hydration

A second order polynomial trendline best represented the scatter of data at release of prestressing. The general trend indicates an improvement in compressive strength up to a hydration temperature of approximately 82.2 °C (180 °F), beyond which compressive strength degrades. Specimens, which were subjected to hydration temperatures above 82.2 °C (180 °F), experienced more severe cracking on the microstructure level. The increased microcracking precipitated the propagation of cracking in the transition zones and, thereby, resulted in failure of the cylinder at a lower compressive strength value. Section 8.3.3 discusses the impact of hydration temperatures in terms of microstructure cracking.

Figure 8.8 illustrates the early-age compressive strength versus the temperature rise of the concrete. When investigating the influence of temperature in terms of the temperature rise of the concrete, the impact of concrete placement temperature and ambient conditions is minimized.

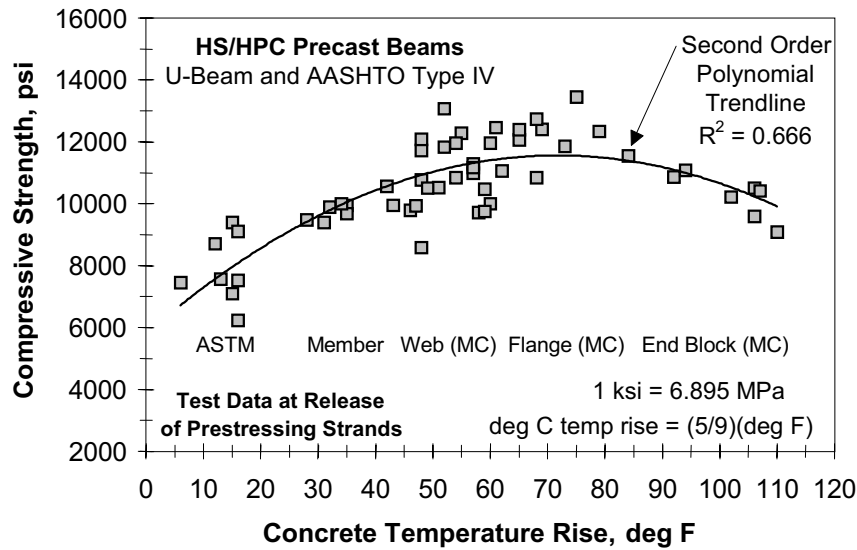


Figure 8.8: Early-Age (at Release) Compressive Strength versus Concrete Temperature Rise after Placement

The ASTM moist-cured cylinders exhibited the lowest temperature rise, from 3.3 °C to 8.9 °C (6 °F to 16 °F), owing to the controlled curing environment and high SA/V ratio of the cylinders. The member-cured cylinders experienced the second lowest temperature rise, generally in the 16.7 °C to 27.8 °C (30 °F to 50 °F) temperature range. The end block locations exhibited the highest temperature rise followed by the flange and web locations. The end block locations for the U-beams experienced a temperature rise between 51.1 °C and 61.1 °C (92 °F and 110 °F), while the AASHTO Type IVs varied between 27.2 °C and 46.7 °C (49 °F and 84 °F). The maximum heat generation for the U-beam and AASHTO Type IV end blocks was 11.14 and 8.55 °F/100 lb of cementitious material per cubic yard of concrete, respectively. Tables A.1 and A.2 in Appendix A summarize the peak temperatures and heat generation for each precast member casting date. The general trend for the temperature rise of the concrete indicates an improvement in compressive strength up to a temperature rise of approximately 38.9 °C (70 °F). Beyond this temperature rise, a decrease in the release strength is exhibited. As noted previously, this may be attributed to increased microcracking resulting in failure of the cylinder at a lower compressive strength value. The only locations that experienced temperature rises above this level were the end block locations, primarily on the U-beams. Owing to the higher SA/V ratio of the AASHTO Type IVs, “burn-out” of the mix design or excessive temperature development during casting did not appear to be as large a concern.

Figure 8.9 illustrates the later-age compressive strength versus the maximum hydration temperature for all of the casting dates and curing locations monitored. A second order polynomial trendline best represented the scatter of data at 56 days. The general trend indicates a linear decrease in compressive strength with an increase in maximum hydration temperature at later ages.

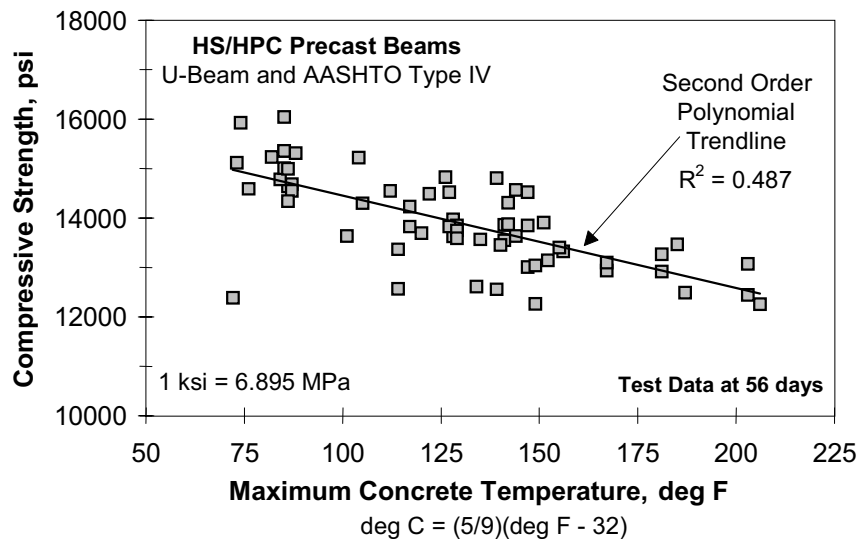


Figure 8.9: Later-Age (56-Day) Compressive Strength versus Maximum Concrete Temperature during Hydration

Figure 8.10 illustrates the later-age compressive strength versus the temperature rise of the concrete. A second order polynomial trendline best represented the scatter of data at 56 days. Consistent with the maximum hydration temperature, the general trend indicates a decrease in compressive strength with an increase in maximum hydration temperature at later ages. The following subsections examine the influence of curing location or regime on the compressive strength development for the two precast beam sections selected for use in the bridges.

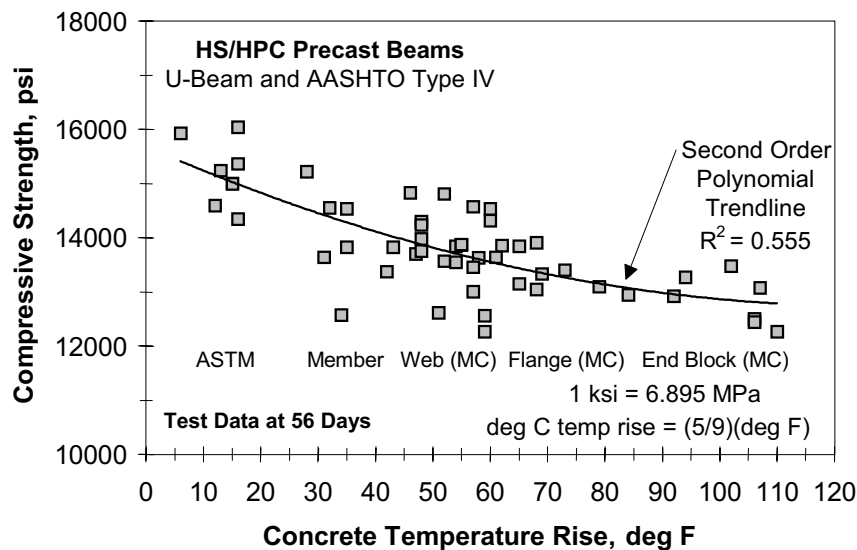


Figure 8.10: Later-Age (56-Day) Compressive Strength versus Concrete Temperature Rise after Placement

8.2.1.1 Louetta Road Overpass Precast/Prestressed U-beam. Casting of the Louetta Road Overpass U-beams was monitored on eight casting dates. Figure 8.11 illustrates a representative hydration temperature profile during the first 24 hours for a U-beam cast on February 15, 1996. On this casting date, the skewed end blocks attained a maximum hydration temperature of 86.1 °C (187 °F) approximately 16 hours after placement of concrete. Figure 8.12 illustrates this skewed end block where the highest hydration temperature occurred. The 457-mm (18-in.) thick regular end blocks attained a maximum hydration temperature of 77.8 °C (172 °F) approximately 21 hours after placement of concrete. The upper flange attained a maximum hydration temperature of 59.4 °C (139 °F), while the web attained the lowest maximum hydration temperature of 56.7 °C (134 °F). The variation between the highest and lowest maximum hydration temperature for the match-cured locations was 29.4 °C (53 °F). These hydration temperatures were primarily influenced by the ambient placement temperatures, the initial concrete temperature at placement, and the SA/V ratio or mass of concrete at the location investigated (Figure 8.29).

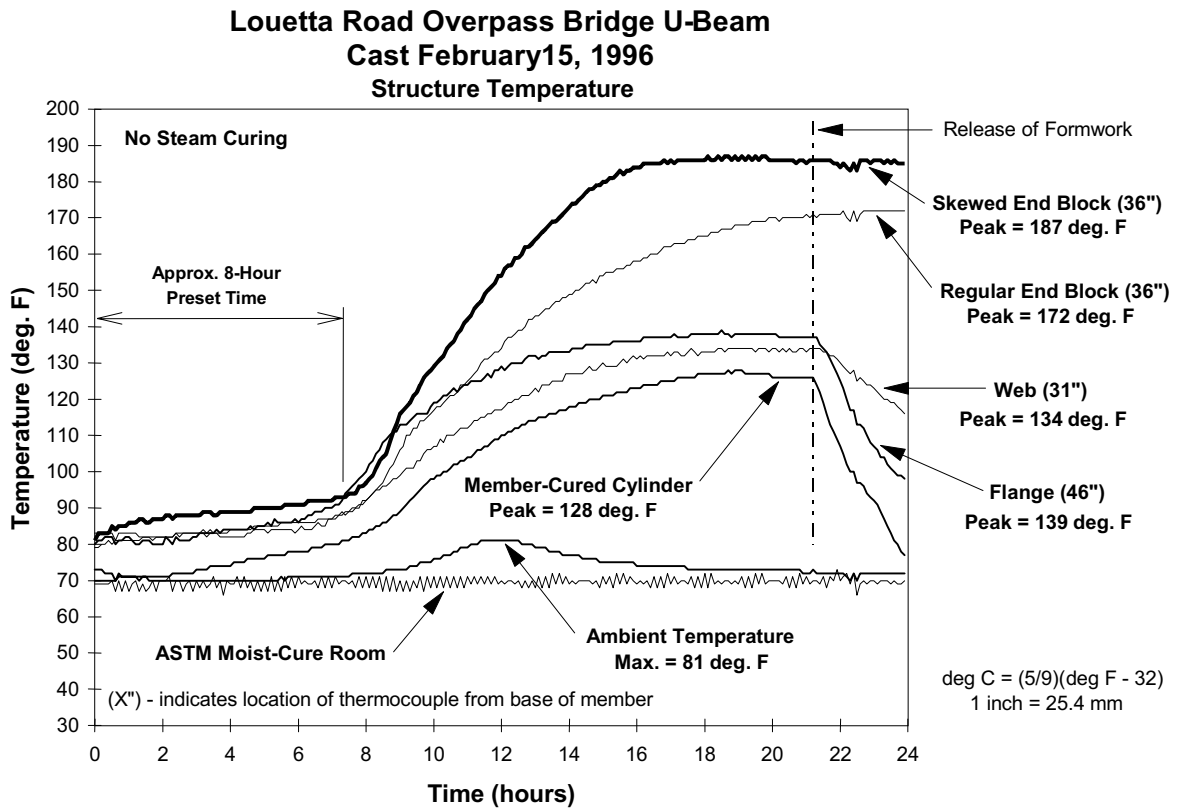


Figure 8.11: Hydration Temperature Profile for U-beam Cast on February 15, 1996

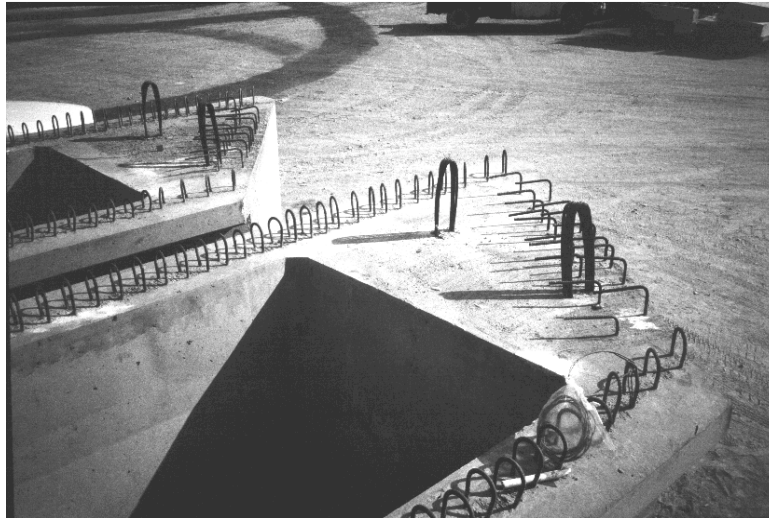


Figure 8.12: Skewed End Block Location for U-beam

The member-cured cylinders attained a maximum hydration temperature of 53.3 °C (128 °F), significantly lower than the hydration temperature of the concrete within the member. These hydration temperatures significantly impacted the compressive strength of the concrete both at early and later-age. Figure 8.13 illustrates the compression strength at release (24 hours) and 56 days for the moist-cured (ASTM), member-cured (MC), and match-cured cylinders. The member-cured cylinders are placed next to the formwork prior to release of the strands as described previously. These cylinders are the standard TxDOT quality control specimen used to determine when the prestressing stands may be released. ASTM moist-cured cylinders are placed in a 21 °C (70 °F) environment for a 24-hour period followed by standard ASTM moist curing.

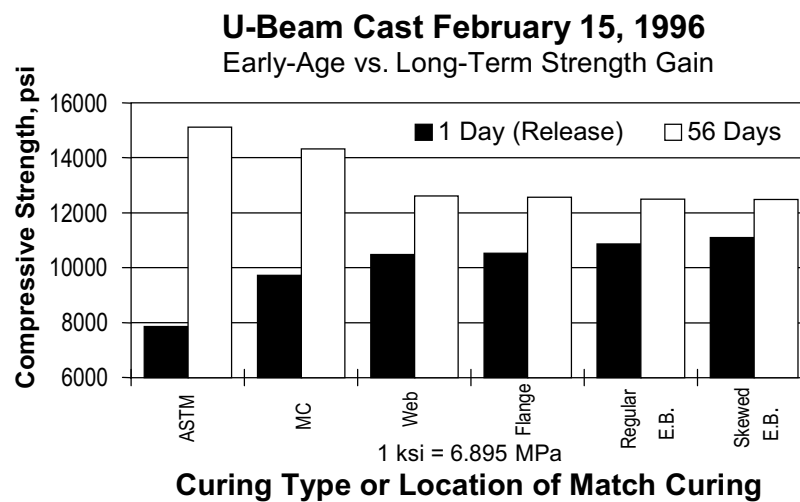


Figure 8.13: Compressive Strength Gain versus Curing Regime or Location for U-beam Cast February 15, 1996

Clearly, the curing condition that attained the highest early temperature displayed the highest early strength, but the lowest later-age (56 days) strength. Note how both the ASTM moist-cured and member-cured cylinders underestimated the strength of the concrete at release, but overestimated the concrete strength at later ages. On this particular casting date, the member-cured cylinders underestimated the strength of the concrete in the member at release by as much as 7.8 percent. The ASTM moist-cured specimens overestimated the concrete strength at 56 days by as much as 17.1 percent. Therefore, the strength of the member is typically underestimated at release, but

overestimated long-term if ASTM moist-cured or member-cured cylinders are used for design strength verification. Hydration profiles for all casting dates are presented in Appendix A.

8.2.1.2 North Concho River Overpass Precast/Prestressed AASHTO Type IV Beam.

Casting of the Louetta Road Overpass U-beams was monitored on twelve casting dates. Figure 8.14 illustrates a representative hydration temperature profile during the first 29 hours for a U-beam cast on February 19, 1997. On this casting date, the square end blocks 914.4, respectively (36 in.) from the base of the member attained the maximum hydration temperature of 63.9 °C (147 °F) approximately 17 hours after placement of concrete. A maximum non end block location hydration temperature of 60.6 °C (141 °F) was attained at the upper flange. The lower flange attained the lowest maximum hydration temperature of 53.9 °C (129 °F). This was a mere 1.1 °C (2 °F) lower than the web location. The variation between the highest and lowest maximum hydration temperatures for the match-cured locations was 10 °C (18 °F), significantly less than for the U-beam shape. The member-cured cylinders attained a maximum hydration temperature of 29.4 °C (85 °F), significantly lower than the hydration temperature of the concrete within the member. It is clear that the higher SA/V ratio of the AASHTO Type IVs did not generate nearly the amount of heat that the U-beam did.

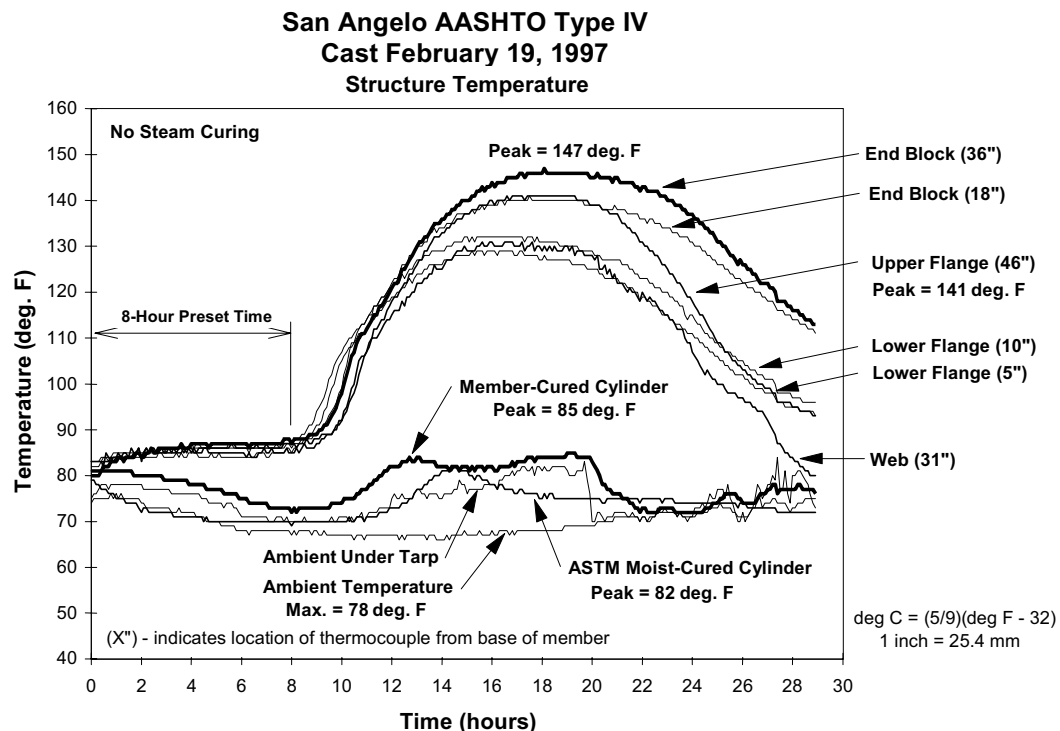


Figure 8.14: Hydration Temperature Profile for AASHTO Type IV Beam Cast on February 19, 1997

Figure 8.15 illustrates the compression strength at release (24 hours) and 56 days for the moist-cured (ASTM), member-cured (MC), and match-cured cylinders. Similar to the U-beam results, the curing condition that attained the highest early temperature displayed the highest early strength. Because the maximum hydration temperatures for the match-cured locations did not vary as dramatically as the U-beam, only 10 °C (18 °F), the variation in the later-age (56 days) strength development for the match-cured specimens was minimal (Figure 8.15). However, in both cases the member-cured cylinders clearly underestimated the strength at release.

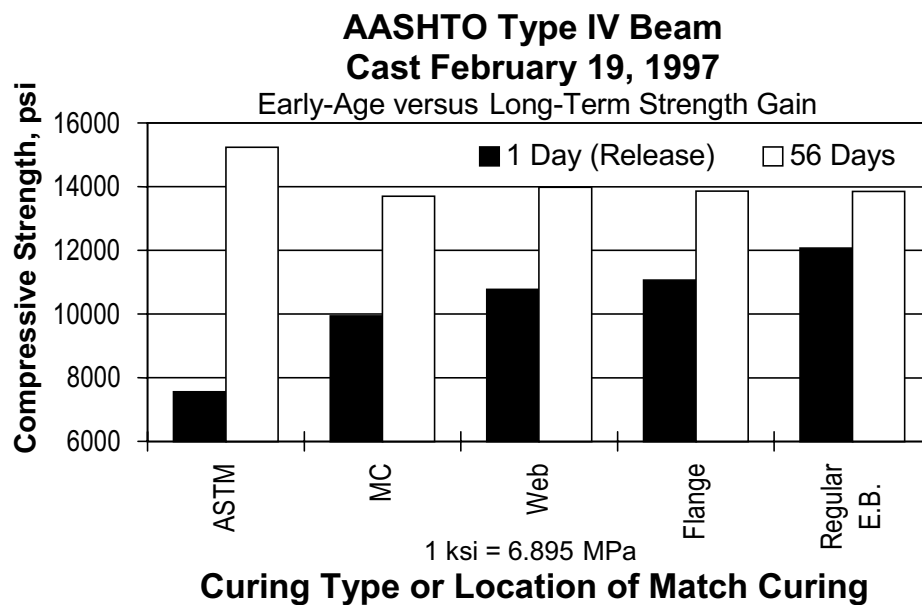


Figure 8.15: Compressive Strength Gain versus Curing Regime or Location for AASHTO Type IV Beam Cast on February 19, 1997

Based on the results of the monitoring for both the U-beam and AASHTO Type IVs, the member-cured cylinders used to determine release of the prestressing strands underestimated the compressive strength of the member by as much as 27.9 percent (19.1 percent on average) when compared to the match-cured cylinders. Using the match-cured system would result in increased plant productivity by allowing plant personnel to release prestressing strands and strip forms sooner. The ASTM moist-cured cylinders used to verify the design strength of the member at 56 days overestimated the compressive strength of the member by as much as 15.9 percent (9.8 percent on average) when compared to the match-cured cylinders. Using the match-cured system results in a more accurate estimate of the actual concrete mechanical properties within the member. This is

particularly important for designers of HPC components in which current code empirical equations are based on a function of the compressive strength of the concrete.

8.2.2 Elastic Modulus Development

The elastic modulus development was monitored both at early and later ages. Figure 8.16 illustrates the early-age elastic modulus versus the maximum hydration temperature for all of the casting dates and curing locations monitored. A second order polynomial trendline best represented the scatter of data at release of prestressing. The general trend indicates an improvement in compressive strength up to a hydration temperature of approximately 76.7 °C (170 °F), beyond which the elastic modulus appears to degrade. In terms of temperature rise, the early-age elastic modulus appears to degrade beyond a temperature rise of 38.8 °C (70 °F), consistent with the compressive strength.

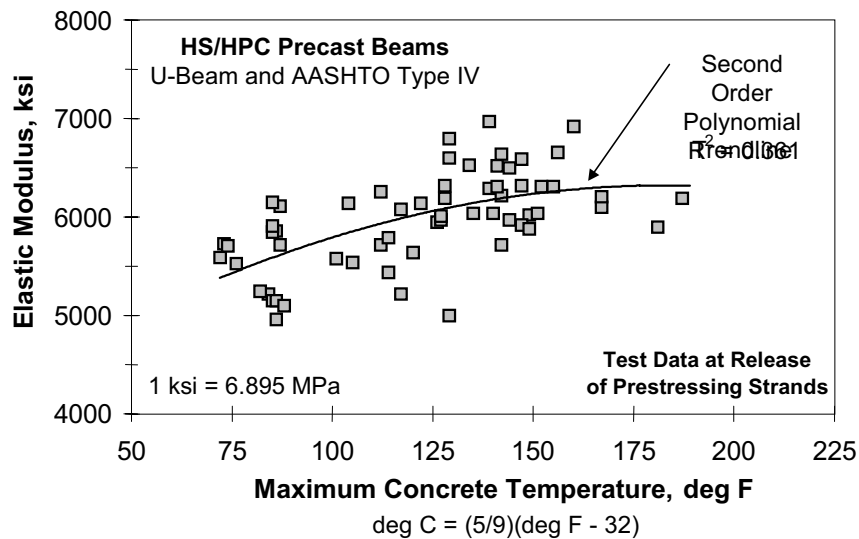


Figure 8.16: Early-Age (at Release) Elastic Modulus versus Maximum Concrete Temperature during Hydration

Figure 8.17 illustrates the later-age elastic modulus versus the maximum hydration temperature for all of the casting dates and curing locations monitored. A logarithmic trendline best represented the scatter of data at 56 days. The general trend indicates a logarithmic decrease in compressive strength with an increase in maximum hydration temperature at later ages.

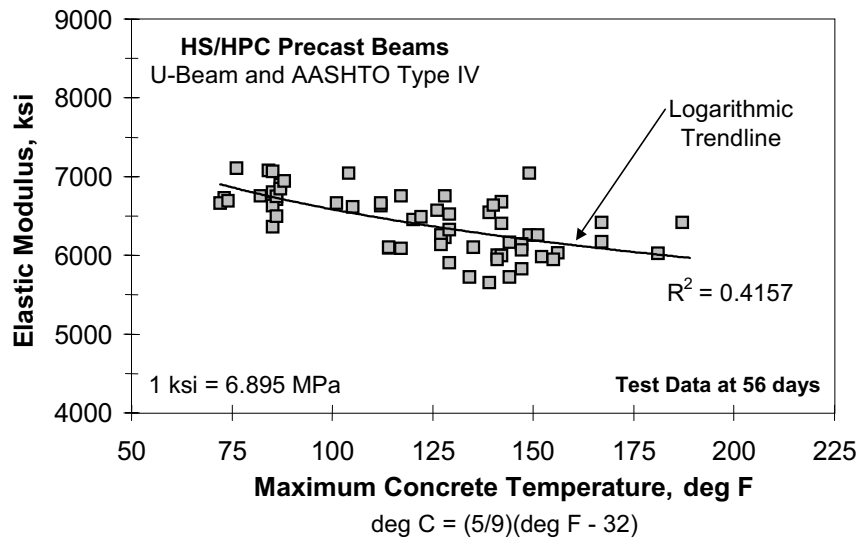


Figure 8.17: Later-Age (56-Day) Elastic Modulus versus Maximum Concrete Temperature during Hydration

8.2.3 Splitting Tensile Strength Development

The splitting tensile strength development was monitored at later ages. Figure 8.18 illustrates the later-age splitting tensile strength versus the maximum hydration temperature monitored for all of the casting dates and curing locations. A second order polynomial trendline best represented the scatter of data at later ages. Due to the wide variability of the splitting tensile strength test and the limited amount of data presented, it is difficult to formulate any conclusions. However, it does appear that the splitting tensile strength is independent of hydration temperatures. At very high temperatures where excessive cracking occurs on the microstructure level, a decrease in splitting tensile strength would be anticipated. Limited data is presented above 82.2 °C (180 °F) to confirm this suspicion.

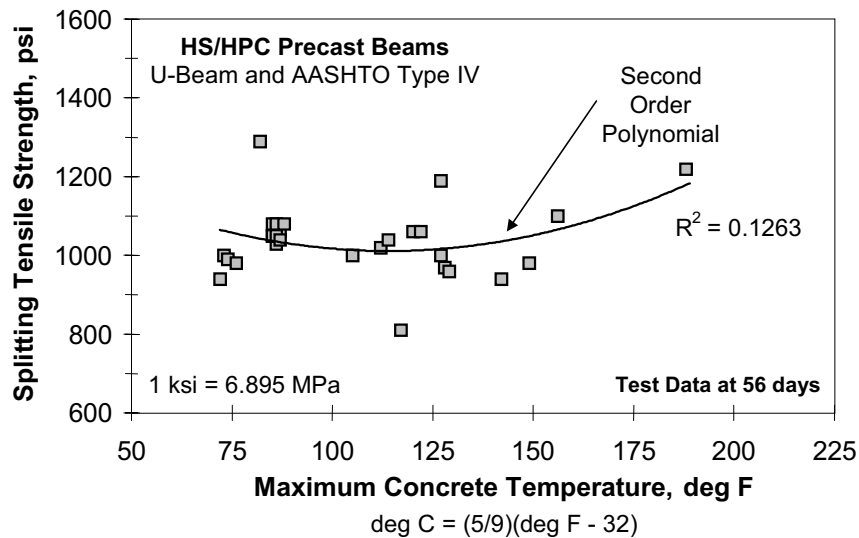


Figure 8.18: Later-Age (56-Day) Splitting Tensile Strength versus Maximum Concrete Temperature during Hydration

Throughout this discussion, the impact of temperature development on mechanical properties and the importance of controlling temperature development during casting of HS/HPC members are clearly evident. Due to the high content of cementitious materials, temperature controls must be considered for most precast sections. Control of temperature development may be accomplished by utilizing pozzolanic materials or by traditional methods such as cooling stock piles, utilizing chilled mixing water, or casting when material and ambient temperatures are minimized (early morning). Whatever method is selected, a general recommendation not to exceed a maximum hydration temperature of 76.7 °C (170 °F) or conversely a temperature rise of 38.8 °C (70 °F) should be considered.

8.3 Influence of Hydration Temperature on the Permeability Performance of HPC Precast/Prestressed Beams

Precast/prestressed concrete has long had the reputation of being a very durable long-lasting concrete as evidenced by data presented on lower structural deficiency of P/C bridges in Chapter One. Much of this reputation has been attributed to the prestressing, which reduces any crack widths, thereby slowing the ingress of chlorides and other harmful chemicals. Furthermore, the durability performance has also been attributed to the higher quality concretes and the tighter quality control measures instituted by many precast plants. This investigation was undertaken to examine if the

higher concrete temperatures during hydration associated with the precast/prestressed concrete members influenced the permeability of the concrete and, thereby, the durability performance.

8.3.1 Rapid Chloride Permeability Test Results

Rapid chloride permeability (RCP) was monitored at 56 days for all casting dates for the precast U-beams and AASHTO Type IV beams. Figure 8.19 illustrates the RCP versus the maximum concrete temperature during hydration. It may be noted that the test results presented are for the same mix design and constituents. Therefore, it is unlikely that chloride binding would impact the general trend of the results presented in Figure 8.19 because the mixture characteristics are identical. It is clearly evident that higher concrete temperatures during hydration reduce the chloride permeability up to a threshold temperature of approximately 87.8 °C (190 °F). In this temperature range, the RCP dramatically increases. It may be noted that when simply examining the temperature rise of the concrete, no trends were observed in terms of permeability performance based on the data collected herein. Clearly, it was the maximum concrete temperature during hydration that affected the permeability performance rather than a specific temperature rise of the concrete.

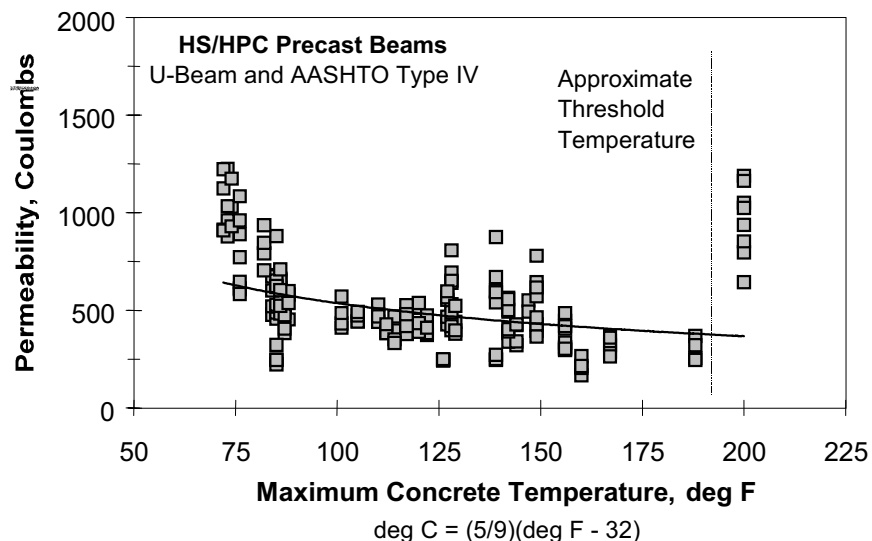


Figure 8.19: Rapid Chloride Permeability versus Maximum Concrete Temperature during Hydration

In order to determine what caused this behavior, the authors conducted a three-phase investigation. This included a low-level microscopic petrographic examination, a high-level

microscopic microstructure examination, and a BSE scanning electron microscope microstructure examination. Concrete from the same cylinders was used for each of these three investigations for consistency in examination. The results and conclusions of each of these investigations is presented in Sections 8.3.2 through 8.3.4.

8.3.2 Petrographic ExaminationLow Level Microscopic Analysis

The first phase was a low-level microscopic petrographic examination. This investigation was conducted by an independent private consultant [141] to examine four representative specimens from the same casting date. The only variation in the four specimens was the maximum hydration temperature encountered during casting (i.e., curing condition). The four specimens attained a maximum concrete temperature during hydration of 26.7 °C, 29.4 °C, 71.1 °C, and 93.3 °C (80 °F, 85 °F, 160 °F, and 200 °F). To conduct an unbiased investigation, no preconditions were specified to the consultant other than to optically inspect the specimens and report any discerning features. The cylinders were examined using methods given in ASTM C856, "Petrographic Examination of Hardened Concrete."

Table 8.1: Summary of Paste Characteristics for the Specimens Investigated [141]

Specimen Number	Curing Condition (Temp °F)	Color of Fractured Surface (Polished Surface)	Nature of Unhydrated Cementitious Material	Hardness of Cement Paste	Luster
1	ASTM Moist (80°F)	Tan (Tan)	Numerous unhydrated fly ash and cement particles. Some of the fly ash particles are etched.	Very Hard	Subvitreous
2	Match (85°F)	Lt Brn to Tan (Tan) ❶	Numerous unhydrated fly ash and cement particles. Some of the fly ash particles are etched.	Very Hard	Subvitreous to Vitreous
3	Member (160°F)	Tan (Tan)	Numerous unhydrated fly ash and cement particles. Some of the fly ash particles are etched.	Very Hard	Subvitreous
4	Match (200°F)	Light Brown (Light Brown) ❷	Numerous unhydrated fly ash and cement particles. Some of the fly ash particles are etched.	Very Hard	Subvitreous
Additional Comments or Notes: ❶ Appears to contain more calcium carbonate crystals than the other samples. ❷ The cement paste on both the polished surface and the fractured surfaces are visibly lighter in color than the other samples. Deg C=(5/9)(deg F-32) 1 inch = 25.4 mm					

To the unaided eye, the only distinguishing feature that was observed by the petrographer was a slightly lighter color of the paste in specimen 4 [maximum concrete temperature during hydration of 93.3 °C (200 °F)]. Close-up photomicrographs of the paste matrix did not show distinguishing features. Past powdered studies showed cement and fly ash particles, as well as normal hydrated products, but there were no distinguishing features that might relate to curing conditions or maximum hydration temperatures.

In summary, the petrographer noted that there were no outstanding optical or material features that appeared to be influenced by the maximum hydration temperature. The results of the low-level microscopic investigation did not indicate any reason for the variation in permeability, as both the paste matrix and material features optically appeared similar.

8.3.3 Microstructure Analysis—High-Level Microscopic Analysis

A higher level microscopic analysis was undertaken to more precisely optically examine the microstructure of the concrete specimens. As noted previously, the same concrete specimens were

investigated. Thin slices were prepared and examined at the TxDOT Materials and Tests Division in Austin, Texas. These slices were impregnated with epoxy and mounted for optical examination. The epoxy impregnated any void space or microcracks present on the thin slice. Figure 8.20 illustrates a mounted thin-sliced specimen that was prepared for microscopic examination. Table 8.2 summarizes the accumulated point-count data and the resulting modified point-count results for the thin-sliced specimens after they were inspected optically. Volume percentages were on the same order for each of the thin slices. Minor variations in the counts may be attributed to sample location and the surface area size of the thin slices.

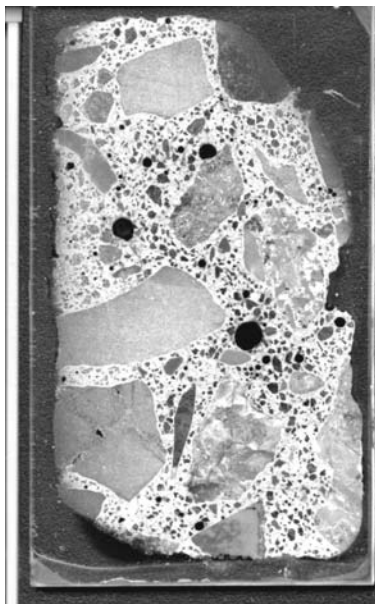
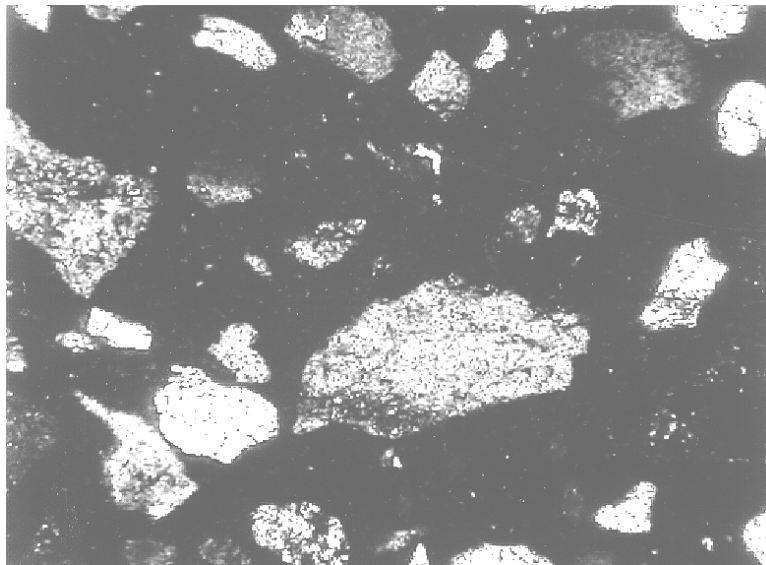


Figure 8.20: Mounted Thin-Sliced Specimen

Two types of imaging were used to inspect and analyze the microstructure: polarized imaging and ephifluorescences imaging. The polarized imaging reflects polarized light on the thin-sliced specimens so that the mineralogical characteristics of the aggregates can be identified. Geologists and material scientists primarily use this method to identify the mineral composition of the coarse and fine aggregates. Figure 8.21 illustrates a thin-sliced specimen subjected to polarized imaging.

Table 8.2: Point-Count Data and Material Volumes for Thin-Sliced Specimens

Curing Regime (Peak Hydration Temperature)	ASTM Moist (80°F)	Match (85°F)	Member (160°F)	Match (200°F)
Point-Count				
Paste	364	377	369	386
Fine Aggregate	327	286	252	268
Coarse Aggregate	461	506	544	517
Air Void	48	31	35	29
Paste Volume (%)	30.3	31.4	30.8	32.2
Fine Aggregate Volume (%)	27.4	23.8	21.0	22.3
Coarse Aggregate Volume (%)	38.4	42.2	45.3	43.1
Percent Entrained Air (%)	3.00	2.08	1.24	1.41
Spacing Factor	0.5125	0.6358	0.6459	0.6595
Deg C=(5/9)(deg F-32)				

**Figure 8.21: Thin-Sliced Specimen under Polarized Imaging**

The ephifluorescences imaging allows for the identification of all air voids and microcracks within the paste matrix. This was the primary imaging used to identify and quantitatively determine the microcrack widths and magnitude of the microcracks. Two methods were selected to quantitatively examine the magnitude of the microcracks. The first method examined the more severe microcrack widths and counted the digital pixels over the width at the same magnification for all specimens. This

was intended to provide a rough estimate of the variation of crack widths between the peak concrete (hydration) temperatures investigated. The second method involved a full count of void space minus actual air voids over the same surface area to quantitatively determine the magnitude of microcracks over the specimen surface. Figures 8.22, 8.23, and 8.24 illustrate the ephifluorescences imaging on thin slices of specimens at 200°F, 160°F, 85°F, and 80°F, respectively. Clearly, the specimen subjected to the highest concrete (hydration) temperature resulted in the most severe microcracking. The microcracks were located through the paste matrix and the transition zones.

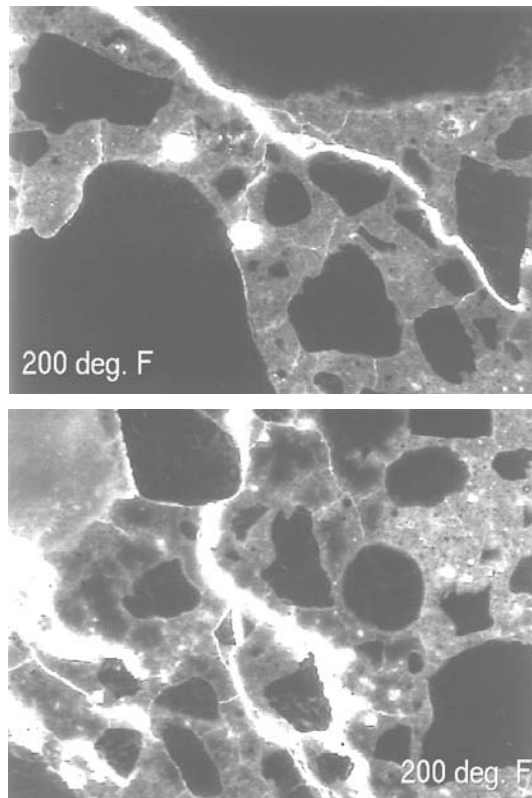


Figure 8.22: 200 deg. F Thin-Sliced Specimen under Ephifluorescences Imaging

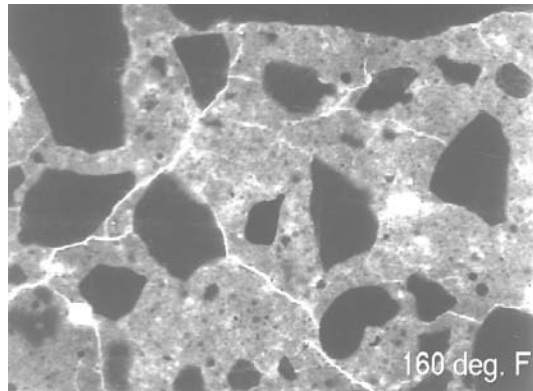


Figure 8.23: 160 deg. F Thin-Sliced Specimen under Ephifluorescences Imaging

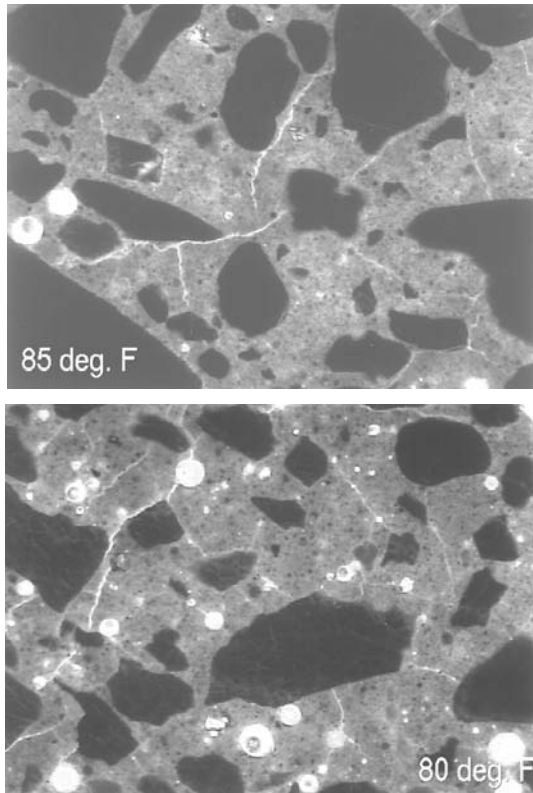


Figure 8.24: 80 & 85 deg. F Thin-Sliced Specimen under Ephifluorescences Imaging

Crack widths for the most severe cracks varied from 18 to 31 pixels wide. The 71.7 °C (160 °F) specimen did not exhibit microcracking nearly as severe as the 93.3 °C (200 °F) specimen. However, the microcracks visually appeared to be wider compared to the 26.7 and 29.4 °C (80 and 85 °F) specimens. The most severe crack widths measured from 11 to 13 pixels in width, while the 26.7 and 29.4 °C (80 and 85 °F) specimen crack widths were not wider than 4 or 5 pixels at any location.

The thin microcracks that are visible in all of the images are associated with drying and shrinkage and are representative of any concrete produced. The quantitative magnitude of microcracking is presented in Table 8.3. The point count confirmed that the higher concrete (hydration) temperatures resulted in a higher magnitude of cracking.

Table 8.3: Quantitative Magnitude of Microcracking

Curing Condition (Temp °F)	Total Point Count of Microcracking ❶	Maximum Crack Widths
ASTM Moist (80°F)	163	4 pixels
Match (85°F)	216	5 pixels
Member (160°F)	309	13 pixels
Match (200°F)	500	31 pixels
❶ Point count measured over the same surface area for each specimen and does not include any air voids.		

8.3.4 Microstructure Analysis—Scanning Electron Microscope Analysis

Finally, a high-level BSE SEM microstructure analysis was undertaken to examine the microstructure of the concrete specimens and the material composition at a higher resolution and power than the previous investigation. The thin slices were impregnated with an Epofix epoxy by Struers to fill any void spaces prior to examination and to further highlight the microcracks. Following this, the specimens were mounted and polished. The thin slices were 30 to 40 microns thick. The specimens were carbon coated prior to SEM examination to prevent damage to the impregnated epoxy under the BSE imaging. The SEM work was performed by staff members from the Geology and Civil Engineering Departments at Texas A&M University in College Station, Texas. Texas A&M University was selected because of its digital graphics capability. Existing hardware at UT consisted of analog technology with no digital graphics capability. Figures 8.25 through 8.27 illustrate the results of the BSE imaging. Results of the BSE imaging indicated more severe microcracking for the specimens subjected to the higher concrete (hydration) temperatures, consistent with the TxDOT thin-slice examination.

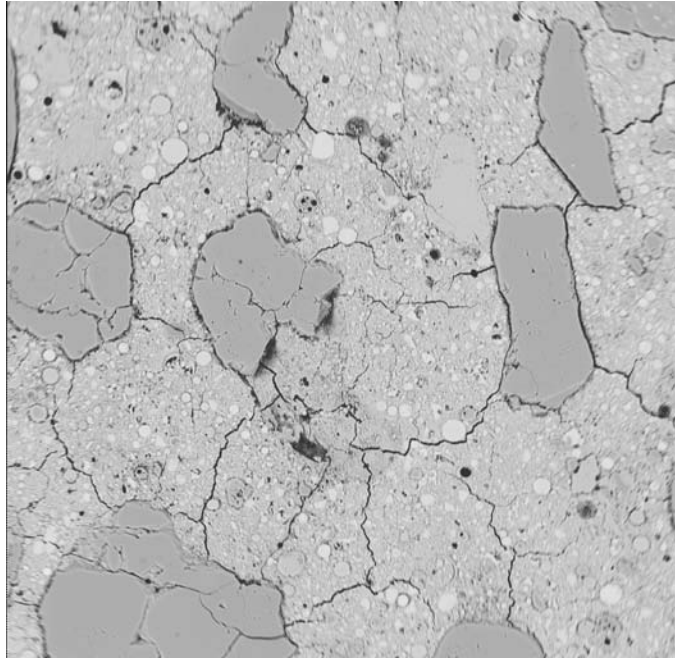


Figure 8.25: BSE Imaging for 200 deg. F Thin-Sliced Specimen

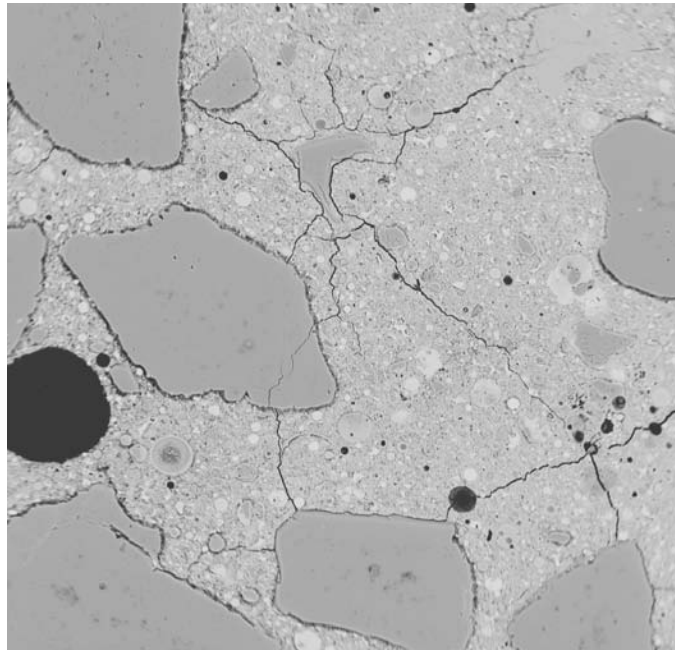


Figure 8.26: BSE Imaging for 160 deg. F Thin-Sliced Specimen

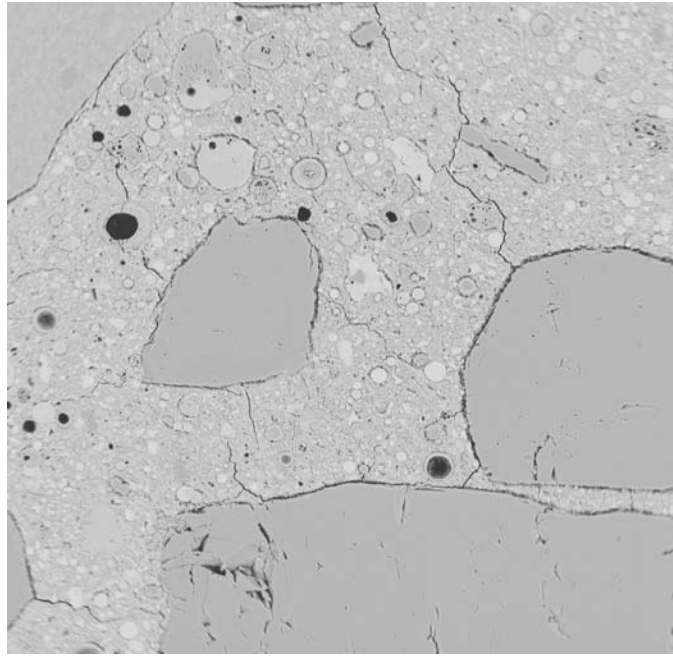


Figure 8.27: BSE Imaging for 85 deg. F Thin-Sliced Specimen

The transition zones and microcracks were much more detailed at a magnification of 100 times using this imaging as illustrated in the figures. This examination detailed the cracking that occurred within the transition zones, which was not visible at 30 times magnification. The severity of the transition zone and paste matrix microcracks appeared greatest for the 93.3 °C (200 °F) specimen (Figure 8.25). The 71.7°C (160 °F) specimen also indicated fairly severe microcracking in the transition zones (Figure 8.26). The 29.4°C (85 °F) specimen clearly indicated the least amount of microcracking (Figure 8.27).

It should be noted that microcracks are already present in the transition zones owing to drying and cooling, even before concrete is subjected to loads. Therefore, it is impossible to eliminate microcracking in the transition zones and more transition-zone cracking should be anticipated in specimens subjected to higher hydration temperatures. In general, the majority of transition-zone microcracking was located around the silicious fine aggregate material as illustrated in Figure 8.28.

Significantly less microcracking was noticeable around the Burnet dolomitic limestone coarse aggregate (Figure 8.28). This is consistent with previous research studies that report a chemical bonding in the transition zone between the hydrated cement paste particles and particles that are largely composed of limestone and dolomitic minerals. At these locations, it was difficult to distinguish the transition zone even at higher magnifications (Figure 8.28). Results from the 26.7 °C (80 °F)

specimen are not presented owing to incongruous results related to inconsistent thin-slice mill thickness and specimen preparation.

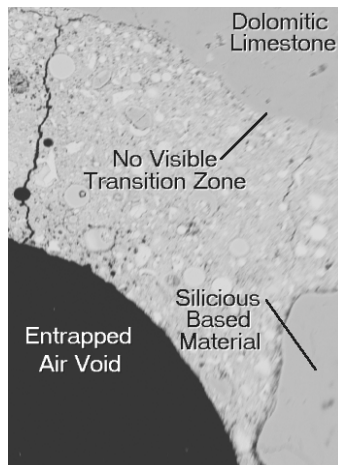


Figure 8.28: Aggregate - Paste Transition Zone

The higher concrete (hydration) temperatures clearly impacted cracking on the microstructure level more severely as illustrated in Sections 8.3.3 and 8.3.4 of this report. Based on the mechanical and material data collected within this investigation as well as the microstructure analysis, it is recommended that excessive hydration temperatures be avoided. To accomplish this, a maximum concrete temperature limitation during hydration in the temperature range of 71.1 to 82.2 °C (160 to 180 °F) is recommended as a specification requirement for future HPC precast components.

8.4 Use of Match-Curing Technology as a Quality Control/Quality Assurance Tool in the Precast Concrete Industry

Current QC/QA procedures within the precast industry produce quality control specimens that are subjected to very different curing temperatures than the actual concrete in the member. This is clearly evident for the HS/HPC precast beams discussed in Section 8.2. Variations in hydration and curing temperatures can dramatically affect the characteristics of the concrete. It has been widely documented that using the match-cured system will increase plant productivity by allowing plant personnel to release prestressing strands and strip forms sooner. On average, the match-cured cylinders [web location (lowest release strength)] satisfied the release strength requirements 2 hours sooner than the standard TxDOT QC/QA member-cured cylinders. Furthermore, using the match-cured system results in a more accurate estimate of the actual concrete mechanical properties within

the member. This is particularly important for designers of HPC components in which current code empirical equations are based on a function of the compressive strength of the concrete. The match-curing technology not only allows the plant to create a database on mechanical property performance, but provides the design engineer with more realistic mechanical properties to predict short and long-term structural performance.

One concern that was raised throughout the production of the U-beams and AASHTO Type IV beams was the type of inherent factor of safety that would exist in the event the match-curing system would be selected as the primary QC/QA tool for release and design strength verification. The two beam locations where concerns were the greatest were at the end block locations and along the bottom flanges where transfer and development stresses for the pretensioned strands and concrete fiber stresses would be the greatest. The two primary variables that directly impact the inherent safety factor of the match-curing system are the match-curing location selected and the time from when the release strength was satisfied (i.e., cylinder meets release strength requirements) to the time when the first strand was released.

First and foremost, it is always recommended that the most conservative location be selected for release strength verification when using a match-cured system. In the case of the U-beam and AASHTO Type IV beams, the location that consistently developed the lowest strength at release (lowest hydration temperature) was the web location at mid-height. Mass concrete or where higher hydration profiles are encountered should not be selected as match-curing locations. This would result in the least conservative QC/QA approach. In the case of the U-beam and AASHTO Type IV, an inherent safety factor can be determined based on the performance of the concrete and the fabrication process at the Texas Concrete Company in Victoria, Texas.

If the web is selected as the match-curing location from a QC/QA standpoint, and the lower flange and end blocks are the critical locations from a concrete compressive stress point of view (transfer and development of prestressing forces), there exists an inherent safety factor because the web is consistently lower in strength than the critical stress location (flange or end block) at release. Based on the results of the data collected within this investigation, the flange location was 4.7 percent higher in strength on average at release compared to the web location. The second inherent factor of safety is the time from which the cylinder is tested and release strength has been verified to the point at which the first prestressing strand is released. The fabricator never releases formwork until release strength has been verified, because it insulates the member, preventing heat loss, resulting in additional strength gain. Based on the fabrication of the HS/HPC beams, it required a minimum of 2 hours for the Texas Concrete Company to strip formwork for the U-beam or AASHTO Type IV before the crew was prepared to release any of the prestressing strands.

Based on the maturity curve and hydration profiles of the HS/HPC, the additional gain in strength of the concrete can be determined for this 2-hour period. This gain in strength was equivalent to 4.45 MPa (645 psi). Factoring in these two inherent factors of safety results in an overall safety factor of approximately 1.11 for the production of the U-beams and AASHTO Type IVs. Although this is a somewhat *crude* determination of the factor of safety specific to the HS/HPC mix design characteristics and the production timetable of the precast plant in Victoria, it does serve to illustrate the inherent safety factor that exists for the match-curing system.

For a more conservative approach, the specifications could specify a minimum time frame that would be required after strength is met for release of the strands. However, this approximate 10 percent factor of safety appeared adequate for the production of the HS/HPC beams because no longitudinal concrete splitting or any other signs of distress were noted prior to or after release. For precast concrete members that do not use a Type III cement, this safety factor would be less owing to the lower hydration temperatures generated and the slower gain in strength. Safety factors for members such as these should be investigated prior to approved use of the match-curing system.

The following are recommendations to be incorporated into the use of match-curing technology as a QC/QA tool in the precast industry to verify release strength requirements.

1. Have a minimum designated period of time (trial phase) in which the fabricator can become familiar with match-curing technology and the system. This could be treated similarly to a precertification requirement in the specifications and/or bid documents. The advantage or motivating factor of utilizing this system for the fabricator is increased plant productivity, thereby, an improved profit margin during production.
2. Specify the lower bound criteria for match-curing locations. Lower bound refers to the match-curing location that attains the lowest maximum temperature during hydration. This would result in the most conservative approach and the highest inherent safety factor for using the match-curing system. For both the U-beam and AASHTO Type IV, this location would be the web at mid-height and centerline of the beam. For shapes other than the U-beam or a standard AASHTO I shape, it is recommended that a trial shape be cast and instrumented with stacked thermocouples to determine this location. Figure 8.29 illustrates how the SA/V ratio of the section or match-curing location selected can dramatically influence the compressive strength results at release.

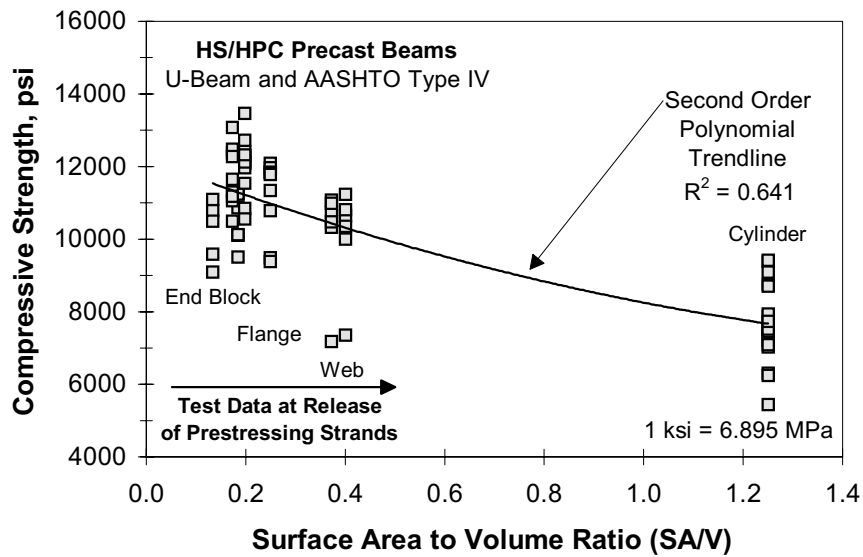


Figure 8.29: Compressive Strength at Release versus Surface Area to Volume Ratio for the Precast Beams

3. Provide a minimum of two member-cured cylinders as a back-up mechanism to verify release strength in the case of a data acquisition (computer) glitch or thermocouple malfunction.
4. Use heavy-duty thermocouple wire in lieu of standard thermocouple wire. This will result in greater system reliability.
5. Use a minimum of two thermocouples at different locations (if possible) in the event that an internal vibrator cuts a thermocouple wire. It may be helpful to mark the location of the thermocouple on the precast forms as a precautionary method for the fabrication crew. Four percent of the thermocouples were accidentally cut or malfunctioned during casting of the U-beams and AASHTO Type IV beams. The majority of these incidents occurred during the casting of the U-beams prior to utilization of a duct system (Figure 8.1).
6. If possible, the fabricator should use a duct system for his/her thermocouple wiring system. Keep external exposure of the thermocouples to a minimum. Access to the internal thermocouple location from the base of the member is recommended if at all possible. This reduces the possibility that the thermocouple wire will be cut accidentally by equipment or fabrication personnel.

7. A climate-controlled room is required for curing the match-curing specimens because the specimens can only heat the cylinders and not cool them. Provisions should be made for a climate-controlled environment.
8. Match-curing technology is more advantageous for mix designs with high cementitious contents and members with smaller SA/V ratios (mass concrete) in which high heat of hydration is generated. For conventional concrete mixes with larger SA/V ratios, member-cured cylinders provide an adequate representation of concrete within the member from a QC/QA standpoint. In this case, match-curing technology may not result in improved productivity owing to the minor variation between member-cured and match-cured cylinders.
9. The use of match-curing cylinders to verify later-age design requirement strengths for the precast members is not particularly advantageous because the existing code methodology addresses design strength variability through the use of phi factors and load factors to ensure a low probability of low-strength concrete. However, later-age match-cured mechanical property results may be valuable to the design engineer for HPC members to better estimate the short and long-term performance aspects of the members on future projects.

CHAPTER 9. USE OF MATURITY METHOD FOR CAST-IN-PLACE HPC BRIDGE DECKS

9.1 Introduction

The maturity method is commonly used to predict strength. This method uses an empirical base to relate the time-temperature profile of a particular mix design to its strength at early ages. Previous research conducted at The University of Texas at Austin by Cetin [15] illustrated how the maturity method underestimated the early strength (release strengths) and overestimated the long-term strength for accelerated heat-cured specimens such as the prestressed/precast U-beams. The high cementitious content of the HPC mix and the mass sections of concrete created a “burn-out” effect (high initial strength gain followed by limited long-term strength gain) as described by Cetin. He concluded that the traditional maturity-strength relationship based on standard-cured cylinders does not appear to be representative of strengths of concretes subjected to accelerated heat curing. The objective of this study was to investigate the applicability of the maturity method for use in HS/HPC and HPC bridge decks. While these mix designs have, like the precast beams discussed herein, a large amount of cementitious materials, the decks generally have a surface-area-to-volume (SA/V) ratio twice as large as their counterpart beams. Because of the larger SA/V, the decks can dissipate heat much more quickly, thereby limiting the development of high internal concrete temperatures. Accordingly, the maturity method may be applicable to HPC bridge decks. This study looks at the applicability of the maturity method for cast-in-place concrete bridge decks.

9.1.1 Description of the Maturity Method

The hydration of cement is affected by both duration and temperature. Much research has been conducted on cement hydration. The concept of maturity or the combined effect of time and temperature on the strength development of concrete dates back to the 1950s.

In the most general sense, maturity is defined as the extent of cement hydration in a cement mixer. Given the set of materials and their proportions, maturity is a function of the concrete's temperature history at a given time, provided there is sufficient moisture present for hydration to proceed. This method requires the determination of a strength-maturity relationship in the laboratory for the mix proportions and constituent materials to be used effectively in the field. Once the strength-maturity relationship for a particular mix design has been developed in the laboratory, the field application only requires monitoring of the temperature history of the in situ concrete. This maturity

index obtained in the field can be translated into strength using the strength-maturity relationship developed in the laboratory as a nondestructive strength evaluation technique.

Also known as the *temperature-time factor*, the Nurse-Saul equation was one of the earliest functions presented to define the maturity index. It is still widely used today and is defined in Equation 9.1.

$$M = \sum (T - T_o) \Delta t \quad (9.1)$$

The *temperature-time factor* is the area between the time-temperature profile and the datum temperature. Figure 9.1 presents a schematic representation of the temperature-time factor. It assumes a linear relationship between the rate constant and the temperature.

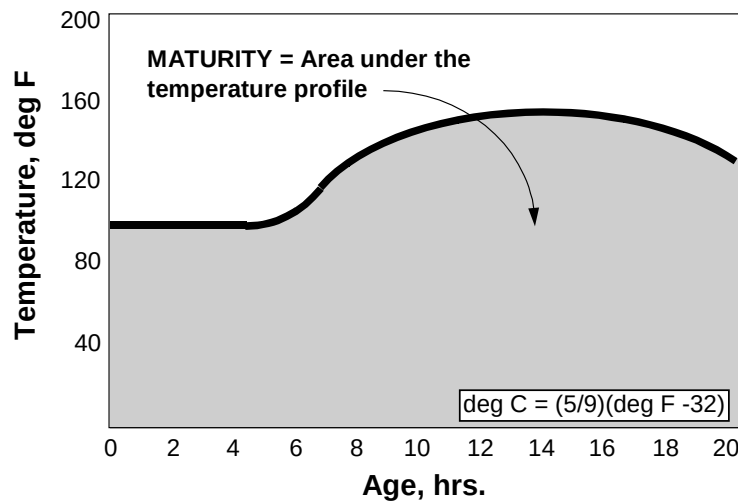


Figure 9.1: Schematic Representation for the Definition of the Maturity Method

Various datum temperatures have been proposed since this relationship was developed. Saul proposed -10.5°C (13.1°F) as the datum temperature, while in North America -10°C (14°F) is most often used [108]. Malhotra [109] presented several constraints for the maturity method to be valid as follows:

1. Initial concrete temperature is between 15.6 and 26.7°C (60 and 80°F).
2. Concrete is maintained in an environment that permits further hydration.
3. Maturity is represented by curing at normal temperatures between the ages of 3 and 28 days.

In 1956, Plowman [110] proposed Equation 9.2 for the strength-maturity relationship based on the strength developments of concretes cured at constant temperatures. The maturity-strength

function is stated in terms of the strength obtained at 28 days of curing at 17.8 °C (64 °F) using a datum temperature of -17.2 °C (11 °F). The variables a and b are constants related linearly to the strength at any age. Plowman stated that this function was valid for times up to at least 1 year and curing temperatures below 38 °C (100 °F). In addition, he proposed values for the constants a and b based on different strength classes of concrete.

$$S = a + b(\log M) \quad (9.2)$$

Another maturity index, called the *equivalent age*, (Equation 9.3) was proposed as an alternative to the temperature-time factor. Equivalent age is defined as the age of the concrete cured at a constant temperature that yields the same relative strength gain as under the actual temperature history. The concept assumes an exponential relationship between the rate constant and temperature based on the Arrhenius equation.

$$t_e = \sum \exp\left(-\frac{E}{R}\left(\frac{1}{T} - \frac{1}{T_s}\right)\right) \quad (9.3)$$

where:

- E = activation energy
- R = gas constant
- T_s = standard temperature in Kelvin

In 1983, Carino [111] proposed a three-parameter model for the relative strength gain-maturity relation as illustrated in Equation 9.4.

$$S / S_u = (M - M_o) / \left[\frac{1}{A} + (M - M_o)\right] \quad (9.4)$$

where:

- S_u = limiting strength as maturity approaches infinity
- M_o = maturity value when rapid strength gain begins
- A = initial slope of the relative strength-maturity curve

9.1.2 Applicability of the Maturity Method for High Performance Concrete

The applicability of the maturity method for HPC was investigated by Carino [108] in 1992. Mortar cubes with w/cm ratios of 0.29 and 0.36 were cured under water at constant temperatures of 7, 23, and 40 °C (44.6, 73.4, and 104°F). The rate constant versus the curing temperature was

represented by an exponential equation and five different models were used to represent the relative strength versus maturity relationship. The following was concluded:

1. The maturity method was applicable to determine the combined effect of time and temperature on the strength development of mixtures with low w/cm ratios.
2. For the mixtures investigated, the linear hyperbolic model appeared to be the most suitable one.
3. Results indicated that the limiting strengths of the low w/cm ratio mixtures did not decrease in a consistent manner, as in conventional concretes, by increasing the curing temperature.

Further investigation on the applicability of the maturity method to HPC was conducted by Cetin [15], as discussed in the introduction of this chapter. He concluded that the traditional maturity-strength relationship based on standard-cured cylinders does not appear to be representative of strengths of concretes subjected to accelerated heat curing for HPC.

9.1.3 Limitations of the Maturity Method

There are a number of limitations to the use of the maturity method for predicting concrete strengths. The following summarizes these limitations [2]:

1. The maturity functions that have been proposed do not take into account the effect of the humidity conditions during curing.
2. The various functions cannot be applied to mass concrete because the rate of heat loss from such structures is much less than that from normal specimens. Only ambient temperatures are considered, while the contributions from the heat of hydration are ignored.
3. The functions are not very useful at low maturities in which significant strength gain has not occurred.
4. The maturity concept is not useful if there are large temperature variations during the curing period because a low initial concrete temperature followed by normal curing will lead to higher concrete strengths than if the concrete had been cured at a normal temperature.
5. Strength is also affected by both the chemical composition and the fineness of the cement because these both affect the rate of hydration. The w/cm ratio may also influence the results.

6. Although there seems to be an initial correlation between maturity and strength for accelerated strength tests, the relationship breaks down when the accelerated cure specimens are subsequently cooled and moist cured.

9.2 Instrumentation of Cast-in-Place Bridge Decks

Six spans were instrumented to monitor concrete maturity development between the two high performance concrete bridge projects, as outlined in Table 9.1. These involved a total of five different concrete mix designs, including normal-strength concrete (NSC), high performance concrete (HPC), and high-strength/high performance concrete (HS/HPC). Three locations within the CIP decks were selected to investigate concrete maturity development throughout this investigation, as illustrated in Figure 9.2. An additional location for the Louetta Road Overpass bridge decks was selected between U-beams at the pier line owing to the more massive section of cast-in-place concrete. These locations were selected because it was anticipated that they would generate the critical temperature development within the deck (minimum and maximum) during hydration of the concrete.

Table 9.1: CIP Decks Instrumented for Concrete Maturity Monitoring

Bridge Project	Span Location	Casting Date	Concrete Mix Design Description
Louetta Road Overpass	Span 3 NB	10-31-96	Class S Modified HPC
Louetta Road Overpass	Span 2 SB	11-8-96	Class K HS/HPC
North Concho River Overpass	Span 1 WB	12-3-96	Class S Modified HPC
North Concho River Overpass	Span 6 WB	2-5-97	Class S NSC
North Concho River Overpass	Span 1 EB	6-12-97	Class K HS/HPC
North Concho River Overpass	Span 4 EB	7-23-97	Class K HS/HPC
NB = Northbound, SB = Southbound, WB = Westbound, EB = Eastbound NSC = Normal-Strength Concrete, HPC = High Performance Concrete, HS/HPC = High-Strength/High Performance Concrete			

The thermocouple located above the precast panel at the centerline of deck steel was expected to generate the lowest peak hydration temperature owing to the reduced mass of concrete (increased SA/V ratio) and heat dissipation provided by the precast panel. The thermocouple, which was centered in the more massive section of CIP concrete at the perimeter location, was expected to generate the highest peak hydration temperature owing to the large mass of concrete (lower SA/V ratio). The third thermocouple was located between panels to investigate the variation of heat dissipation through the precast beams. Figure 9.3 illustrates a fully instrumented deck with

thermocouples, prior to placement of concrete. Standard, commercially available, shielded thermocouple wire was selected for monitoring concrete temperature, as illustrated in Figure 9.4.

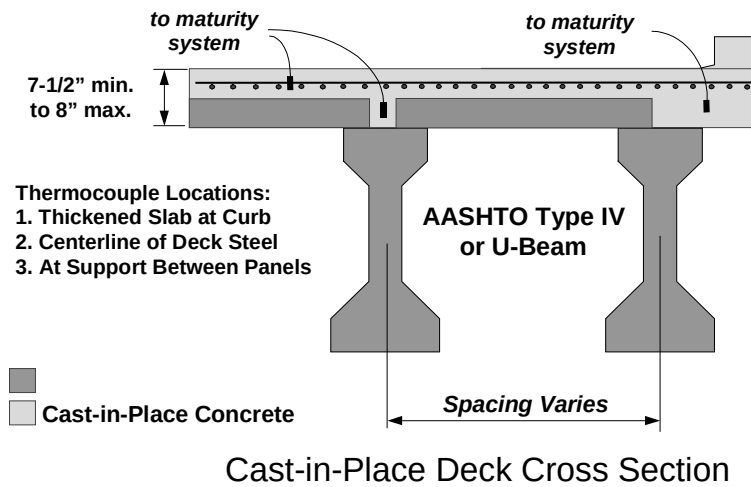


Figure 9.2: Schematic Representation of Cast-in-Place Deck Thermocouple Locations

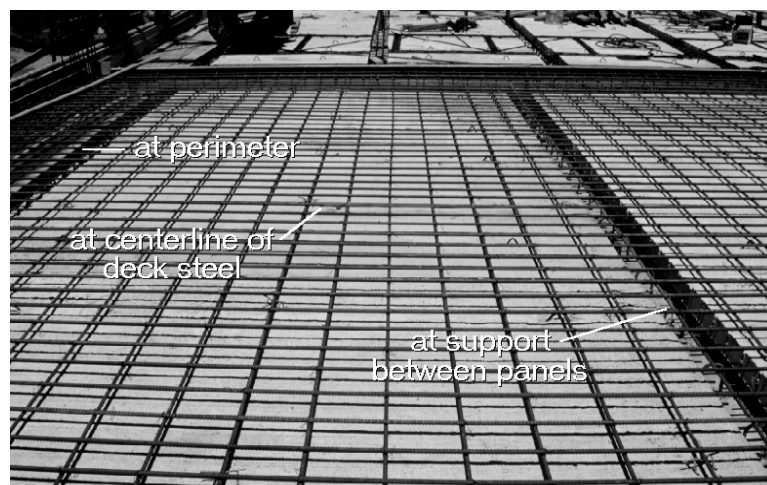


Figure 9.3: Cast-in-Place Deck Instrumentation

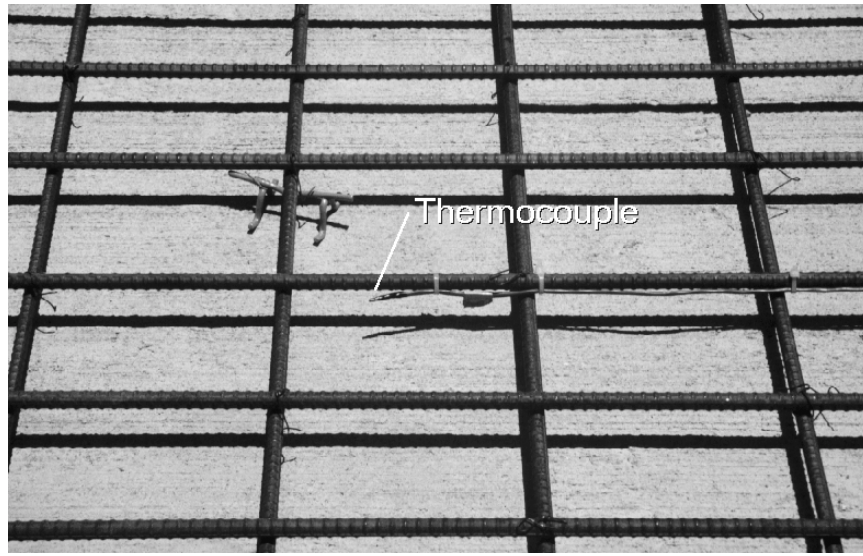


Figure 9.4: Shielded Thermocouple Prior to Placement of Deck Concrete

The thermocouple wire was attached to the data acquisition system at the abutment or pier cap, as shown in Figure 9.5. The researchers accessed the data acquisition system with the use of a “cherry picker” or extendible aluminum ladder, as illustrated in Figure 9.6. Concrete maturity was monitored by the commercially available data acquisition system described in Section 5.3.4.3 of this report.



Figure 9.5: Location of Data Acquisition System at Bridge Abutment



Figure 9.6: Access to Data Acquisition System

In order to compare the relative strength gain of the cast-in-place decks using the maturity method, all tests were conducted in accordance with ASTM C1074-93, *Standard Practice for Estimating Concrete Strength by the Maturity Method*. Laboratory mixes were duplicated in the field with identical mixture proportions and constituents to develop the natural maturity curve (strength versus temperature-time factor) for each of the five different mix designs involved in this investigation. Compressive strength was evaluated by averaging the results for three representative 100-mm x 200-mm (4-in. x 8-in.) cylinders at test ages of 1, 3, 7, 14, 28, and 56 days in accordance with ASTM test method C39-94. The test results for the laboratory mixes and the natural maturity curves are reported in Section 9.3, along with the test results for the concrete CIP bridge decks.

9.3 Test Results of Maturity Investigation

Table 9.2 summarizes the concrete maturity results for the five different mix designs that were selected for use in the CIP bridge decks. Specimens were moist cured upon stripping of the molds at 20–23 °C (68–73 °F) in accordance with ASTM C1074-93 until test age. The mix design constituents and proportions as well as the performance-related test results of these mix designs are reported in Chapter 6 of this report.

Table 9.2: Temperature-Time Factor, degree C-days for Field Mix Designs as Developed in the Laboratory

Time		Temperature-Time Factor, degree C-days				
		Louetta Road Overpass		North Concho River Overpass		
Days	Hours	Class S Modified	Class K	Class S	Class S Modified	Class K
1	24	498	746	556	533	588
3	72	1420	2122	1542	1616	567
7	168	3232	4900	3499	3510	3476
14	336	6489	9765	6976	6860	6963
28	672	13125	17950	13480	12880	13775
56	344	26397	33695	26867	25057	27877

The Class K mix designs generated the higher maturity values as anticipated owing to their increased hydration temperatures from the higher cementitious contents. The Class S mix design developed higher maturity values initially (1 day) when compared to the Class S Modified mix design (North Concho River Overpass) — a result of the higher relative cement content. Each of these mix designs was identical except that the Class S Modified mix design had 30 percent fly ash replacement. The fly ash replacement resulted in lower concrete temperatures during hydration and strength gain during the first 24 hours, but greater later-age strength gain after 48 to 72 hours.

9.3.1 Louetta Road Overpass Maturity Investigation

Two decks were instrumented at the Louetta Road Overpass to investigate the use of the maturity method for HPC and HS/HPC bridge decks. Figure 9.7 illustrates the compressive strength gain with time of the two bridge decks instrumented for the maturity investigation at the Louetta Road Overpass. The improved strength gain characteristics of the southbound deck may be attributed to the use of an HRWR and reduced w/cm ratio.

9.3.1.1 Louetta Road Overpass Northbound HPC CIP Deck. Figure 9.8 illustrates the concrete hydration temperature development at four locations within the deck upon placement of concrete for the northbound Class S Modified HPC deck. The more massive 190-mm (7.5-in.) thick sections of CIP concrete, including the thickened slab at the pier cap support and perimeter overhang for the guardrail, attained the highest peak concrete temperature during hydration as anticipated. The thin 102-mm (4-in.) CIP concrete deck above the precast panel attained the lowest peak concrete temperature during hydration as anticipated. It may be noted that concrete was placed during mild ambient conditions, as shown in Figure 9.8. Thus, the precast panels and U-beams were able to act as a “heat sink” to dissipate heat related to concrete hydration quite effectively. As discussed

previously, controlling or maintaining lower hydration temperatures during placement only improves the quality and later-age strength gain of the concrete.

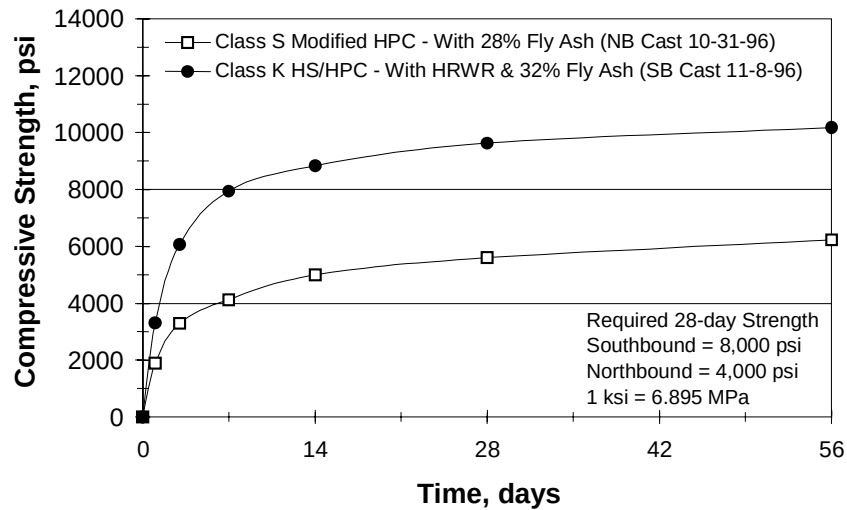


Figure 9.7: Compressive Strength Gain with Time of Louetta Road Overpass CIP Bridge Decks

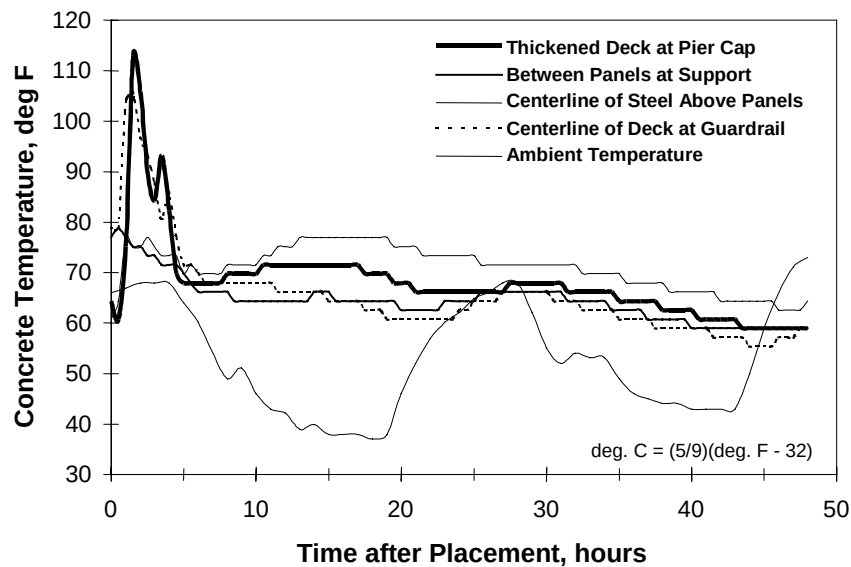


Figure 9.8: Concrete Hydration Temperature Development of Class S Modified HPC Louetta Road Overpass Northbound Span 3, Cast October 31, 1996

The concrete maturity results for the northbound CIP deck at 7, 14, and 28 days versus the natural maturity curve for the mix design are illustrated in Figure 9.9. Although the initial concrete temperatures varied within the deck during the first 24 hours after the placement (Figure 9.8), a very minor variation in maturity development was noted at the various locations. The maturity variation between the 28-day results from the bridge deck and the natural maturity curve were 3.4 percent on the conservative side. The results of the maturity method for the Class S Modified HPC mix design were within the ± 10 percent variation typically associated with the maturity method.

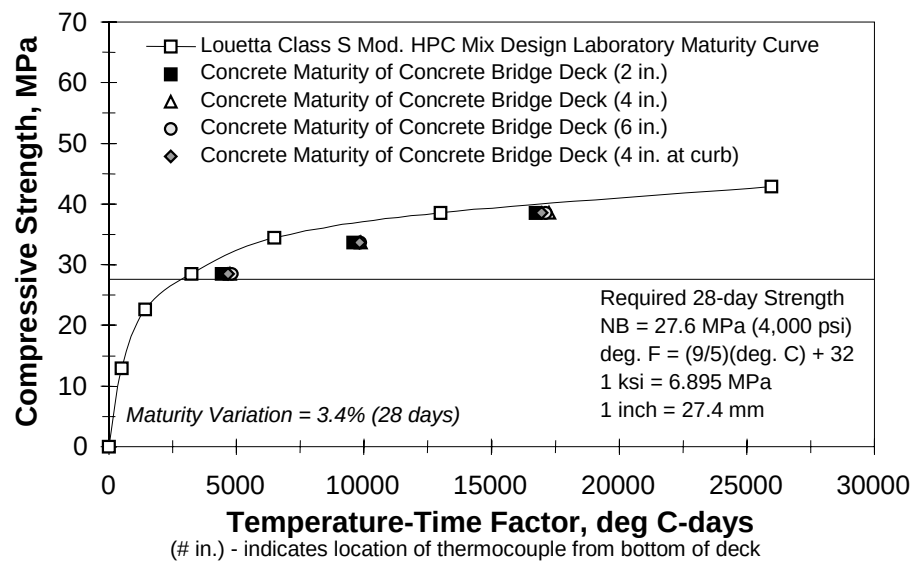


**Figure 9.9: Concrete Maturity CIP Deck Field Results—Class S Modified HPC Louetta
Road Overpass Northbound Span 3, Cast October 31, 1996**

9.3.1.2 Louetta Road Overpass Southbound HS/HPC CIP Deck. Figure 9.10 illustrates the concrete hydration temperature development at four locations within the deck upon placement of concrete for the Southbound Class K HS/HPC deck. As with the northbound deck, the more massive sections of CIP concrete attained the highest peak concrete temperature during hydration while the thinner CIP concrete attained the lowest peak concrete temperature during hydration. The southbound deck was also placed during mild ambient conditions, as shown in Figure 9.10. Thus, the precast panels and U-beams acted as a heat sink and dissipated heat related to concrete hydration quite effectively. The southbound deck attained a significantly higher maximum concrete temperature during hydration of 67 °C (152.6 °F) compared to the northbound deck of 45 °C (113 °F) under very similar placement conditions (ambient temperature). The much higher concrete temperature of the

southbound deck during hydration may be attributed to the much higher cementitious content. The concrete maturity results for the northbound CIP deck at 7, 14, and 28 days versus the natural maturity curve for the mix design are illustrated in Figure 9.9. Although the initial concrete temperatures varied within the deck during the first 24 hours after the placement (Figure 9.10), a very minor variation in maturity development was noted at the various locations.

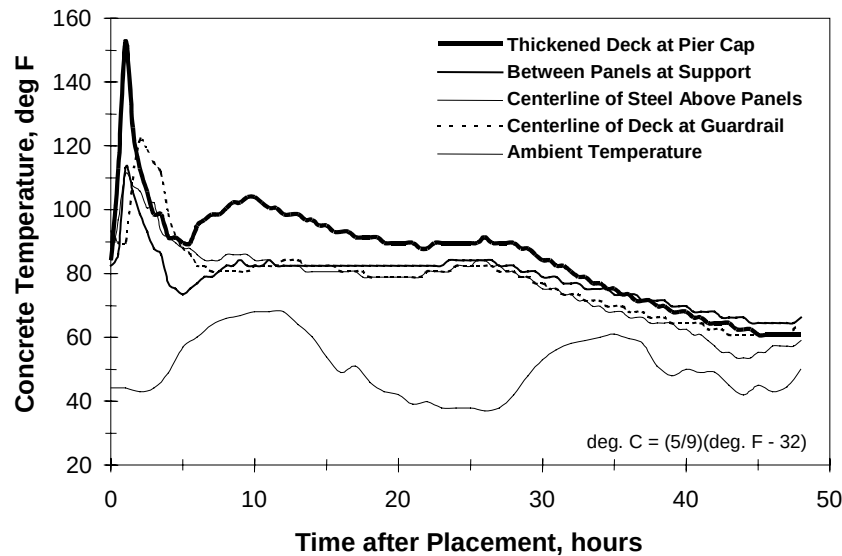


**Figure 9.10: Concrete Hydration Temperature Development of Class K HS/HPC
Louetta Road Overpass Southbound Span 2, Cast November 8, 1996**

The maturity variation between the 28-day results from the bridge deck and the natural maturity curve was 4.1 percent, on the conservative side. The results of the maturity method for the Class K HS/HPC mix design were within a ± 10 percent variation, even though the concrete temperature exceeded 26.7°C (80°F), the maximum internal concrete temperature proposed by Malhotra [109] for maturity method validity.

9.3.2 North Concho River Overpass Maturity Investigation

Four decks were instrumented at the North Concho River Overpass to investigate the use of the maturity method for HPC and HS/HPC bridge decks. Figure 9.12 illustrates the compressive strength gain with time of the three different mix designs selected for use in the maturity investigation at the North Concho River Overpass. The improved strength gain characteristics of the Class K eastbound deck may be attributed to the use of an HRWR and reduced w/cm ratio. The improved

later-age strength gain of the Class S Modified concrete may be attributed to the ASTM Class C fly ash replacement. Note the lower initial strength gain (1 to 3 days) when compared to the Class S concrete without fly ash replacement. This may be attributed to the hydration characteristics of the ASTM Class C fly ash.

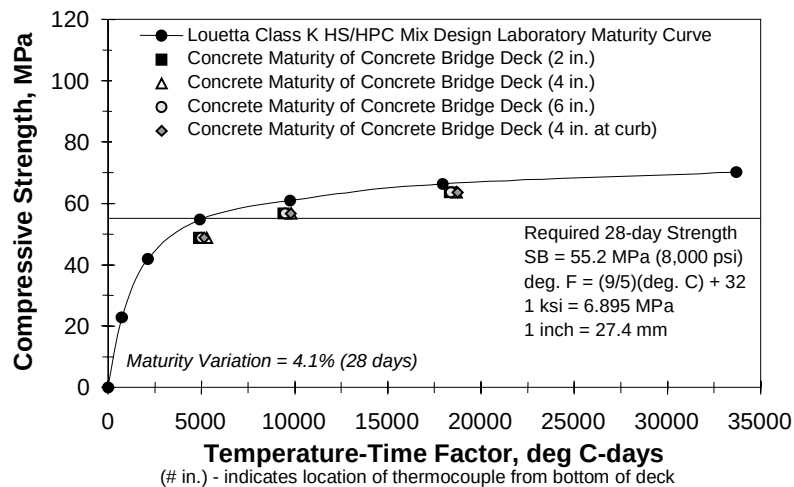


Figure 9.11: Concrete Maturity CIP Deck Field Results—Class K HS/HPC
Louetta Road Overpass Southbound Span 2, Cast November 8, 1996

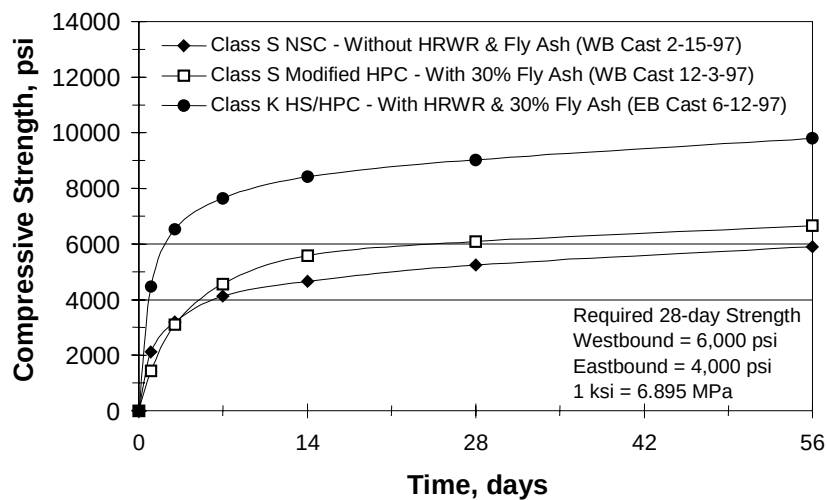


Figure 9.12: Compressive Strength Gain with Time of North Concho River Overpass CIP
Bridge Decks

9.3.2.1 North Concho River Overpass Westbound NSC CIP Deck. Figure 9.13 illustrates the concrete hydration temperature development at three locations within the deck upon placement of concrete for the Westbound Class S NSC deck. As with the Louetta Road Overpass Bridge, the more massive sections of CIP concrete attained the highest peak concrete temperature during hydration while the thinner CIP concrete attained the lowest peak concrete temperature during hydration. The deck attained a maximum concrete temperature of 30 °C (86 °F).

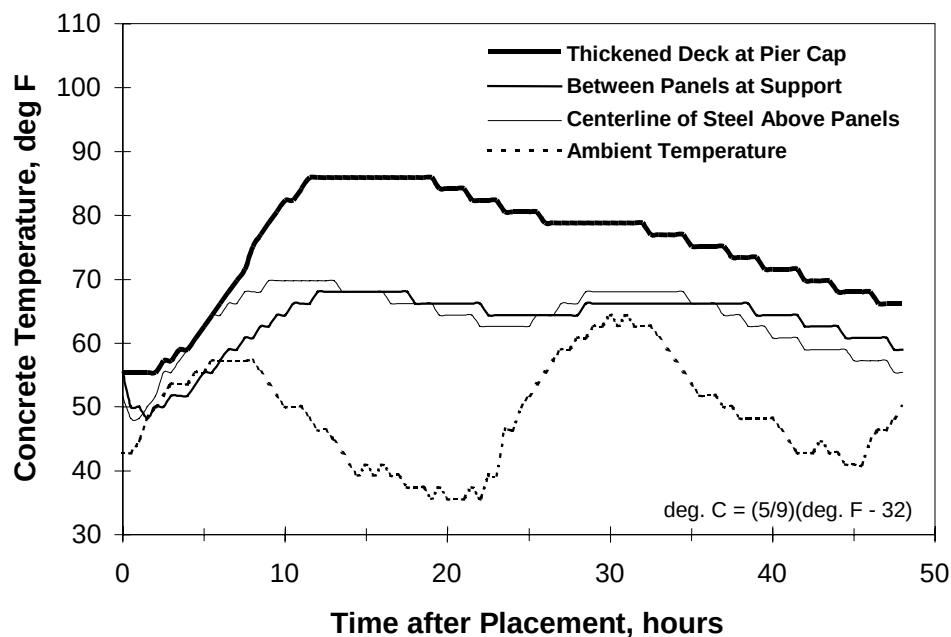


Figure 9.13: Concrete Hydration Temperature Development of Class S NSC North Concho River Overpass Westbound Span 6, Cast February 6, 1997

The concrete maturity results for Span 6 of the westbound CIP deck at 3, 7, 14, and 28 days versus the natural maturity curve for the mix design are illustrated in Figure 9.14. Although the initial concrete temperatures varied within the deck during the first 24 hours after the placement (Figure 9.13), a very minor variation in maturity development at the various locations was noted. The maturity variation between the 28-day results from the bridge deck and the natural maturity curve was 0.8 percent on the conservative side. The results of the maturity method for the Class S NSC mix design were well within the ± 10 percent variation typically associated with the maturity method.

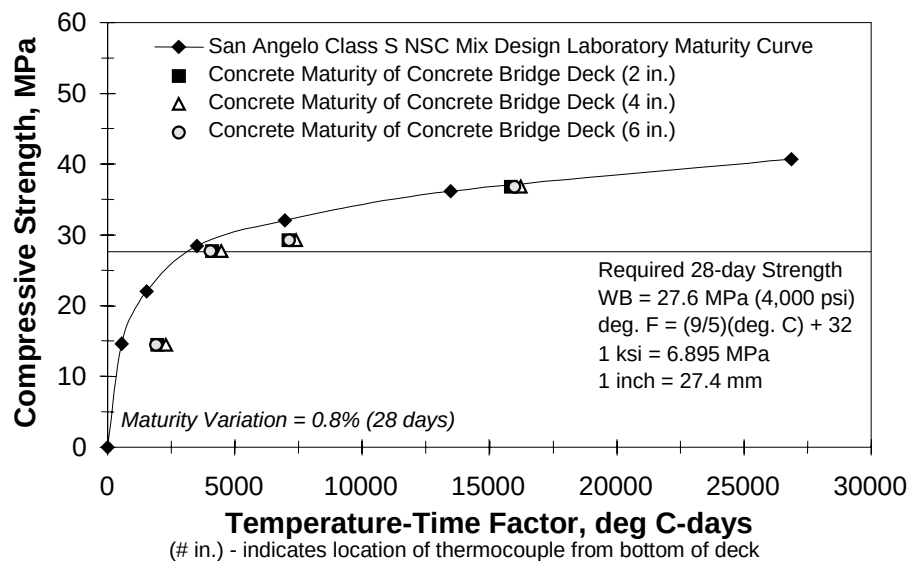


Figure 9.14: Concrete Maturity CIP Deck Field Results—Class S NSC
North Concho River Overpass Westbound Span 6, Cast February 5, 1997

9.3.2.2 North Concho River Overpass Westbound HPC CIP Deck. Figure 9.15 illustrates the concrete hydration temperature development at three locations within the deck upon placement of concrete for the westbound Class S Modified HPC deck. The deck attained a maximum concrete temperature during hydration of 34 °C (93.2 °F). The concrete maturity results for Span 1 of the westbound CIP deck at 7, 14, and 28 days versus the natural maturity curve for the mix design are illustrated in Figure 9.16. Although the initial concrete temperature during hydration varied within the deck during the first 24 hours after the placement (Figure 9.15), a very minor variation in maturity development was noted at the various locations. The maturity variation between the 28-day results from the bridge deck and the natural maturity curve was 0.2 percent on the conservative side. The results of the maturity method for the Class K HS/HPC mix design were well within the ± 10 percent variation typically associated with the maturity method.

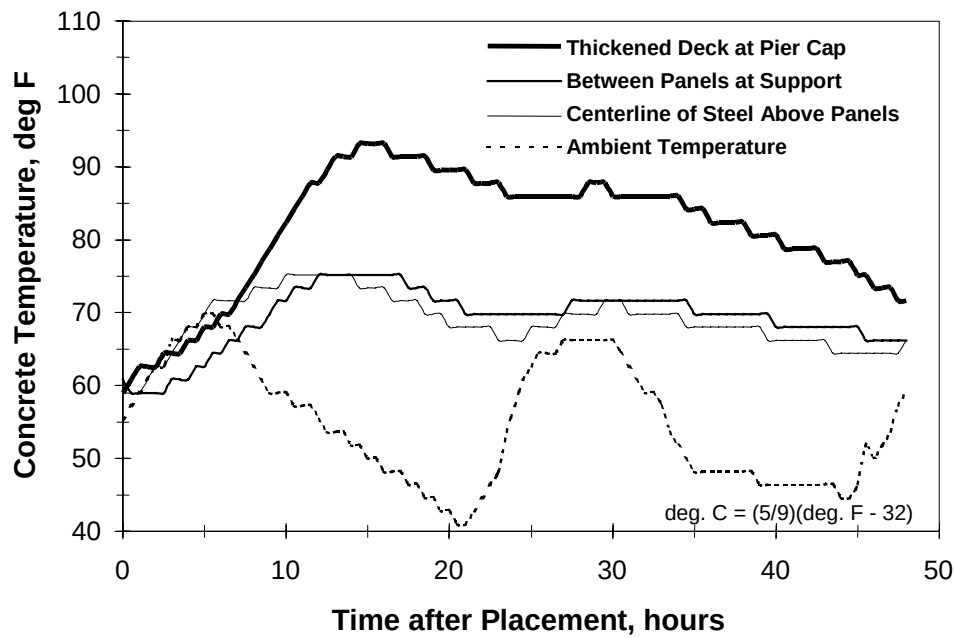


Figure 9.15: Concrete Hydration Temperature Development of Class S Modified HPC
North Concho River Overpass Westbound Span 1, Cast December 3, 1996

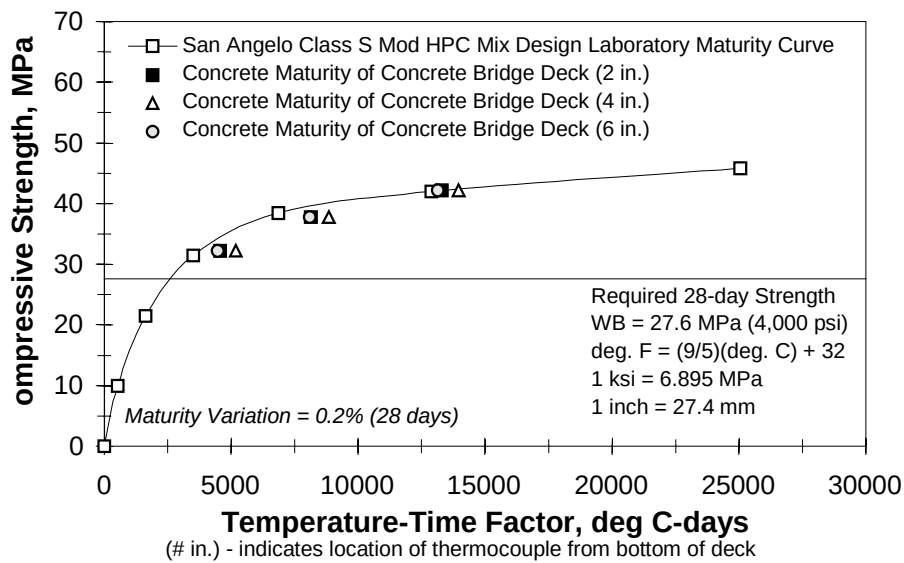


Figure 9.16: Concrete Maturity CIP Deck Field Results—Class S Modified HPC
North Concho River Overpass Westbound Span 1, Cast December 3, 1996

9.3.2.3 North Concho River Overpass Eastbound HS/HPC CIP Deck. Figures 9.17 and 9.18 illustrate the concrete hydration temperature development at three locations within the deck upon placement of concrete for Spans 1 and 4 of the westbound Class K HS/HPC decks. Span 1 and 4 decks attained a maximum concrete temperature during hydration of 51 °C (123.8 °F) and 53 °C (127.4 °F), respectively. As with the Louetta Road Overpass, the precast panels and AASHTO Type IV beams acted as a heat sink and effectively dissipated heat related to concrete hydration. However, the configuration and spacing of the U-beam appeared to dissipate heat more efficiently than the AASHTO Type IV under similar casting conditions (ambient temperature). It is difficult to conclude this with certainty because the rate and heat of hydration was undoubtedly influenced by the chemical admixtures, mineral admixtures, and the type and fineness of the cement particles used in the mix design. It should be noted that a similar dosage of retarder was used when casting these decks.

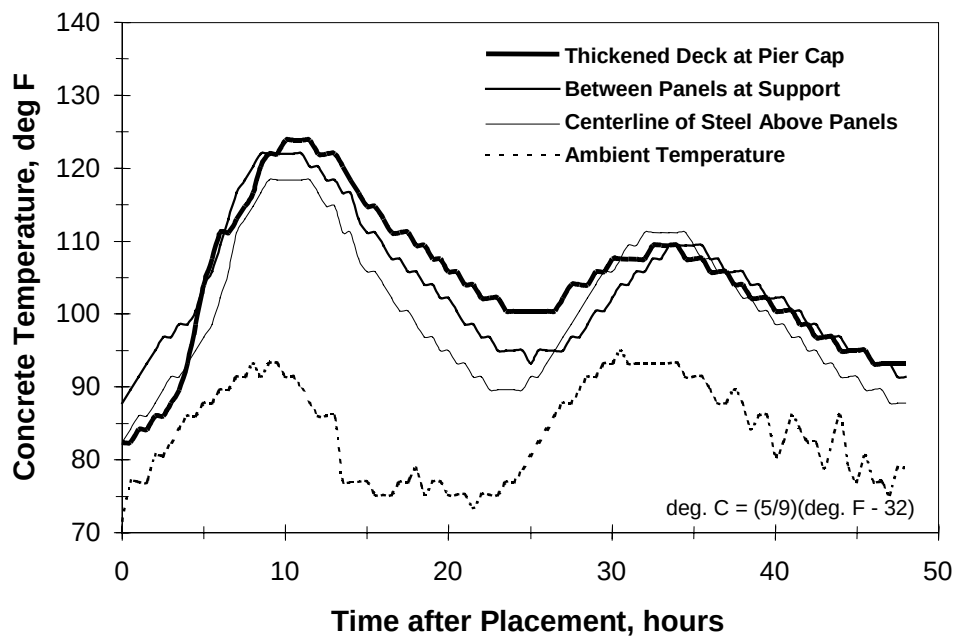


Figure 9.17: Concrete Hydration Temperature Development of Class K HS/HPC North Concho River Overpass Eastbound Span 1, Cast June 12, 1997

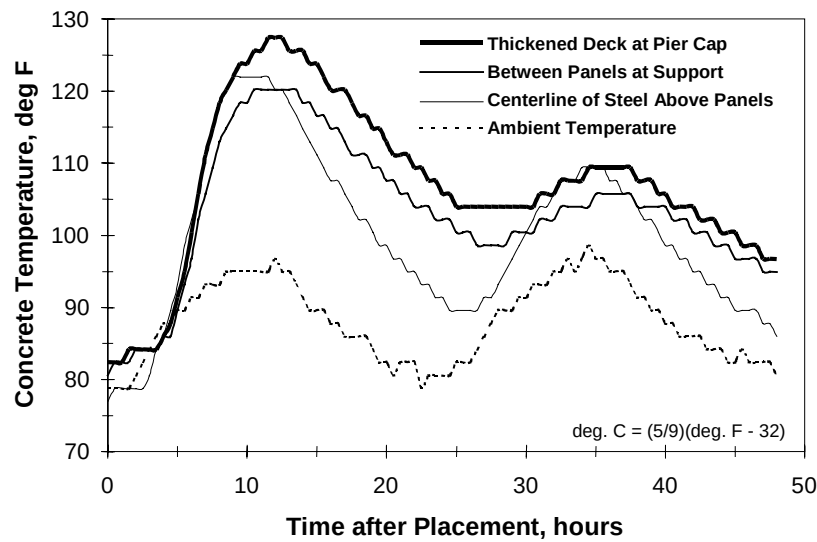


**Figure 9.18: Concrete Hydration Temperature Development of Class K HS/HPC
North Concho River Overpass Eastbound Span 4, Cast July 23, 1997**

The concrete maturity results for the eastbound CIP deck at 3, 7, 14, and 28 days versus the natural maturity curve for the mix design are illustrated in Figure 9.19.

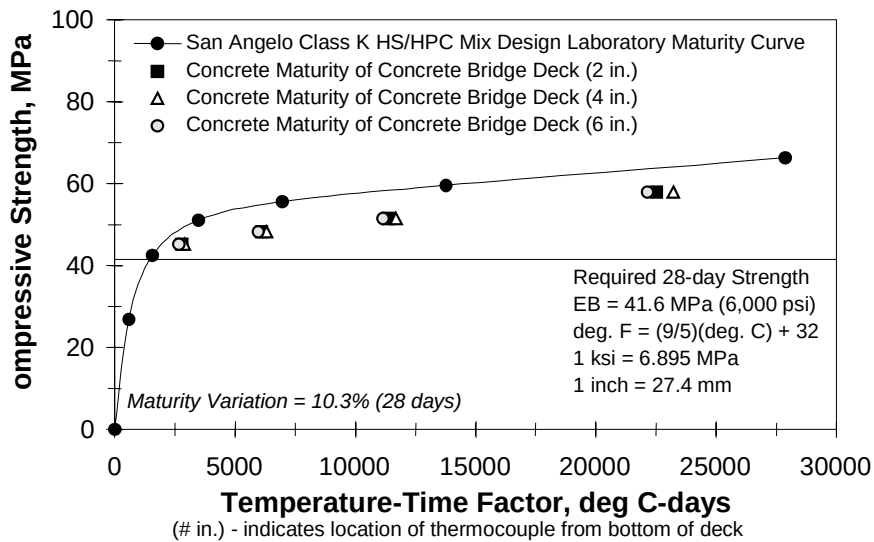


**Figure 9.19: Concrete Maturity CIP Deck Field Results—Class K HS/HPC
North Concho River Overpass Eastbound Span 1 and 4
Cast June 12, 1997, and July 23, 1997**

Although the initial concrete (hydration) temperatures varied throughout the deck during the first 24 hours after placement (Figures 9.17 and 9.18), a minor difference in maturity development was noted at the various locations that were monitored. The maturity variation between the 28-day results from the bridge deck and the natural maturity curve was 10.3 percent on the conservative side. The results of the maturity method for the Class K HS/HPC mix design were at the outer limit of the standard variation of the ± 10 percent typically associated with the accuracy of the method. While both Class K decks that were monitored far exceeded the maximum peak hydration temperature of 37.8 °C (100 °F) recommended by Plowman [110] at all locations in the deck, the maturity method predicted the concrete strength within acceptable levels on the conservative side. The high relative SA/V ratio for these CIP decks compared to other structural components undoubtedly allowed for sufficient heat dissipation to avoid a burn out effect. Based on the results of the five HPC CIP decks investigated, it is evident that the use of the maturity method is acceptable for CIP HPC bridge decks. However, it should be stated that temperature control for CIP decks is advised for placement temperatures of HPC. The current TxDOT specification for bridge decks states that “the temperature of cast-in-place concrete in bridge slabs and top slabs of direct traffic structures shall not exceed 29.4 °C (85 °F) when placed” [72]. This requirement is also recommended for all HPC decks. During hot-weather concreting, the cooling of material stockpiles, cooled mixing water, or ice replacement may be necessary to satisfy temperature placement requirements. In the case of the North Concho River Overpass, approximately one-third ice replacement was used for the eastbound CIP decks in order to meet the TxDOT specification placement temperature requirement. These decks were cast during the summer months. All other decks cast for both bridges satisfied the TxDOT placement temperature requirements without adjusting material temperatures.

The use of the maturity method requires modifications to project specifications. The specifications should, at a minimum, address the development of the maturity curve, the frequency of testing, the acceptance criteria if strength is not met, and any secondary QC/QA measures. The specifications must require that the maturity curve for the mix design be re-established as mix constituents or mix contents are modified by the concrete producer. The frequency of testing should, at a minimum, meet the current sampling frequency specified. Owing to the ease of thermocouple installation, concrete maturity development may be monitored even more frequently than previously specified. Such monitoring would result in a higher level of quality control. The specifications should also address acceptance criteria (such as cores) if strength is not met. A limited number of representative cylinders may be specified as a secondary quality control measure. When not used to

verify strength, these specimens may be used to periodically correlate field-produced concrete with the natural maturity curve of the concrete.

CHAPTER 10. HPC PERMEABILITY CORRELATION STUDY—AASHTO T277 AND AASHTO T259

10.1 Introduction

Workability and strength have long been the two major properties used for the specification and design of durable structural concrete. This practice has been based on the assumption that strength relates to all properties of concrete and, thus, can also be used as a durability indicator. This general association has been seriously questioned, and recently engineers have started to specify concrete for durability as well as for strength. This change can be traced in part to the rising cost of repair and maintenance of existing concrete structures, many of which are experiencing serious structural distress long before their design service life is reached. While high-strength concrete enables the design of structures with higher load capacity and smaller members, it does not automatically imply a more durable structure. It is the responsibility of the designer to specify a concrete mixture that combines both strength and durability under aggressive environments. This task is further complicated by the use of several mineral and chemical admixtures in the production of high-strength concrete. Such a durable mixture is classified as an HPC, which includes any concrete with an enhanced property, be it strength, durability, economics, or constructability.

Specifying concrete directly on the basis of performance rather than indirectly by strength requires a suitable durability test measure that is simple and reliable. While accelerated durability tests exist, they are costly, time consuming, and limited to laboratory use. On the other hand, correlations between permeability and durability have long been justified and studied. Permeability has been shown to be a major factor affecting many types of concrete distress, including sulfate attack, frost resistance, alkali-aggregate attack, carbonation, corrosion of reinforcing steel, and other durability properties. As a result, permeability is an aspect of concrete performance that can be specified, designed for, and monitored in the production of concrete.

Two tests are generally used to determine concrete's permeability to chloride ions and other aggressive solutions: The AASHTO test method T277, "Rapid Determination of the Chloride Permeability of Concrete," and test method T259, "Resistance of Concrete to Chloride Ion Penetration." The AASHTO T259 test has long been favored among the concrete corrosion specialists for assessing the chloride penetration of concrete. On the other hand, with a growing need for a faster and less-expensive indicator of chloride permeability, AASHTO T277 is becoming more popular among owners and specifiers. Today, the rapid 6-hour test (AASHTO T277) is a common requirement in construction project specifications in which engineers are specifying low values between 700 and 1,000 coulombs to ensure durable concrete. The following discusses the original

correlation developed between these aforementioned tests and investigates the correlation for a series of HS/HPC mix designs developed in the laboratory.

10.2 Historical Perspective on Permeability Evaluation and Testing

10.2.1 Original Development of AASHTO T277

The process of chloride ion diffusion into concrete is a very slow one, even when dealing with conventional concrete having a high water-to-cement ratio. Thus, in 1981, the Portland Cement Association (PCA) headed a research program sponsored by the FHWA to develop a test method for determining the chloride permeability of various concretes. The original test method is listed under FHWA/RD-81/119, "Rapid Determination of the Chloride Permeability of Concrete." AASHTO adopted the same test in 1983 under AASHTO T277, "Rapid Determination of the Chloride Permeability of Concrete," and in 1991, ASTM designated it as ASTM C1202, "Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration." ASTM limited the use of this test to types of concrete in which correlations to long-term chloride ponding procedures, such as AASHTO T259, have been established [119].

Originally, the FHWA wanted a test that was fast, could be performed in place (without disturbing concrete) or on cores in the laboratory, had a good correlation to data obtained from AASHTO T259, and measured the degree of impermeability. The research program, conducted in 1981, showed that the rate of chloride migration through a concrete specimen accelerates under the influence of an electric potential. Researchers also found that a good correlation existed between the measured amount of coulombs through the concrete specimens and the results obtained from the long-term ponding tests. Based on these findings, the rapid chloride permeability test was developed to measure the concrete resistivity to chloride migration. It measures neither the concrete permeability nor what actually occurs to concrete in the field. It gives only an indication as to how a certain concrete mixture is relating to the original set of mixtures listed in Table 10.1 [119].

Table 10.1: Charge Passed versus Total Integral Chloride [119]

Charge Passed (Coulombs)	Chloride Permeability	Type of Concrete	Total Integral Chloride to 40.6-mm (1.6-in.) Depth After 90-day Ponding Test
>4000	High	High water-to-cement ratio, conventional (≥ 0.6) PCC *	>1.3
2000-4000	Moderate	Moderate water-to-cement ratio, conventional (0.4 to 0.5) PCC *	0.8 to 1.3
1000-2000	Low	Low water-to-cement ratio, conventional (< 0.4) PCC *	0.55 to 0.8
100-1000	Very Low	Latex-modified concrete, internally sealed concrete	0.35 to 0.55
<100	Negligible	Polymer impregnated concrete, polymer concrete	<0.35

* portland cement concrete

10.2.2 Correlation between AASHTO T277 and AASHTO T259

While the literature involving the use of AASHTO T277 and AASHTO T259 with low to midrange strength concrete is voluminous, few research programs dealt with the use of these tests with HS/HPC. The following papers were found to be most relevant to the current research and very necessary to assess the validity of the two tests.

In the original report “Rapid Determination of the Chloride Permeability of Concrete” by Whiting [119] that was published in 1981 and included the results of a research program undertaken by the FHWA, Whiting outlined the development of a test procedure for a rapid assessment of the permeability of concrete to chloride ions (rapid 6-hour test method). He conducted a test program that included the concrete mixtures listed in Table 10.2 and established a correlation (Figure 10.1) between the 6-hour test method and AASHTO T259, the 90-day ponding test, as shown in Table 10.1. The correlation was based on tests conducted on a single core of each of the concrete mixtures. None of the concrete mixtures tested contained mineral admixtures or were high strength.

Whiting introduced the parameter of “total integral chloride,” I , which is a dimensionless value representing the integral of chloride concentration versus unit depth increments of 5.1 mm (0.2 in.) rather than actual depth. The variable I was used to represent the results of the 90-day ponding test in only one value. The total integral chloride is represented by Equation 10.1.

$$I = \frac{\text{total area under the line representing chloride content vs. depth}}{5.1 \text{ mm (0.2 in.) depth increments}} \quad (10.1)$$

Table 10.2: Concrete Mixtures Used in the FHWA’s 1981 Study [119]

Concrete Type	Water to Cement Ratio (by Weight)
Conventional Portland Cement Concrete, PCC	0.60, 0.50, 0.40
Latex Modified, LMC	0.24
Internally Sealed Wax Bead, WBC (heated and not heated)	0.55
Polymer Impregnated Concrete, PIC	0.50
Iowa Low Slump, IOWA	0.33
Polymer Concrete, PC	N.A.

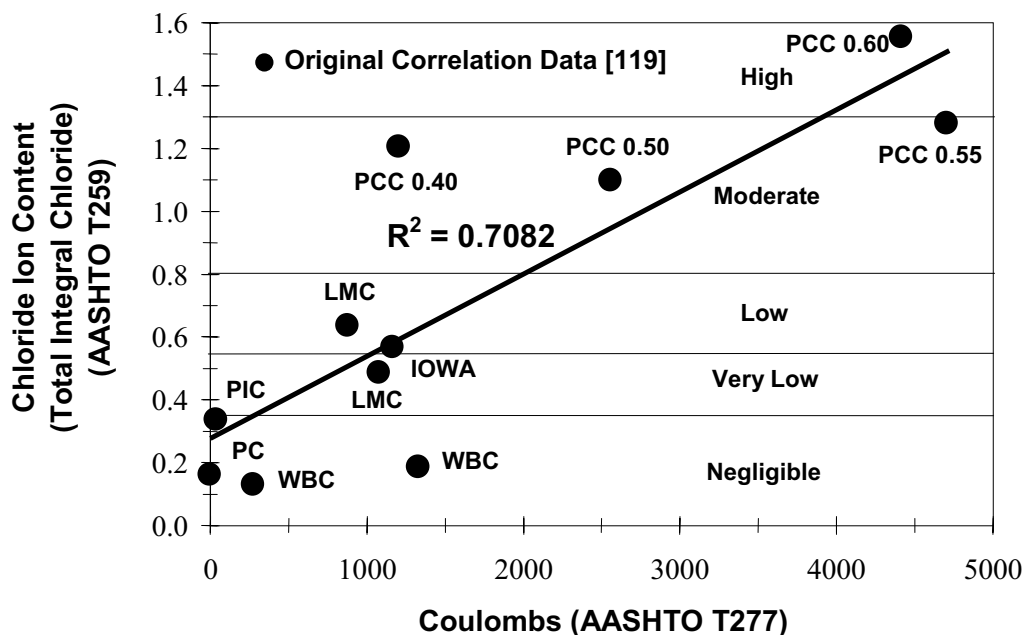


Figure 10.1: Correlation Data from Whiting [119] of ASTM C1202 Document [120]

Using a linear regression analysis, Whiting developed a correlation between the 90-day ponding test and the rapid 6-hour permeability test with an R^2 coefficient of 0.708. The linear regression line of his data is reproduced in Figure 10.1, where it can be seen that the data show a poor correlation in the 700 to 1,300 coulomb range. Whiting also concluded that polymer concrete and polymer-impregnated concrete had “negligible” chloride permeability based on the 6-hour test. Yet these same concrete mixtures showed I values of 0.16 and 0.34 according to the 90-day ponding tests. These I values are relatively high compared with the 6-hour test results.

Whiting and Dziedic published “Resistance to Chloride Infiltration of Superplasticized Concrete as Compared with Currently Used Concrete Overlay Systems” [120] in 1989. In it, they discussed the effectiveness of various materials used in bridge overlays at construction sites in Ohio and developed a correlation between the rapid (6 hour) and 90-day ponding tests. From each of the concrete mixtures listed in Table 10.3, two specimens were tested for AASHTO T277 and a single specimen was tested for AASHTO T259. Concrete mixtures containing HRWRs and silica fume were moist cured for 42 days and the latex-modified concrete (LMC) was moist cured for two days then left to air dry for 40 days. Whiting and Dziedic found a correlation coefficient, R^2 , of 0.81 for data from both the rapid and the 90-day ponding tests. The long-term ponding tests on all twenty-two concrete

mixtures resulted in low total integral chloride values, I , ranging from 0.05 to 0.15. The corresponding coulomb values ranged from 250 to 5,000. The latex-modified concretes had coulomb values between 700 and 2,200, while the HRWR concretes had coulomb values between 250 and 5,000.

Table 10.3: Concrete Mixtures Used in the 1989 Paper by Whiting and Dziedic [120]

Concrete Type	Usual w/c	Number of Mixes	Source of Concrete Mixtures
HRWR modified	0.30 to 0.32	10	2 mixtures from each of 5 sites
Latex modified	<0.39	10	2 mixtures from each of 5 sites
Silica Fume modified	0.30	2	1 site

In a paper published in 1992, Mitchell and Whiting [121] stated that results of the AASHTO T277 test are not as reliable as they were thought to be. The authors stated that the development of the original correlation did not investigate variables such as aggregate type and size, cement content and composition, and density, in addition to other factors. In the authors' opinion, the correlation did not represent a large database of concrete, only single cores taken from FHWA original slabs. The authors recommended that individuals using the RCPT establish their own correlation between charge passed and known chloride permeability for their own particular materials.

In 1994, Pfeifer, McDonald, and Krauss published "The Rapid Chloride Permeability Test and Its Correlation to the 90-day Chloride Ponding Test" [123], in which they seriously critiqued the use of the AASHTO T277 test. The paper included a review of the five references [119, 124, 125, 126, and 127] that substantiate the use of the ASTM C1202 and several other papers concerned with the application of the RCPT. The authors of the paper calculated and compared total integral values, I , from all five references. They concluded that these values exhibit wide variance and no correlation with Table 10.1, originally devised by Whiting in 1981. Results of that comparison are presented in Table 10.4, which shows that there is no consistency in rating the chloride penetrability of the concrete mixtures tested.

Table 10.4: Comparison of the Ratings of Chloride Penetrability Based Upon Results from the Five ASTM C1202 References [123]

Coulombs	Ref 1	Ref 2	Ref 3	Ref 5
5000	High	Low	Negligible	Low
2500	Moderate	Very Low	Negligible	Very Low
1500	Low	Very Low	Negligible	Very Low
500	Very Low	Negligible	Negligible	Very Low
<100	Negligible	Negligible	Negligible	Negligible

10.3 Discussion of Test Results

10.3.1 Mechanical and Material Property Results

10.3.1.1 Fresh Concrete Properties. Thirty-one concrete mixtures were tested, in the plastic state, for slump and air content. These mixtures were designated AELS1 through AELS9, AETR1 through AETR9, AERG1 through AERG9, SSP-3, SSP-32, HP-3, and HP-32. Slump tests were performed before and after adding superplasticizers and in accordance with ASTM C143, “Standard Test Method for Slump of Portland Cement Concrete.” Air content was tested in accordance with AASHTO T152, “Air Content of Freshly Mixed Concrete by the Pressure Method,” immediately before placement of the concrete. Fresh concrete property results for each concrete mixture are presented in Tables 6.6 through 6.9 of this report.

10.3.1.2 Compressive Strength ASTM C39-93a. The compressive strength of each mixture was determined at 7, 28, and 56 days of age in accordance with ASTM C39-93a, “Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens.” The compressive strength was reported as the average of three companion cylinders. Compressive strength test results for each concrete mixture at the specified test ages are presented in Tables 6.40 through 6.43 of this report.

10.3.1.3 Splitting Tensile Strength ASTM C496-96. The splitting tensile strength was determined for each mixture at 28 and 56 days of age in accordance with ASTM C496-96, “Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens.” The average of two companion cylinders was reported as the splitting tensile strength of the mix design. Test results for the splitting tensile strength at 56 days for all the mixtures are illustrated in Figure 10.2. This represents the concrete properties when the 90-day ponding cycle was initiated. The data is generally

concentrated in the bands between 622 to 830 sqrt f'_c , in kPa (7.5 to 10 sqrt f'_c , in psi), which exceeds the empirical equation for HSC recommended by ACI Committee 363.

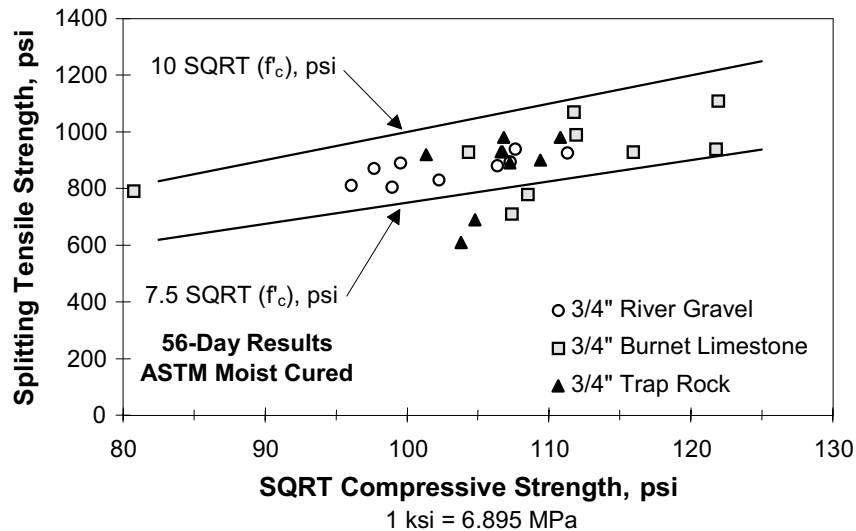


Figure 10.2: Splitting Tensile Strength Test Results

10.3.1.4 Modulus of Elasticity ASTM C469-94. ASTM C469-94, “Standard Test Method for Static Modulus of Elasticity and Poisson’s Ratio of Concrete in Compression” was used to determine the elastic moduli of 101.6-mm x 203.2-mm (4-in. x 8-in.) cylinder specimens. Three cylinders of each mixture were used to obtain an elastic modulus measurement. Figure 10.3 illustrates the 56-day test results for all of the concrete mixtures. The elastic modulus generally satisfied the empirical equation recommended by ACI Committee 318 for conventional concretes. All of the mixes exceeded the empirical equation recommended by ACI Committee 363 for high-strength concretes.

10.3.1.5 Rapid Chloride Permeability AASHTO T277. AASHTO T277, “Standard Method of Test for Rapid Determination of the Chloride Permeability of Concrete,” was used to determine the chloride ion permeability. The permeability of each mixture was considered to be the average coulomb results of four 51-mm (2-in.) thick disks (i.e., two 100-mm x 200-mm cylinders from each mixture) that were prepared and tested following the AASHTO T277 procedure.

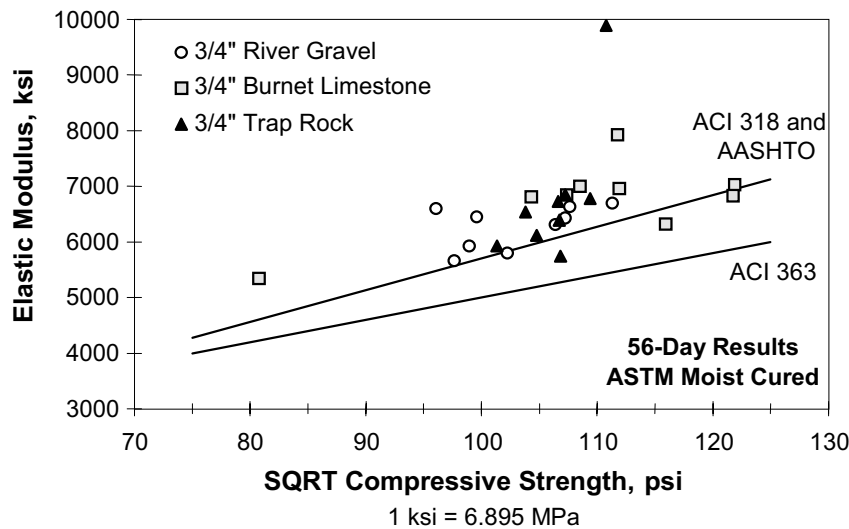


Figure 10.3: Elastic Modulus Test Results

In this study, the researchers desired to investigate the correlation between AASHTO T277 and AASHTO T259 for HS/HPC. Therefore, specimens from both tests were cured identically and the RCPT was performed at the same time the 90-day ponding cycle for the ponding block specimens was initiated. Following the 365-day ponding cycle, RCPT testing was conducted on cores taken from the ponding block specimens (see Section 10.3.4). This was done to ensure that specimens for both tests were subjected to identical curing conditions. The RCPT was also performed on specimens moist cured for 56 days to investigate the effect of curing on the correlation data.

10.3.1.6 Chloride Ponding AASHTO T259. Diffusion of chloride ions into the slab specimens was determined using a CL-1000 chloride test system manufactured by James Instruments. After 90 days of continuous ponding, powdered samples were collected from each of the slabs by drilling three holes to various depths: from 1.6 mm to 12.7 mm (0.0625 in. to 0.5 in.), 12.7 mm to 25.4 mm (0.5 in. to 1.0 in.), and 25.4 mm to 38.1 mm (1.0 in. to 1.5 in.). The powdered concrete samples from each slab for a given depth were mixed together and then divided into two 1.5-gram samples. This resulted in a total of six samples per mixture at each depth (three slabs per mixture). The powder was then mixed with a 15 percent acetic acid extraction solution that reacted with the chloride ions and produced an electrochemical reaction, the degree of which was measured using the CL-1000 electrode setting. All drilled holes were then closed using a rapid setting epoxy and the slabs were subjected to an additional 90-day period of ponding as part of the ongoing study. Measurements were converted into percent chloride ion content using a mathematical model presented in AASHTO

T260, "Sampling and Testing for Chloride Ions in Concrete and Concrete Raw Materials," Procedure C. Assuming a concrete density of 4,000-pounds per cubic yard, the chloride percentage was translated into chloride content in pounds of chloride per cubic yard. For the purpose of comparing these results to literature values, the total integral chloride, I , factor was computed.

10.3.2 Variation between RCPT Curing Conditions

In order to investigate the correlation between AASHTO T277 and AASHTO T259, it was important to determine the effect of the length of moist curing on the AASHTO T277 results. Therefore, specimens from all the mixtures were cured under two different conditions: (1) 56 days moist curing, and (2) 14 days moist curing plus 42 days air drying. Clearly the specimens moist cured for 56 days were less permeable than those cured with the ponding blocks owing to the additional cement hydration associated with the added moist curing. However, the difference between the two permeability results was not large enough to cause a shift in the permeability levels defined in AASHTO T277 (Table 10.1). Specimens from the same mixture, but cured under the two conditions, exhibited the same permeability, either "Low" or "Very Low." From this evaluation, it was concluded that for the low water-to-cementitious material ratio mixtures used in this study (0.25 through 0.33), the length of moist curing had a relatively minor effect on the permeability results of AASHTO T277. However, the correlation relationship was affected by the curing condition as discussed in subsequent sections of this report.

10.3.3 90-Day Correlation between AASHTO T277 and AASHTO T259

10.3.3.1 General Results. As expected from HS/HPC mixtures with low water-to-cementitious material ratios, the measured coulomb values were relatively low, and the 90-day chloride contents were concentrated in the uppermost 12.7 mm (0.5 in.) of all the blocks subjected to ponding. Chloride content at deeper depths was negligible, and in some cases, was zero. The RCPT results ranged from 384 to 1,956 coulombs, while the corresponding integral chloride ranged from 0.250 to 1.214.

10.3.3.2 Permeability Classifications as Defined in the FHWA-81 Original Report. Using the permeability definitions presented and discussed in the FHWA-81 report, Table 10.1, the mixtures were classified based upon their total integral chloride and their coulomb values. Fifty percent of the mixtures exhibited identical total integral chloride and coulomb classifications, while the other 50 percent showed some discrepancies: eight mixtures exhibited "Very Low" coulomb values while exhibiting "Low" total integral chloride values, three mixtures exhibited "Low" coulomb values

while exhibiting “Moderate” integral chloride values, and two mixtures exhibited “Very Low” coulomb values while exhibiting “Moderate” total integral chloride values. As a result, it was concluded that for the high-strength concrete mixtures tested, the two tests were not consistent in predicting the permeability levels defined in the original test development report of AASHTO T277. However, it is important to note that in the case of this study, all of the AASHTO T277 results were conservative and resulted in a lower permeability classification than the 90-day ponding test results. All the discrepancies arose from the fact that the total integral chloride resulted in a higher classification.

10.3.3.3 Linear Regression Analysis. The correlation between AASHTO T277 and AASHTO T259 for the HS/HPC mixtures used in this experimental program was further investigated by performing a linear regression analysis on the two sets of data. Two cases were considered: (1) the 90-day total integral chloride values, I values, were coupled with their corresponding coulomb values; and (2) the 90-day chloride content results in percent by weight of concrete in the uppermost 12.7 mm (0.5 in.) of the blocks were coupled with their corresponding coulomb values. The uppermost 12.7-mm (0.5-in.) chloride content resulted in an R^2 correlation coefficient of 0.71 and exhibited a slightly better correlation to the AASHTO T277 results than the total integral chloride values that displayed an R^2 coefficient of 0.65. Results of both regression analyses are shown in Figures 10.4 and 10.5 for the 90-day results. In the original FHWA-81 report, the authors used the total integral chloride data and reported an R^2 correlation coefficient of 0.70, which is equivalent to the coefficient calculated in this study. However, the correlation was quite different owing to the following: (1) thirty-one mixtures were investigated in the current study as compared to eleven mixtures in the original report, and (2) results obtained from the RCPT in this experimental program were all relatively low and did not cover a wide range of values. Figure 10.4 compares the total integral chloride data generated in the course of the current study to data used in the development report of AASHTO T277.

The following remarks pertain to Figure 10.4 and the total integral chloride:

1. For the range of coulomb values used, between 384 and 1,904 coulombs, the original data exhibited a very poor correlation, while data produced during this study displayed good correlation ($R^2 = 0.65$).
2. Seventy-seven percent of the data points were higher than the original regression line, which was anticipated owing to the mixture types originally investigated. However, the location of the original portland cement concrete data (w/cm ratio = 0.04 to 0.60) relates well to the water-to-cement ratios (w/cm ratio = 0.28 to 0.33) investigated in this study.

Figure 10.5 compares the chloride ion content from percentage by weight of concrete to data used in the development report of AASHTO T277 [119]. In this manner, a better correlation is

represented for the HS/HPC mixes because chlorides below 19-mm (0.75-in.) depths are generally not present after the initial 90-day ponding cycle.

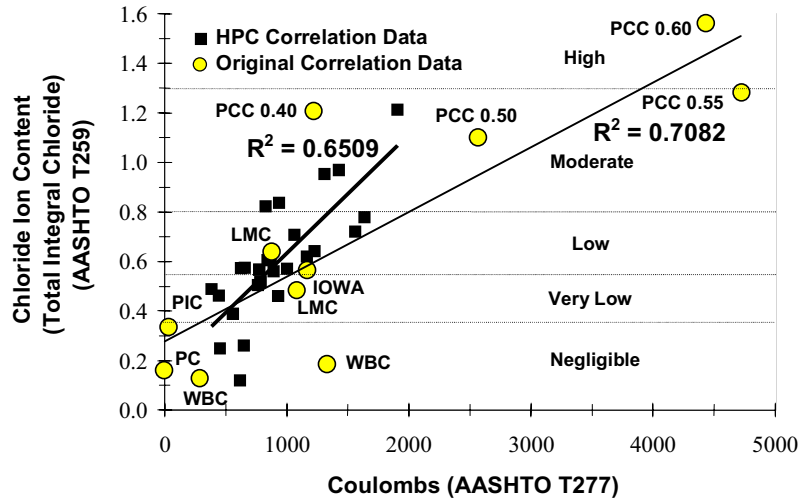


Figure 10.4: Total Chloride Ion Content by Integral—90-day Correlation Data

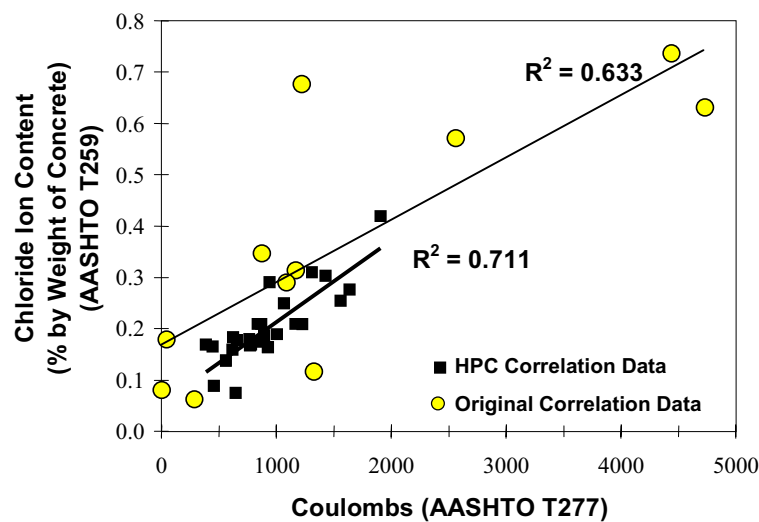


Figure 10.5: Total Chloride Content % by Weight—1.6 mm to 12.7 mm Correlation Data at 90 Days

The chloride integral value was averaged to the same depth (40-mm) as the original correlation study even though chlorides were not present at this depth. In order to determine how

changing different constituents in the mix design affected the correlation between the two tests, mixtures were divided into six groups: mixtures containing river gravel, mixtures containing Burnet limestone, mixtures containing trap rock, mixtures containing fly ash, mixtures with 0 percent fly ash content, and mixtures with 0 percent entrained-air content. For each group of mixtures, linear regression analysis was performed. The following sections address each of these results in detail.

10.3.3.4 Effect of Entrained-Air Content on the Results of the Two Tests. The first step in investigating the correlation between AASHTO T277 and AASHTO T259 was to monitor the effect of the entrained-air content on the results of the two tests. For the levels of entrained-air content used in this study (between 0 and 6 percent), both tests reinforced the fact that the permeability of HS/HPC is not affected by the addition of air entrainment. Figure 10.6 illustrates the AASHTO T277 results of identical mixtures with the only difference being the entrained-air content.

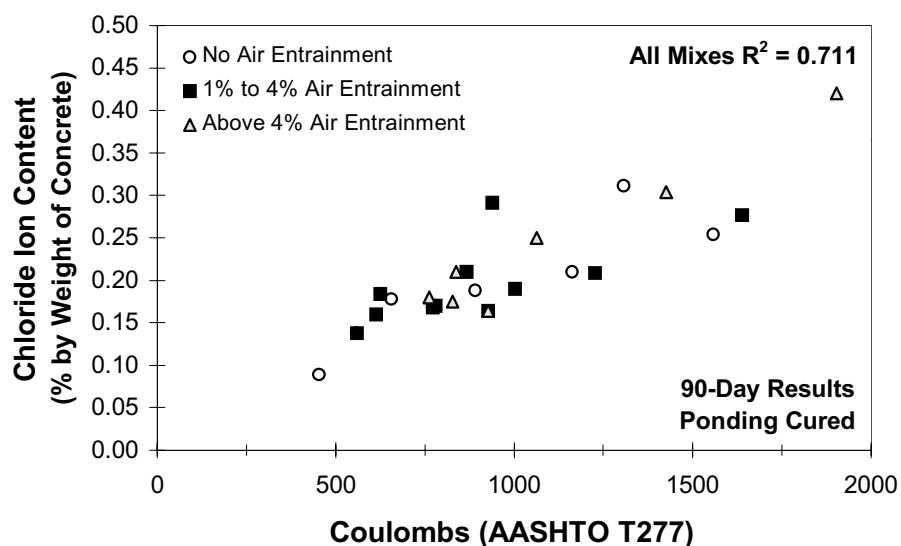


Figure 10.6: Air-Entrainment Correlation Data at 90 Days
Total Chloride Ion Content by Weight—1.6 mm to 12.7 mm

The data indicated that 44 percent of the mixtures containing entrained air had higher permeability than the corresponding mixtures without air, while 56 percent of these mixtures had lower permeability. The AASHTO T259 results for the same mixtures are also illustrated in Figure 10.6, where it can be seen that the addition of air entrainment caused an increase in chloride content in 69 percent of the mixtures, while 31 percent of these mixtures showed a decrease after the 90-day ponding cycle. It is clear from this evaluation that the addition of entrained air did not exhibit a definite

trend on the chloride content and the coulomb values of the mixtures involved and, thus, the conclusion was reached that an increase in the entrained-air content had no effect on the chloride permeability of high-strength concrete.

10.3.3.5 Effect of Coarse Aggregate Type on the Results of the Two Tests. The effect of three aggregate types on the permeability of HS/HPC was also investigated using both tests (Figure 10.7) after the 90-day ponding cycle. The aggregates were 19-mm (0.75-in.) river gravel, 19-mm (0.75-in.) Burnet limestone, and 19-mm (0.75-in.) trap rock. Based on the AASHTO T277 results, mixtures containing trap rock exhibited higher permeability than mixtures containing river gravel, which in turn exhibited higher permeability than the Burnet limestone mixtures. The variation between the coulomb values of the river gravel and Burnet limestone mixtures was minimal. Based on the AASHTO T259 results, the trap rock mixtures also exhibited higher permeability than the Burnet limestone mixtures, which in turn exhibited higher permeability than the river gravel mixtures.

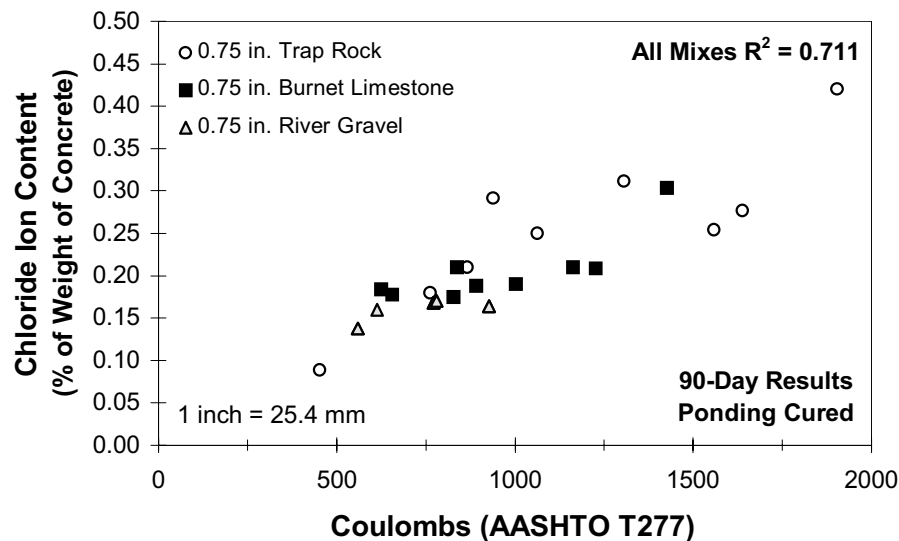


Figure 10.7: Aggregate Type Correlation Data at 90 Days
Total Chloride Ion Content by Weight—1.6 mm to 12.7 mm

In the same manner, total chloride integrals of the river gravel and Burnet limestone mixtures were comparable with minimal differences. Thus, both tests classified the trap rock mixtures as having the highest permeability and determined that the river gravel and Burnet limestone mixtures exhibited comparable permeability. The two tests had contradicting results on the classification of the river gravel and Burnet limestone mixtures.

10.3.3.6. Effect of Fly Ash Replacement on the Results of the Two Tests. The effect of fly ash replacement on the permeability of HS/HPC was also investigated using both tests. Three variations of fly ash replacement were selected, 0, 25, and 35 percent. Figure 10.8 illustrates the results of both AASHTO T277 and T259. Each series of replacement percentages exhibited very good grouping correlation after the 90-day ponding cycle, as illustrated in Table 10.5. Clearly, as the Class C fly ash replacement was increased, both the chloride content and coulombs passed decreased. The results after the initial 90-day ponding cycle indicated that the use of a Class C fly ash reduced permeability and enhanced durability performance.

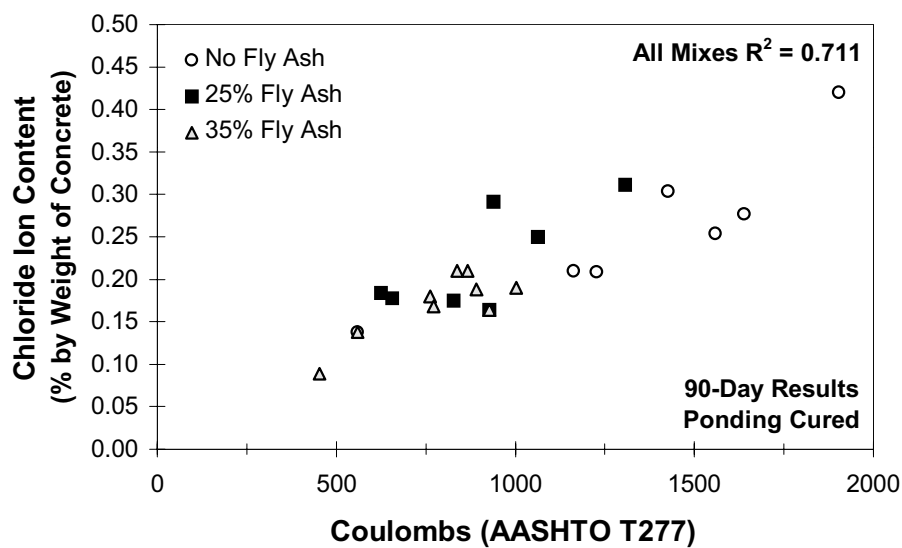


Figure 10.8: Fly Ash Replacement Correlation Data at 90 Days
Total Chloride Ion Content by Weight—1.6 mm to 12.7 mm

Table 10.5: Correlation of Mixes with and without Fly Ash Replacement

Fly Ash Replacement (Percentage)	R^2 Correlation Value
No Fly Ash Replacement	0.8094
25 percent Fly Ash Replacement	0.6445
35 percent Fly Ash Replacement	0.6852

10.3.4 365-Day Results—AASHTO T277 and AASHTO T259

After the initial 90-day ponding cycle and chloride testing, the specimens were repounded for a second ponding cycle totaling 365 days. This long-term ponding cycle was performed to investigate the rapid chloride permeability and chloride penetration at later ages owing to the slow intrusion of chlorides for these HS/HPC mix designs at depths greater than 19-mm (0.75-in.). RCPT specimens were core-drilled from the control blocks and ponding blocks after the 365-day ponding cycle to ensure similar curing conditions for the concrete specimens sampled under both test methods. Results of these tests are as follows.

10.3.4.1 General Results. As expected from HS/HPC mixtures with low water-to-cementitious material ratios, the measured coulomb values were relatively low and the 365-day chloride contents were concentrated in the uppermost 12.7-mm (0.5-in.) of all the blocks subjected to ponding. Chloride content at deeper depths was negligible and, in some cases, was zero. The RCPT results ranged from 129 to 1,852 coulombs, while the corresponding integral chloride ranged from 0.507 to 1.794 after the 365-day ponding cycle. Figure 10.9 illustrates the chloride ion penetration for a representative laboratory trial mix. While the chloride percentages increased in the uppermost 19 mm (0.75 in.) after the 365-day cycle, no chlorides were noted beyond a depth of 31.75-mm (1.25-in.), significantly less than clear cover requirements for most structural components. This was typical of all mix designs. Results for each mix design and test method are reported in Appendix D.

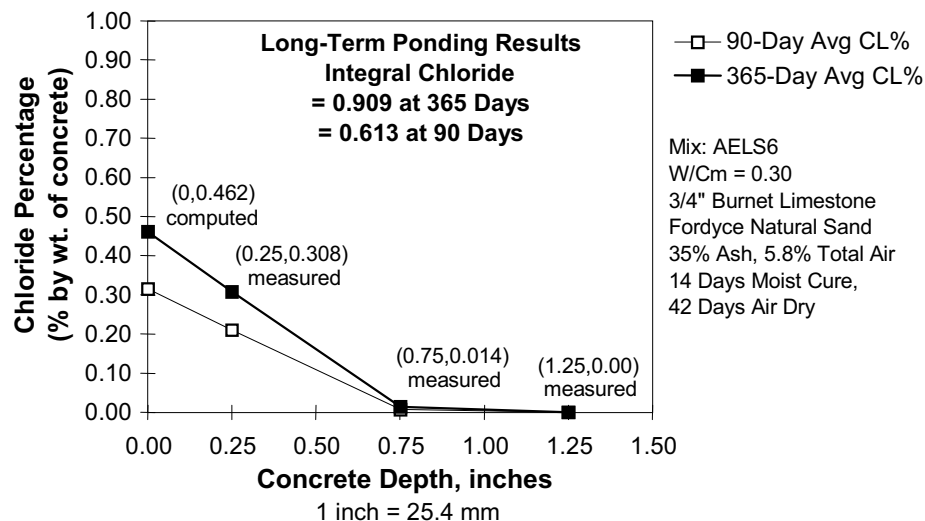


Figure 10.9: Chloride Ion Penetration for Laboratory Trial Mix AELS6

10.3.4.2 Effect of Entrained-Air Content on the Results of the Two Tests. For the levels of entrained-air content used in this study (between 0 and 6 percent), both tests were consistent with the 90-day ponding cycle results and reinforced the fact that the permeability of HS/HPC is not affected by the addition of air entrainment. Figure 10.10 illustrates the AASHTO T277 results of identical mixtures with the only difference being the entrained-air content after the 365-day ponding cycle. Figures 10.11 and 10.12 illustrate the total air content of the laboratory mixes versus chloride content and RCPT, respectively. It is clear from the scatter illustrated in both of these figures that the addition of entrained air did not exhibit a definite trend on the chloride content and the coulomb values of the mixtures involved. Results from both tests at both cycle intervals exhibited similar behavior and conveyed the same conclusion. This is expected behavior from HS/HPC mixtures, which can be explained by the fact that the capillary porosity is usually very low (generally disconnected at very early ages) and the addition of air entrainment for HS/HPC mixes is not expected to drastically affect the permeability.

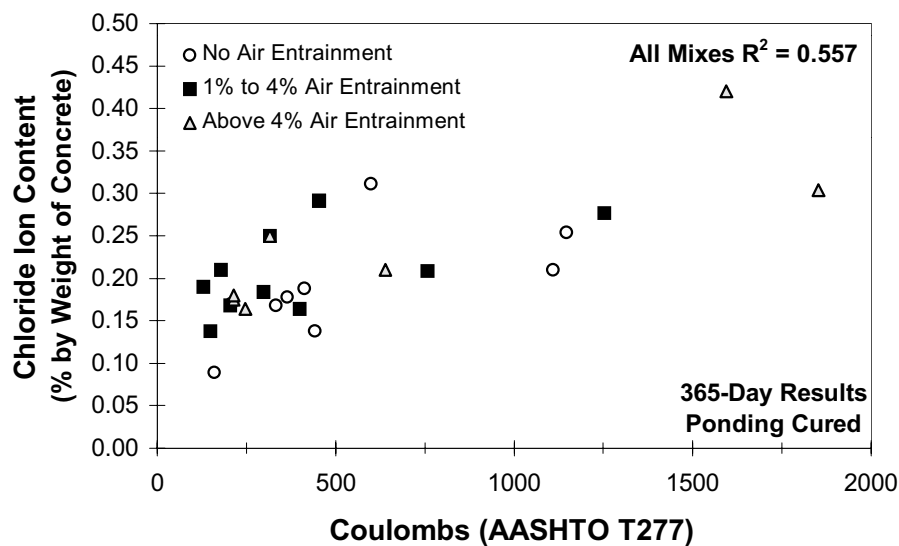


Figure 10.10: Air-Entrainment Correlation Data at 365 Days
Total Chloride Ion Content by Weight—1.6 mm to 12.7 mm

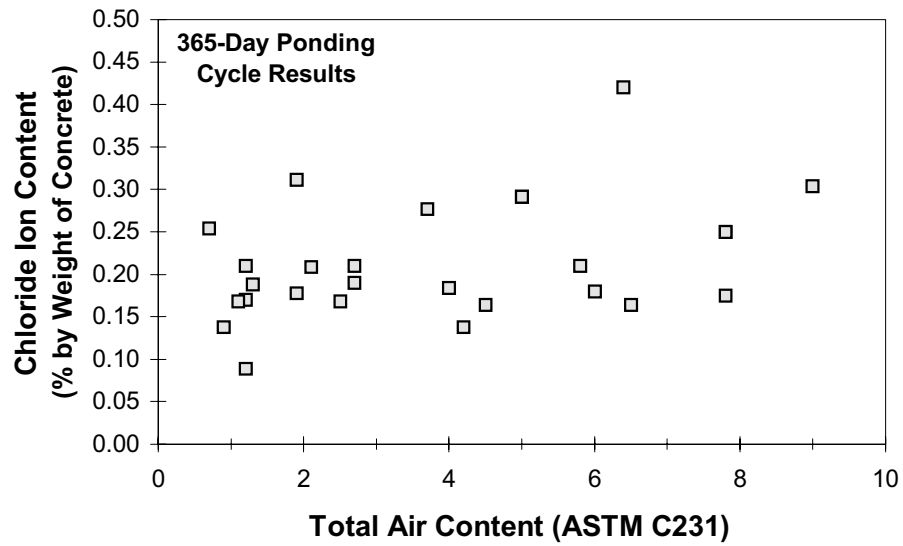


Figure 10.11: Total Air Content versus Chloride Content at 365 Days
Total Chloride Ion Content by Weight—1.6 mm to 12.7 mm

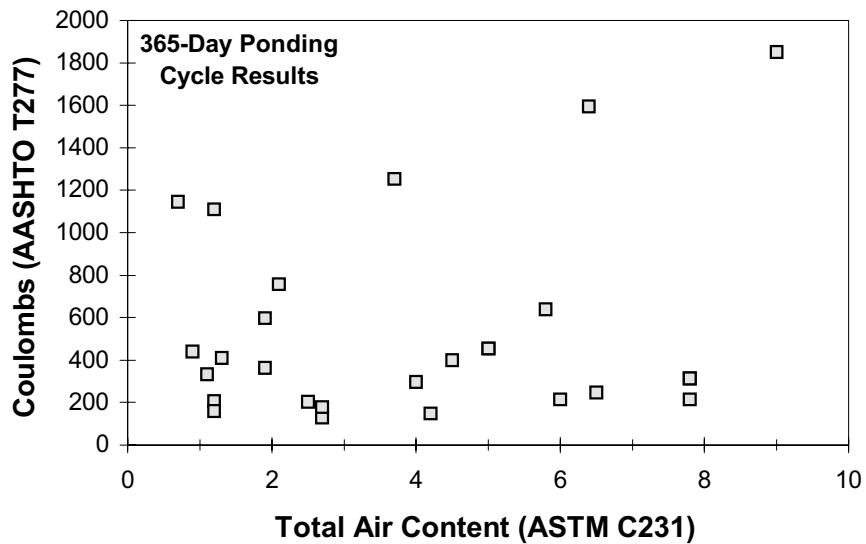


Figure 10.12: Total Air Content versus RCPT at 365 Days

10.3.4.3 Effect of Coarse Aggregate Type on the Results of the Two Tests. The effect of three aggregate types on the permeability of HS/HPC was also investigated using both tests after the 365-day ponding cycle, as illustrated in Figure 10.13. Based on the AASHTO T277 results, mixtures

containing trap rock exhibited higher permeability than mixtures containing river gravel, consistent with the 90-day ponding results. However, the variation between the coulomb values of the river gravel and Burnet limestone mixtures was consistently higher than the 90-day ponding results. Based on the AASHTO T259 results, the trap rock mixtures also exhibited higher permeability than the Burnet limestone mixtures, which, in turn, exhibited higher permeability than the river gravel mixtures. In the same manner, total chloride integrals of the river gravel and Burnet limestone mixtures were comparable with minimal differences. Thus, both tests classified the trap rock mixtures as having the highest permeability and determined that the river gravel and Burnet limestone mixtures exhibited comparable permeability. The two tests had contradicting results on the classification of the river gravel, trap rock, and Burnet limestone mixtures.

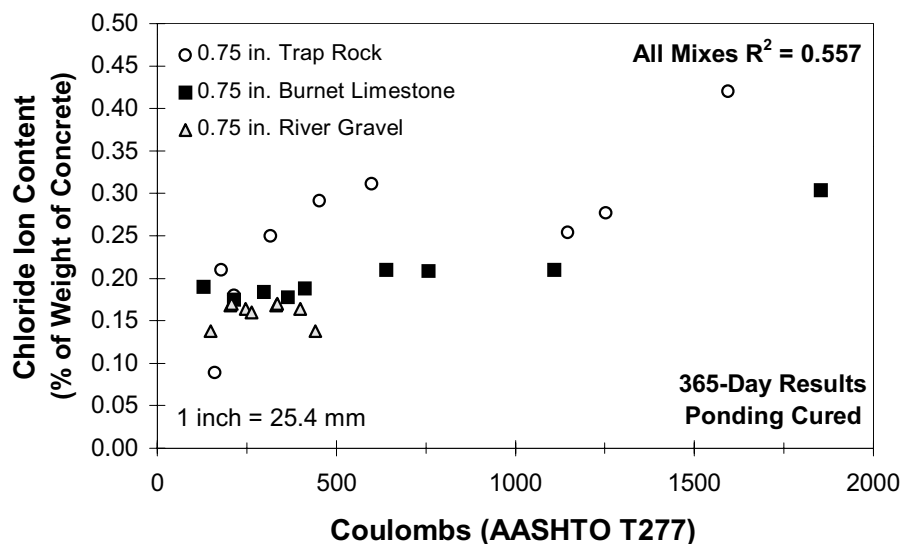


Figure 10.13: Aggregate Type Correlation Data at 365 Days
Total Chloride Ion Content by Weight—1.6 mm to 12.7 mm

10.3.4.4 Effect of Fly Ash Replacement on the Results of the Two Tests. The effect of fly ash replacement on the permeability of HS/HPC was also investigated using both tests after the 365-day ponding cycle. Figure 10.14 illustrates the results of both AASHTO T277 and T259. Clearly, as the Class C fly ash replacement was increased, both the chloride content and coulombs passed decreased.

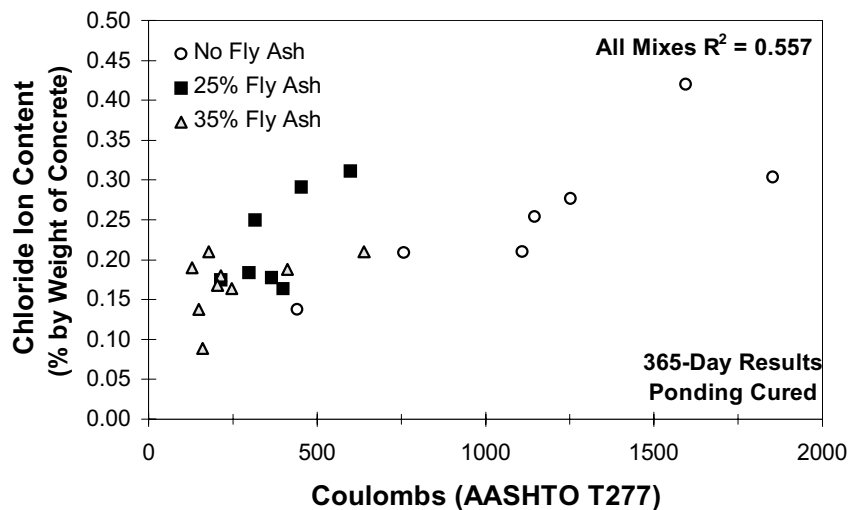


Figure 10.14: Fly Ash Replacement Correlation Data at 365 Days
Total Chloride Ion Content by Weight—1.6 mm to 12.7 mm

Based on the RCPT and ponding test data presented, it may be concluded that the ASTM Class C fly ash reduced the permeability of the concrete at both 90 days and 365 days.

10.4 Summary

Permeability of high-strength concrete, which is considered to be a good indicator of its durability and long-term performance, has been the topic of a large number of research studies and investigations. The primary reason for this concern is that permeability has been shown to directly affect most of the durability problems of concrete. The purpose of this study was to investigate the correlation between AASHTO T277 and AASHTO T259 for HS/HPC and to determine which constituents and percentages of materials affect its permeability. Thirty-one high-strength concrete mixtures were cast to investigate the influence of varying several constituents within the mix design. The variables included entrained-air content, coarse aggregate type, and length of moist curing. The water-to-cementitious material ratio varied between 0.28 and 0.33 and the 28-day compressive strength varied between 48.3 and 93.1 MPa (7,000 and 13,500 psi). All mixtures were tested for compressive strength, splitting tensile strength, elastic modulus, and permeability. The permeability was evaluated using both the Rapid Chloride Penetration Test (AASHTO T277) and the 90-day ponding test (AASHTO T259). The conclusions drawn from this investigation are summarized in Chapter 12 of this report.

CHAPTER 11. CONSTRUCTION AND FABRICATION-RELATED ISSUES AND CONCERNS

11.1 Introduction

Throughout the design and construction phases of the Louetta Road Overpass and North Concho River Overpass, many production, fabrication, and construction issues arose, many of which were specific to the use of HPC. Several of these issues were discussed at the prebid and preconstruction meetings, while some issues were addressed during fabrication and construction of the bridges. The general concerns or issues are separated into two primary categories; fabricator and contractor-related issues. Many of these issues need to be addressed by the owner, fabricator, concrete producer, and contractor should HPC be specified for use in a bridge project.

The precast/prestressed beam fabricator was faced with three primary issues related to the use of HPC:

1. Precasting bed capacity
2. Revised QC/QA program and practices for HPC
 - ◆ QC/QA testing equipment capability for HPC
 - ◆ Match-curing technology
 - ◆ accelerated heat curing
3. Transportation/shipping of long-span members
 - ◆ lateral stability and rotation concerns

Because the U-beams and AASHTO Type IV beams required very high prestressing forces (4 million lb maximum) to maximize the advantage of the HPC, the fabricator was required to either retool the stressing hardware, completely redesign the casting bed, or redesign the members. In the case of the U-beam, the fabricator elected to redesign the casting bed to accommodate the U-beam shape that was recently developed by TxDOT. In the case of the AASHTO Type IV beams, the fabricator performed a redesign of the member that included a two-stage stressing operation of partial prestress and partial post-tensioning to satisfy the required stressing force. The fabricator was also faced with QC/QA issues related to the use of HPC, including testing equipment, casting procedures, and testing procedures and practices. Finally, the fabricator was faced with the transporting and shipping of these long-span members that posed lateral stability and rotation concerns. The routes to the bridge sites also had to be verified for the increased turning radius required for the trucks transporting the longer members. Several

of these structural issues are discussed in greater detail by Gross [136], while two of these QC/QA issues are discussed in greater detail in Section 11.2.

The contractor was faced with two primary issues related to the use of HPC:

1. Erection of long-span members
2. Additional placement and curing concerns
 - ♦ control of concrete temperatures (cooling of stock pile, cooled mixing water, and ice replacement)
 - ♦ environmental conditions (RH, wind speed, and ambient temperature)
 - ♦ fogging of concrete decks, wet mat curing, etc.

The contractor was faced with the erection of the long-span members. Because some of the members were 50 percent longer and, therefore, 50 percent heavier than traditional precast beams, the contractor was required to utilize cranes with significantly larger capacity to erect and support the overturning moment and higher axial load of the members. The contractor and concrete producer were also required to address additional placement and curing concerns as discussed in Section 10.3.

11.2 Production of U-Beams—Fabrication-Related Issues

11.2.1 Casting Sequence of the U-Beams

Because of the unique shape of the U-beams, special attention was required during casting of these members. A high coarse aggregate content was selected for placement reasons and to satisfy the elastic modulus requirements. The fabricator elected to use a continuous inner steel form that required the concrete to flow across the bottom flange and up the adjacent web to avoid consolidation and possible air voids in the bottom flange. The casting sequence is illustrated in Figure 11.1.

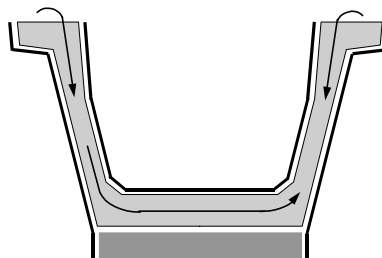


Figure 11.1: U-Beam Casting Sequence

The natural tendency for concrete is to flow in the path of least resistance, which would naturally be down the strands rather than across the strands. This natural tendency would result in air voids in the bottom flange. It was critical to develop a mix design that would create a bulkhead during placement of the concrete. Sandy mixes or mixes with lower coarse aggregate contents would not be able to create a bulkhead. The final mix design developed in the field trial mixes used a slightly higher aggregate content to accomplish this. Concrete was placed from the top web (Figure 11.2), vibrated down the web and across the bottom flange (Figure 11.3). Form and handheld vibrators were used to consolidate the concrete and to prevent voids and honeycombing, as illustrated in Figures 11.3 and 11.4. Wet mats were placed prior to covering the U-beams until release, as illustrated in Figure 11.5.



Figure 11.2: Placement of U-beam Concrete



Figure 11.3: Internal Vibration of Concrete for U-beam



Figure 11.4: External Vibration of Concrete for U-beam



Figure 11.5: Wet Mat Moist Curing of U-beam

11.2.2 Cold-Weather Casting of the U-Beams

Owing to the high content of cementitious materials and the resulting high heat of hydration generated during casting of these members, accelerated heat curing or steam curing during cold-weather concreting was not required during casting to attain release strength requirements. However, steam curing was selectively used during three casting dates when ambient temperatures approached freezing. Concerns related to thermal shock of the member arose during cold-weather casting because it was realized that the temperature differential between the beam and ambient conditions could be as high as 54.4 °C (130 °F). By the same

token, the contractor and researchers wanted to avoid traditional steam curing that would have had a detrimental effect on the later-age strength gain of the members. Therefore, the steam curing was not used in a traditional sense (up to 73.9 °C [165 °F]) to enhance early-age strength gain, but rather to prevent heat loss from the member and curb thermal shock concerns. The temperature of the steam curing used on these dates varied between 32.2 and 48.9 °C (90 and 120 °F) and was selected to insulate the member or provide a “buffer zone” between the member and ambient conditions so that thermal shock would not be an issue. This “light” steam curing was used until the concrete temperature within the member approximately reached equilibrium (± 16.7 °C [30 °F]) with the external environment. By this procedure, heat loss from the member was prevented and there was not a detrimental effect on the later-age strength gain. Figure 11.6 illustrates a U-beam during cold-weather casting.



Figure 11.6: Light Steam Curing of U-Beam during Cold-Weather Casting

It is recommended that for fabrication of the HPC U-beams in Texas, cold-weather steam-curing temperatures be limited to a maximum of 48.9 °C (120 °F) to prevent burn out of the mix design based on the temperature profiles monitored. Alternatively, if the fabricator has the capability to monitor the concrete temperature at midheight location of the web during casting, a recommendation can be made to limit steam curing so that the temperature at the web location (U-beam) does not exceed 54.4 °C (130 °F). This recommendation would be 60 °C (140 °F) for the HPC AASHTO Type IV beams. If these recommendations are followed, the steam curing will not have a detrimental effect on the long-term strength gain. These recommendations are specific to HPC mix designs in excess of nine sacks of cementitious materials.

11.3 Construction of Cast-In-Place HS/HPC Decks

Because of the unique nature of HPC, incorporating chemical and mineral admixtures, special attention is required during casting of these decks. Many HS/HPC deck mixes utilize low w/cm ratios in the 0.30 to 0.35 range where minimal bleed water or none occurs from the mix design during casting. When this limited amount of bleeding occurs in conjunction with high temperatures, high wind speeds, and low relative humidity, plastic shrinkage cracking of the deck is a serious concern. Plastic shrinkage cracks form while the concrete is plastic and when the evaporation of the free surface moisture occurs. This concern may be addressed by fogging of the deck during placing, finishing, and curing of the concrete while the concrete is plastic, as illustrated in Figure 11.10. This procedure prevents the evaporation of surface moisture and avoids plastic shrinkage cracking. In addition, membranes such as a curing compound may be applied to the surface after finishing to help prevent surface moisture loss. After the concrete hardens, curing of the decks becomes a primary concern to enhance the long-term durability performance and strength gain characteristics of the concrete.

For HPCs that include mineral admixtures, moist curing is even more critical than in conventional concretes for the development of the long-term strength gain characteristics required of pozzolans. Many publications [2] recommend a minimum of 7 days of moist curing for conventional concrete and suggest even longer periods of time (2 weeks) for concretes with pozzolans. In many cases, as described in Chapter 2, sufficient attention is not paid to the post-casting procedures and conditioning of the decks. Contractors are often more concerned with construction schedules and minimizing costs. This is one area in which specifications should clearly dictate recommended practices for postcasting procedures and conditioning of the concrete. The owner should also always hire an inspection agency to verify compliance with the specifications. After all, a 3-day period of moist curing will only allow the concrete to reach 75 to 80 percent of the potential 28-day strength of what can be achieved with continuous curing [2].

The following recommended procedure for the construction of HS/HPC bridge decks addresses the aforementioned issues. This procedure was followed for the casting of the Louetta Road Overpass and North Concho River Overpass and is in conformance with the current TxDOT specifications for casting of bridge decks.

1. Precast panels and CIP deck steel are placed (Figure 11.7).



Figure 11.7: CIP Deck Prior to Placement of Concrete

2. Prior to placement of the CIP deck, the precast panels are saturated to prevent loss of mix water while the concrete is in the plastic state (Figure 11.8).



Figure 11.8: Saturation of Deck Panels Prior to Placement of Concrete

3. Concrete deck is placed via concrete bucket or pump. The concrete is internally vibrated to provide proper consolidation and avoid internal segregation (Figure 11.9).



Figure 11.9: Placement of Deck Concrete and Internal Concrete Vibration

4. Fogging of the concrete deck while it is in the plastic state (as required, based on environmental conditions) during placement and finishing of the deck is performed throughout to avoid surface moisture evaporation and plastic shrinkage cracks (Figure 11.10). This procedure is particularly important for HPC mix designs with low w/cm ratios in which very little bleed water or none comes to the surface during placement.



Figure 11.10: Fogging of Concrete throughout Placement and Finishing Operation of Concrete Deck Construction

5. Concrete is distributed by the mechanical spreader (Figure 11.11).



Figure 11.11: Mechanical Spreader Distributes Concrete

6. A final troweled finish is applied (Figure 11.12) followed by the tining for enhanced abrasion resistance (Figure 11.13).



Figure 11.12: Trowel Finish Applied to Fresh Concrete Deck

7. A curing compound is applied (Figure 11.13) in addition to the continuous fogging (as required) to prevent moisture loss from the surface. This method may be considered a “belt and suspenders” approach or technique to avoid moisture loss from the surface. This approach is recommended for HS/HPC because it has been

documented that many curing compounds do not provide a continuous surface membrane due to the roughness of the concrete surface.



Figure 11.13: Placement of Tining and Curing Compound

8. Wet mats are placed (Figure 11.14). The mats should be placed and continuously moist for a minimum of 10 days after casting for HS/HPC decks with pozzolans. Polypropylene sheeting may be placed on top of the wet mats to prevent moisture loss from the mat. This procedure is particularly important for HS/HPC mix designs that incorporate fly ash because the pozzolanic reaction does not occur until 48 to 72 hours after casting. The additional moist curing provided improved later-age strength gain and durability as discussed in Chapter 2.



Figure 11.14: Placement of Wet Mats

11.4 Concrete Cracking-Related Issues

Concrete deck cracking has long been a concern for bridge decks because the existence of cracks can have a significant effect on the long-term durability performance of the decks. Most experts [139] agree that concrete cracks allow chloride, oxygen, water, and carbon dioxide to reach the steel and, therefore, decrease the time to corrosion. Depending on the orientation, length, depth, and location relative to the reinforcing steel, the impact varies on the time to corrosion and the severity of the associated corrosion. Cracks that are transverse to the steel expose only a small area of steel to corrosive conditions, while longitudinal cracks expose large lengths of steel to corrosive conditions, leading to extensive corrosion and spalling of the concrete cover. Even on the microstructure level, cracks can influence the long-term durability performance, as investigated and described in Section 8.3.3. Concrete cracking may be classified as an even larger concern in some respects for HPC decks because many HPC decks utilize concrete mix designs that have been optimized to provide an impermeable barrier between the steel and corrosive conditions. The weak link created by a concrete crack at the surface eliminates this barrier on the local level of the crack length.

Several inspection trips to the Louetta Road Overpass in Houston and the North Concho River Overpass in San Angelo were conducted by CMRG personnel after the bridges were completed to inspect and map crack patterns that were observed on the surface of the decks. The following sections detail and discuss any typical patterns observed for the two projects and provide recommendations to avoid the formation of similar crack patterns in future HPC bridges.

11.4.1 Cracking of Cast-in-Place Decks—Louetta Road Overpass

The inspection and mapping of crack patterns for the Louetta Road Overpass CIP decks were performed on two occasions in June and July 1998, approximately 19 months after the decks were cast, in conjunction with CTR research project 7-3993, which involves the long-term monitoring of the Louetta Road Overpass. Similar crack patterns were noted for both the southbound and northbound bridge decks. However, the extent of longitudinal cracking appeared to be more severe in the southbound HS/HPC decks. The southbound HS/HPC decks had span lengths similar to those of the northbound HPC decks, but had significantly wider beam spacings, resulting in a more flexible system. The crack widths varied between 25 and 30 mil on the surface of the concrete deck. Crack widths were similar for both mainspans.

The crack patterns that occurred may be classified into three categories. The first is the typical “staircase” pattern, which occurred at all of the interior bent lines. The configuration of the overpass and intersecting Louetta Road required all of the bent lines to be constructed on a skew, which contributed to the extent of these cracks. The second category of cracking that occurred may be classified as interior transverse cracks. These interior transverse cracks were located above U-beam diaphragms, between precast deck panels, and at the precast deck panel-thickened CIP deck interface. Finally, the third category of cracking that occurred may be classified as longitudinal cracks. These longitudinal cracks occurred primarily along the interior and exterior face of the upper U-beam flange and precast deck panel interface. Figure 11.15 schematically illustrates these typical crack patterns mapped at Bent Line 2 of the southbound mainspans.

The following sections discuss the typical crack patterns in greater detail and suggest recommendations for preventing similar crack patterns from developing on future projects.

The term *staircase* is used to describe the first category of cracking because the cracks step or zigzag along the width of the bridge deck. They occurred between U-beams at the interior bent supports and along the skew between beam centerlines. Figure 11.16 illustrates the deck cracking between U-beams at the support.

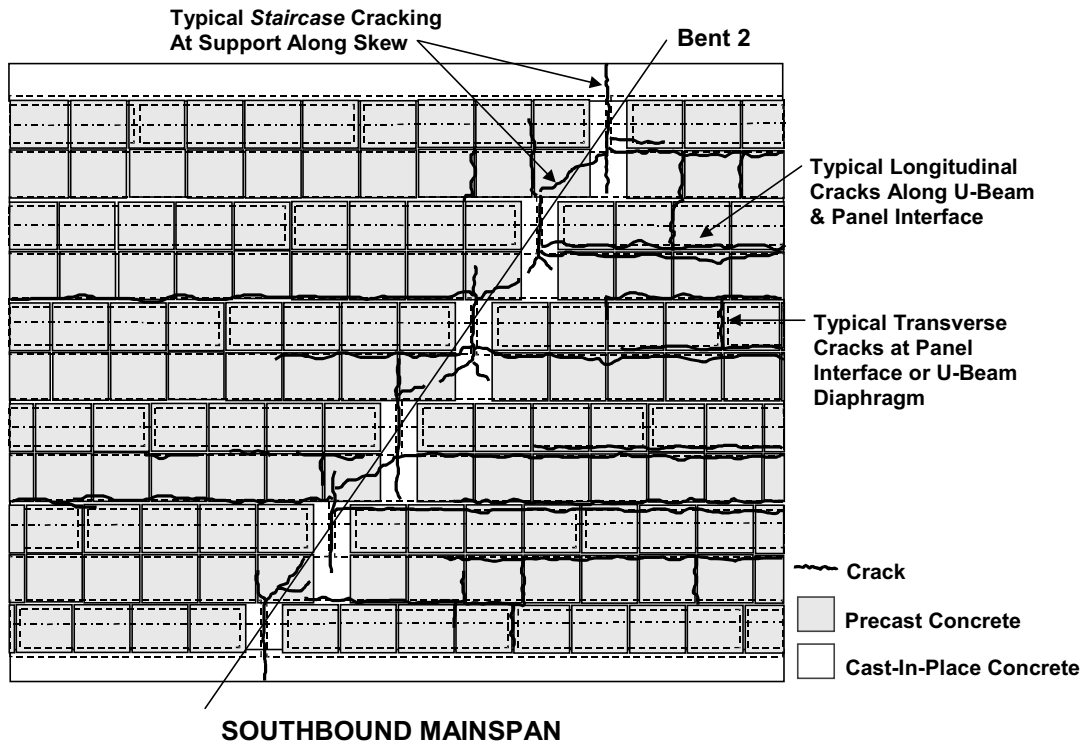


Figure 11.15: Typical Crack Patterns for Louetta Road Overpass (Southbound Mainspan at Bent Line 2)

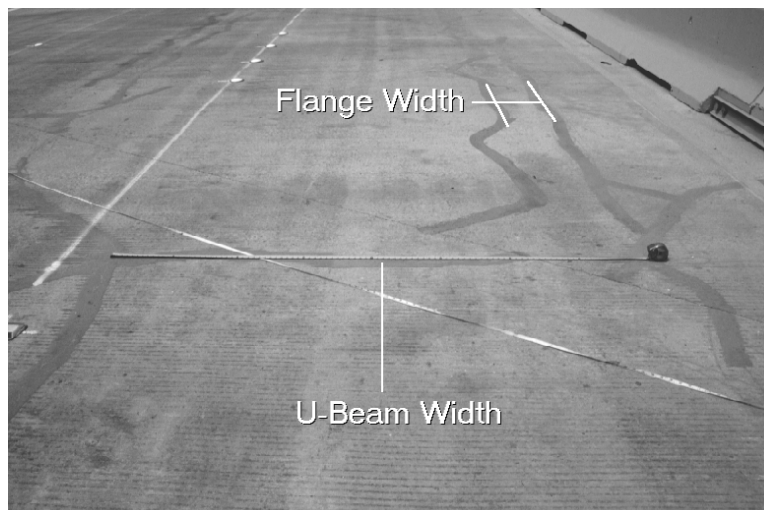


Figure 11.16: Louetta Road Overpass Bridge Deck Cracking at Interior Bent Support

The cracks are transverse over the width of the U-beam (2.44 m [8 ft]) and then continue to propagate or follow the bent skew between U-beam centerlines. The deck cracking is full depth, as illustrated in Figures 11.17 and 11.18. Figure 11.18 illustrates how the crack pattern follows the skew between U-beam centerlines. It may be noted in Figures 11.17 and 11.19 that

the contractor did not properly locate the tooled control joint at the centerline of the skew. A properly located control joint might have reduced the level of cracking between U-beam centerlines, but would not have prevented the cracks from forming between U-beams at the support (Figure 11.16). The pattern of cracking may be classified as shrinkage cracks. The location and extent of these cracks were caused by the significant change in stiffness and the geometrical layout of the bridge. To control the location and extent of these staircase cracks at the bent line, the designer could have required a skewed end block and skewed deck panels at all interior bent locations. The resulting configuration of the beam and deck panels would have resulted in a change in stiffness or discontinuity directly at the centerline of the skew.



Figure 11.17: Blow-Up Detail of Deck Cracking between U-Beams at Interior Bent Support



Figure 11.18: Deck Cracking at Skew between Mainspan U-Beams

In addition to the staircase cracking, interior transverse cracks were located above U-beam diaphragms, between precast deck panels, and at the precast deck panel-thickened CIP deck interface. In addition, longitudinal cracks occurred primarily along the interior and exterior face of the upper U-beam flange and precast deck panel interface. Figure 11.20 illustrates these typical crack patterns that occurred in the deck.

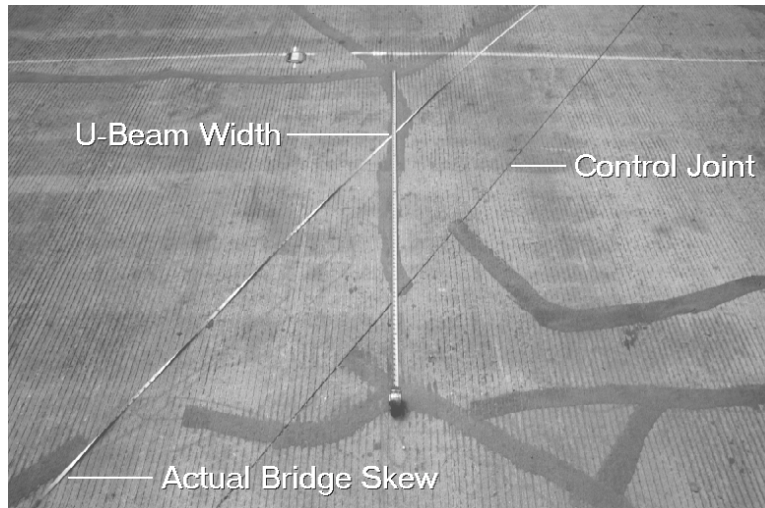


Figure 11.19: Misaligned Skewed Control Joint at Interior Bent

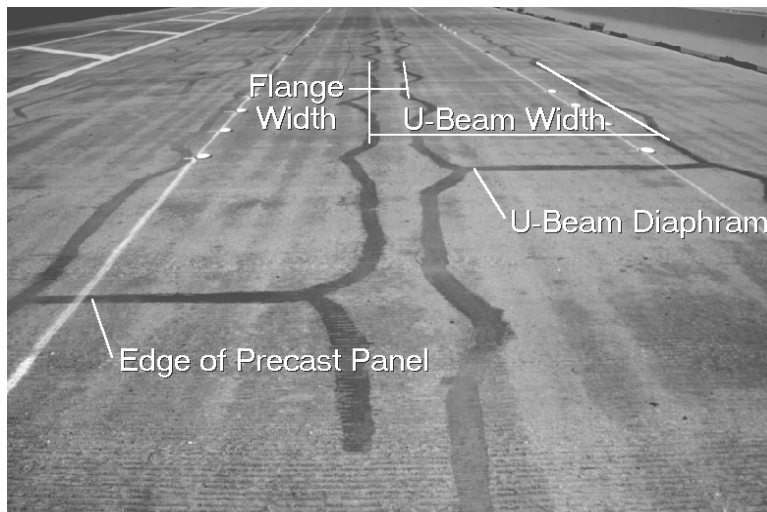


Figure 11.20: Longitudinal and Transverse Interior Cracking

These cracks were induced because of a combination of a change in relative stiffness (discontinuity), the structural configuration of the bridges, and a significant variation in the coefficient of thermal expansion between the deck panels and CIP deck. The combination of wide beam spacings, long span supporting elements, and a relatively thin deck resulted in a more flexible structural system, compared with those in conventional bridges. The flexibility of the system undoubtedly contributed to these cracks, as did the discontinuities and thermal effects.

Figure 11.21 illustrates the coefficient of thermal expansion (CTE) by aggregate types for the primary mix designs used in the two bridge projects. In particular, note the difference in the CTE between the Louetta Road Overpass jobsite concrete (CIP deck concrete) and the Louetta Road Overpass panels (precast deck panels). The CTE of the precast panels is substantially higher, so that as the panels try to expand, they are restrained by the CIP portion of the deck. The variation in the coefficients of thermal expansion undoubtedly contributed to the Louetta Road Overpass deck cracking.

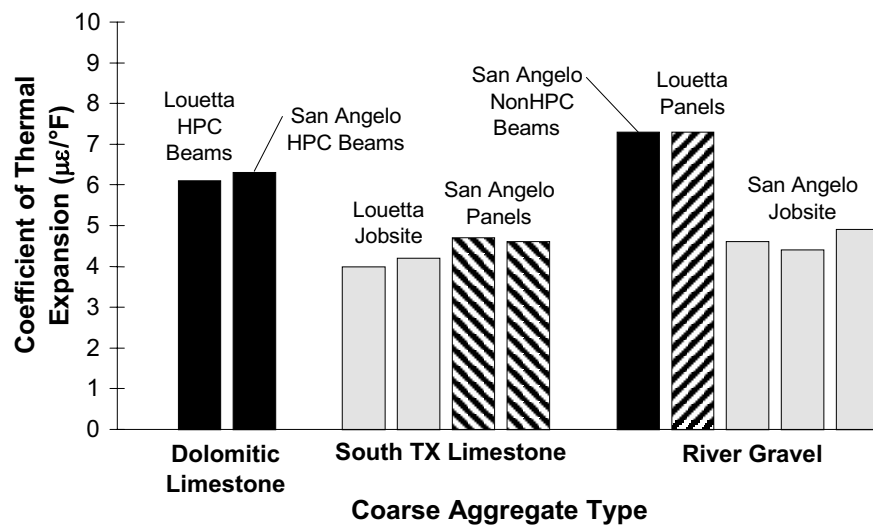


Figure 11.21: Coefficient of Thermal Expansion by Aggregate Type and Source [136]

While selecting materials with similar coefficients of expansion will help prevent the formation of such cracks, it will not eliminate the potential completely. Designers must also pay close attention to the flexibility and configuration of the bridge. This is particularly important for HPC bridge systems that are often optimized for span length, beam spacing, and member efficiency. It may be noted that the level of cracking observed for the Louetta Road Overpass was significantly less for Span 1 compared to Spans 2 and 3 of the northbound and southbound decks. Span 1 had a span length of 37.0 m (121 ft-6 in.) while Spans 2 and 3 had span lengths of 41.3 m (135 ft-6 in.) and 40.8 m (134 ft-0 in.), respectively. Span 1 was less flexible than Spans 2 and 3 and the resulting deck cracking was less severe. It should also be noted that none of the crack patterns discussed herein were present while the concrete was in the plastic state.

It may be concluded that the crack patterns observed in the decks were not related to the HPC mix designs per se, but rather to a combination of the aggregate sources and the structural configuration of the bridge. Long-term monitoring of the Louetta Road Overpass is currently ongoing under CTR research project 7-3993. This project is cosponsored by the FHWA and TxDOT through the year 2000. Under this research study, deck cracking continues to be

monitored on an annual basis. The impact and effect of thermal gradients and workmanship on the Louetta Road Overpass deck cracking will be discussed thoroughly under this research study.

11.4.2 Cracking of Cast-in-Place Decks—North Concho River Overpass

After a visual inspection of both the Westbound and Eastbound bridge decks, only one *typical* location was observed for deck cracking. This occurred at two locations along Bents 7 and 9 of the westbound bridge where continuous deck construction occurred (Table 6.71). At no other location on either the westbound or eastbound bridges was continuous construction used. Spans 6 and 7 and 8 and 9 of the westbound deck were cast as single-pour with continuous construction. These cracks involved the Class S NSC. No deck cracking was observed in the Class K HS/HPC eastbound decks or the Class S Modified HPC (westbound spans 1 to 5) decks. Figure 11.22 schematically represents where these “typical” crack patterns occurred at Bent locations 7 and 9.

At a location of continuous construction, the TxDOT typical detail calls for a tooled control joint at the centerline of the bent. The recommended detail to control the extent and location of shrinkage cracks would be a dual tooled control joint or dual continuous zip strips on either side of the bent centerline directly above the edge of the precast deck panel where the discontinuity occurs. A second option would be to extend the precast deck panels to the centerline of the bent. This would locate the discontinuity directly above the bent line. However, this option would reduce the composite deck action that occurs at the bent line as a result of the pier cap stirrups. The cracks may be attributed to shrinkage-related stresses at a location where a discontinuity occurs. The variation in CTE for the North Concho River Overpass precast panel and CIP deck concrete were of the same magnitude (Figure 11.21), suggesting that thermal-related stresses played a minor role in the deck cracking.

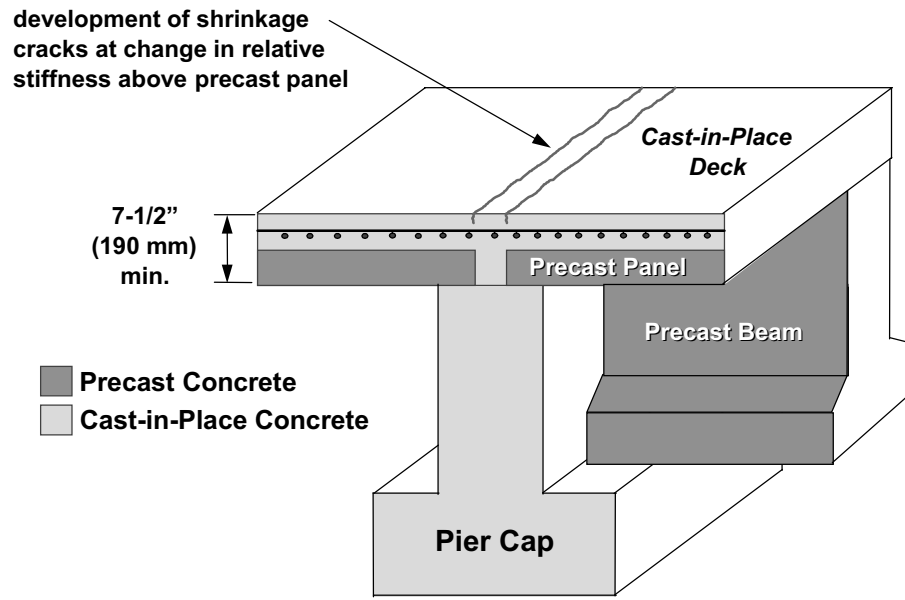


Figure 11.22: Schematic Representation of Deck Cracking for the North Concho River Overpass Bridge Cast-in-Place Deck

CHAPTER 12. SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

12.1 Summary

HPC is becoming more widely utilized as designers, contractors, and producers become more familiar with its production and property performance. Strength levels, which were thought to be impossible to obtain 15 years ago, are now commonplace. Today, high performance concrete is being considered for use in a wide variety of applications, including bridges. With the use of HPC comes concerns related to concrete properties, mix design performance, QC/QA, the applicability of current code requirements and construction practices, and the predictability of short- and long-term performance. To investigate many of these concerns and to promote the use of HPC within the infrastructure, the FHWA initiated a series of HPC showcase bridge projects in 1993. Two of these projects, the Louetta Road Overpass in Houston, Texas, and the North Concho River Overpass in San Angelo, Texas, were investigated within the scope of this study.

The results of this investigation illustrated the importance of material selection and material interaction in the optimization of HPC mix designs, both from the mechanical and material performance standpoint. Laboratory studies investigated the impact of aggregates, temperature, air entrainment, mineral admixtures (fly ash), and chemical admixtures on the performance of HPC. Furthermore, various curing regimes were investigated to examine the impact of curing on HPC properties and note the variation between concrete most representative of the structure and the “standard” quality control specimens. Quality control/quality assurance issues were also investigated including mix design performance in the field, the use of match-curing technology, and the application of the maturity method for HPC bridge decks.

During the course of this investigation various mechanical and material properties of high performance and conventional concrete were investigated. As discussed in Chapter 3, the Strategic Highway Research Program initiated a study to develop an HPC definition based on performance. Tables 12.1 and 12.2 summarize the HPC grade performances for the Texas HPC bridge components. Tables 3.2 and 3.3 may be referenced for the definition of the performance characteristics and test methods.

It is clearly evident from the Texas bridge data that the performance characteristic grade for a particular mix design varied depending on the characteristic for which the mix design was optimized. For example, strength was not particularly critical for many of the bridge decks. However, chloride permeability was important in order to extend the time-to-corrosion for the bridge decks and to reduce the long-term maintenance costs. While strength may have resulted in a Grade 1 performance

classification based on the design requirements, chloride permeability of the mix design was optimized to satisfy Grade 2 requirements as part of the project objectives. Rather than using traditional methods, such as manipulating the w/cm ratio to improve the overall performance of the concrete (including the impermeability), a specific performance characteristic — such as permeability — can be optimized to satisfy specific project requirements. Using this performance grade system, a pozzolan replacement could be selected to meet chloride permeability requirements rather than a reduced w/cm ratio (increase in cement), which is how many specifications would approach satisfying a durability requirement. The objective of a reduced permeability would be satisfied with the pozzolan; moreover, the mix design would be more economical and provide the concrete producer with greater flexibility than could be achieved by simply reducing the w/cm ratio. The methodology of using performance grades for defining a high performance concrete is advantageous to all parties involved, including the designer and concrete producer.

Table 12.1 HPC Performance Grades for the Louetta Road Overpass

Performance Characteristics	Northbound				Southbound			
	CIP Deck	Deck Panels	Beams	Substr.	CIP Deck	Deck Panels	Beams	Substr.
Freeze-Thaw Durability	②	③	③	③	②	③	③	③
Scaling Resistance	②	③	③	③	②	③	③	③
Abrasion Resistance	②	③	③	③	②	③	③	③
Chloride Penetration	2	2	3	3	2	2	3	3
Strength ①	N/A	2	2-3	3	2	2	2-3	3
Elastic Modulus	1	1	2	2	1	1	2	2
Shrinkage ④ [136]	3	3	3	3	3	3	3	3
Creep ④ [136]	2	4	3	4	2	4	3	4
① Strength Grades are based on design requirements. Grades for other performance characteristics are based on test results. ② Testing is in progress and associated with the Louetta Road Overpass widening project. The research study is associated with Center for Transportation Research Project No. 9-3993. ③ No testing planned or performed. ④ Report by Gross [136] details curing and evaluation criteria that differed from standard ASTM procedures.								

Table 12.2 HPC Performance Grades for the North Concho River Overpass

Performance Characteristics	Eastbound				Westbound Span 1-5		Westbound Span 6-9	
	CIP Deck	Deck Panels	Beams	Substr.	CIP Deck	Deck Panels	CIP Deck	Deck Panels
Freeze-Thaw Durability	2	②	②	②	2	② ③	2	② ③
Scaling Resistance	1-3	②	②	②	3	② ③	1	② ③
Abrasion Resistance	1-2	②	②	②	1	② ③	1	② ③
Chloride Penetration	3	2	3	3	1	< 1	< 1	< 1
Strength ①	1	1	1-4	1-2	N/A	N/A	N/A	N/A
Elastic Modulus	1	1	2	1-2	1	1	1	1
Shrinkage ④ [136]	3	2	3	②	2	2	2	2
Creep ④ [136]	3	1	4	②	< 1	1	2	1
① Strength Grades are based on design requirements. Grades for other performance characteristics are based on test results. ② No testing planned or performed. ③ Non-HPC (Conventional Concrete) ④ Report by Gross [136] details curing and evaluation criteria that differed from standard ASTM procedures.								

12.2 Conclusions

Conclusions have been formulated based on the results of the various investigations conducted during this research study. They have been separated into six primary subsections. These include Phases I and II of the research program, as well as four related studies that comprise Phase IV of the research program. Phase III of the research program involves an in situ durability study to compare the long-term performance of various concretes and steel deck reinforcement. A durability performance study of the North Concho River Overpass may be conducted in 15 to 20 years to assess the durability performance of the various mix designs and steel reinforcing under field conditions. After that time, final recommendations and conclusions related to this investigation may be drawn.

12.2.1 Conclusions — HPC Trial Mixes and Laboratory Studies

Based on test results obtained from Phase I of the laboratory investigations, the following conclusions were drawn:

Coarse Aggregate Effects — Aggregate Replacement Study

1. Smaller maximum size aggregate gradations yielded higher compressive strengths (4.5 percent on average). The concrete tended to behave as a more homogenous mixture.
2. Concretes incorporating crushed river gravels exhibited lower compressive strengths. This was caused by poor mechanical bond resulting from the river gravel shape and texture.
3. Aggregates that are significantly harder than the mortar (e.g., trap rock) caused stress concentrations at higher stress levels, introducing microcracking in the transition zones. Compatibility of the aggregate and mortar matrix moduli resulted in optimal strength performance for a particular mix design with the same mixture proportions.
4. Mineralogical characteristics of the coarse aggregates influenced the mechanical properties of the concrete. Lower modulus and flexural strength resulted for concretes incorporating a high abundance of calcite (calcitic limestone).
5. Increasing aggregate content resulted in a lower flexural strength for a given aggregate size.
6. The elastic modulus appeared to be independent of aggregate size for a given aggregate content. Aggregate type and aggregate content primarily influenced the elastic modulus for high-strength/high performance concretes.

Temperature Effects — Temperature-Related Study

1. HPC specimens subjected to accelerated heat-curing temperatures generated higher early strengths (56 percent on average), but lower long-term strengths (25 percent on average). In general terms, the higher the accelerated heat or steam curing, the higher the early-age strength and the lower the later-age strength.
2. The use of Class C fly ash replacement improved the later-age compressive strength and elastic modulus of the HPC over those for similar mix designs without fly ash when all were subjected to accelerated heat curing. The fly ash replacement also reduced the high heat of hydration associated with many HPC mixtures. Excessively high heat of hydration can have an adverse effect on both mechanical and material property performance at later ages, as described in Section 12.2.3.
3. Accelerated heat curing also improved the elastic modulus of the HPC at early ages, but resulted in a reduced elastic modulus at later ages when compared with nonaccelerated heat-cured specimens.

4. When subjected to accelerated heat curing, the majority of the elastic modulus (95 percent on average) for HPC is developed within 3 days after casting.

Fly Ash Effects — Permeability-Related Study

1. The ASTM Class C fly ash reduced the cost of the cementitious material and resulted in improved mechanical and material property performance. Improvements included greater strength and elastic modulus and reduced concrete permeability.
2. The rapid chloride permeability test results exhibited the formation of bands or regions of permeability performance for concrete mix designs with and without fly ash.
3. The results illustrated that a water-to-cementitious ratio can no longer be used effectively to predict the durability or permeability performance of a concrete mix design with the use of today's mineral and chemical admixtures. Varying degrees of permeability performance (from low to high) at any single w/cm ratio were exhibited, with the extent of the variability depending on the mixture constituents and characteristics.

TxDOT Replacement Study for Non-HRWR HPC Mixes

1. Optimized mix designs can be developed to improve mechanical and material performance at a reduced cost when compared with current TxDOT Standard Class S and C concrete specification requirements. This can be done through the utilization of an ASTM Class C fly ash. These optimized mix designs resulted in a cementitious material cost saving ranging from 20 to 50 percent. The use of fly ash also improved pumpability and finishability characteristics.
2. A performance-based specification system is recommended as an alternative to the current prescriptive specification system. The potential cost savings for the concrete producer would provide incentive or motivation to stimulate the use of an alternate performance-based system. The performance-based system would also allow the concrete producer to more directly optimize mix designs for a specific durability criterion, which could be specified in the alternate system. Such an approach represents an alternative to the existing methodology involving the indirect prescriptive w/cm ratio requirement to address concrete durability.

12.2.2 Conclusions — Performance-Related Test Results of Plant-Produced Concrete

Based on test results obtained from Phase II of this study, the following conclusions were drawn:

1. All of the project's mechanical and material property specification requirements and objectives were satisfied.
2. The ASTM Class C fly ash replacement improved the long-term mechanical properties of the concrete and reduced the permeability of the concrete.
3. The ASTM Type F HRWR improved the early and long-term mechanical properties of the concrete.
4. No one empirical equation accurately represents the elastic modulus, flexural strength, or splitting tensile strength characteristics of an HS/HPC mix design. Any elastic modulus, flexural strength, or splitting tensile strength that is assumed, based on a function of the compressive strength for the design of HS/HPC members by the design engineer, should be verified through a series of field trial batches for the specific mix design or by the documented historical properties performance for that mix design.
5. In addition to satisfying concrete property specifications, allowances for unexpected occurrences during production must be anticipated owing to the nature of most high performance mix designs and placement requirements. This begins with close monitoring of a well-developed QC/QA program. Allowances for a reduction in mechanical properties should be accounted for when a laboratory mix design is used in the field. It is recommended that a field trial mix on a production-sized batch be conducted for all HS/HPC mix designs developed in the laboratory.
6. For precast/prestressed beams, the match-cured cylinders best represented the mechanical and material properties of the beams at early and later age.
7. HPC mixtures resulted in reduced ultimate creep and shrinkage. No significant effects owing to the high curing temperatures were observed.
8. Concretes without fly ash and HRWRs resulted in the largest variability between curing conditions (moist and field performance) for permeability, while concretes with fly ash and or HRWRs were significantly closer in relationship. This should be considered when specifying a permeability rating or classification within a performance-based specification. For example, a 2,000-coulomb rapid ion permeability requirement for an ASTM moist-cured cylinder may result in a significantly higher field permeability of 4,200 coulombs for conventional concrete without fly ash and without an HRWR. Similarly, for an HPC with fly ash, the resulting field permeability may be 2,500 coulombs, clearly within the same classification of performance as desired by the designer.
9. HS/HPC with a low w/cm ratio and HRWRs can be used successfully for cast-in-place bridge decks. These mix designs result in the highest mechanical properties and best

overall durability performance (material property) compared to conventional concretes and concretes without HRWRs. However, these mix designs do require that recommended placing, finishing, and curing practices be followed more precisely to avoid potential field-related problems.

10. The HS/HPC and HPC resulted in improved freeze-thaw resistance compared to conventional concretes. The HS/HPC and HPC have resulted in a higher durability factor and a reduced change in mass compared to conventional concretes.
11. Enhanced strength characteristics associated with the HS/HPCs had a minor influence on the overall freeze-thaw resistance of the concrete as long as the concrete incorporated sound materials and an adequate well-distributed air void system. The high strength associated with many HPCs resulted in slightly less visible surface damage, but similar freeze-thaw durability performance.
12. Higher strength concrete can be expected to result in reduced wear resistance for mix designs with similar constituents, assuming they are finished and cured under similar conditions.
13. Strength had little influence on the scaling resistance of the concrete. The concrete, which resulted in the lowest strength, actually exhibited the best scaling resistance. Rather than strength, the primary factors affecting scaling resistance are mixture characteristics, curing and curing procedures, air entrainment, aging effects, and sealers.

12.2.3 Conclusions — Early-Age Curing Aspects of Prestressed/Precast Beams

Based on the testing performed during the course of this study, the following conclusions were drawn:

1. Member-cured cylinders used to determine release of the prestressing strands for the U-beam and AASHTO Type IV beams underestimated the compressive strength of the member by as much as 27.9 percent (19.1 percent on average) when compared with the match-cured cylinders. Using the match-cured system would result in increased plant productivity.
2. ASTM moist-cured cylinders used to verify the design strengths for the U-Beam and AASHTO Type IV beams at 56 days overestimated the compressive strength by as much as 15.9 percent (9.8 percent on average) compared with the match-cured cylinders. Using the match-cured system resulted in a more accurate estimate of the actual mechanical properties of the concrete member.

3. Based on the results of the beam shapes investigated, and given the precast plant casting procedures and the HS/HPC mix design utilized, the match-cured system resulted in an inherent safety factor of 1.11 when the lowest peak hydration temperature location within the member was selected for monitoring. It should be emphasized that this inherent safety factor is specific to the mix design used, to the beam shapes investigated, and to the casting procedures followed. If the level of the inherent safety factor is of a primary concern to the fabricator/specifier, a trial casting should be made from the mix design to be used in production.
4. Match-curing technology increased plant productivity and allowed for the monitoring of the concrete maturity to more accurately estimate concrete strength at release. Release strengths were attained 2 hours sooner on average with the match-cured cylinders compared with the standard QC/QA member-cured cylinders. This is based solely on the HS/HPC mix design used in production of the HS/HPC precast beams. For conventional precast concrete mix designs and members with larger SA/V ratios, the increase in plant productivity will not be as significant owing to the lower hydration temperatures generated during casting.
5. HPC hydration temperatures that exceeded 76.7 °C (170 °F) during casting detrimentally affected the concrete performance both from a mechanical and material performance standpoint. These detriments include reduced compressive strength, increased microcracking, and increased permeability at later ages. Higher hydration temperatures caused more extensive and wider cracking at the microstructure level. The increased microcracking precipitated the propagation of cracking in the transition zones and, thereby, resulted in failure of the cylinder at a lower compressive strength. A reduction in elastic modulus and increased microcracking were also noted above this temperature range.
6. For optimal mechanical and material property performance of concrete, specifications should address a maximum hydration temperature for precast/prestressed elements. This is particularly important for HPC components in which high heat of hydration is generated. To prevent detrimental temperature effects and increased microcracking, a maximum hydration temperature limitation in the temperature range of 71.1 to 82.2 °C (160 to 180 °F) is recommended. Alternatively, a limitation of 38.9 °C (70 °F) on the maximum temperature rise is also recommended.

12.2.4 Conclusions — Use of Maturity Method for CIP HPC Bridge Decks

Based on the testing performed during the course of this study, the following conclusions were drawn:

1. The location selected within the deck for monitoring of concrete maturity did not impact the concrete maturity values in any significant manner. For standard TxDOT bridge deck construction, including cast-in-place decks cast on precast panels, any convenient monitoring location is acceptable. However, as a rule of thumb, a middepth location in the cast-in-place deck above the precast panel is recommended as the *standard* location. This location generates the lowest heat of hydration and, consequently, the lowest concrete maturity values. It is thus the most conservative location that can be selected.
2. In all cases, the concrete maturity values generated in the field were on the lower bound or conservative side compared with the natural maturity curve of the concrete. The maturity variation between the bridge decks and the natural maturity curves ranged from 0.2 percent to 10.3 percent generally within the ± 10 percent associated with the accuracy of the evaluation method.

12.2.5 Conclusions — HPC Permeability Correlation Study

Based on the testing performed during the course of this study, the following conclusions were drawn:

1. The addition of air entrainment had no appreciable effect on the permeability of HS/HPC.
2. The length of moist curing for the HS/HPC did not significantly affect the results of AASHTO T277. Mixtures moist cured for 56 days exhibited slightly lower coulomb values than mixtures moist cured for 14 days and then air dried for 42 days. This difference was not significant and did not cause a shift between the permeability levels defined in AASHTO T277.
3. AASHTO T277 and T259 indicated that mixtures containing trap rock aggregate were the most permeable. Based upon the AASHTO T277 results, the river gravel mixtures displayed slightly larger coulomb values than the Burnet limestone mixtures. Based on the AASHTO T259 results, the Burnet limestone mixtures exhibited slightly higher total integral chloride values than the river gravel mixtures. Both tests however, indicated that the variation of the permeability resulting from changing the coarse aggregate type was minimal.

4. Both AASHTO T277 and T259 indicated that the use of a Class C fly ash replacement reduced the permeability of the HS/HPC mixes. Mixes that incorporated 35 percent fly ash replacement exhibited generally lower permeability than was observed with 25 percent fly ash replacement.
5. Both AASHTO T277 and T259 indicated that decreasing the w/cm ratio from 0.28 to 0.22 while keeping all other properties constant was not effective in decreasing the permeability of high-strength concrete.
6. There exists a linear regression correlation between AASHTO T259 and T277. For specimens cured under the same conditions, namely 14 days of moist curing followed by 42 days of air drying, the 90-day chloride content in percent by weight of concrete in the uppermost 12.7 mm (0.5 in.) exhibited an R^2 value of 0.71 when coupled with the corresponding coulomb values. The 90-day total integral chloride and the corresponding coulomb values exhibited an R^2 value of 0.65 with a 33 percent standard error.
7. When coulomb values of specimens moist cured for 56 days are coupled with total integral chloride values of specimens moist cured for 14 days and air dried for 42 days, the R^2 correlation coefficient decreases to 0.42, indicating a poor correlation.
8. When compared against data used in the original development report, the total integral chloride (I) results obtained during this study displayed a better correlation in the range of coulomb values used (400 to 2,000 coulombs). The current regression line was steeper, reflecting the fact that relatively large I values were recorded for low coulomb values.
9. When separated into groups by aggregate types, mixtures containing the river gravel aggregate indicated a very poor correlation with the AASHTO T277 and T259 results.
10. The original correlation between AASHTO T259 and T277 in the FHWA-81 report could not be used to accurately predict the permeability of HS/HPC. However, the RCPT can serve as a general tool to compare the relative permeability of similar mix designs with similar constituents and admixtures, in this case HS/HPC. An initial correlation to accurately predict permeability for mix designs developed for field use is recommended.
11. Results from the 90-day and 365-day ponding cycles were consistent with respect to aggregate type, air content, and fly ash replacement. Chloride ions for the HS/HPC mix designs were not encountered beyond a level of standard minimum specified clear cover for most structural components, 37 to 50.8 mm (1.5 to 2 in.), indicating very impermeable concrete with enhanced durability performance.

12.2.6 Conclusions — Construction- and Fabrication-Related Issues and Concerns

Based on the HPC fabrication and construction concerns that arose during construction of the bridges, the following conclusions were drawn:

1. A prefabrication meeting should be conducted to address precasting bed capacity requirements and fabrication options, as well as any revised QC/QA practices for the evaluation of HPC.
2. A preconstruction meeting should also be conducted to address special construction requirements and any additional placement and curing concerns associated with the HPC mix designs.
3. For HPC mix designs in excess of 9 sacks of cementitious materials, it is recommended that temperature limitations be set for accelerated heat or steam curing to prevent burn-out of the mix design. It is recommended that for fabrication of the HPC U-beams in Texas, cold-weather steam-curing temperatures should be limited to a maximum of 48.9 °C (120 °F) based on the temperature profiles monitored. Alternatively, if the fabricator has the capability to monitor the concrete temperature at the midheight location of the web during casting, a recommendation can be made to limit steam curing so that the temperature at the web location for the U-beam does not exceed 54.4 °C (130 °F). This recommendation would be 60 °C (140 °F) for the HPC AASHTO Type IV beams.

Based on the visual inspection of the in-place bridges and on the mapping of deck crack patterns during the course of this study, the following conclusions were drawn:

1. Deck cracking that occurred in the Louetta Road Overpass Bridge decks can be attributed to the geometrical layout of the bridge, the change in cross-sectional stiffnesses, the thin cast-in-place decks, and to the flexibility of the system. The cracks that developed are related to shrinkage and thermal stresses associated with the above constraints and not to the HPC mix designs.
2. Deck cracking that occurred in the North Concho Overpass Bridge decks can be attributed to a change in the cross-sectional stiffness at a location of continuous construction above a bent line. The cracks can be attributed primarily to shrinkage stresses. A dual control joint at continuous construction would better control the location of the shrinkage cracks at the precast panel-thickened cast-in-place deck transition location at the bent line.

12.3 Recommendations for the Application of HPC in Bridge Structures

The following recommendations are suggested to assist in implementing the conclusions made in this study:

1. **Selection of HPC Materials:** Interaction among mix constituents is as important as the water-to-cement ratio. The interaction between mineral admixtures such as fly ash and portland cement on strength is highly unpredictable and can only be determined by conducting laboratory and field trial mixes. To control high heat of hydration, mineral admixtures such as fly ashes can be utilized. Fly ash also benefits the later-age strength gain and finishability characteristics of the concrete. The use of a higher coarse aggregate content and a coarser gradation of coarse aggregates with a lower percent voids generally results in the production of a higher compressive strength concrete owing to a decreased water demand for a given workability. For precast/prestressed construction, smaller maximum aggregate size, which is often required owing to steel congestion, can improve aggregate-mortar bond characteristics. The shape and texture of the aggregate can also improve the strength characteristics (for example, dolomitic limestone has better bond characteristics than river gravel). The use of natural river sands with a fineness modulus in the range of 2.70 to 3.10 results in HPC mixtures with good workability. Avoid manufactured sands whose high absorption capacities can create problems in the consistent production of a mix design on a daily basis. The initial slump of the concrete should be 25 to 50 mm (1 to 2 in.) prior to the addition of an HRWR. This is particularly important because placement problems and plastic shrinkage cracking can result if the mix is initially too dry. Placement problems and poor strength gain characteristics can also result if the initial slump is too high.
2. **QC/QA:** Promote the use and incorporation of performance-based specification systems over prescriptive-based specification systems. Through this methodology, “engineered concrete” mix designs are more likely to result, ensuring a mix that has been optimized to satisfy a specific mechanical and/or material criterion. The resulting optimized mix designs represent a more economical use of materials.
3. **Concrete Temperature Controls for HPC Precast Beams:** Temperature control for HPC precast beams is recommended to prevent high hydration temperatures and excessive cracking at the microstructure level. A recommended maximum temperature control between 71.1 and 82.2 °C (160 and 180 °F) is recommended to prevent excessive microstructure cracking.

4. Match-Curing Technology as QC/QA: Promote the use of match-curing technology as a quality control/quality assurance tool for the precast industry. The system is beneficial to the fabricator because it increases plant productivity and allows for the monitoring of the concrete maturity to more accurately estimate concrete strength at release. Match-curing technology also results in a more accurate representation of mechanical and material properties, which allows the design engineer to more precisely predict short- and long-term structural performance. Recommendations for the incorporation of match-curing technology into a QC/QA program are presented in Section 8.4 of this report.
5. Concrete Temperature Controls for HPC Bridge Decks: Temperature control for cast-in-place HPC bridge decks is recommended to prevent excessively high hydration temperatures. The existing TxDOT requirement of 29.4 °C (85 °F) is appropriate for both HPC and conventional concrete.
6. Maturity for Cast-in-Place HPC Bridge Decks: Use of the maturity method for strength prediction is one option in lieu of traditional QC/QA methods. This method is particularly attractive when traffic access is required on short timetables for either new bridge decks or resurfaced bridge decks with high performance or conventional concretes. This method allows traffic to access the deck as soon as strength has been met.
7. Casting of HPC Bridge Decks: When casting concrete bridge decks during hot-weather, select casting times that correspond to the concrete material stock pile minimum daily temperature, usually from 5 to 6 a.m. in Texas. Fogging to prevent plastic shrinkage cracks is recommended for mix designs subjected to such environmental conditions as high wind speed, high temperature, and low relative humidity. Fogging is typically more critical for lower w/cm ratio concrete where limited bleed water comes to the surface.
8. Curing of HPC Decks: Special attention should be paid to the curing of HPC decks, particularly those that include pozzolans. Proper curing, one of the least expensive techniques that can be practiced, has a significant impact on later-age mechanical property development. A minimum of 10 days moist curing or wet mat curing is recommended for HPC decks with pozzolans.
9. Cracking of Concrete Decks: Designers must pay more attention to the detailing of control, construction, and expansion joints owing to the thinner bridge decks, longer mainspans, and wider beam spacings associated with many HPC bridge projects. Changes in cross-sectional stiffnesses and bridge constraints, which are generally not a concern for conventional concrete bridges, are more critical for HPC bridges owing to the greater flexibility of the system as a result of optimizing the structural members. These

transition locations should be where the relative change in stiffness occurs. This may require dual control or tooled joints at support (bent) locations in some cases.

12.4 Recommendations for Future Research

The following recommendations are made regarding areas requiring further research:

1. Investigate the impact of peak hydration temperatures on the durability of HS/HPC and HPC at the microstructure level more fully, so that a direct relationship between hydration temperature and microstructure damage can be ascertained.
2. Using match-curing technology, investigate the combination of standard and high performance mix designs combined with various “standard shape” surface-area-to-volume (SA/V) ratios to determine where match-curing technology results in increased plant productivity. For some combinations of high SA/V ratios and conventional concretes, member-cured cylinders sufficiently represent the concrete properties in the member.
3. Develop improved methods of permeability testing and evaluation. These include investigating porosity methods and more fully investigating the impact of fly ash on permeability, including chloride binding.
4. Develop an analytical model for the Louetta Road Overpass or similar long-span bridges to predict locations of high tensile stresses where appropriate detailing can be recommended to prevent extensive deck cracking. This is particularly important if the U-beam shape is to be used in other projects incorporating long spans with wide beam spacings and relatively thin decks.
5. Monitor the existing HPC bridges on a long-term basis, both from a mechanical and material property standpoint, to further document the field performance of high performance concrete subjected to various environmental exposures.
6. Conduct a durability study of the North Concho River Overpass in 15 to 20 years to assess the durability performance of the various mix designs under field exposure.
7. Further investigate the freeze-thaw resistance of HPCs with and without air entrainment to establish a w/cm ratio threshold where a discontinuous capillary system exists and at which point air entrainment is no longer required for adequate freeze-thaw resistance.

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APPENDIX

***Appendix A: Hydration Temperature Profiles for Precast/Prestressed U-Beams,
AASHTO Type IV, and Type A by Casting Date
and Temperature Development of Prestressed/Precast Beams***

Table A.1: Temperature Development of Prestressed/Precast U-Beams

Casting Date (Mo-Day-Yr)	"Light" Steam Curing	Peak Temperature, deg F	Temperature Rise, deg F	Heat Generation deg F/100 lbs. Cementitious/yd ³
9-23-94	No	206	110	11.14
9-30-94	No	203	106	10.74
10-7-94	No	203	107	10.84
10-28-94	Yes	185	102	10.33
11-3-94	No	184	100	10.13
11-10-94	Yes	181	94	9.52
2-15-96	No	187	106	10.74
2-26-96	No	189	98	9.93
3-8-96	No	188	98	9.93
Bold text indicates the maximum values encountered during casting of this member shape. All peak temperature locations were monitored in the end blocks. deg C = 5/9(deg F –32)				

Table A.2: Temperature Development of Prestressed/Precast AASHTO Type IVs

Casting Date (Mo-Day-Yr)	"Light" Steam Curing	Peak Temperature, deg F	Temperature Rise, deg F	Heat Generation deg F/100 lbs. Cm material/yd ³
4-1-96	No	126 ❶	46	4.68
2-19-97	No	147	65	6.61
2-25-97	Yes	129	54	5.49
3-3-97	No	147	60	6.10
3-8-97	No	142	55	5.60
3-15-97	No	142	55	5.60
3-22-97	No	156	69	7.02
3-29-97	No	160	75	7.63
4-7-97	No	167	84	8.55
4-12-97	No	114	49	4.98
4-18-97	No	151	68	6.92
4-28-97	No	167	79	8.04
Bold text indicates the maximum values encountered during casting of this member shape. All peak temperature locations were monitored in the end blocks unless noted otherwise. ❶ Match curing not performed, value based on member-cured cylinder. deg C = 5/9(deg F –32)				

Louetta Road Overpass Bridge U-Beam Cast February 15, 1996

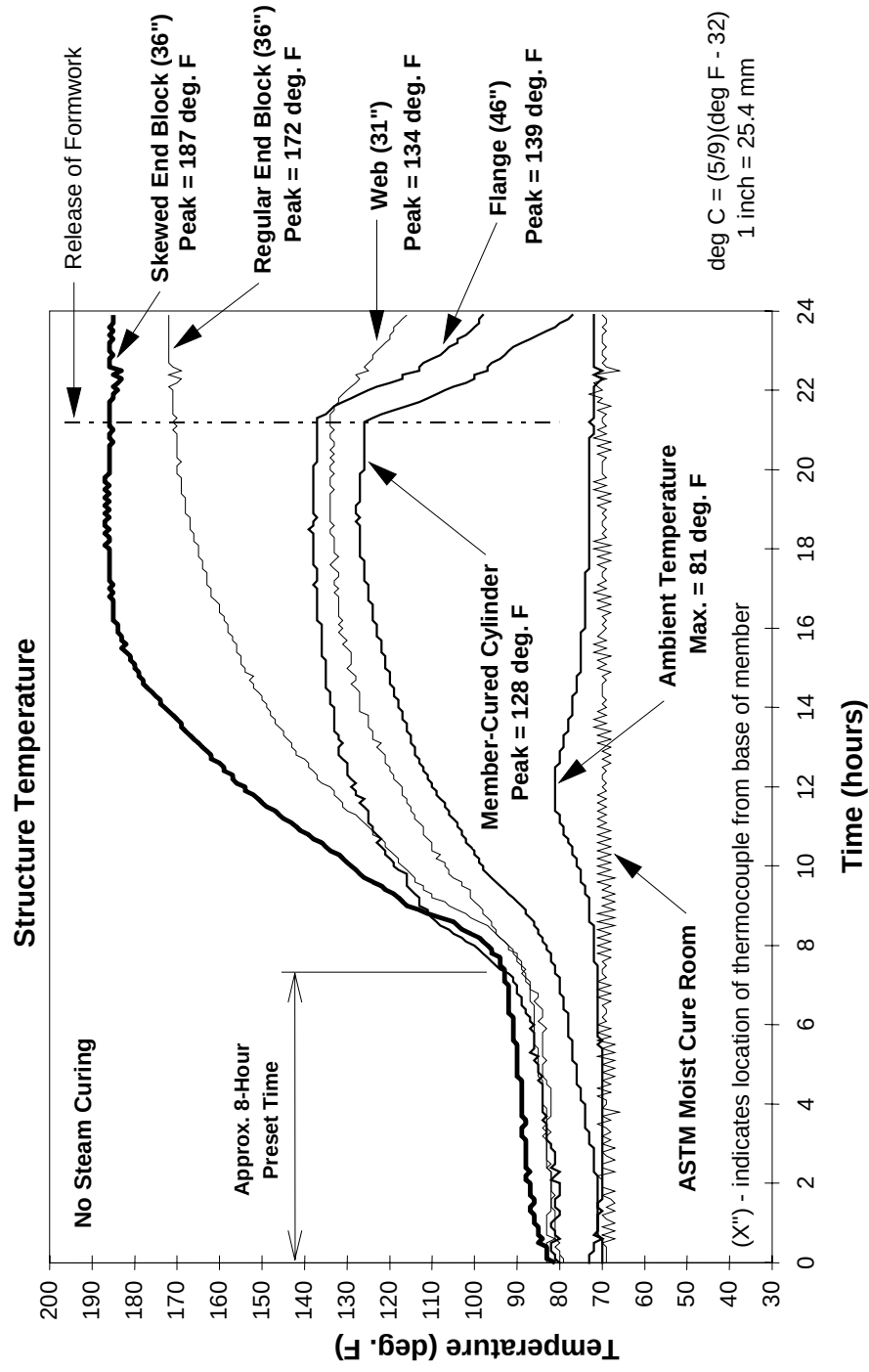


Figure A.1: Hydration Temperature Profile for Louetta Road Overpass Bridge U-Beam
Cast February 15, 1996

Louetta Road Overpass Bridge U-Beam Cast February 26, 1996

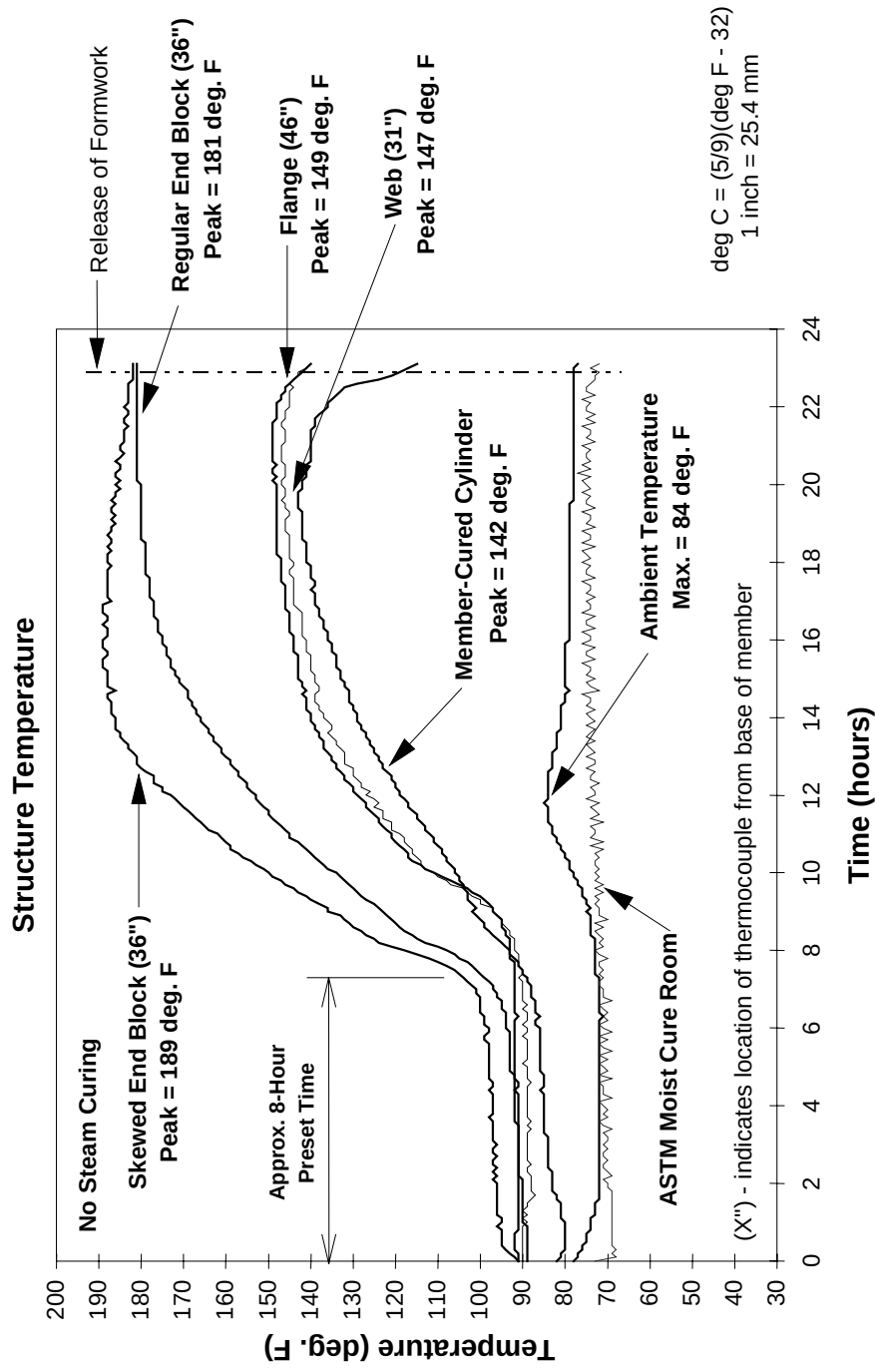


Figure A.2: Hydration Temperature Profile for Louetta Road Overpass Bridge U-Beam
Cast February 26, 1996

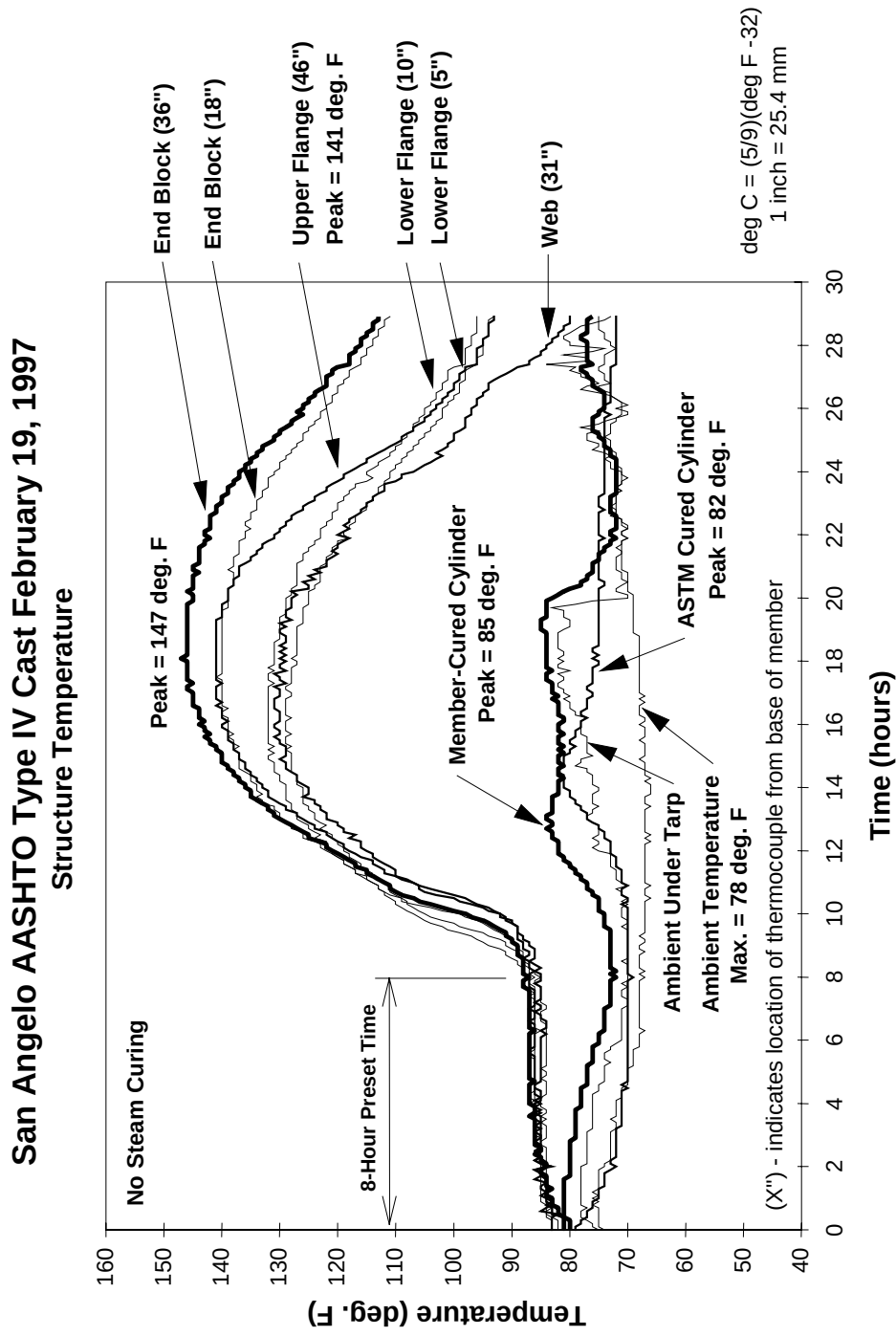


**Figure A.3: Hydration Temperature Profile for San Angelo Type IV Beam
Cast February 19, 1997**

San Angelo AASHTO Type IV Cast February 25, 1997

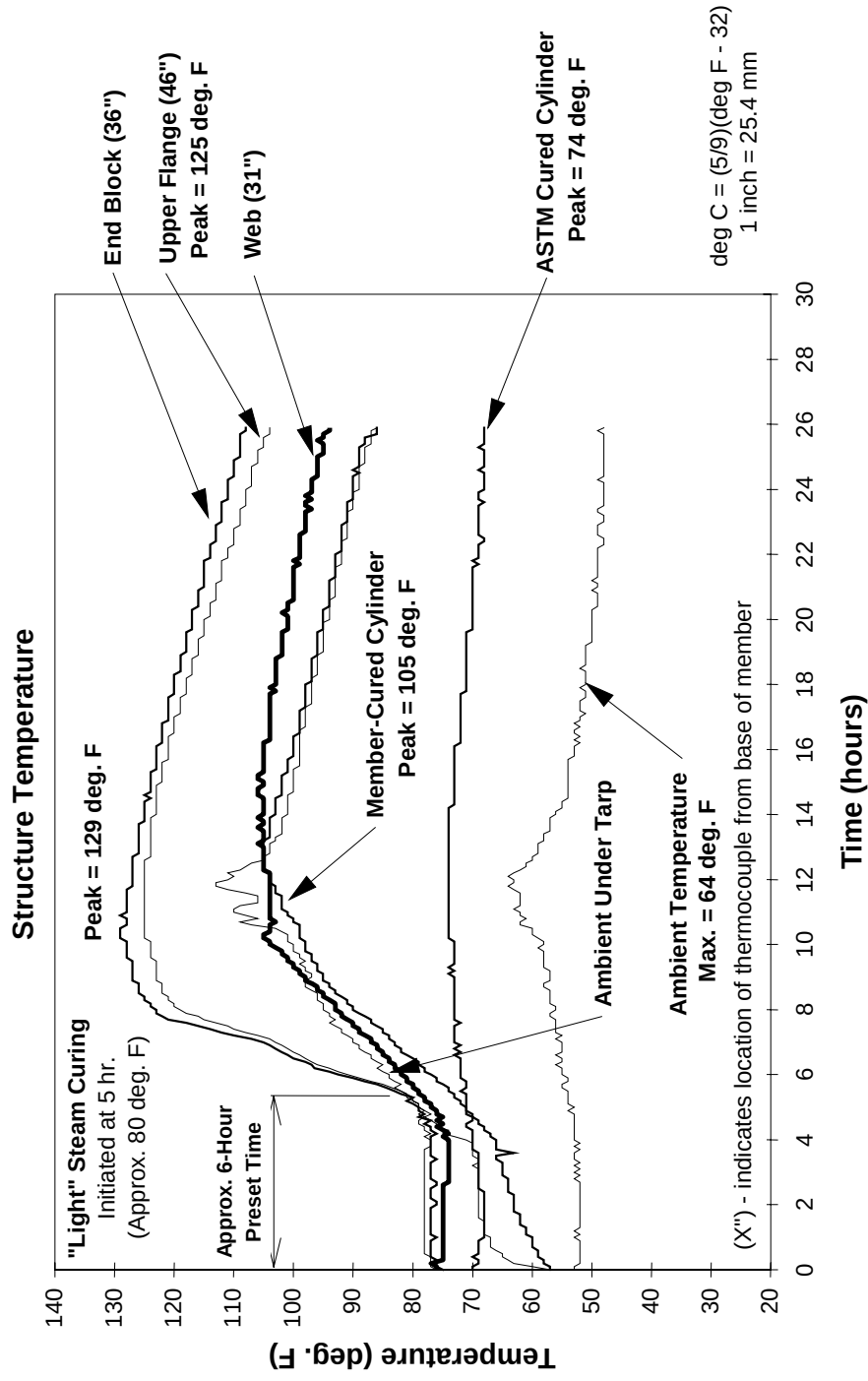


Figure A.4: Hydration Temperature Profile for San Angelo Type IV Beam Cast February 25, 1997

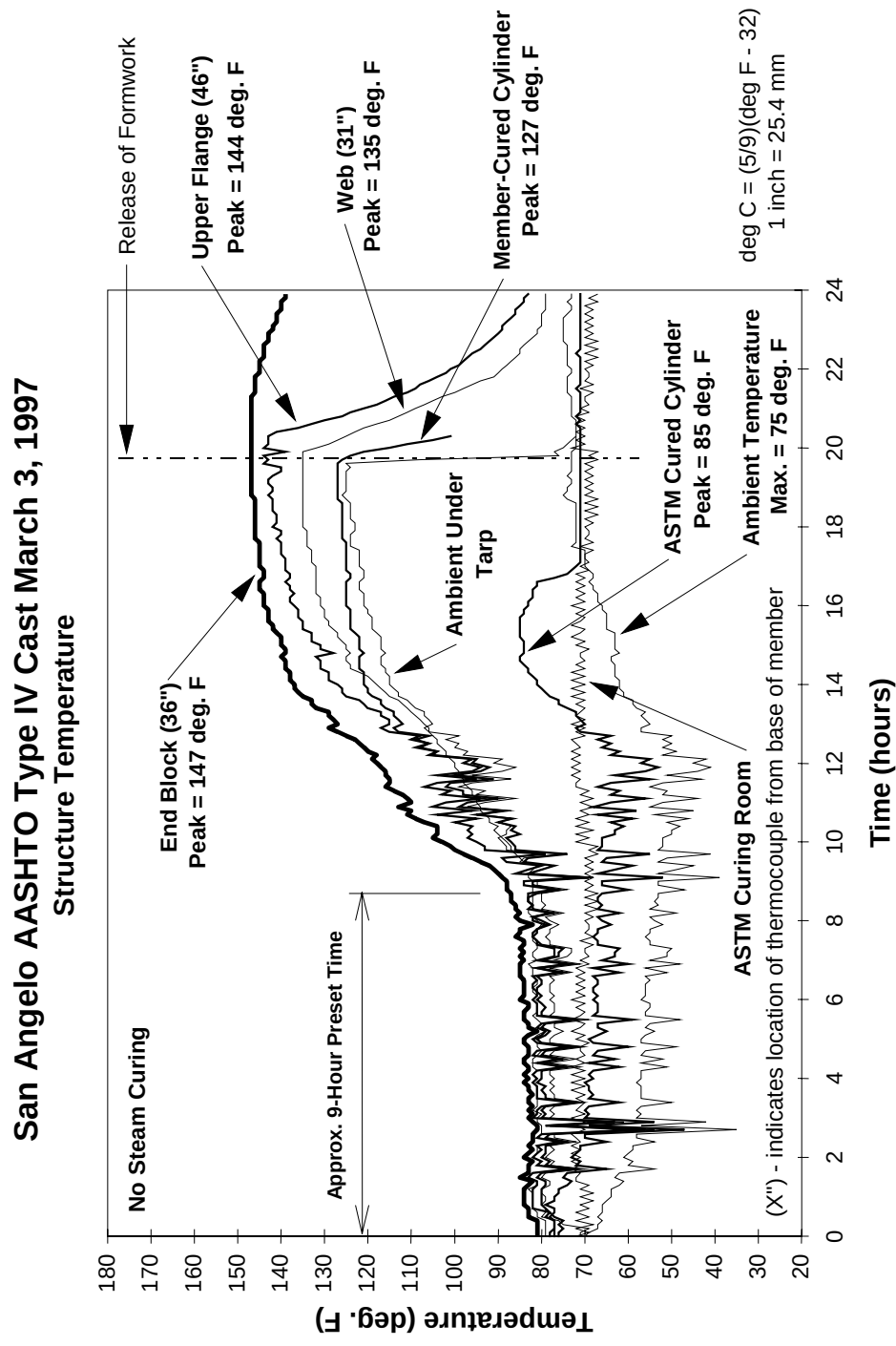


Figure A.5: Hydration Temperature Profile for San Angelo Type IV Beam
Cast March 3, 1997

San Angelo AASHTO Type IV Cast March 8, 1997

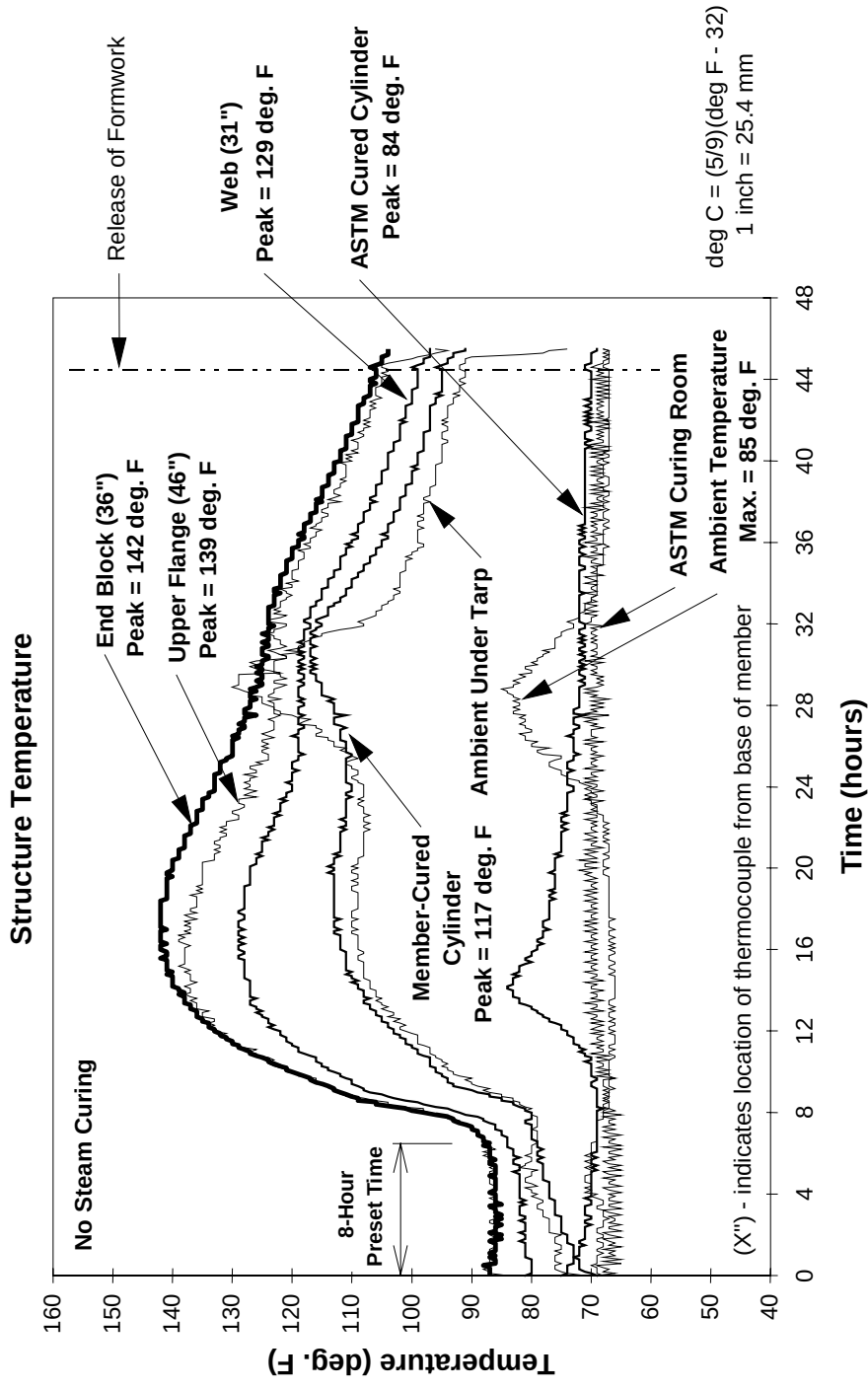


**Figure A.6: Hydration Temperature Profile for San Angelo Type IV Beam
Cast March 8, 1997**

San Angelo AASHTO Type IV Cast March 22, 1997

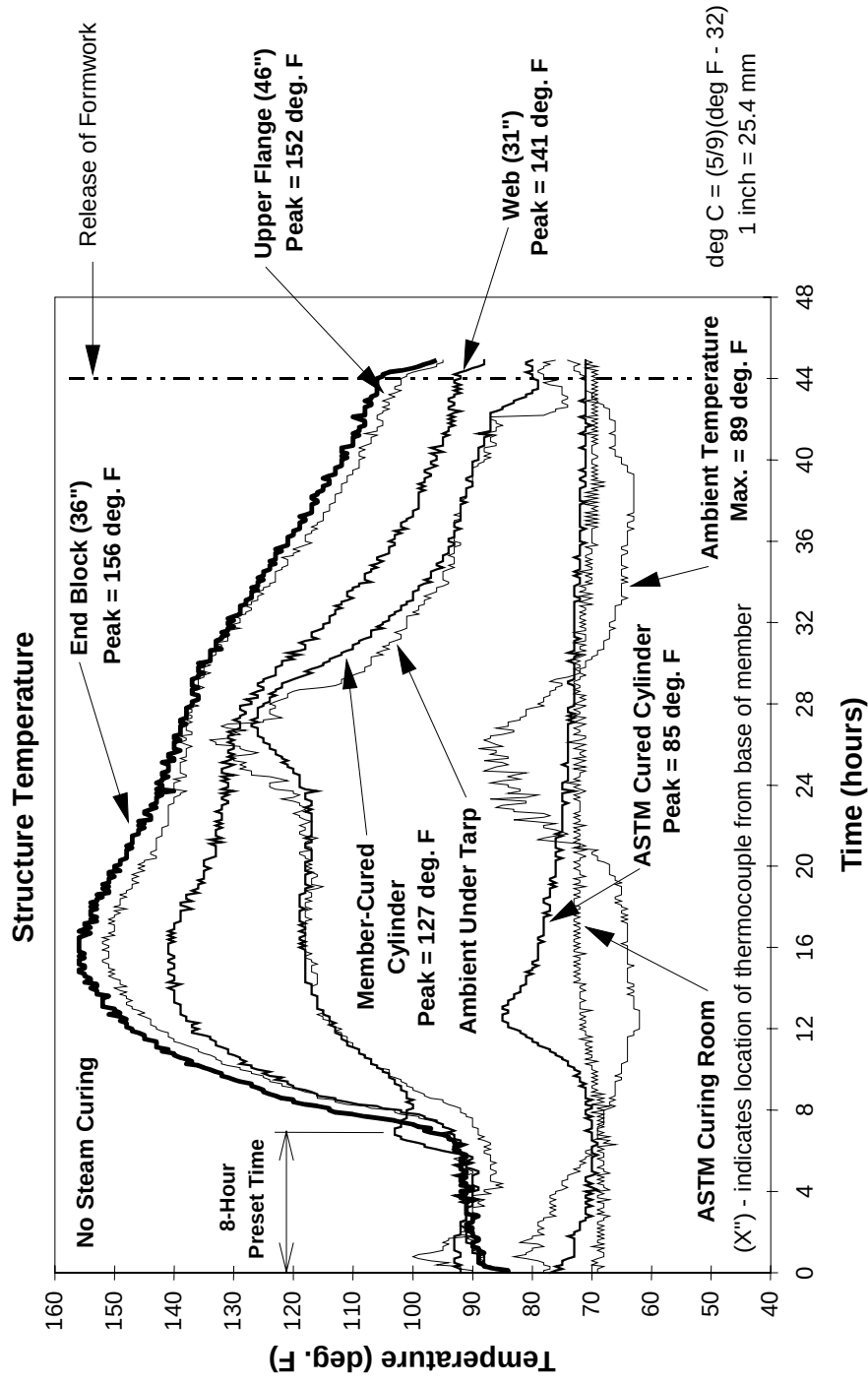


Figure A.7: Hydration Temperature Profile for San Angelo Type IV Beam Cast March 22, 1997

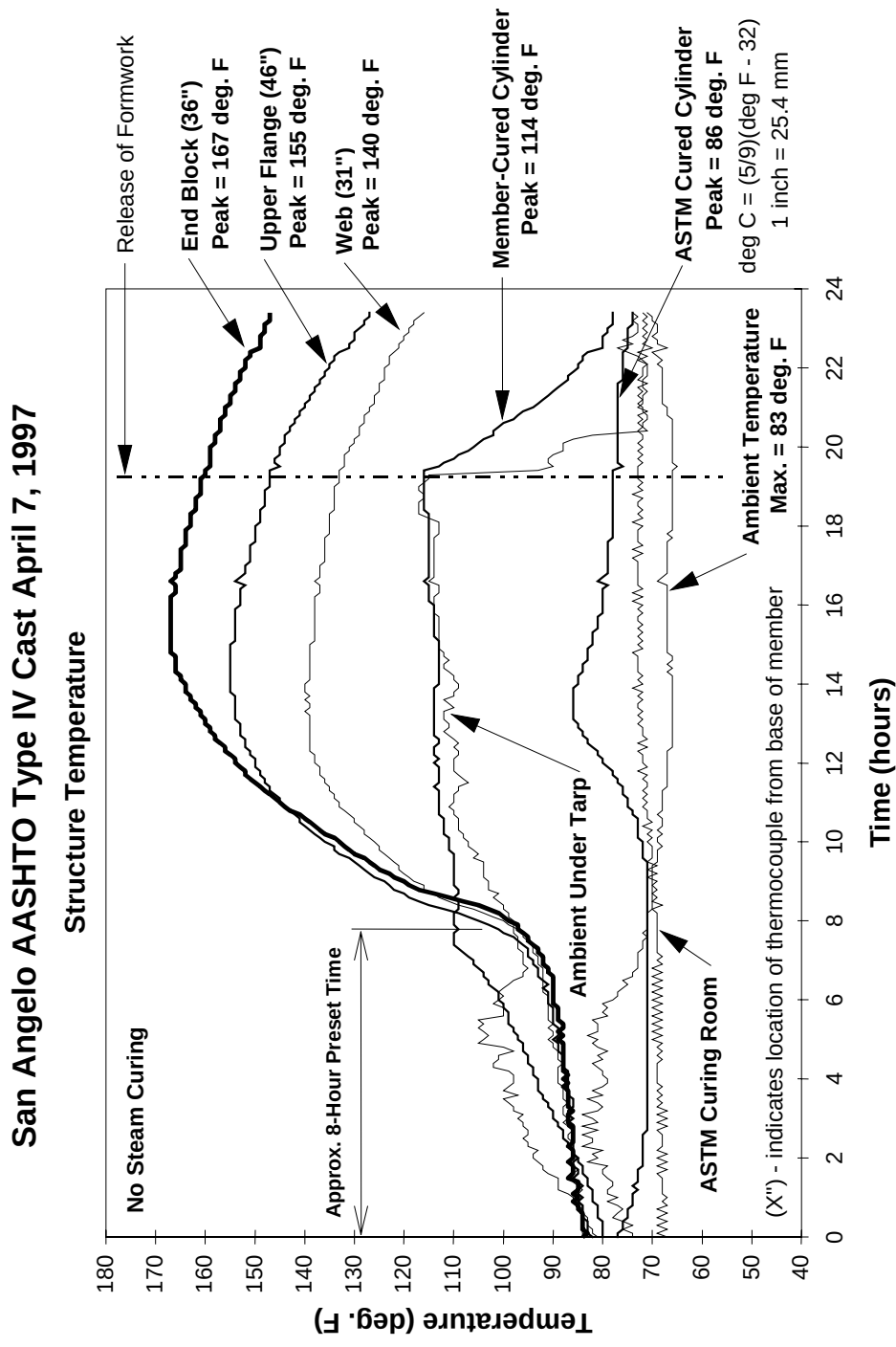


**Figure A.8: Hydration Temperature Profile for San Angelo Type IV Beam
Cast April 7, 1997**

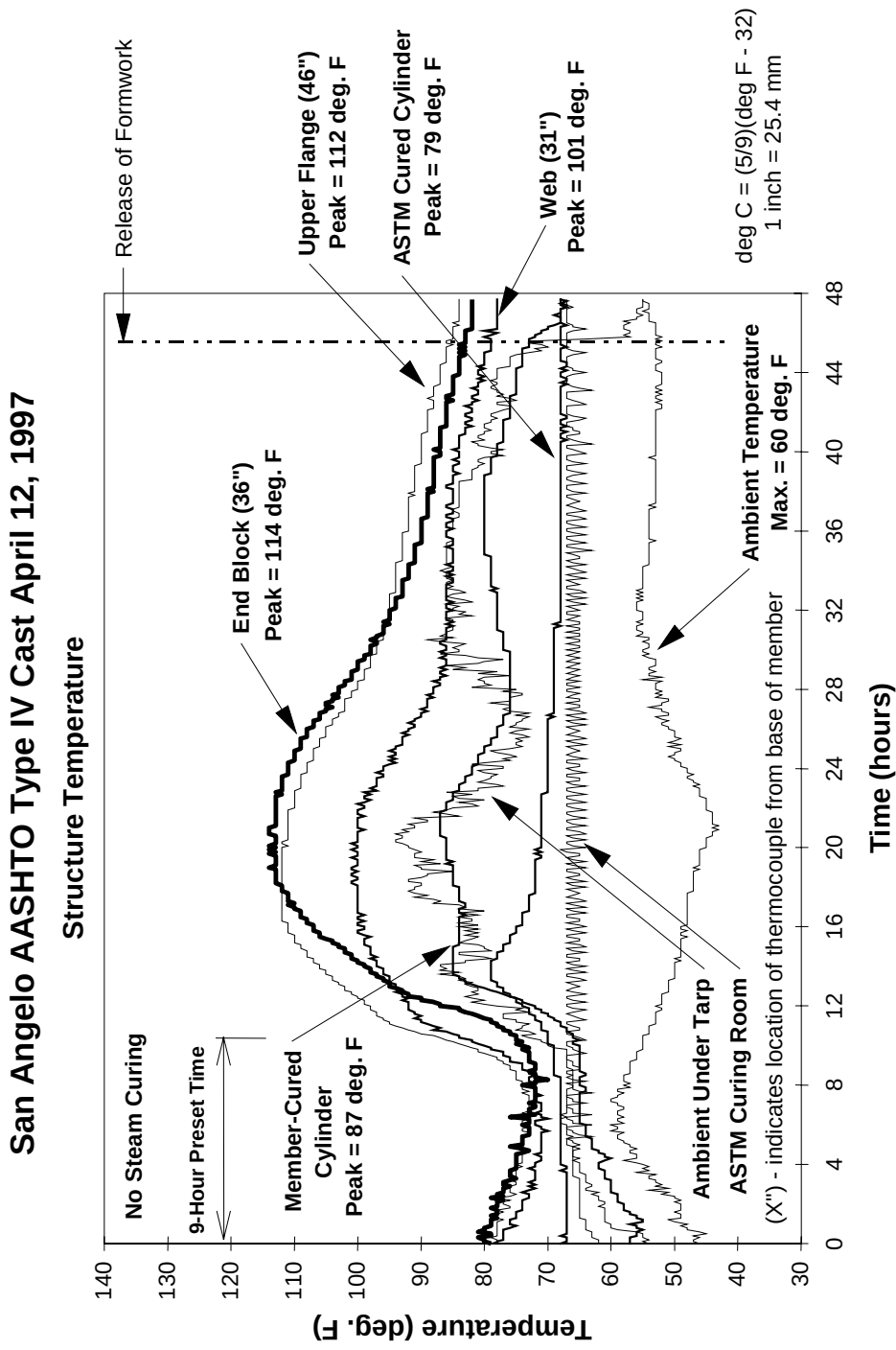


**Figure A.9: Hydration Temperature Profile for San Angelo Type IV Beam
Cast April 12, 1997**

San Angelo AASHTO Type IV Cast April 18, 1997

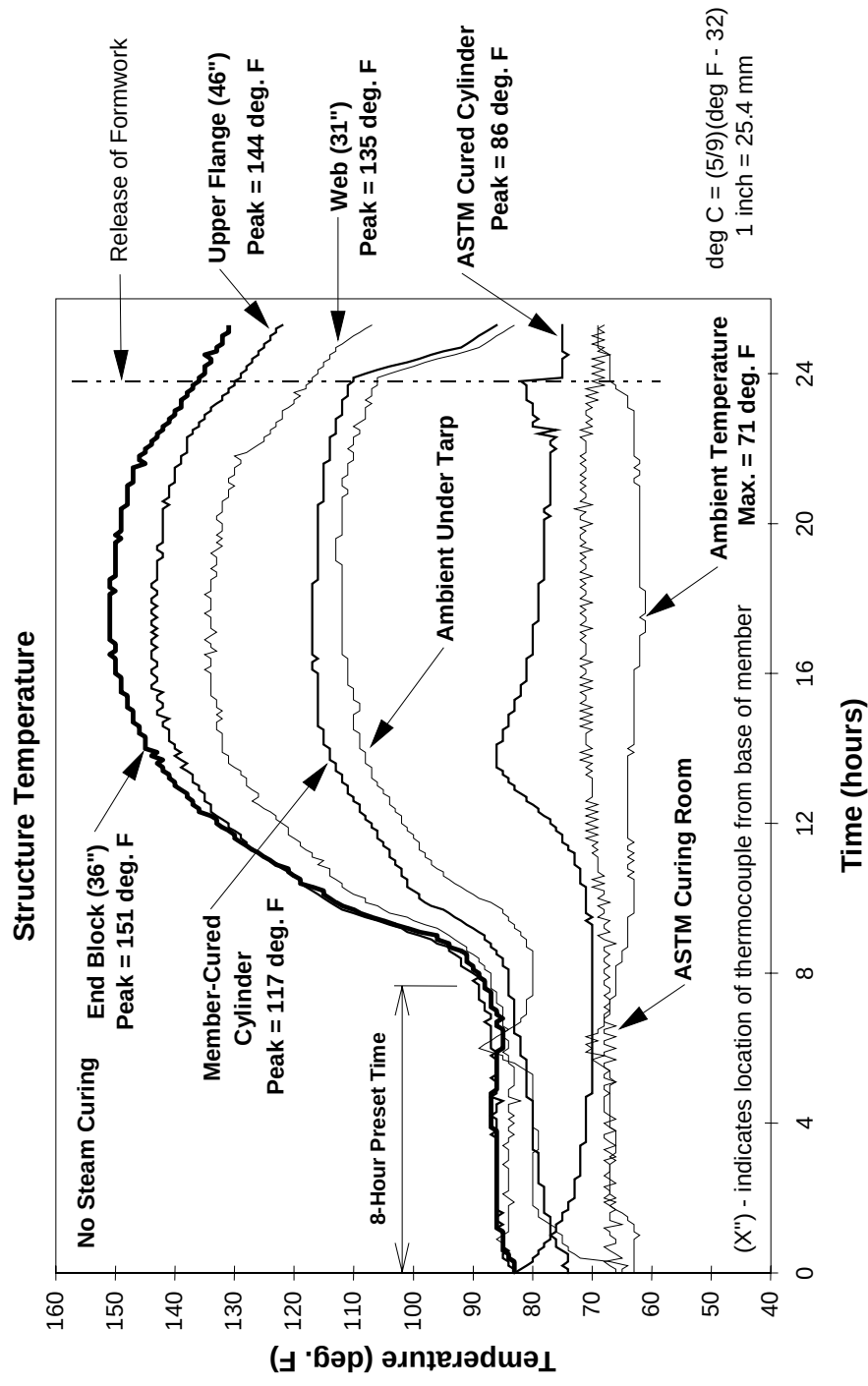


Figure A.10: Hydration Temperature Profile for San Angelo Type IV Beam
Cast April 18, 1997

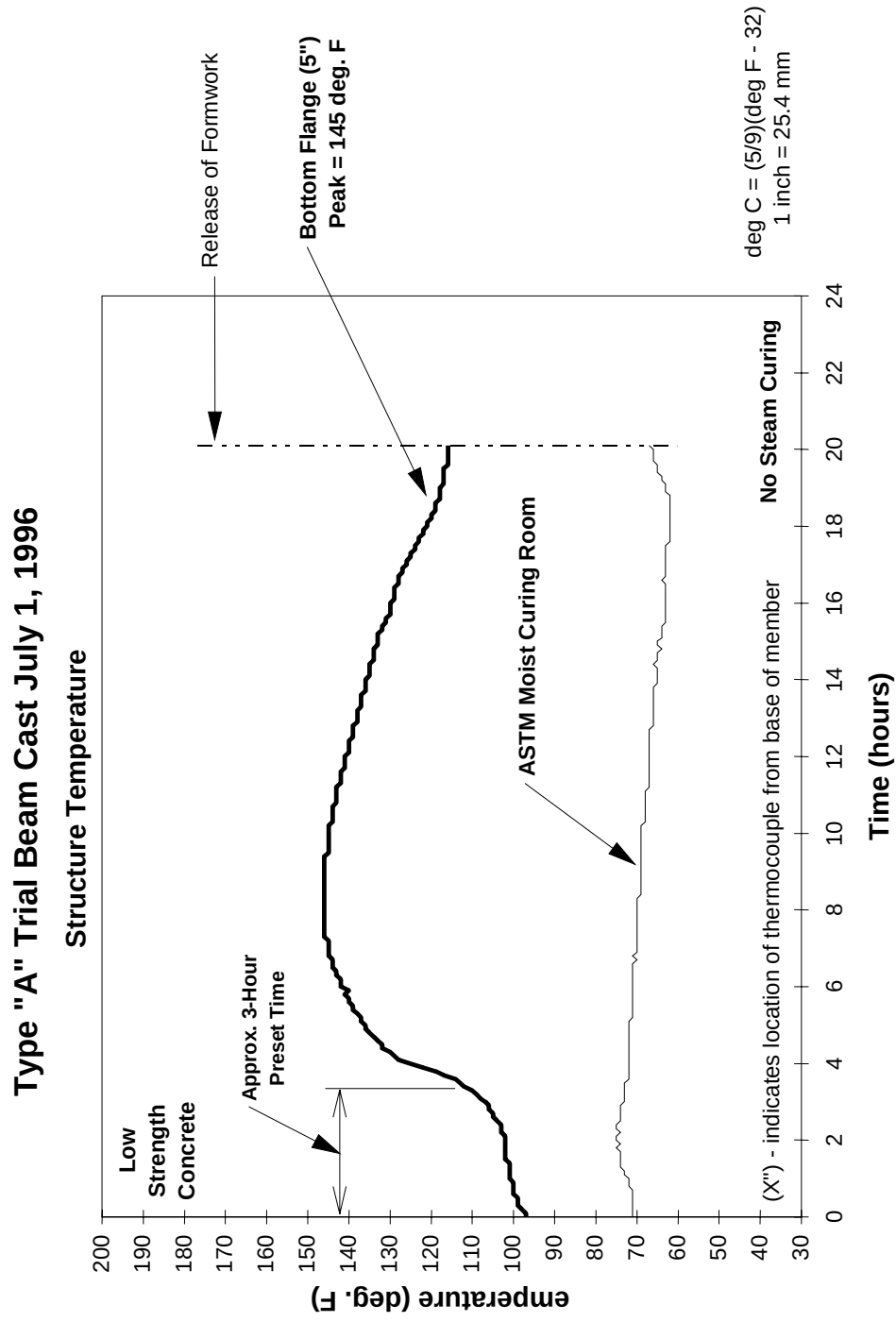


Figure A.11: Hydration Temperature Profile for Type "A" Trial Beam Cast July 1, 1997

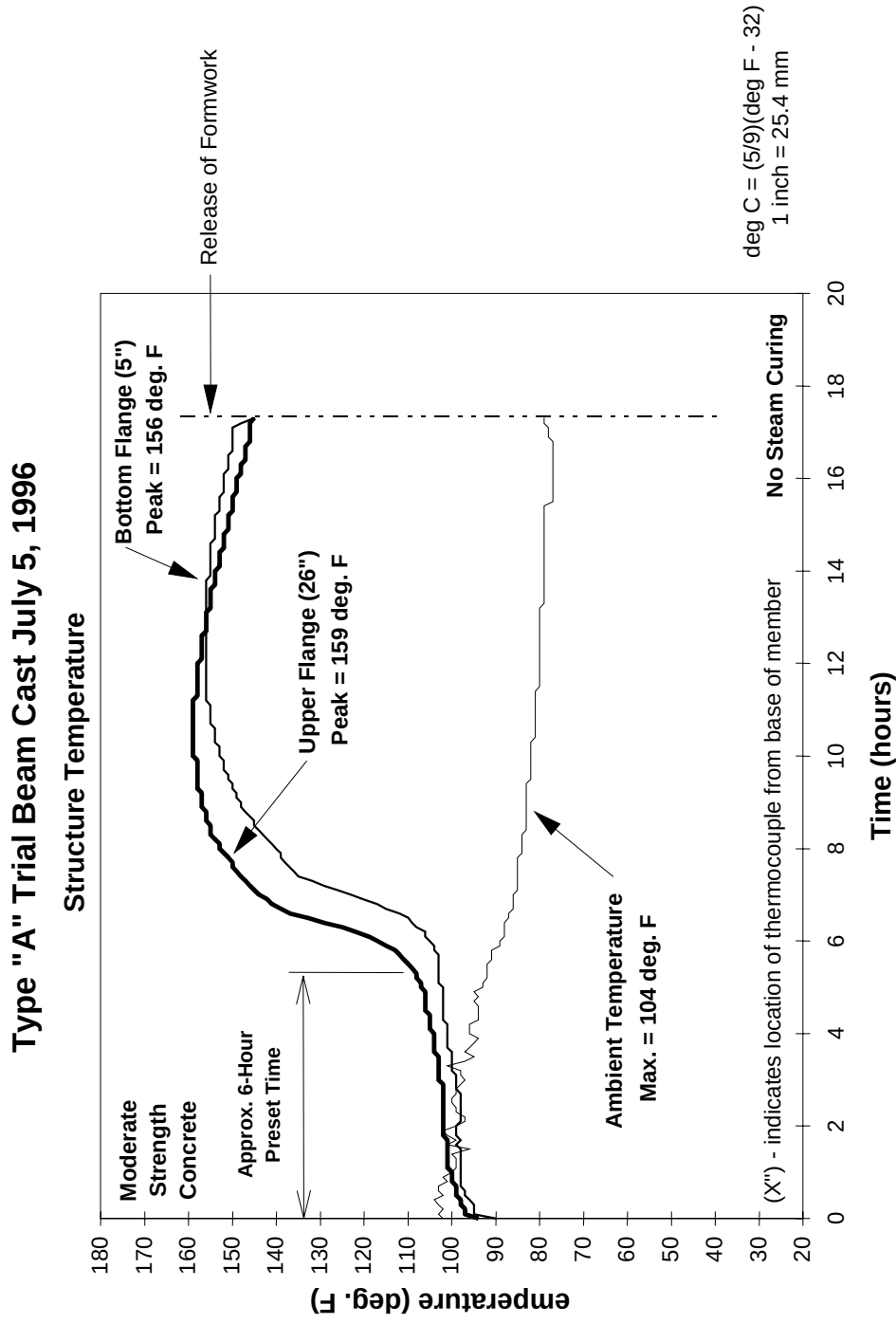


Figure A.12: Hydration Temperature Profile for Type "A" Trial Beam Cast July 5, 1997

Appendix B: Rapid Ion Permeability Test Results by Casting Date

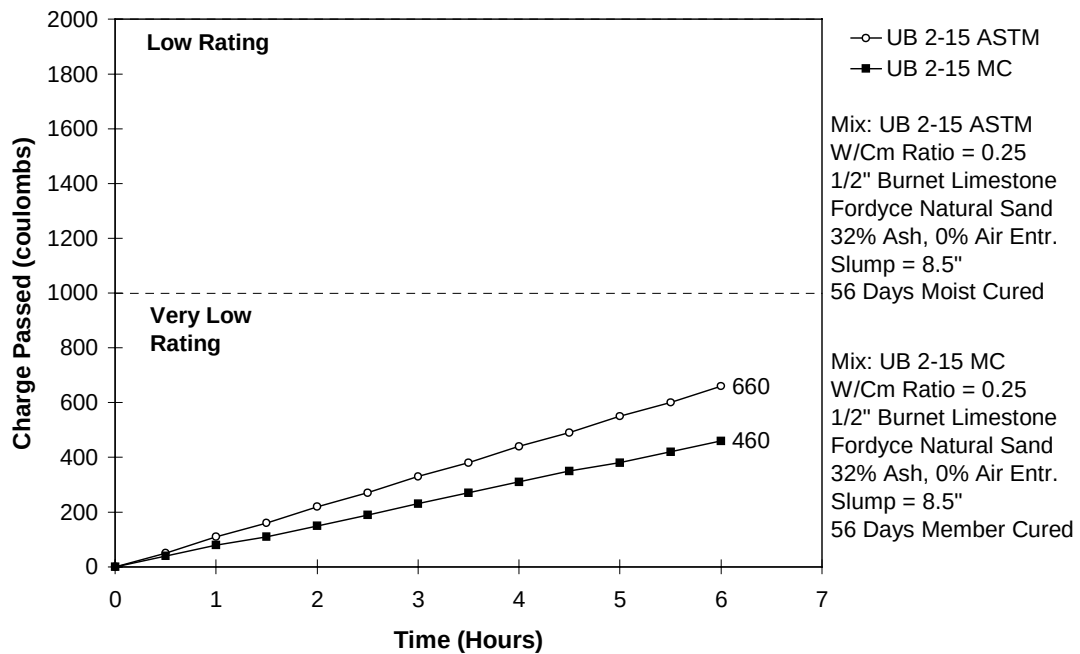


**Figure B.1: Rapid Ion Permeability for Louetta Road Overpass Bridge U-Beam
Cast February 15, 1996**

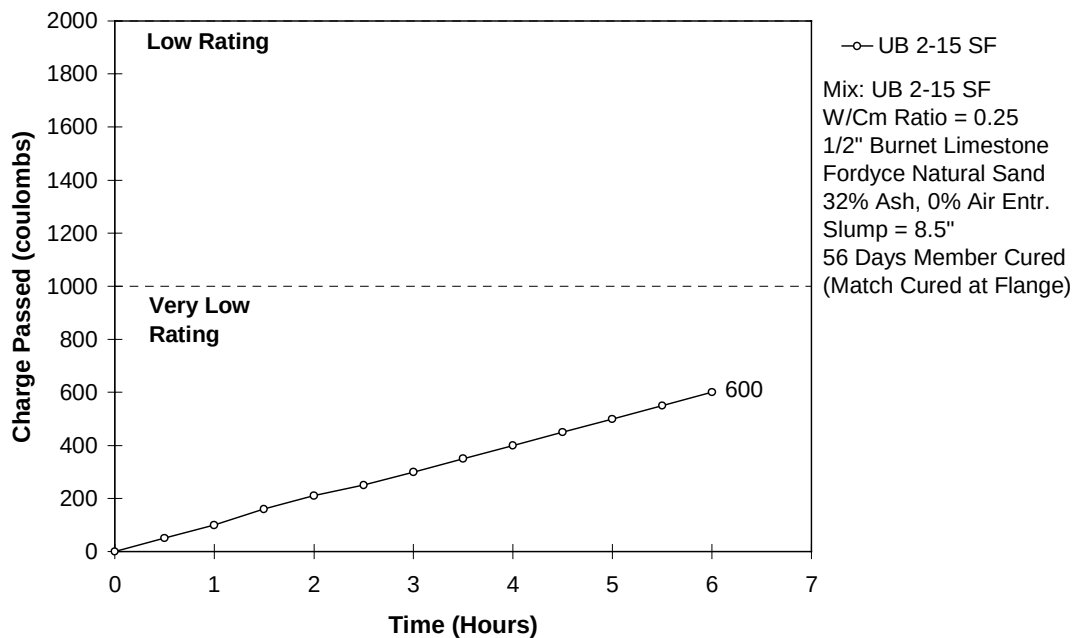


**Figure B.2: Rapid Ion Permeability for Louetta Road Overpass Bridge U-Beam
Cast February 15, 1996**

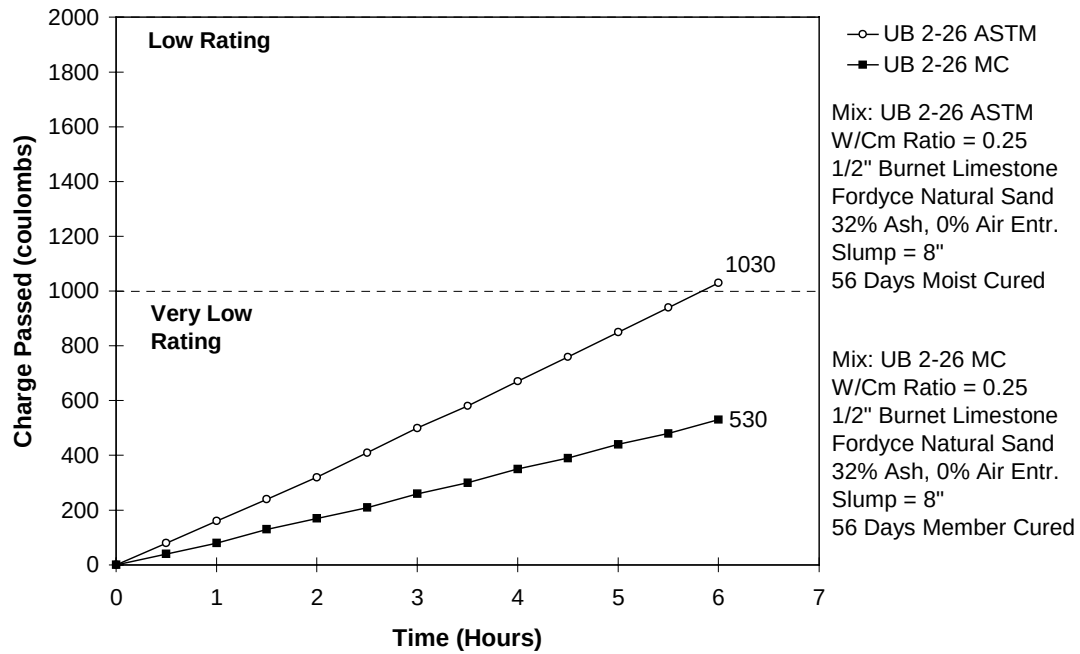


**Figure B.3: Rapid Ion Permeability for Louetta Road Overpass Bridge U-Beam
Cast February 26, 1996**

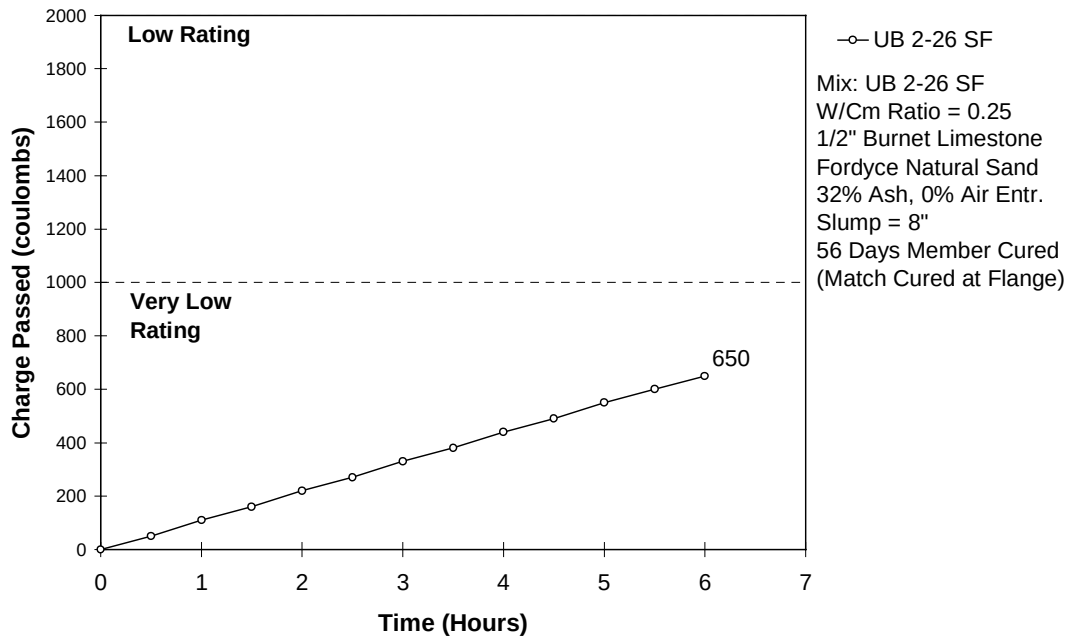


**Figure B.4: Rapid Ion Permeability for Louetta Road Overpass Bridge U-Beam
Cast February 26, 1996**

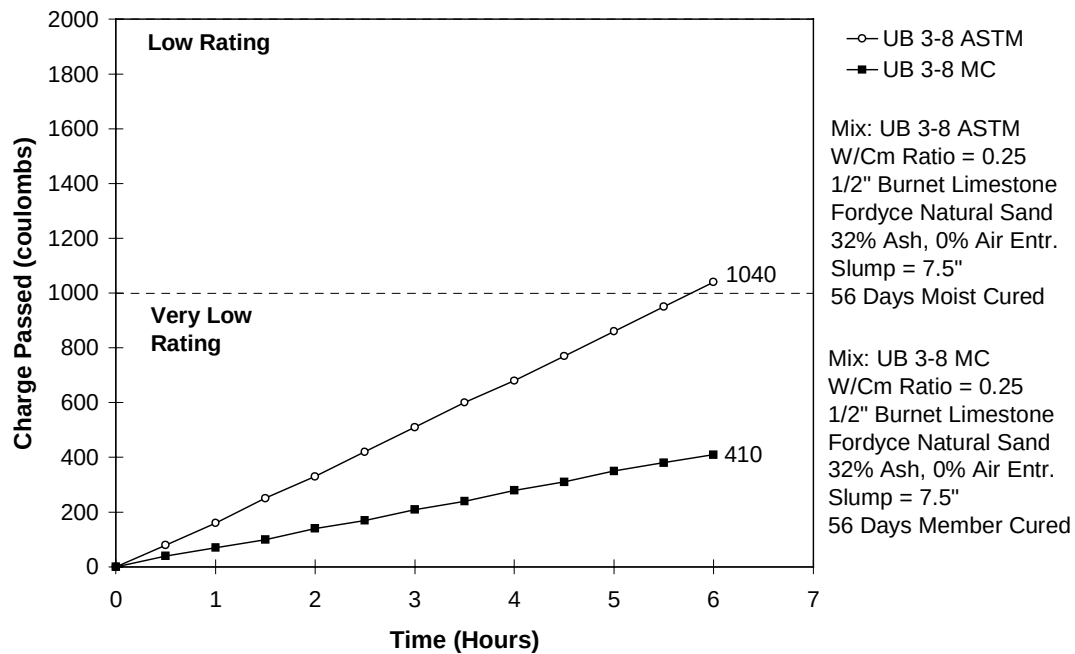


**Figure B.5: Rapid Ion Permeability for Louetta Road Overpass Bridge U-Beam
Cast March 8, 1996**

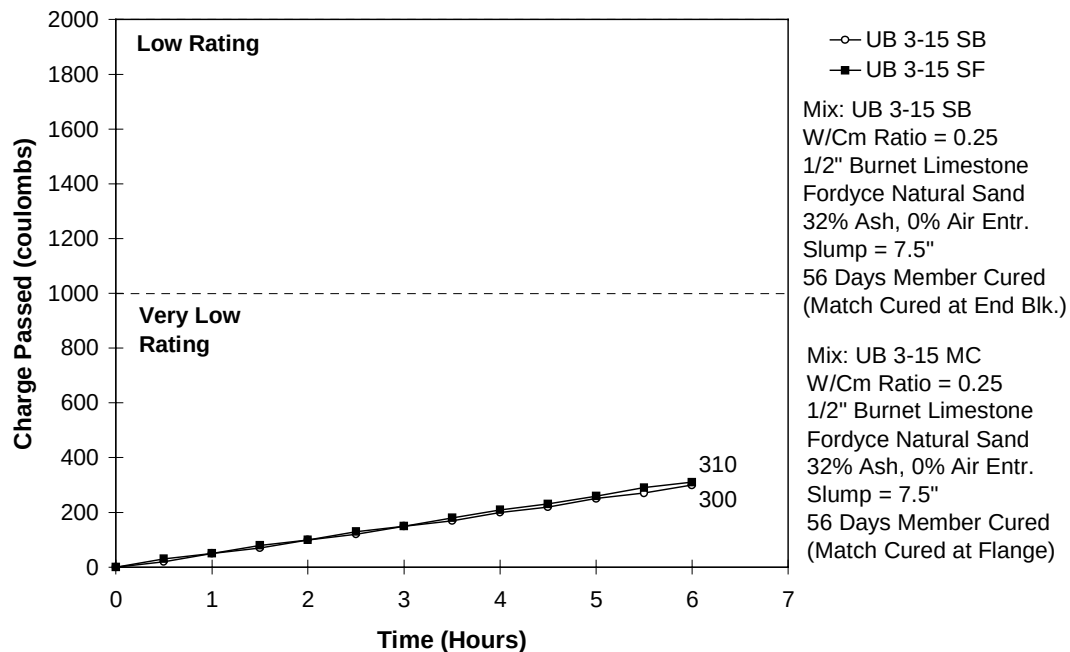


**Figure B.6: Rapid Ion Permeability for Louetta Road Overpass Bridge U-Beam
Cast March 15, 1996**

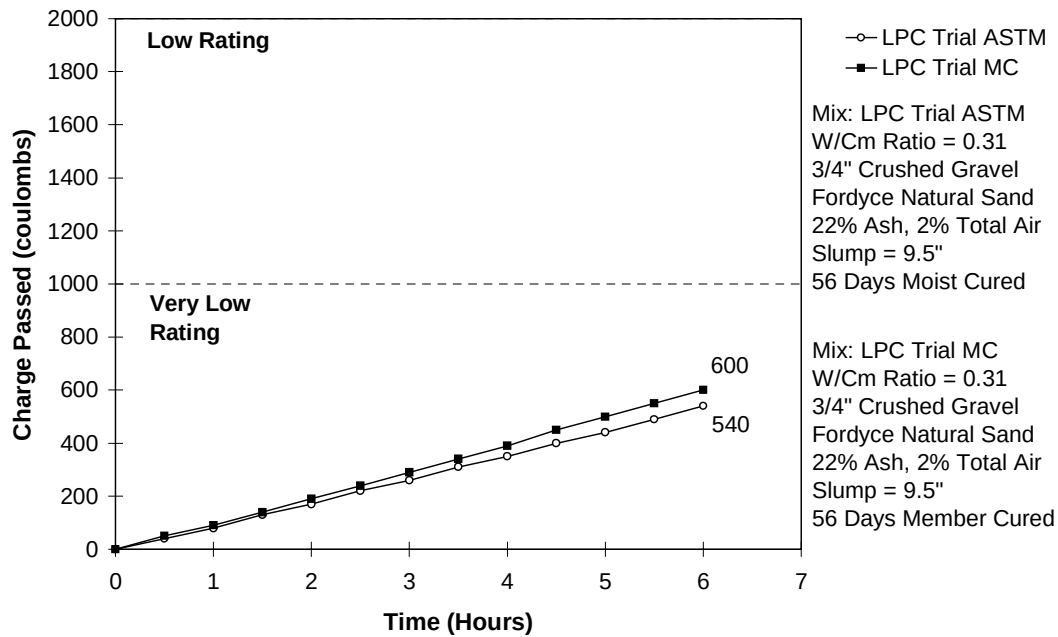


**Figure B.7: Rapid Ion Permeability for Louetta Road Overpass Bridge
Precast Column Field Trial Mix 1**

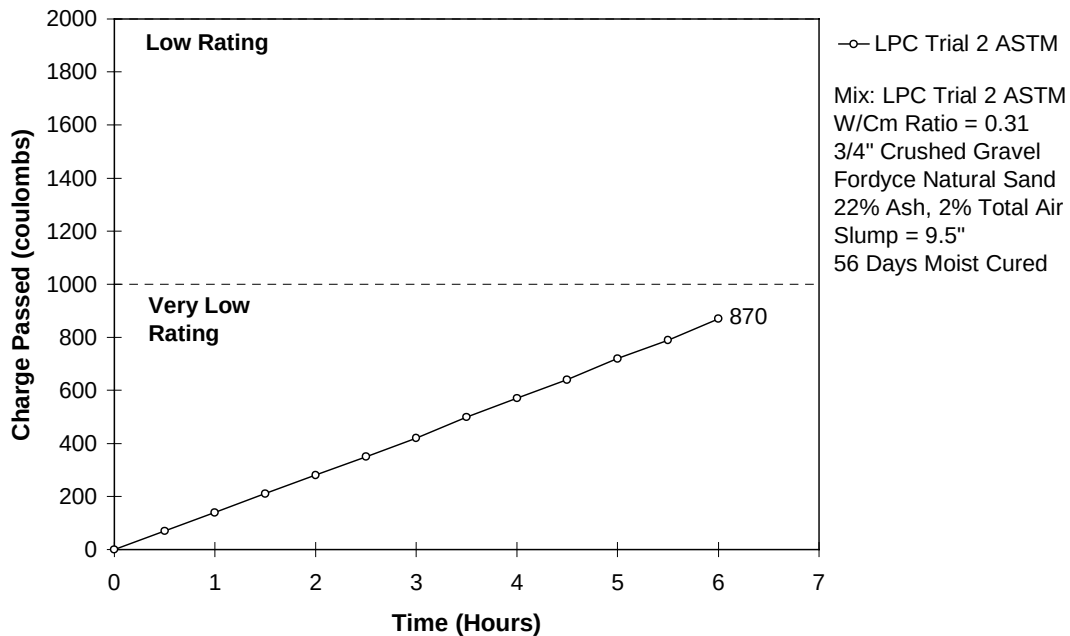


**Figure B.8: Rapid Ion Permeability for Louetta Road Overpass Bridge
Precast Column Field Trial Mix 2**

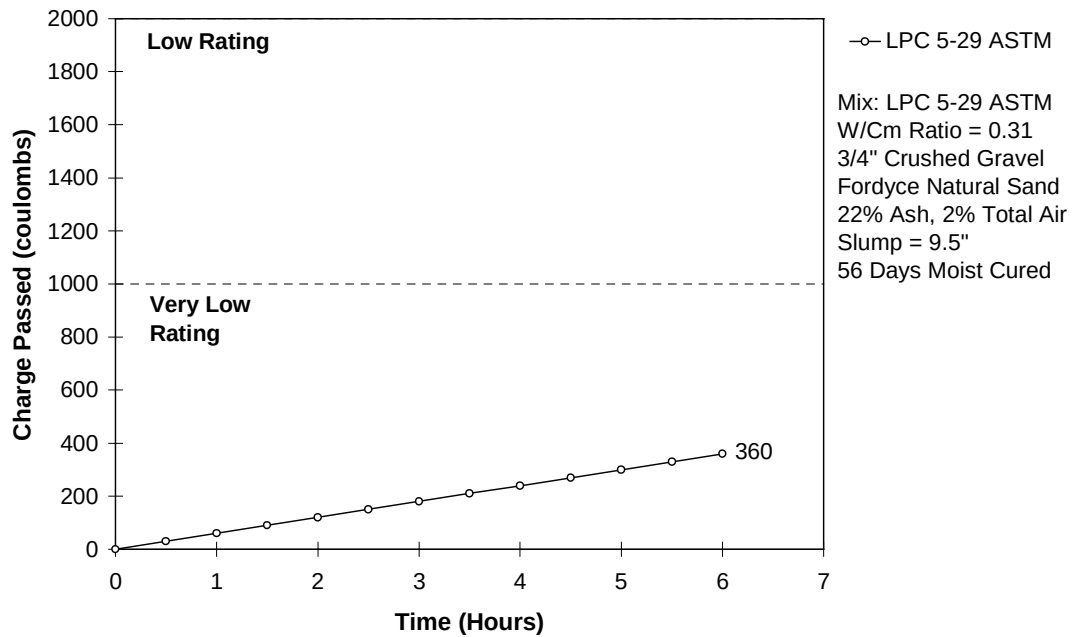


**Figure B.9: Rapid Ion Permeability for Louetta Road Overpass Bridge
Precast Column Cast May 29, 1996**

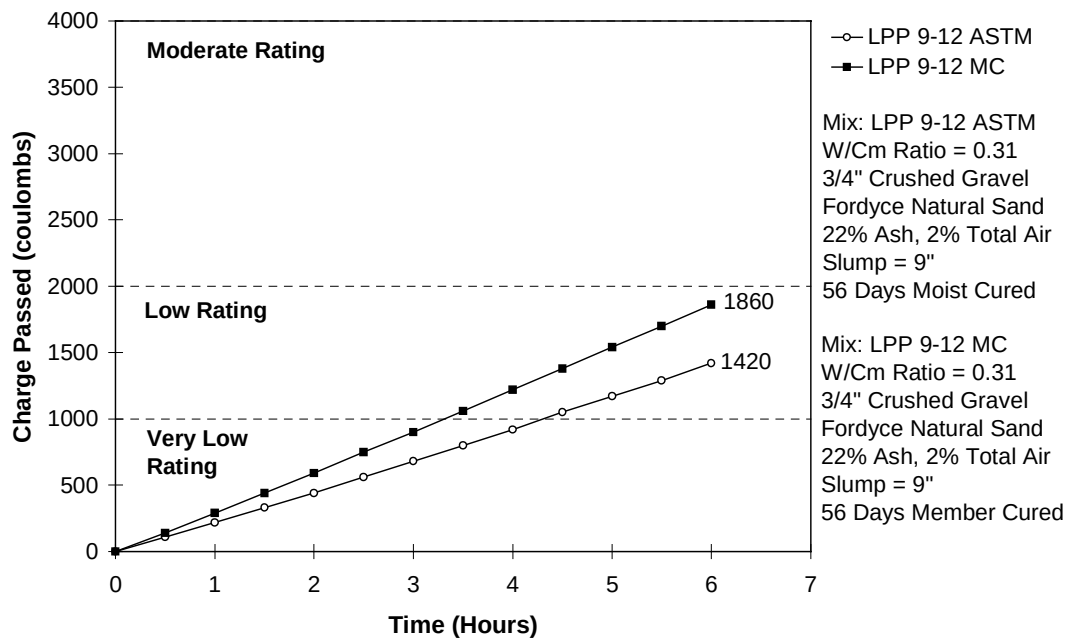


**Figure B.10: Rapid Ion Permeability for Louetta Road Overpass Bridge
Precast Panel Cast September 12, 1996**

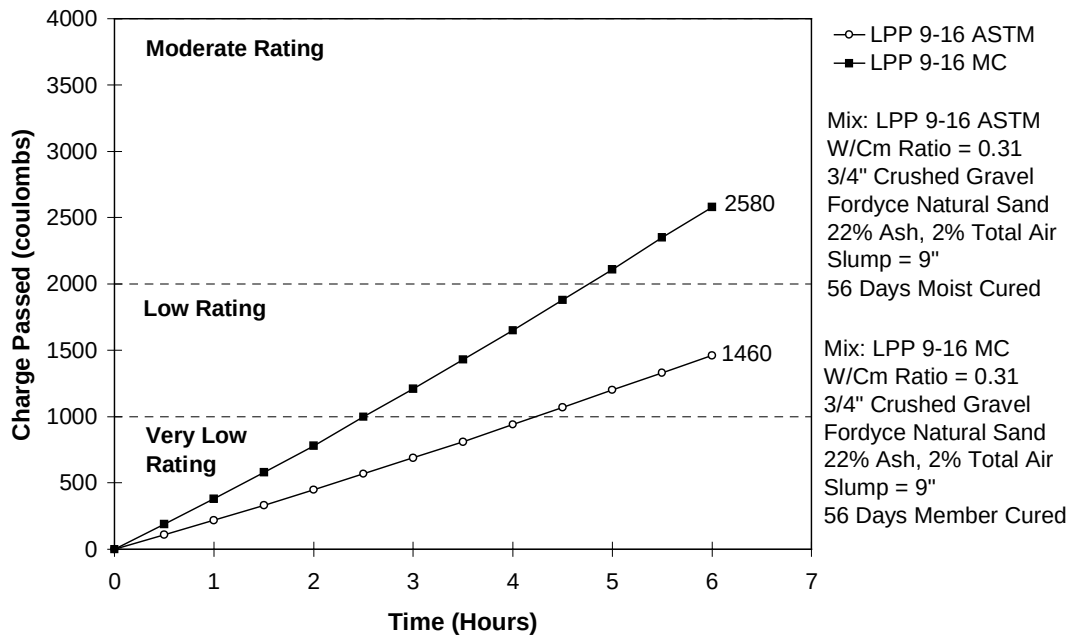


**Figure B.11: Rapid Ion Permeability for Louetta Road Overpass Bridge
Precast Panel Cast September 18, 1996**

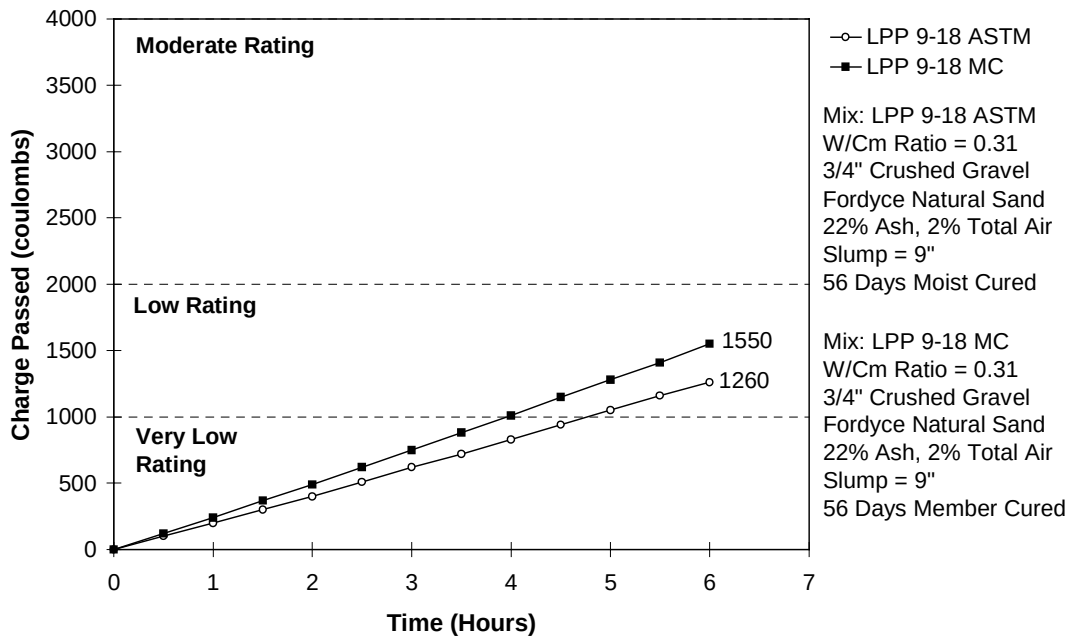


**Figure B.12: Rapid Ion Permeability for Louetta Road Overpass Bridge
Precast Panel Cast September 18, 1996**

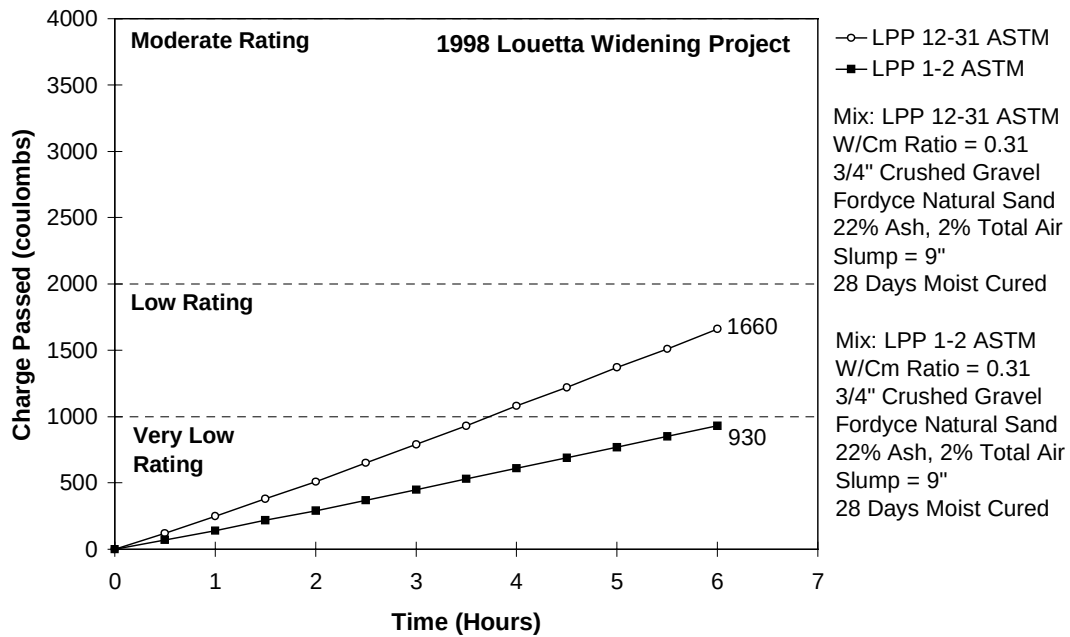


Figure B.13: Rapid Ion Permeability for Louetta Road Overpass Bridge
Precast Panel Cast December 31, 1997 and January 2, 1998

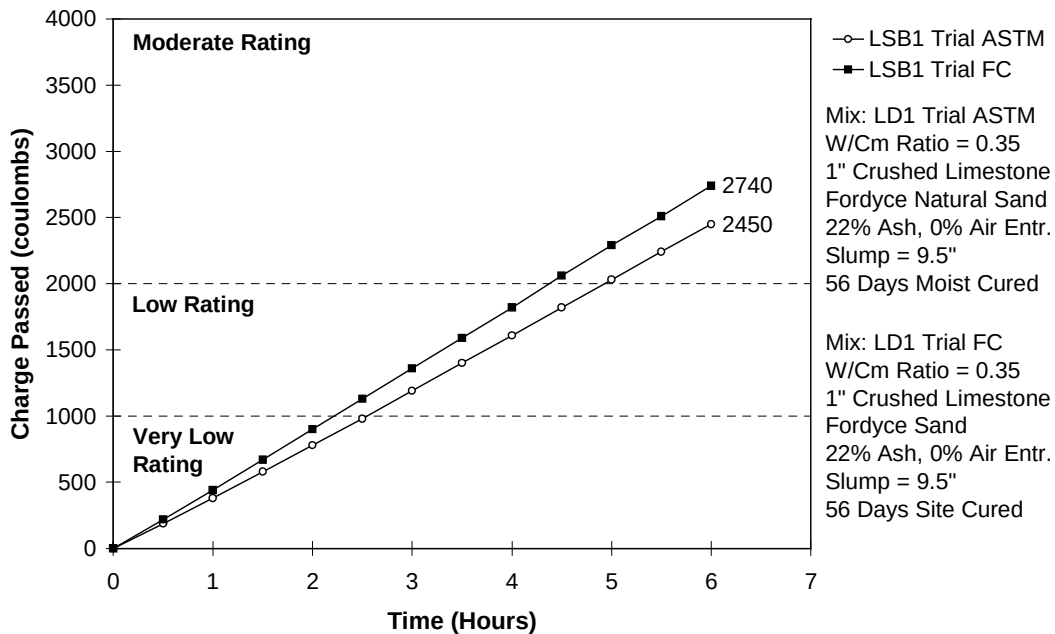


Figure B.14: Rapid Ion Permeability for Louetta Road Overpass Bridge
Southbound Deck Field Trial Mix 1

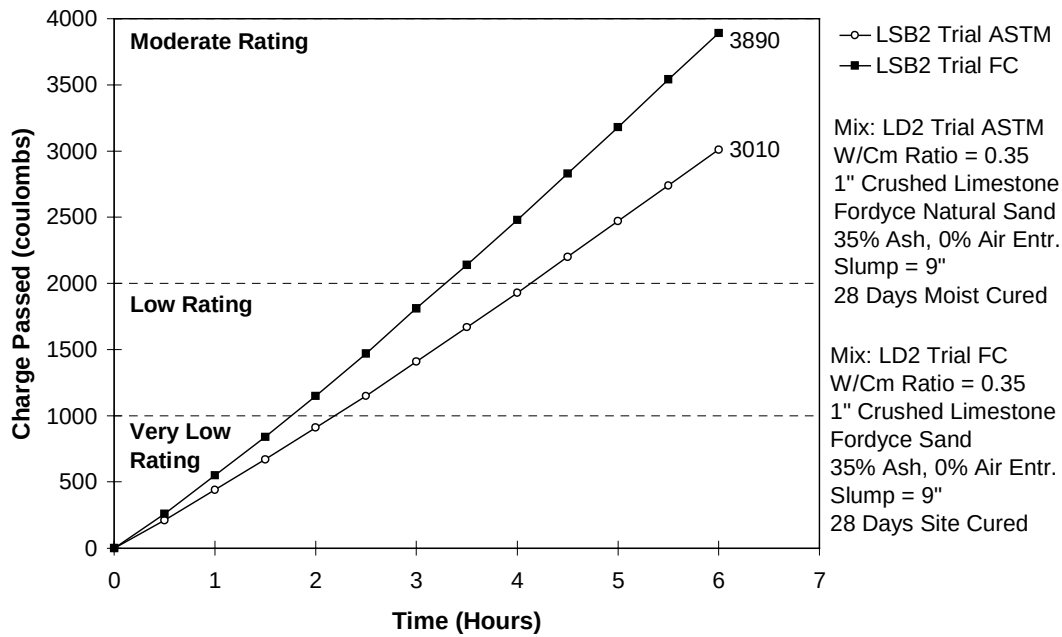


Figure B.15: Rapid Ion Permeability for Louetta Road Overpass Bridge Southbound Deck Field Trial Mix 2

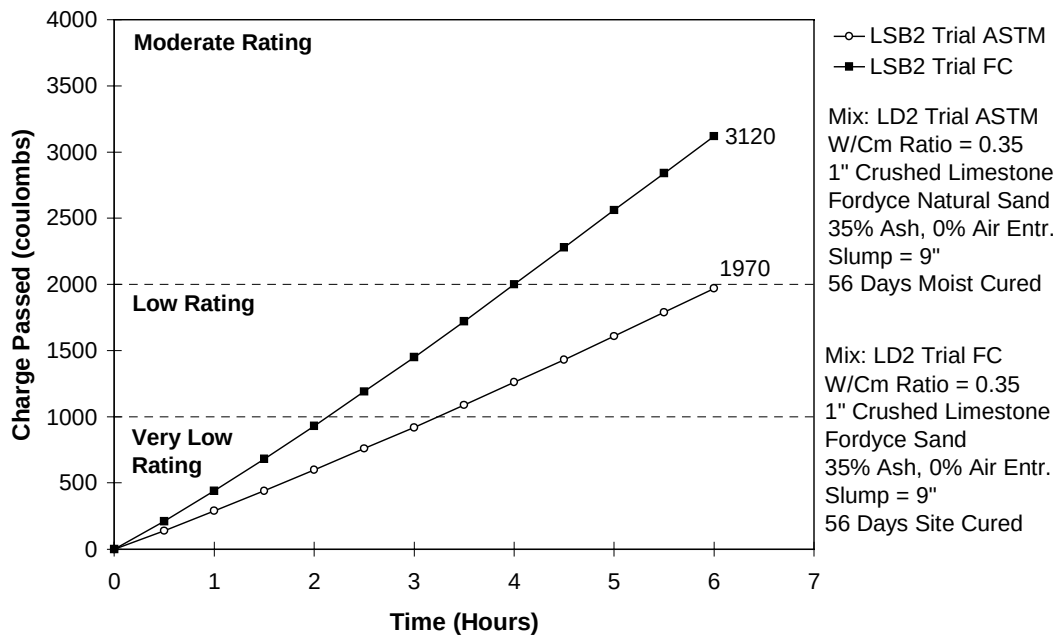


Figure B.16: Rapid Ion Permeability for Louetta Road Overpass Bridge Southbound Deck Field Trial Mix 2

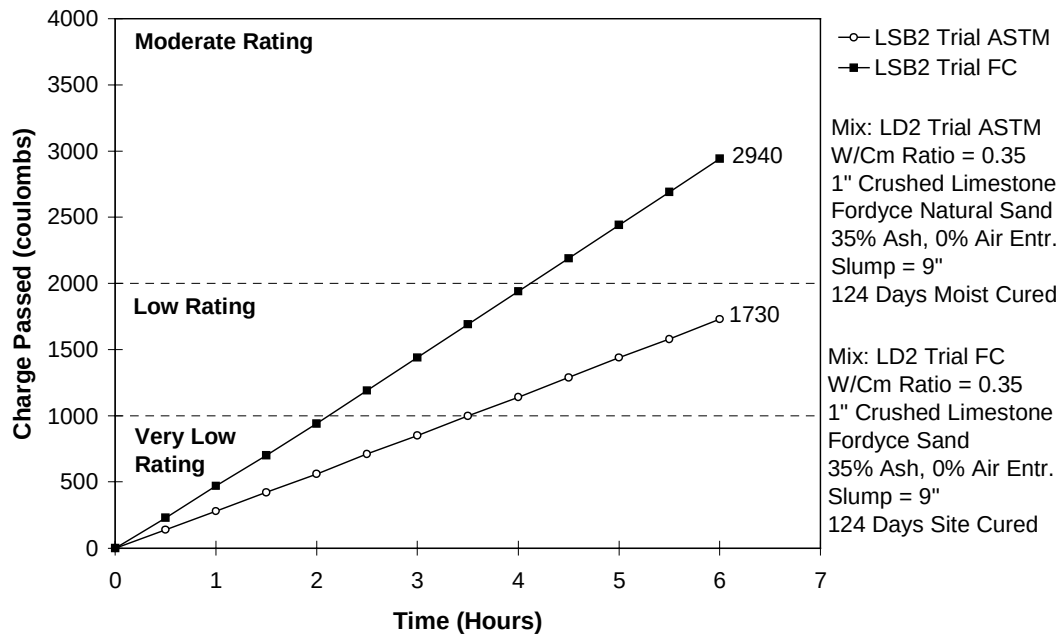


Figure B.17: Rapid Ion Permeability for Louetta Road Overpass Bridge Southbound Deck Field Trial Mix 2

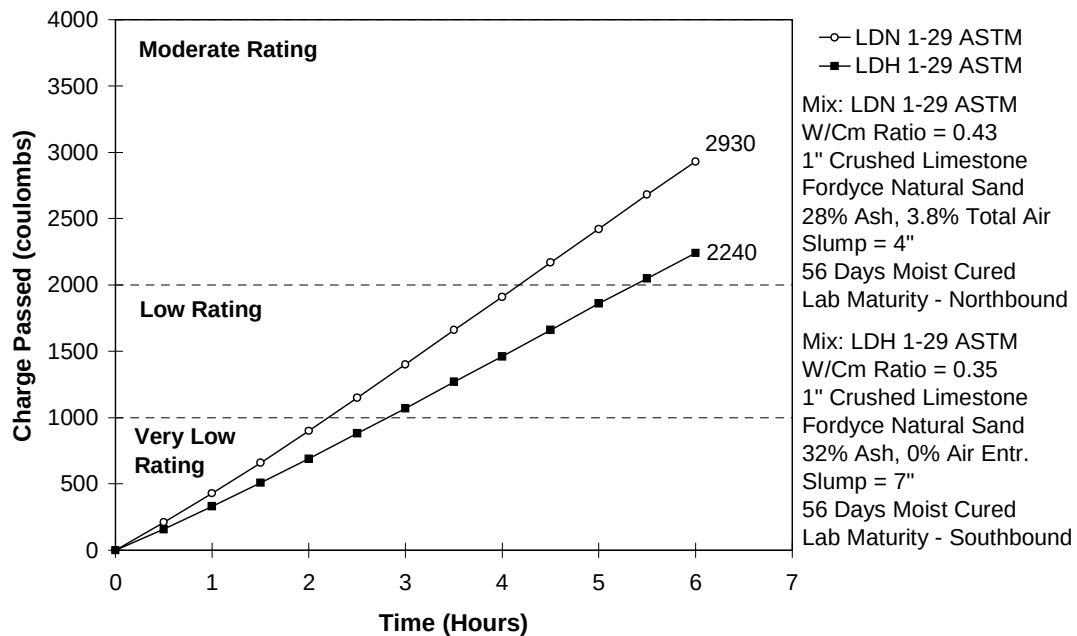


Figure B.18: Rapid Ion Permeability for Louetta Road Overpass Bridge Decks – Laboratory Maturity

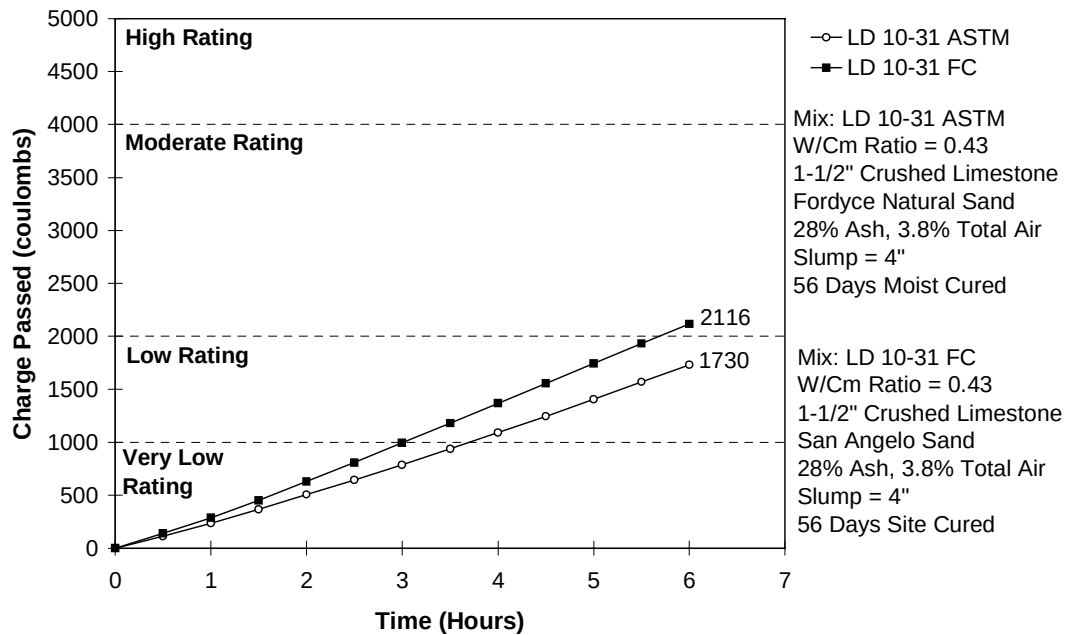


**Figure B.19: Rapid Ion Permeability for Louetta Road Overpass Bridge
Southbound Deck Cast October 31, 1996**

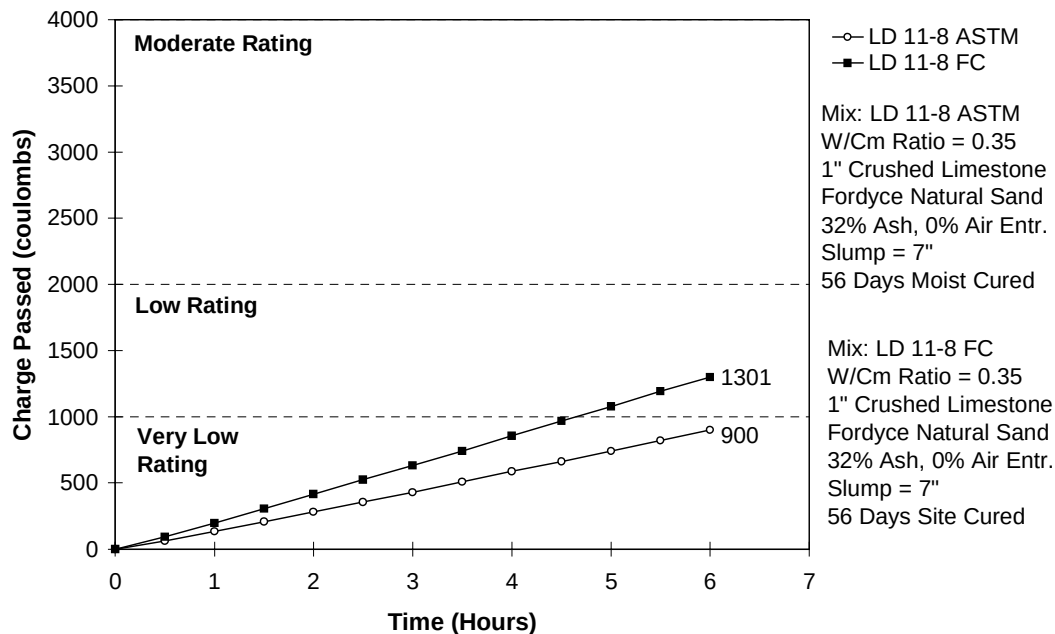


**Figure B.20: Rapid Ion Permeability for Louetta Road Overpass Bridge
Northbound Deck Cast November 8, 1996**

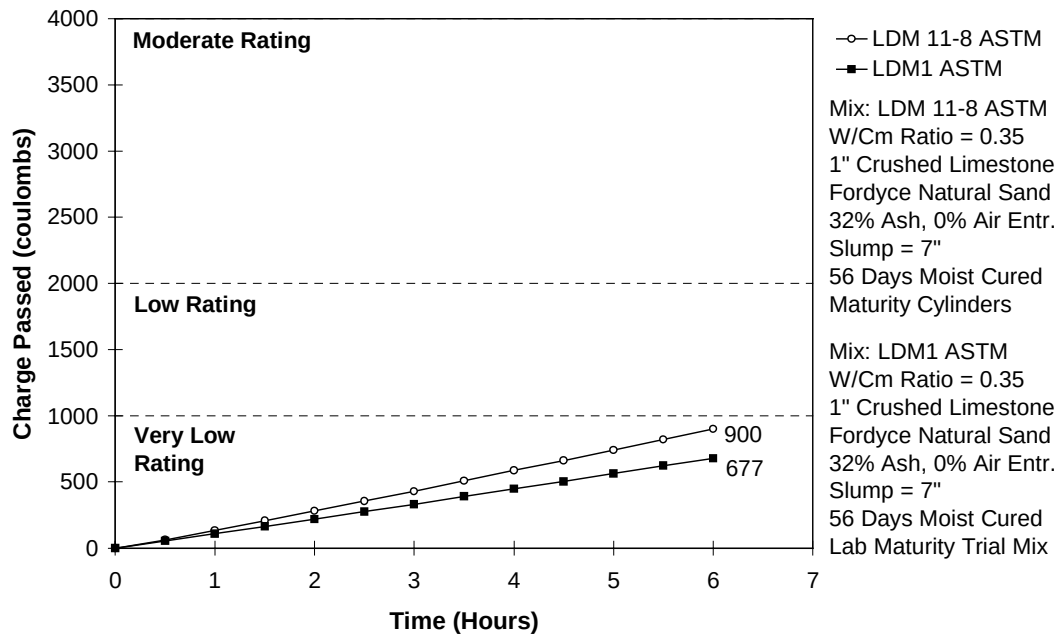


**Figure B.21: Rapid Ion Permeability for Louetta Road Overpass Bridge
Northbound Deck Cast November 8, 1996**

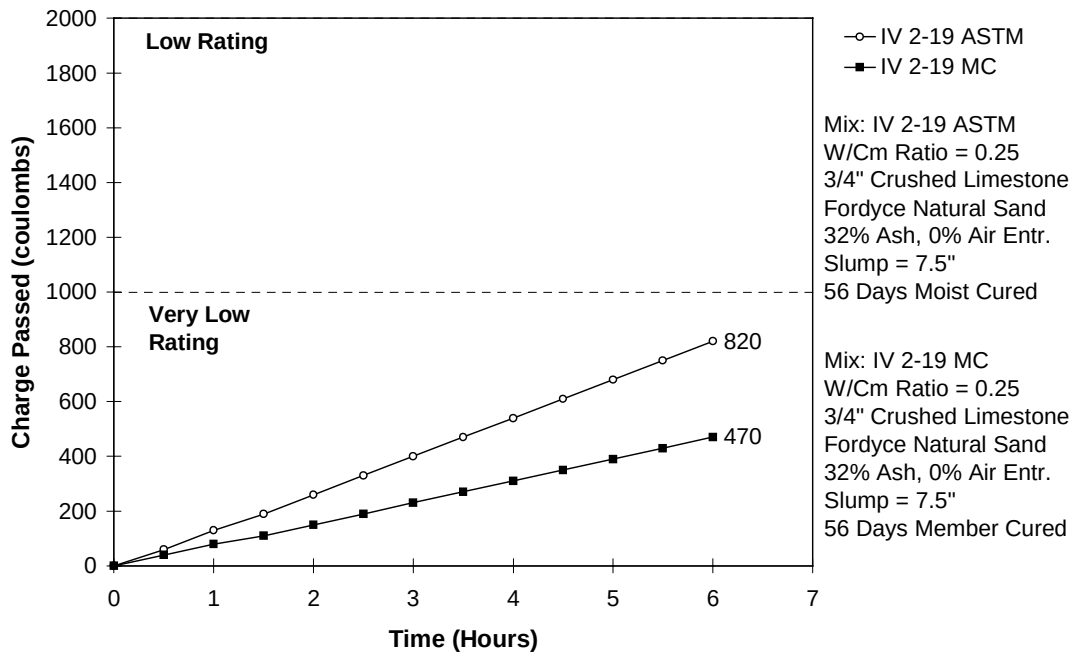


**Figure B.22: Rapid Ion Permeability for San Angelo Type IV Beam
Cast February 19, 1997**

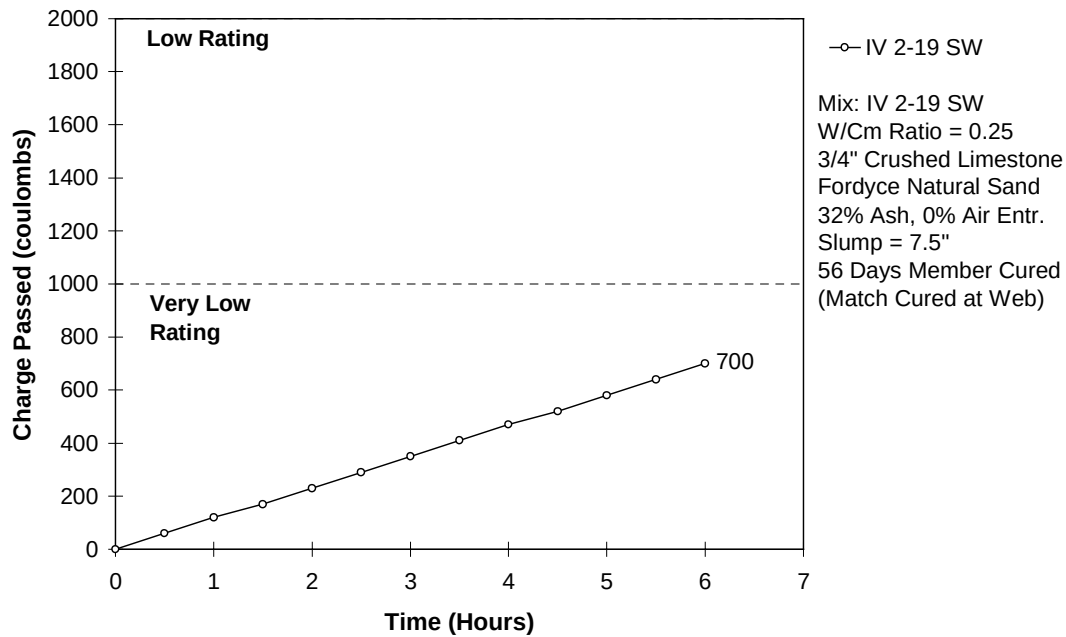


**Figure B.23: Rapid Ion Permeability for San Angelo Type IV Beam
Cast February 19, 1997**

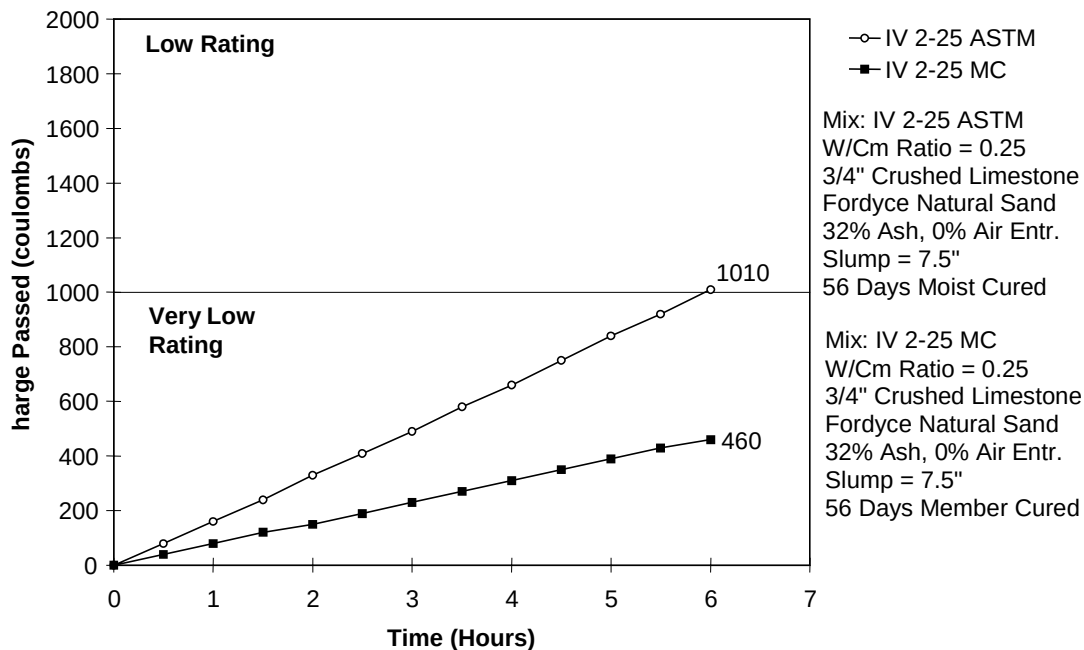


**Figure B.24: Rapid Ion Permeability for San Angelo Type IV Beam
Cast February 25, 1997**

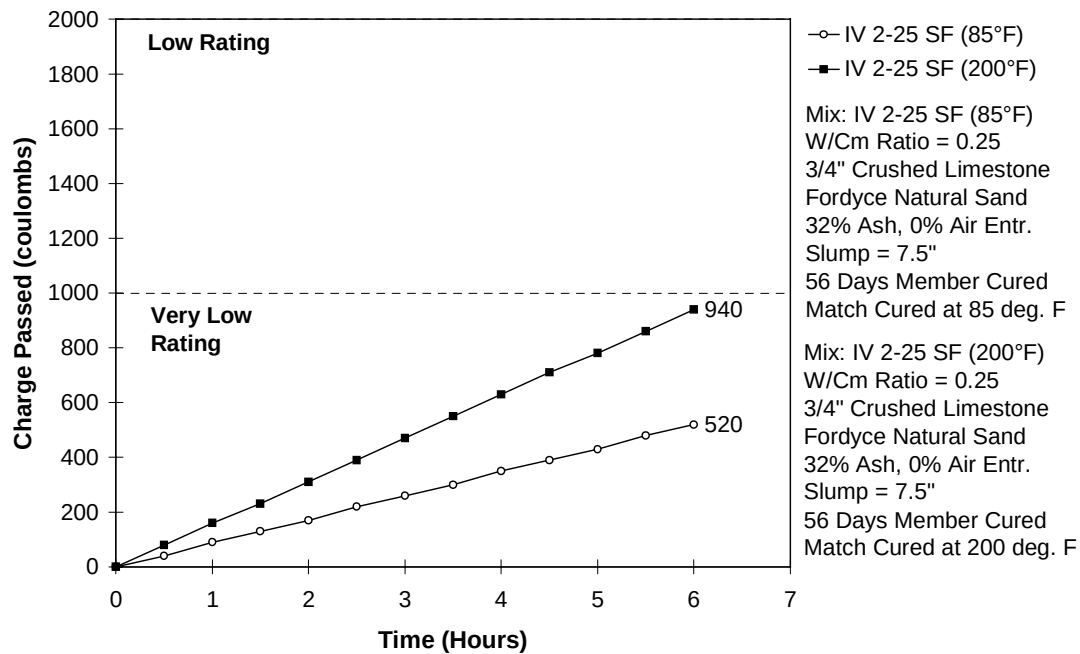


**Figure B.25: Rapid Ion Permeability for San Angelo Type IV Beam
Cast February 25, 1997**

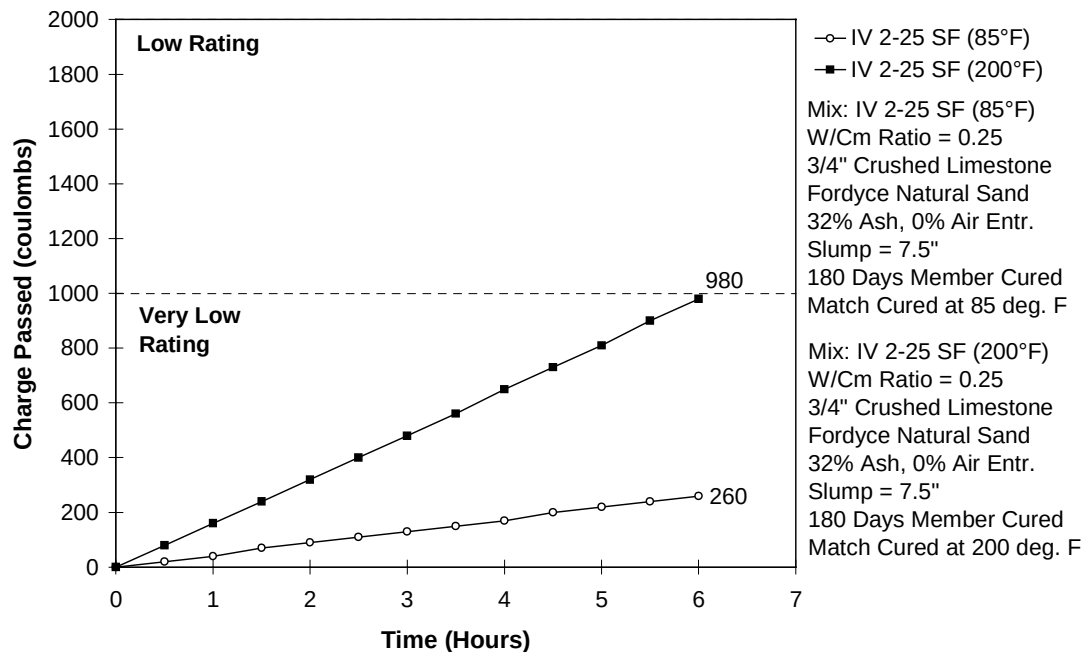


**Figure B.26: Rapid Ion Permeability for San Angelo Type IV Beam
Cast February 25, 1997**

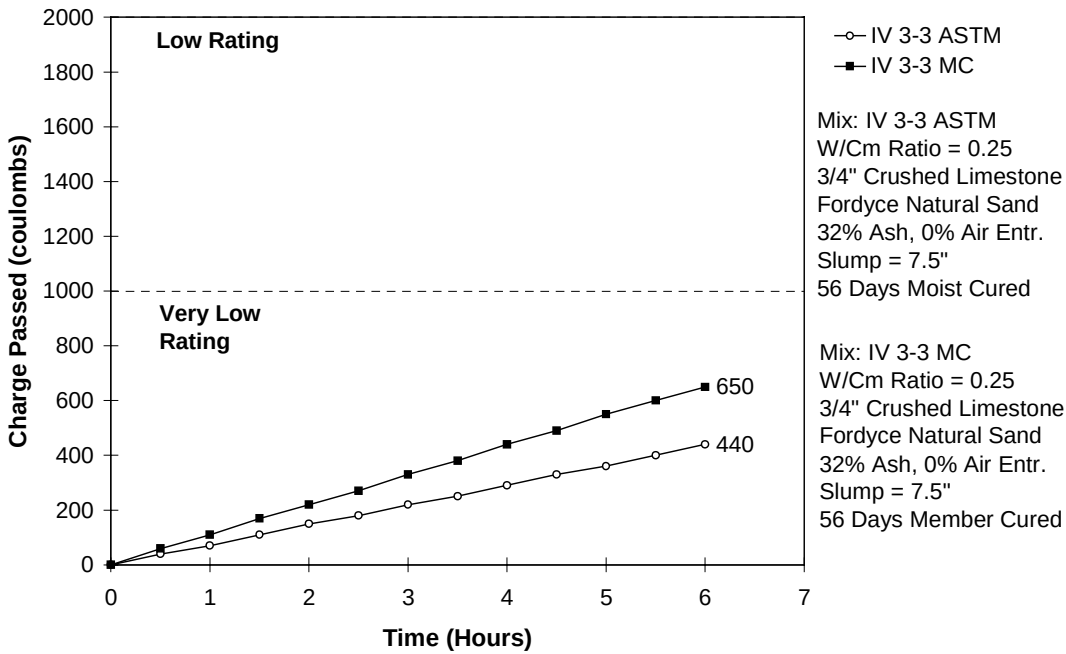


**Figure B.27: Rapid Ion Permeability for San Angelo Type IV Beam
Cast March 3, 1997**

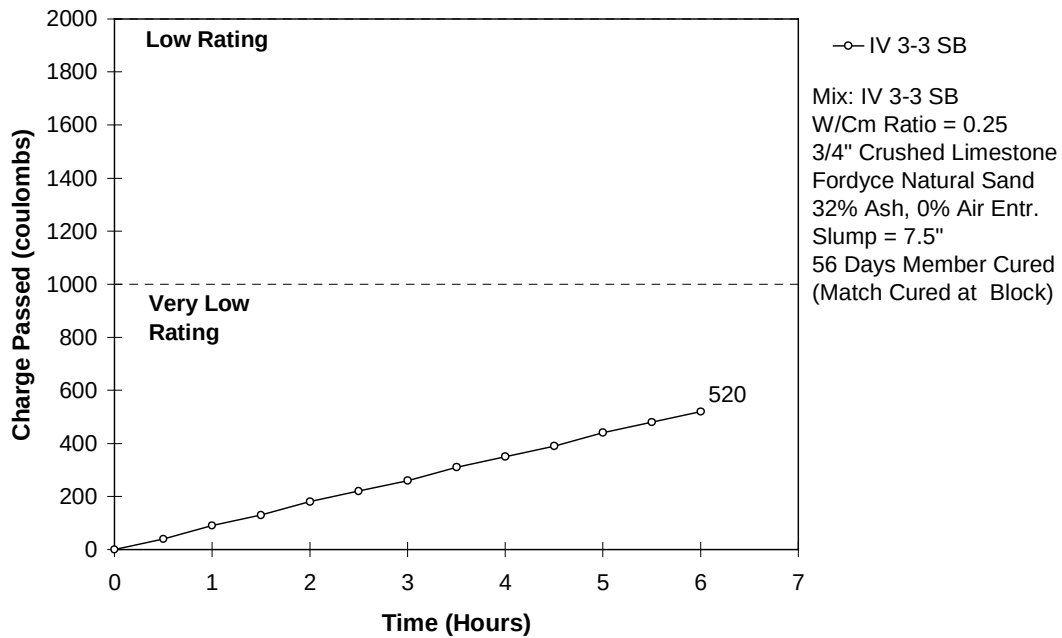


**Figure B.28: Rapid Ion Permeability for San Angelo Type IV Beam
Cast March 3, 1997**

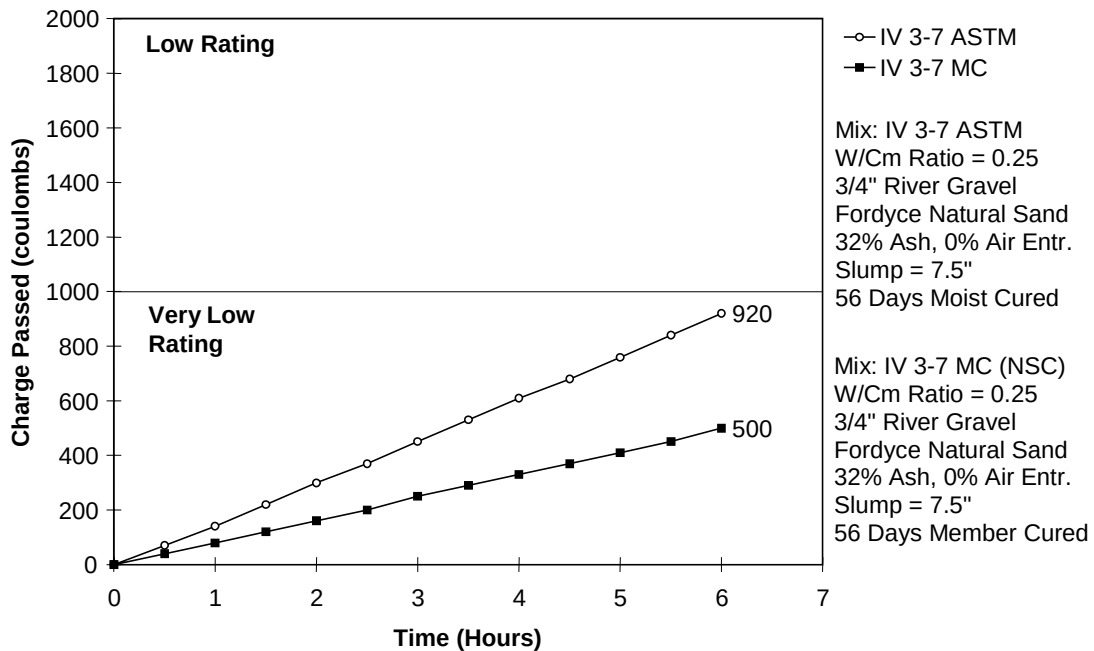


**Figure B.29: Rapid Ion Permeability for San Angelo
Type IV Beam (NSC) Cast March 7, 1997**

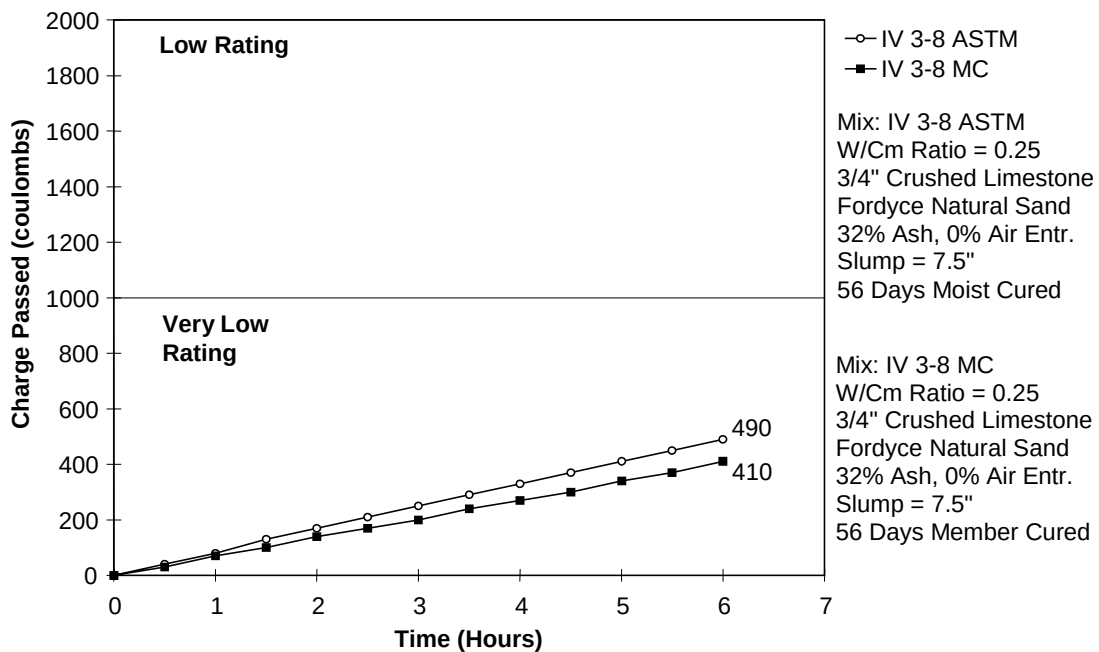


**Figure B.30: Rapid Ion Permeability for San Angelo Type IV Beam
Cast March 8, 1997**

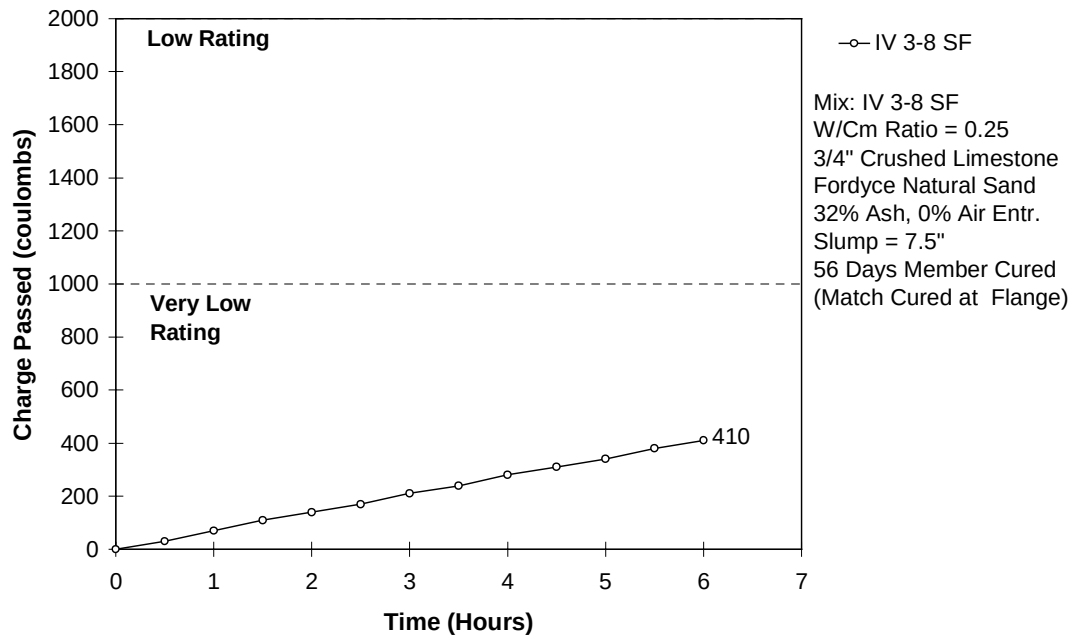


**Figure B.31: Rapid Ion Permeability for San Angelo Type IV Beam
Cast March 8, 1997**

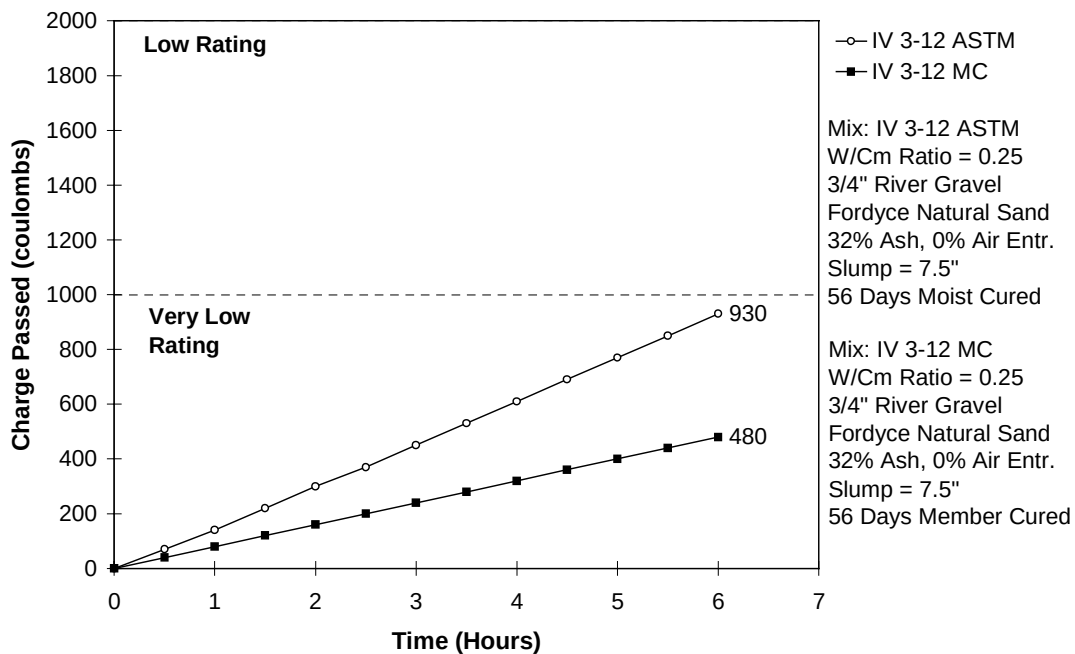


**Figure B.32: Rapid Ion Permeability for San Angelo
Type IV Beam (NSC) Cast March 12, 1997**

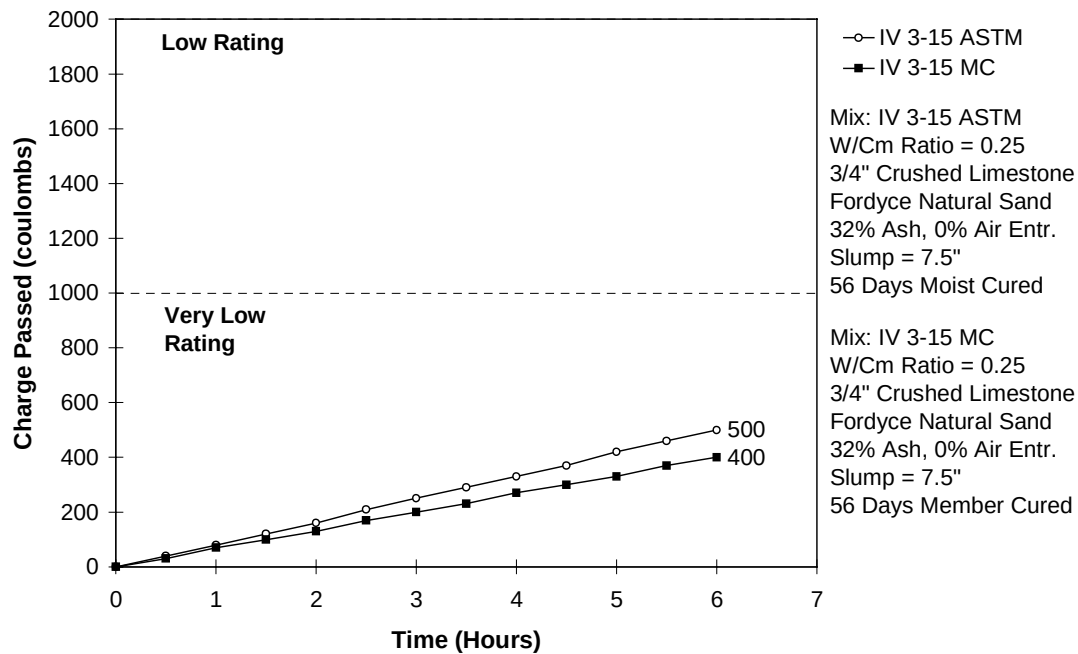


**Figure B.33: Rapid Ion Permeability for San Angelo Type IV Beam
Cast March 15, 1997**

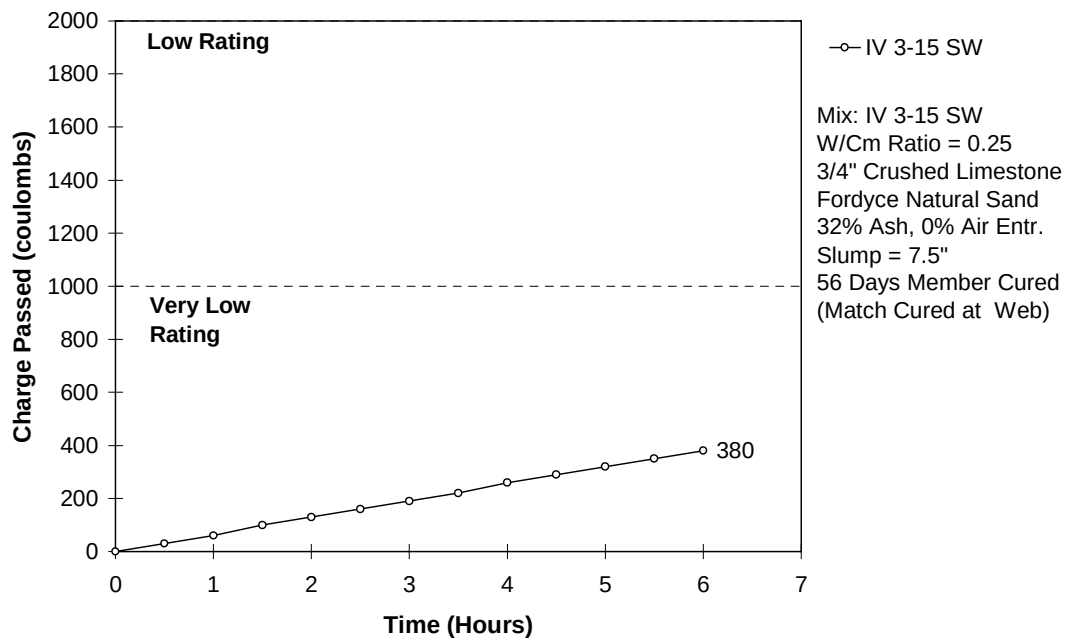


**Figure B.34: Rapid Ion Permeability for San Angelo Type IV Beam
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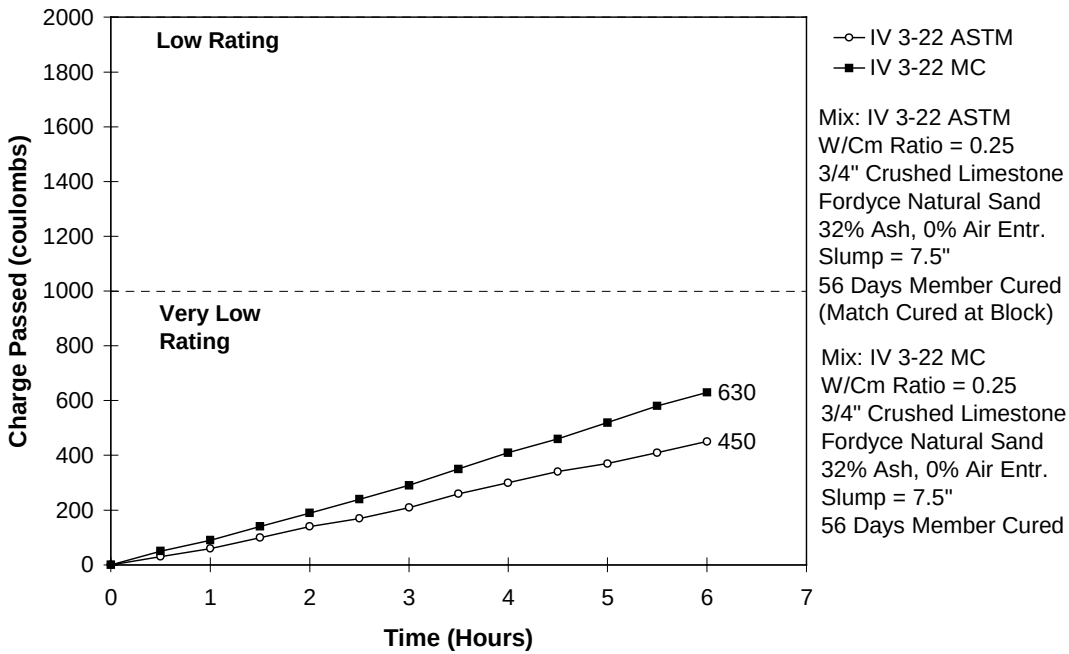


**Figure B.35: Rapid Ion Permeability for San Angelo Type IV Beam
Cast March 22, 1997**

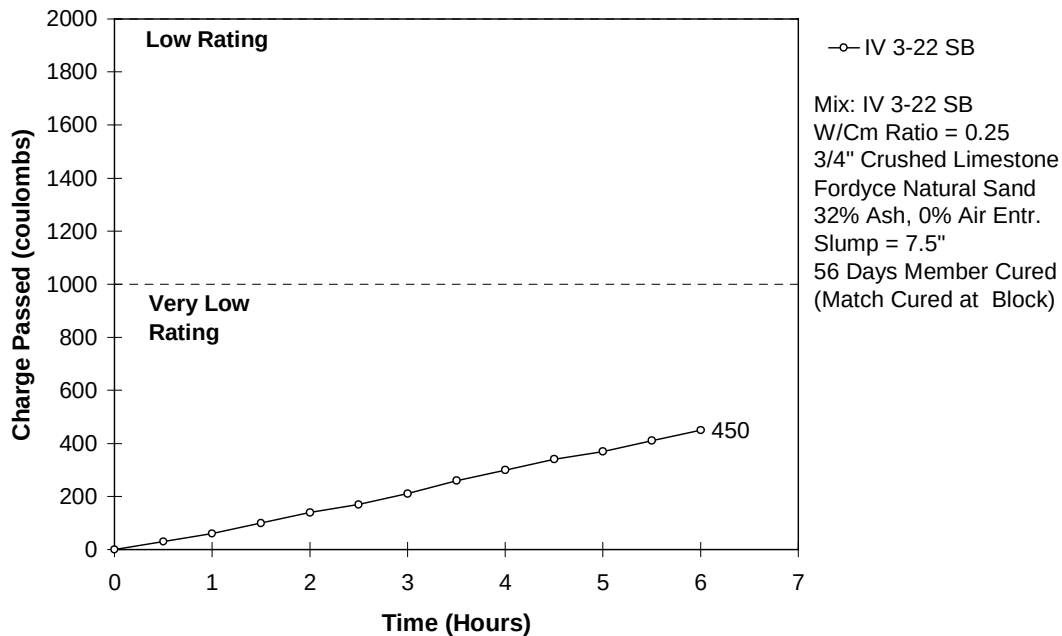


**Figure B.36: Rapid Ion Permeability for San Angelo Type IV Beam
Cast March 22, 1997**

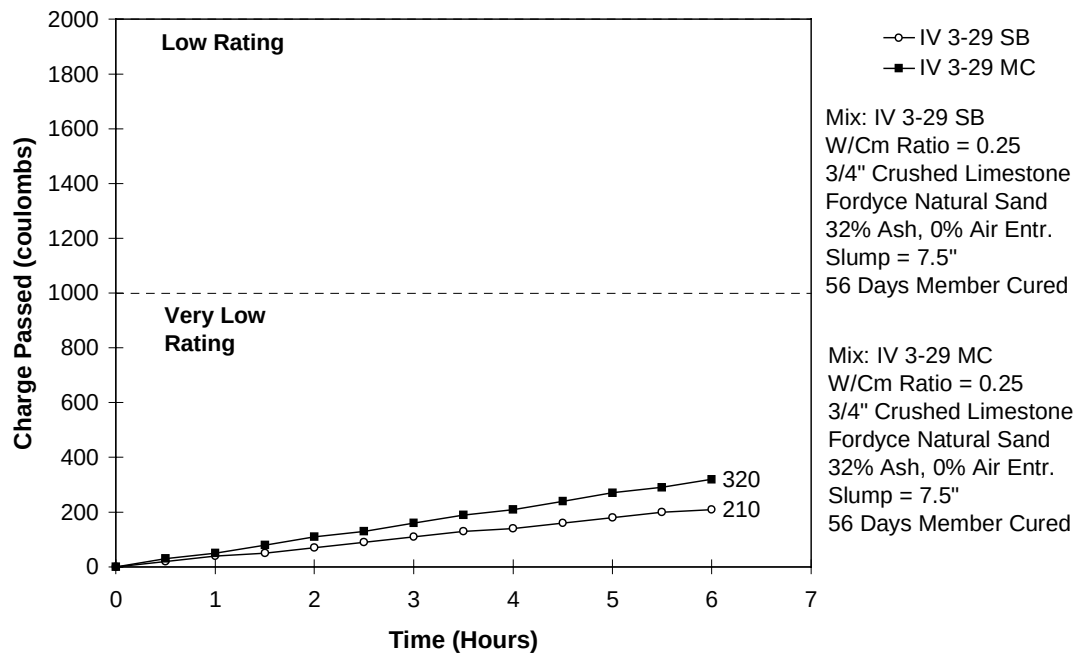


**Figure B.37: Rapid Ion Permeability for San Angelo Type IV Beam
Cast March 29, 1997**

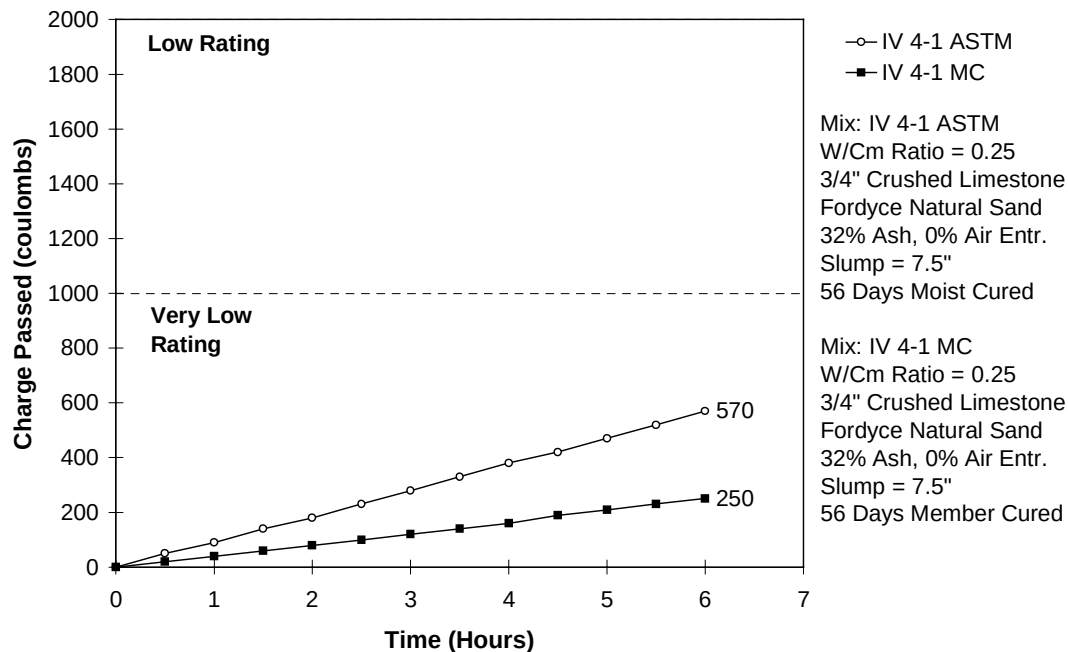


**Figure B.38: Rapid Ion Permeability for San Angelo Type IV Beam
Cast April 1, 1997**

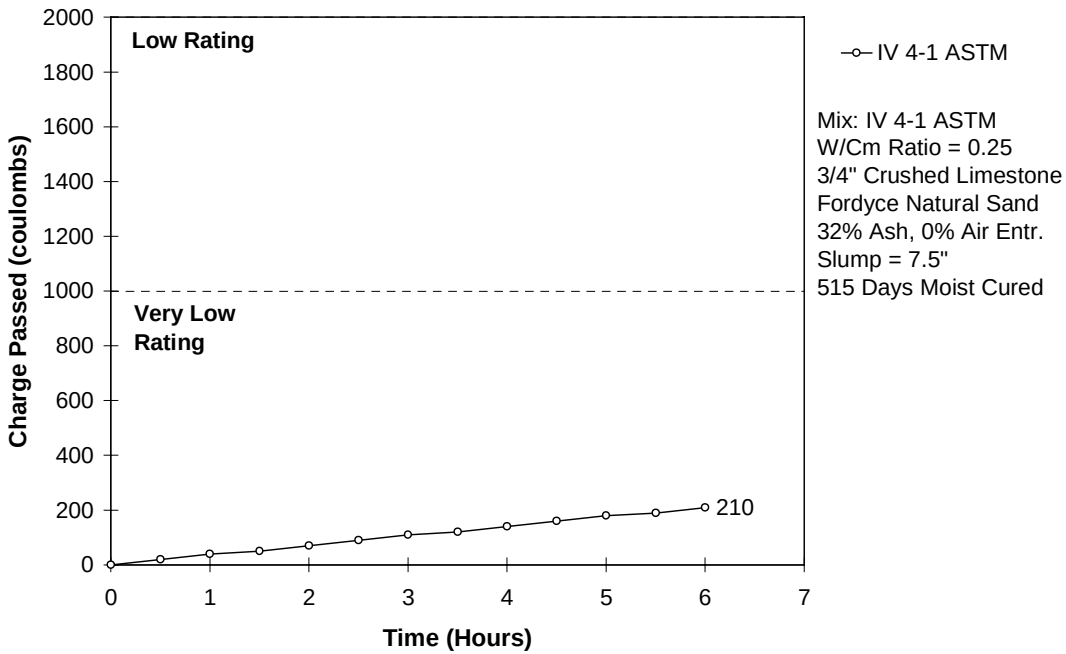


**Figure B.39: Rapid Ion Permeability for San Angelo Type IV Beam
Cast April 1, 1997**

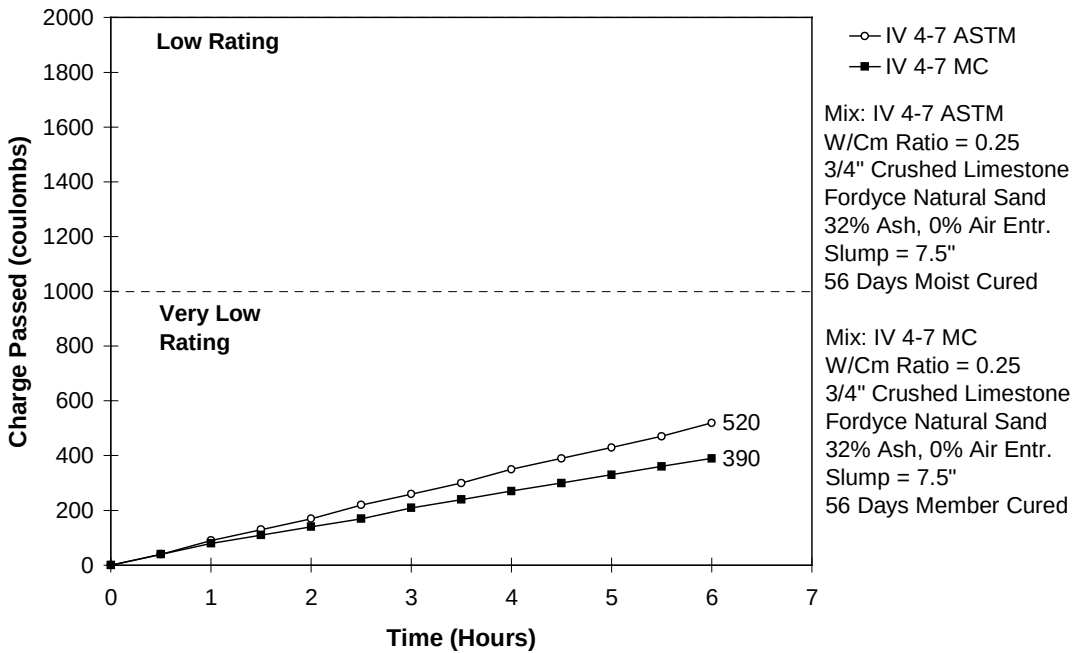


**Figure B.40: Rapid Ion Permeability for San Angelo Type IV Beam
Cast April 7, 1997**

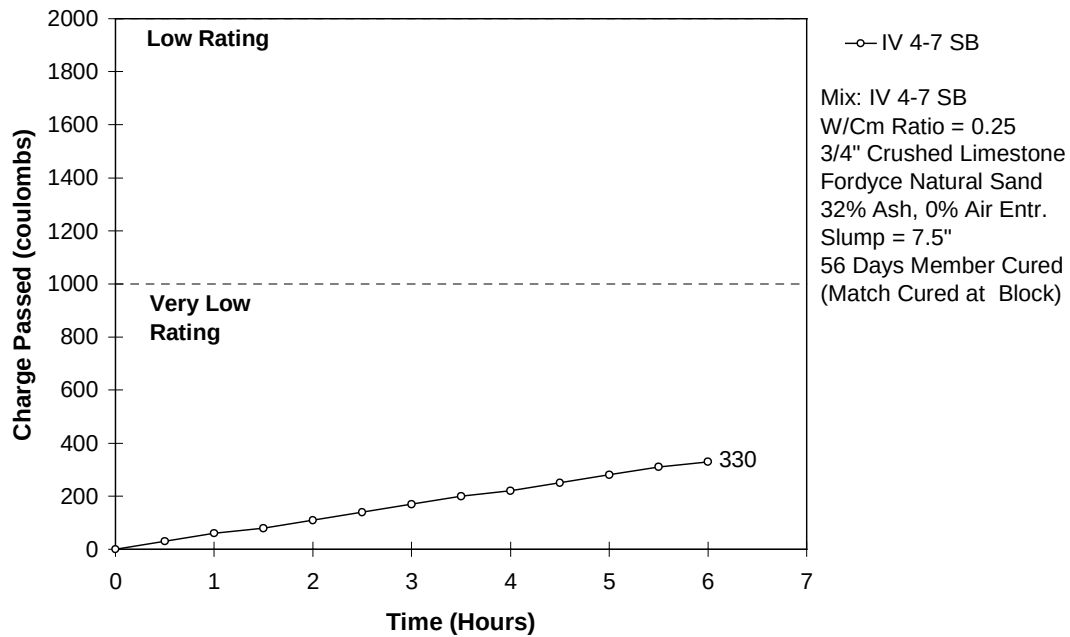


**Figure B.41: Rapid Ion Permeability for San Angelo Type IV Beam
Cast April 7, 1997**

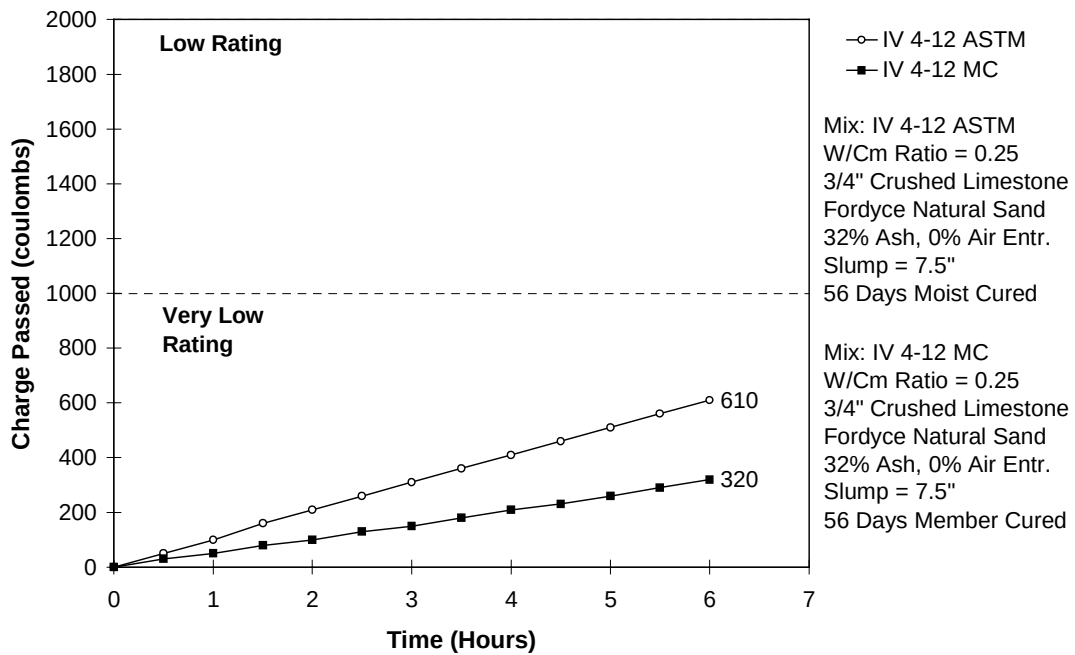


**Figure B.42: Rapid Ion Permeability for San Angelo Type IV Beam
Cast April 12, 1997**

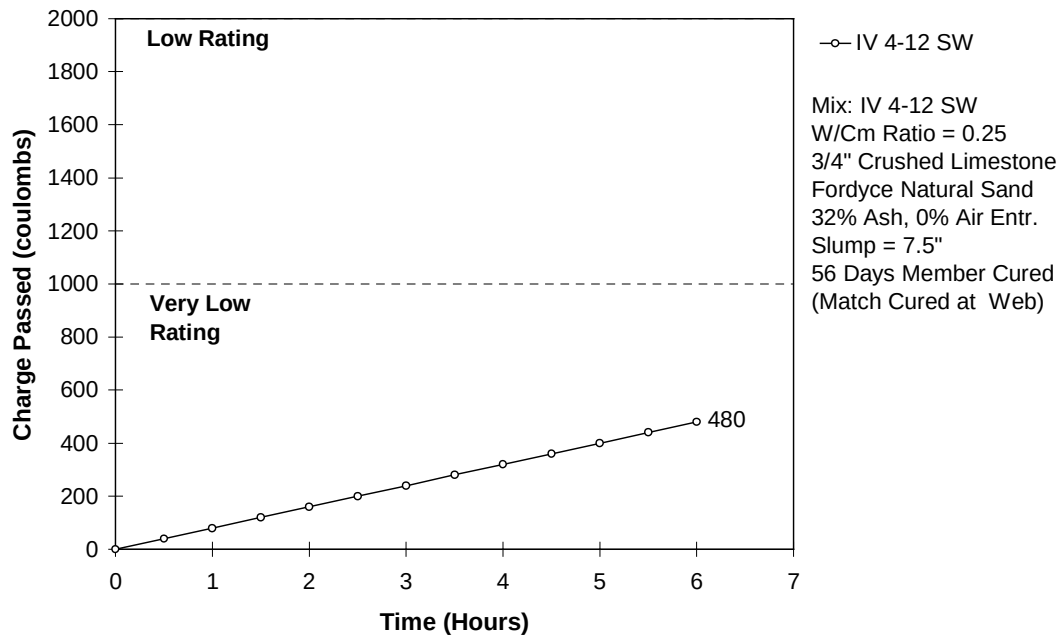


**Figure B.43: Rapid Ion Permeability for San Angelo Type IV Beam
Cast April 12, 1997**

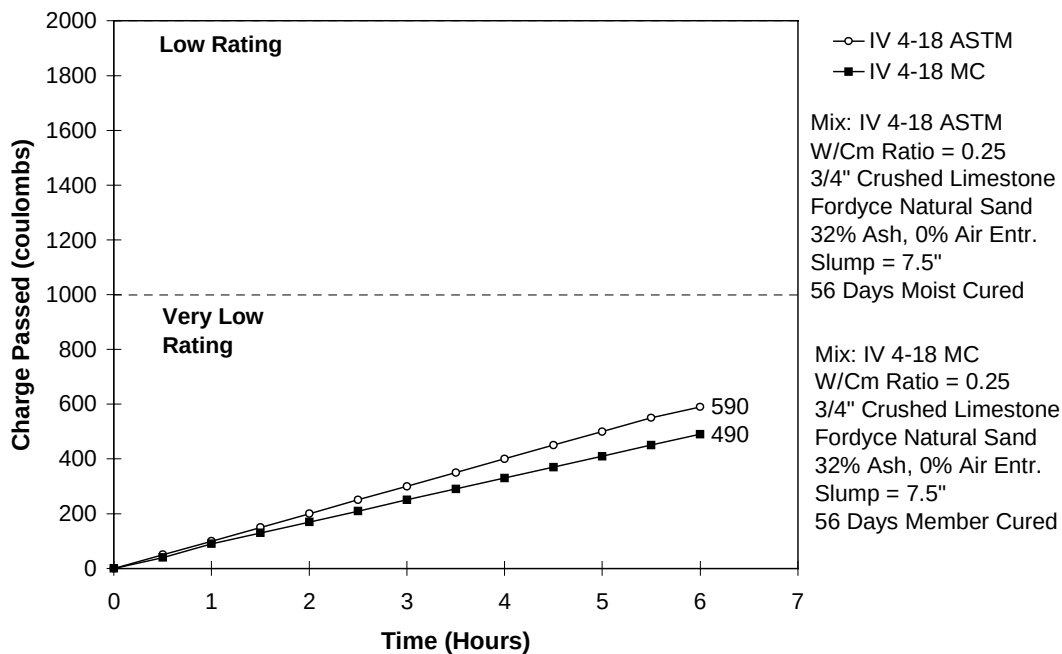


**Figure B.44: Rapid Ion Permeability for San Angelo Type IV Beam
Cast April 18, 1997**

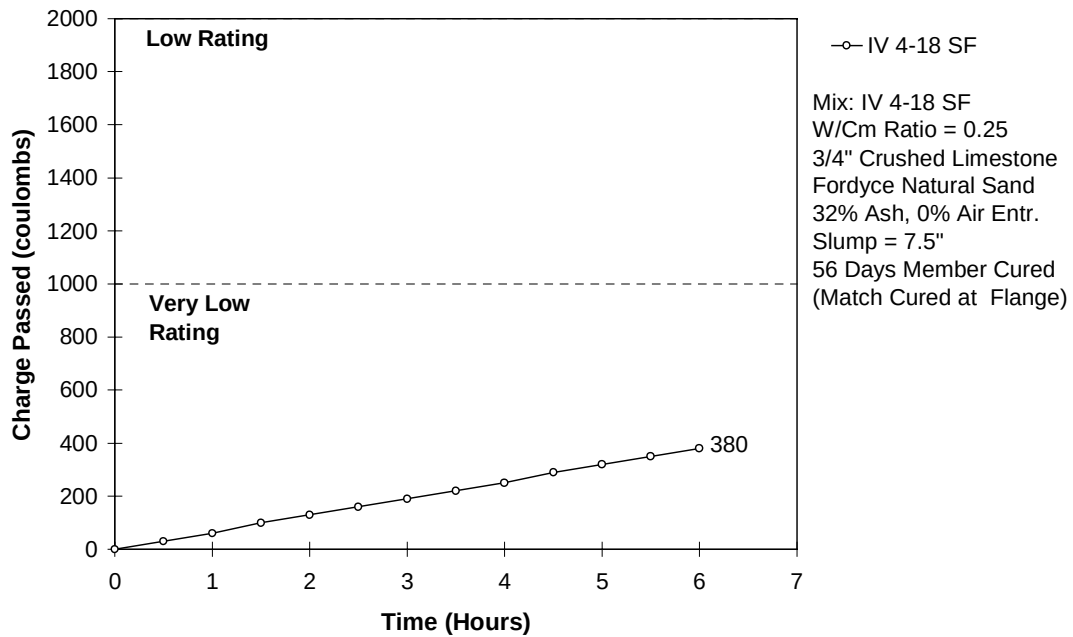


**Figure B.45: Rapid Ion Permeability for San Angelo Type IV Beam
Cast April 18, 1997**

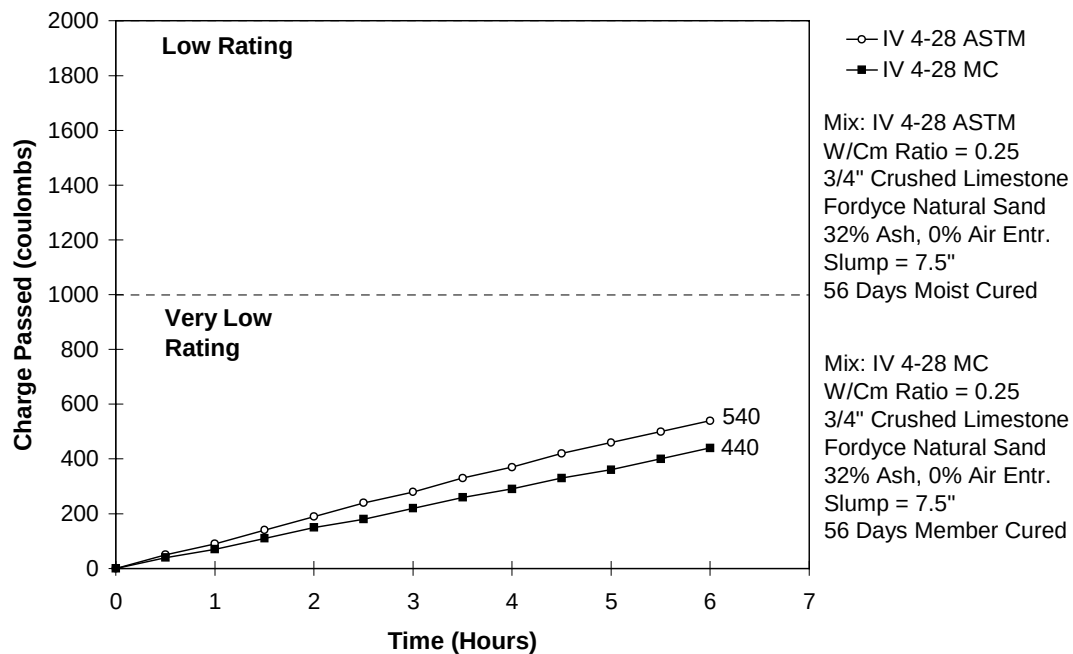


**Figure B.46: Rapid Ion Permeability for San Angelo Type IV Beam
Cast April 28, 1997**

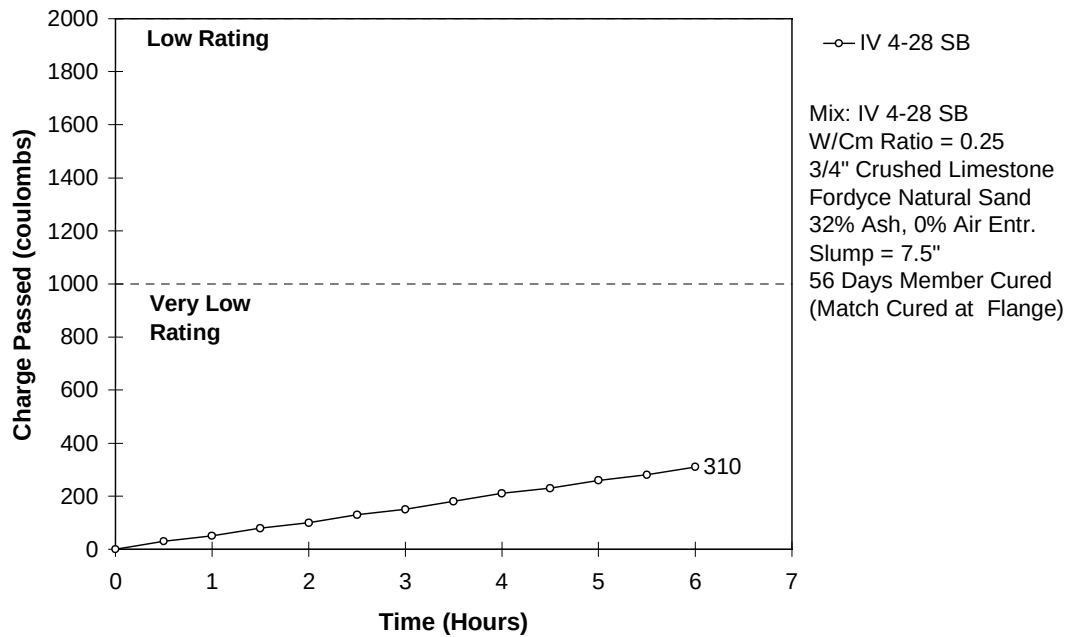


**Figure B.47: Rapid Ion Permeability for San Angelo Type IV Beam
Cast April 28, 1997**

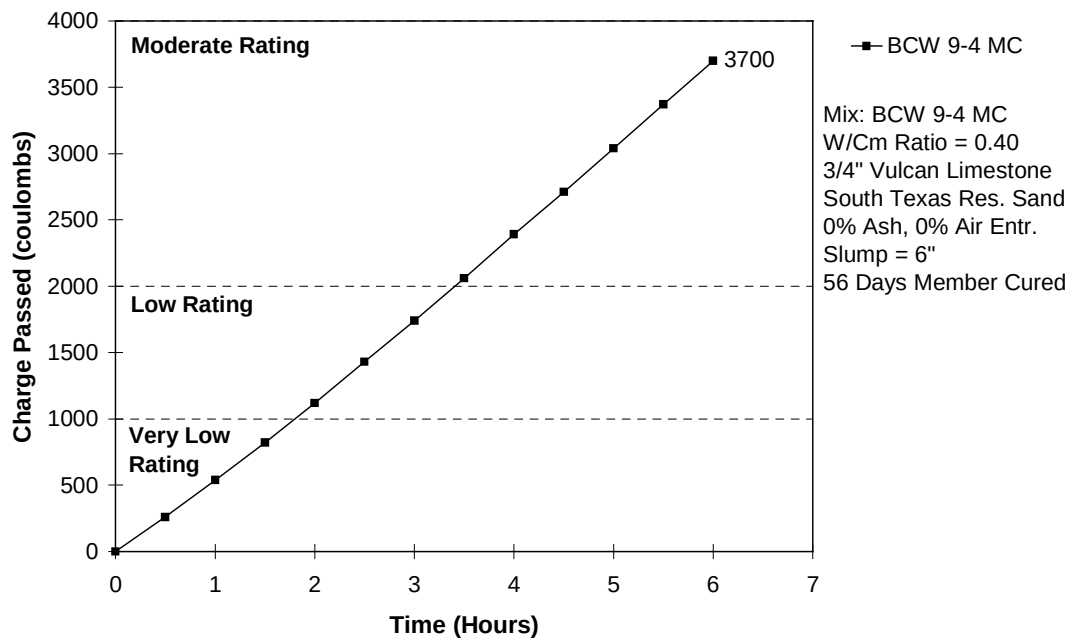


**Figure B.48: Rapid Ion Permeability for San Angelo Precast Panels
Cast September 4, 1996**

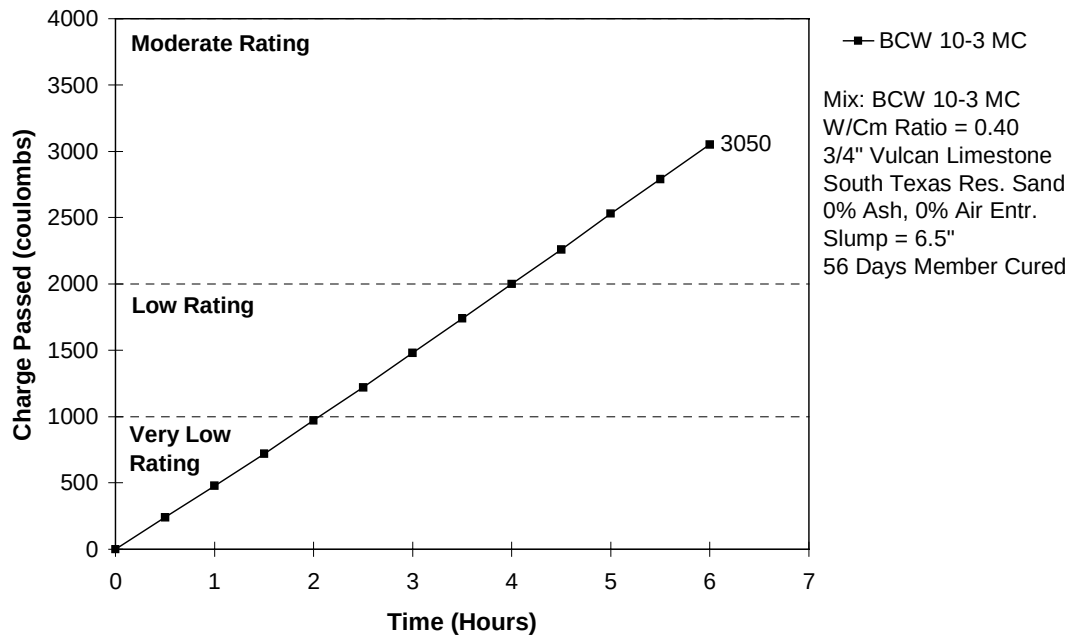


**Figure B.49: Rapid Ion Permeability for San Angelo Precast Panels
Cast October 3, 1996**

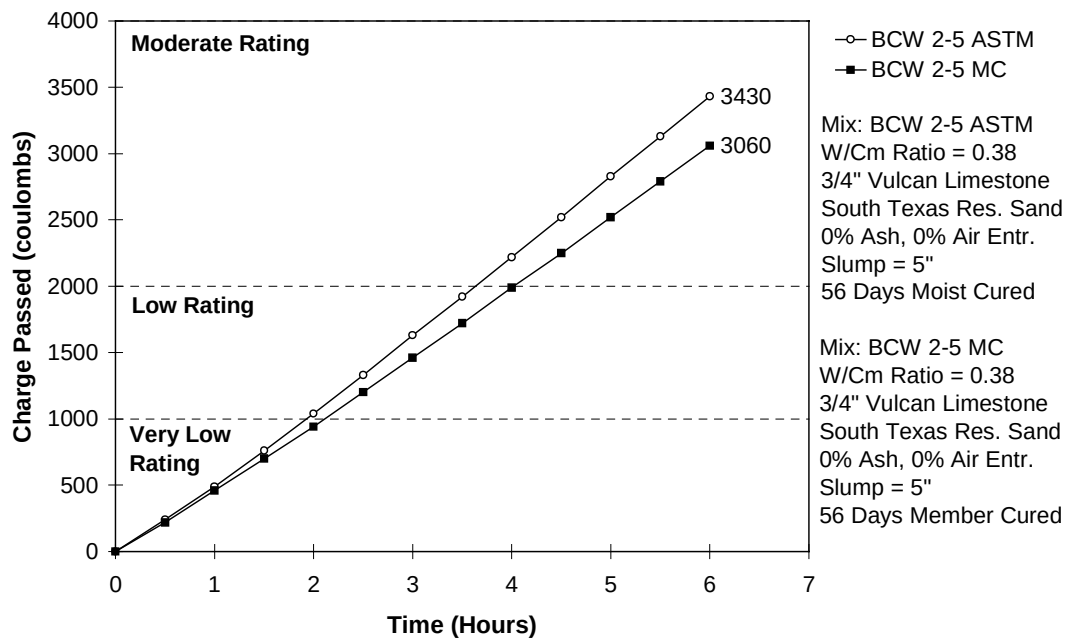


**Figure B.50: Rapid Ion Permeability for San Angelo Precast Panels
Cast February 5, 1997**

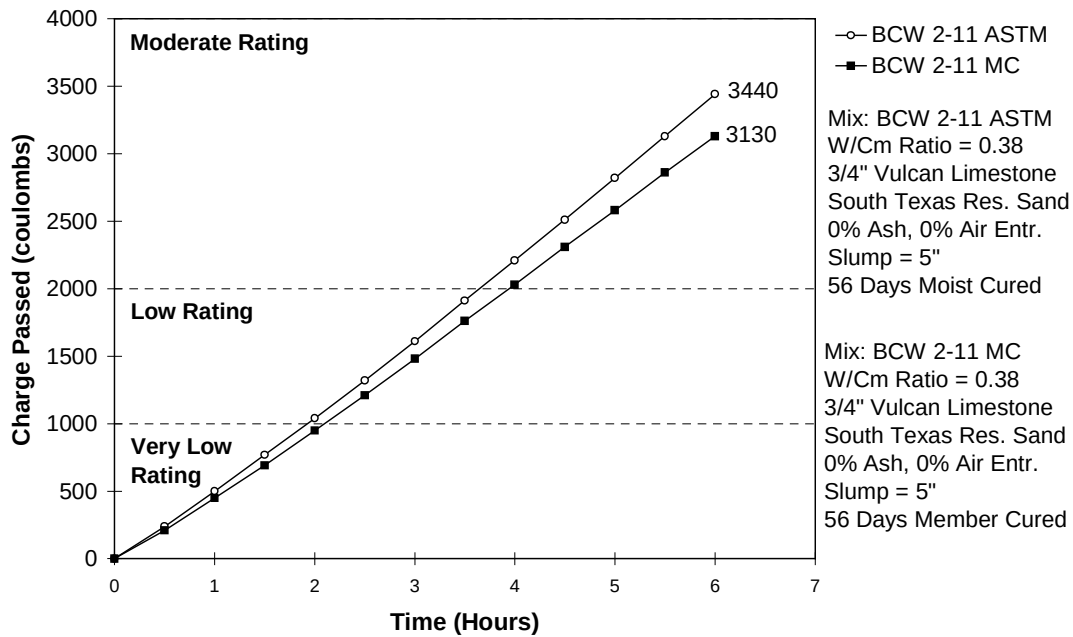


**Figure B.51: Rapid Ion Permeability for San Angelo Precast Panels
Cast February 11, 1997**

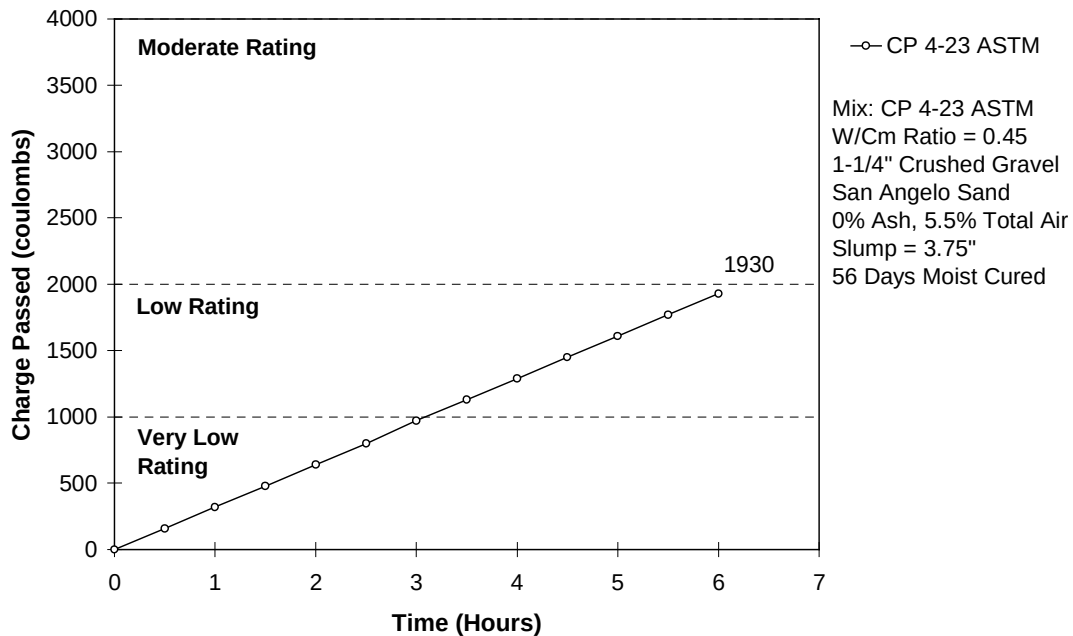


**Figure B.52: Rapid Ion Permeability for San Angelo Class C Pier
Cast April 23, 1996**

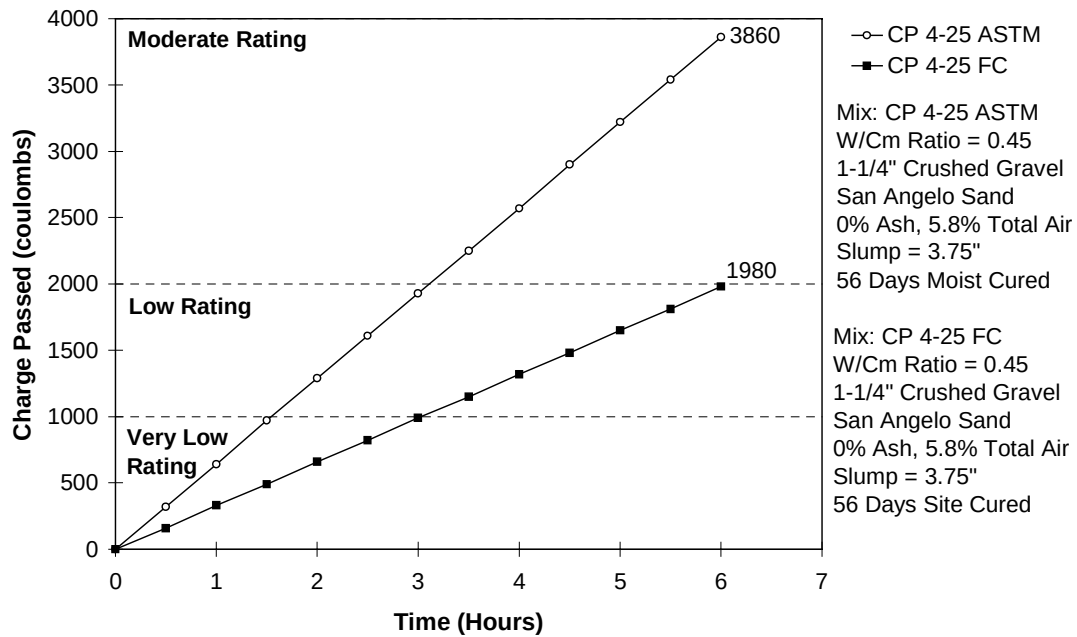


**Figure B.53: Rapid Ion Permeability for San Angelo Class C Pier
Cast April 25, 1996**

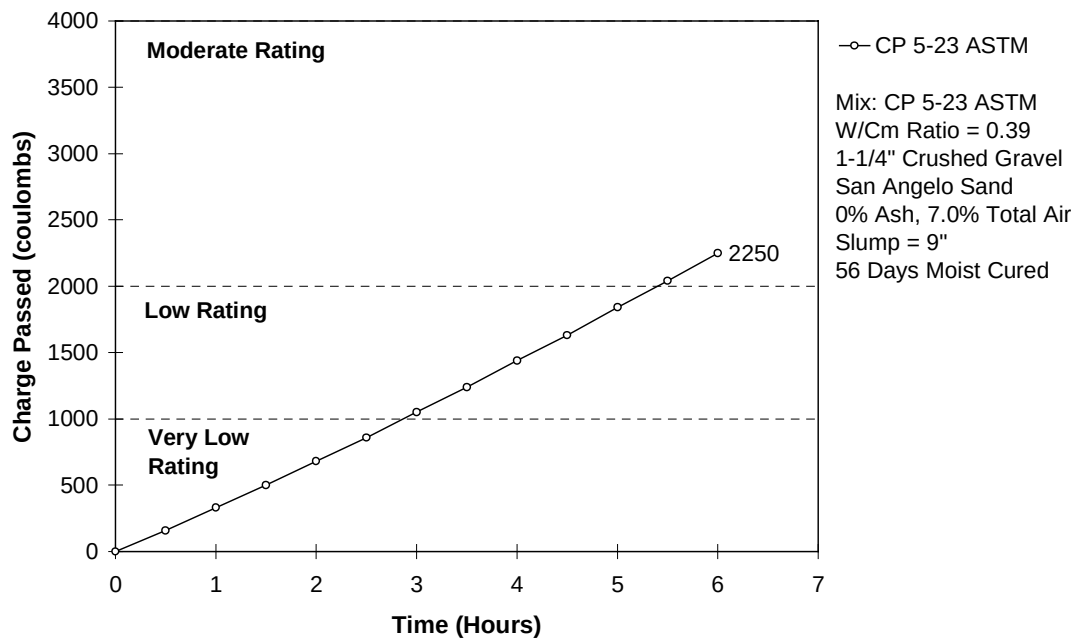


**Figure B.54: Rapid Ion Permeability for San Angelo Class C Pier
Cast May 23, 1996**

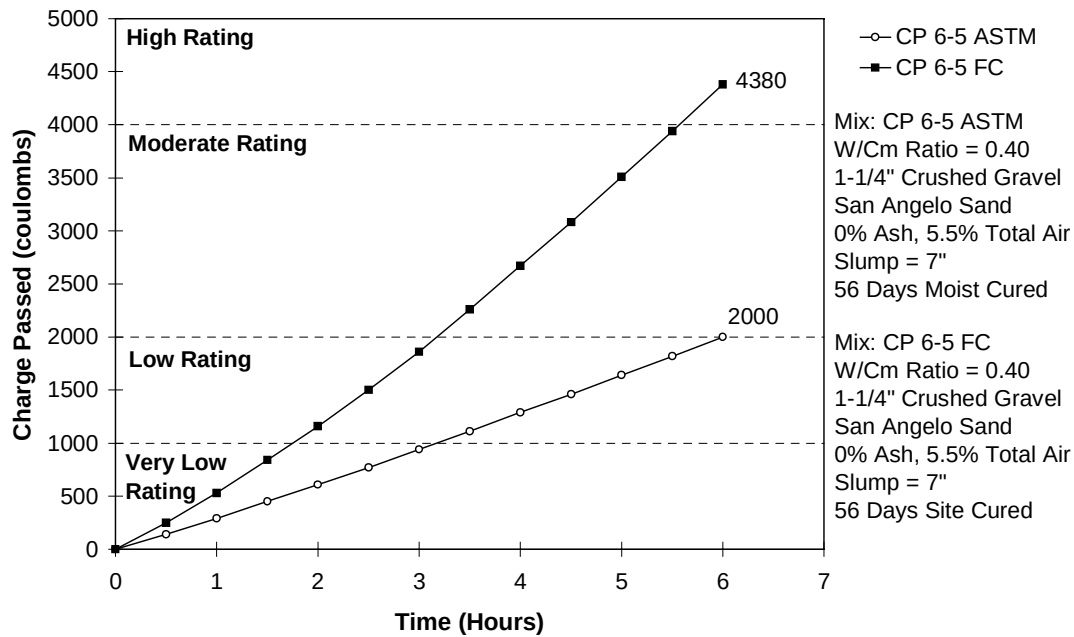


**Figure B.55: Rapid Ion Permeability for San Angelo Class C Pier
Cast June 5, 1996**

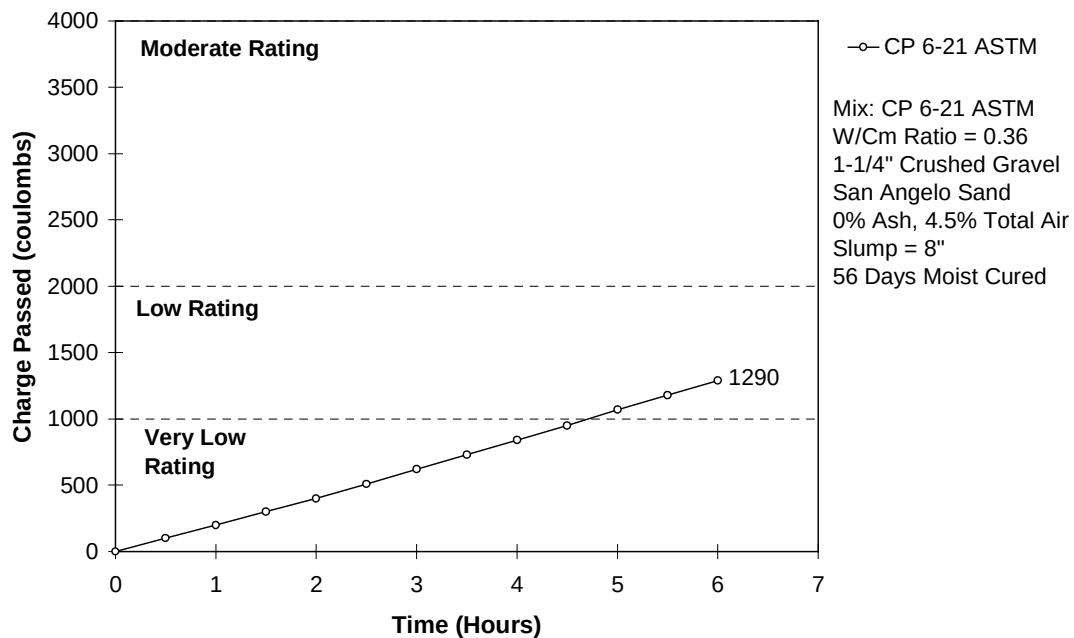


**Figure B.56: Rapid Ion Permeability for San Angelo Class C Pier
Cast June 21, 1996**

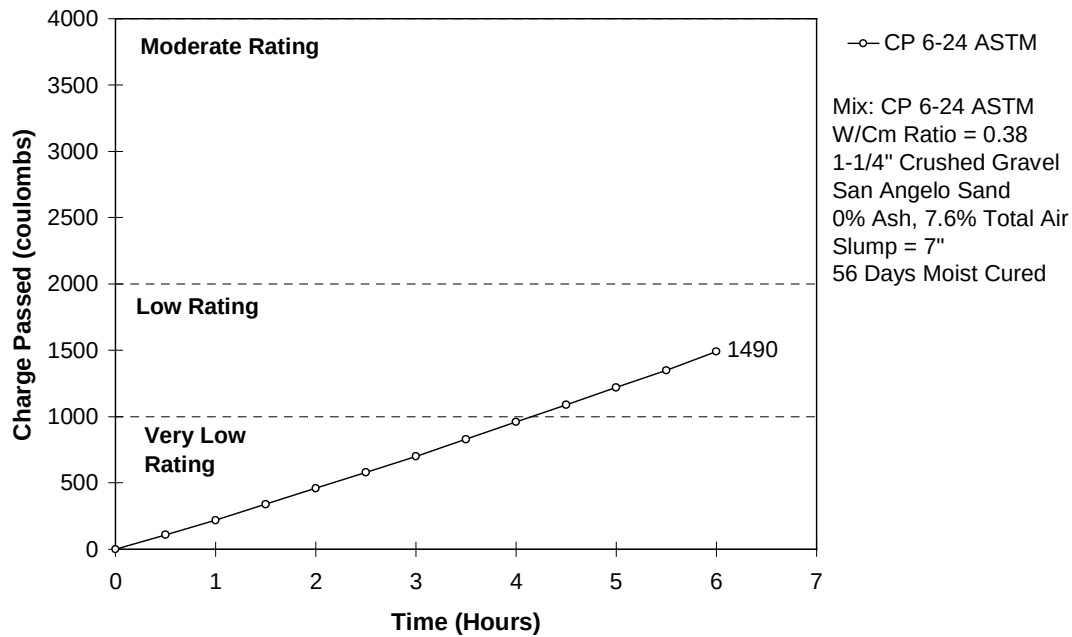


**Figure B.57: Rapid Ion Permeability for San Angelo Class C Pier
Cast June 24, 1996**

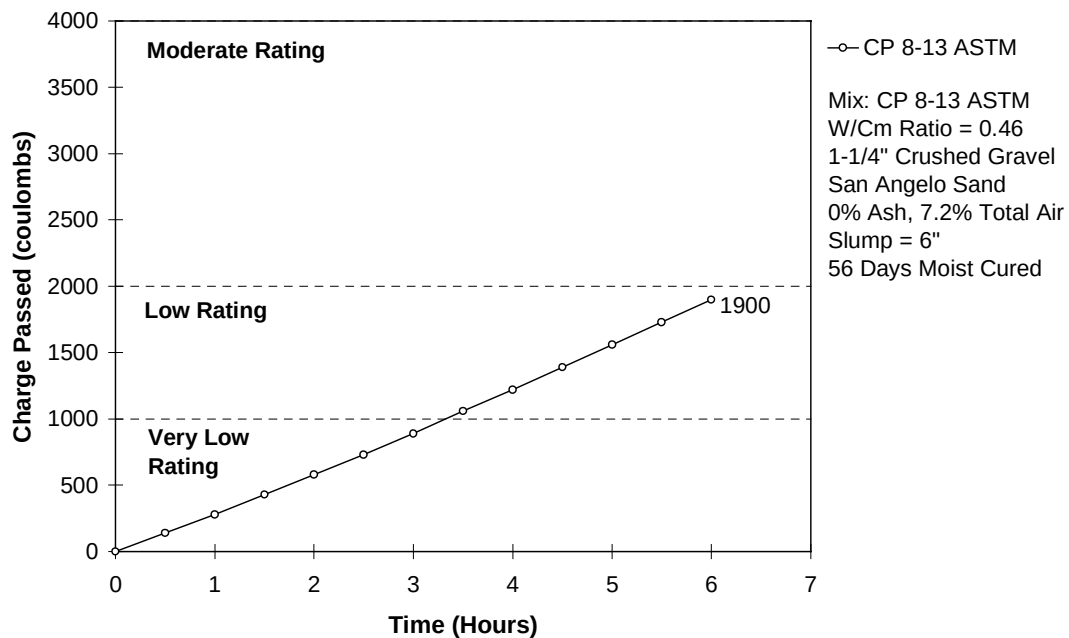


**Figure B.58: Rapid Ion Permeability for San Angelo Class C Pier
Cast August 13, 1996**

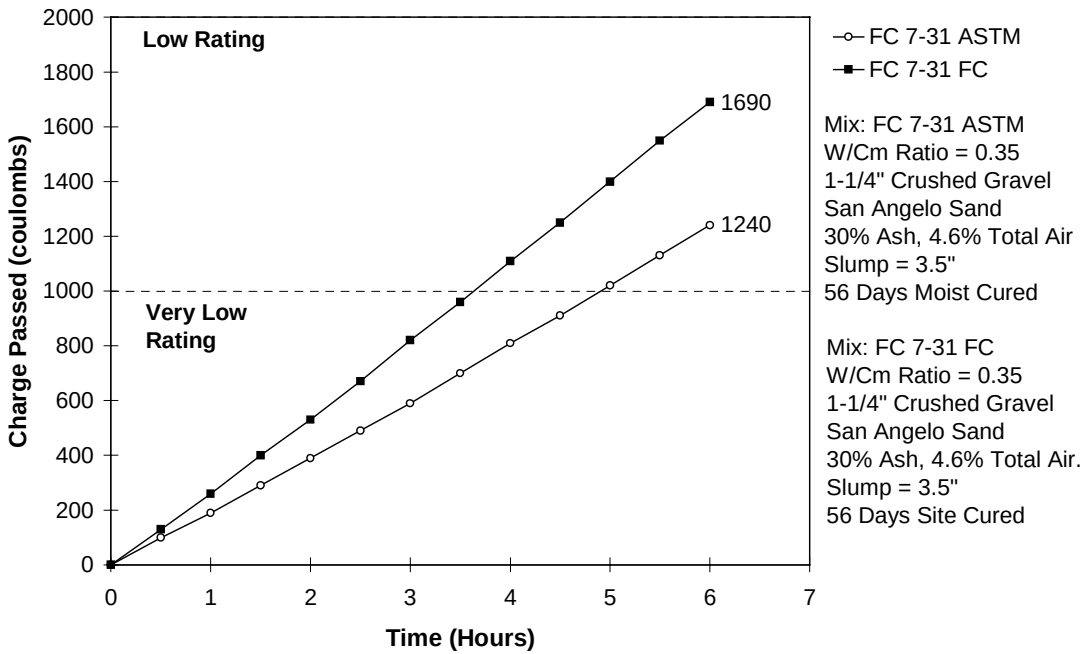


**Figure B.59: Rapid Ion Permeability for San Angelo Class F Pier Cap
Cast July 31, 1996**

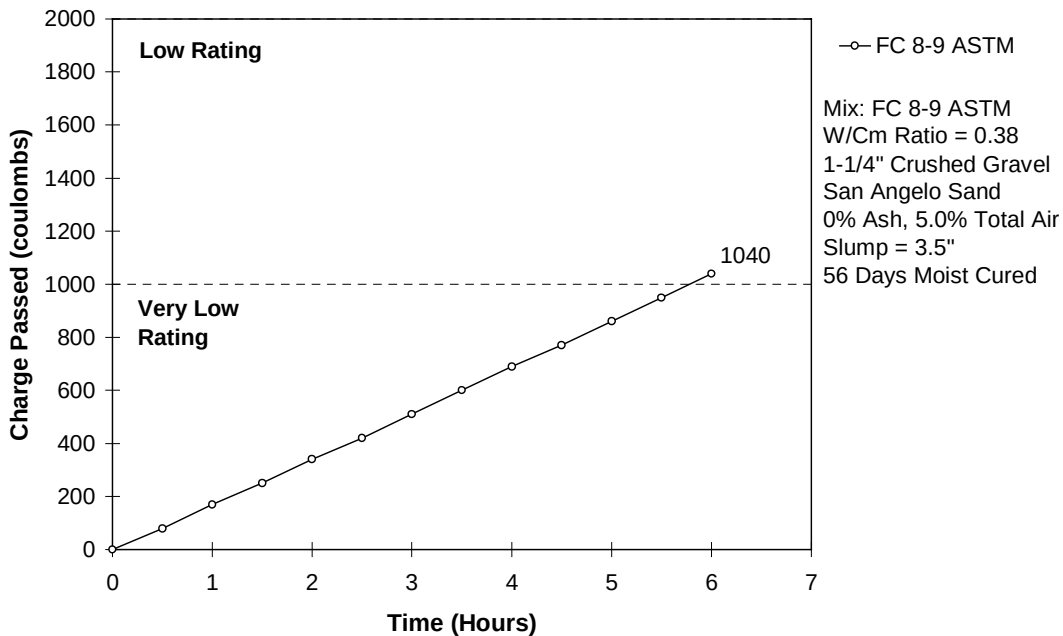


**Figure B.60: Rapid Ion Permeability for San Angelo Class F Pier Cap
Cast August 9, 1996**

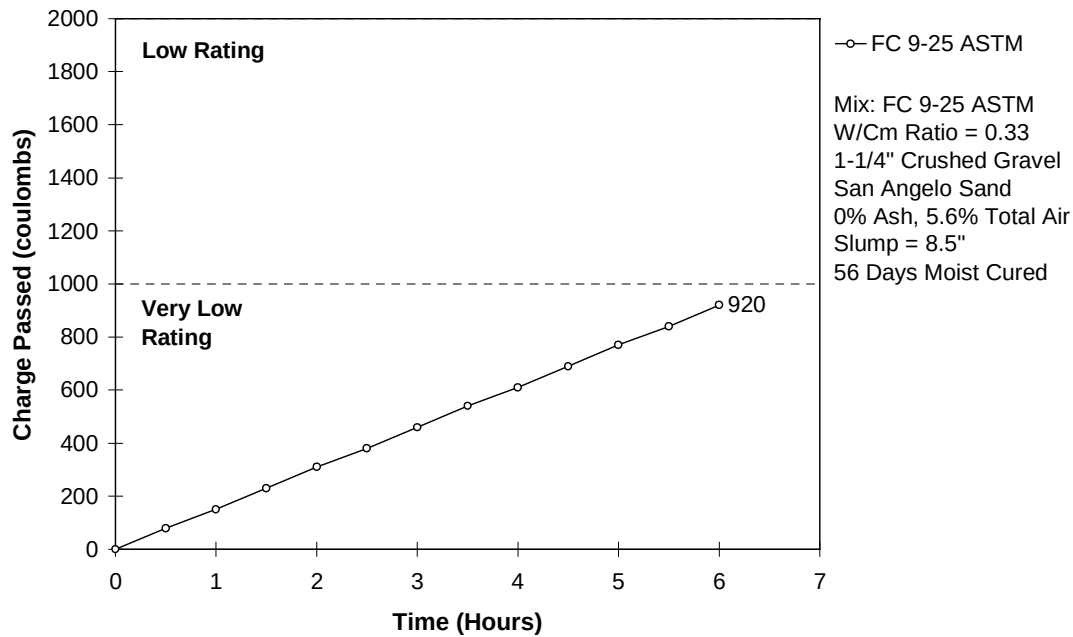


**Figure B.61: Rapid Ion Permeability for San Angelo Class F Pier Cap
Cast September 25, 1996**

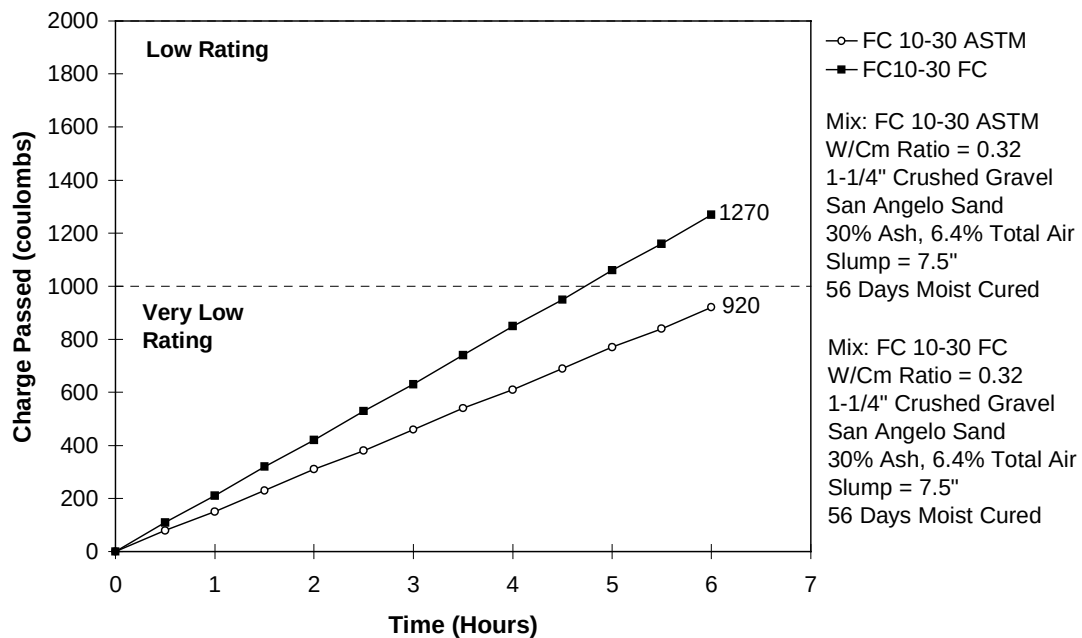


**Figure B.62: Rapid Ion Permeability for San Angelo Class F Pier Cap
Cast October 30, 1996**

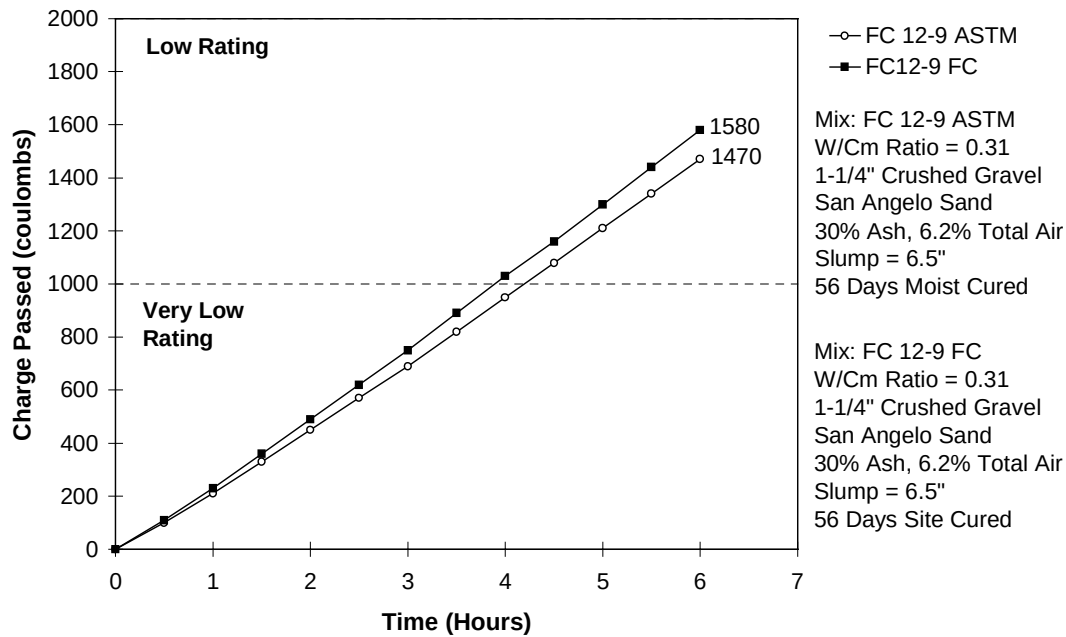


**Figure B.63: Rapid Ion Permeability for San Angelo Class F Pier Cap
Cast December 9, 1996**

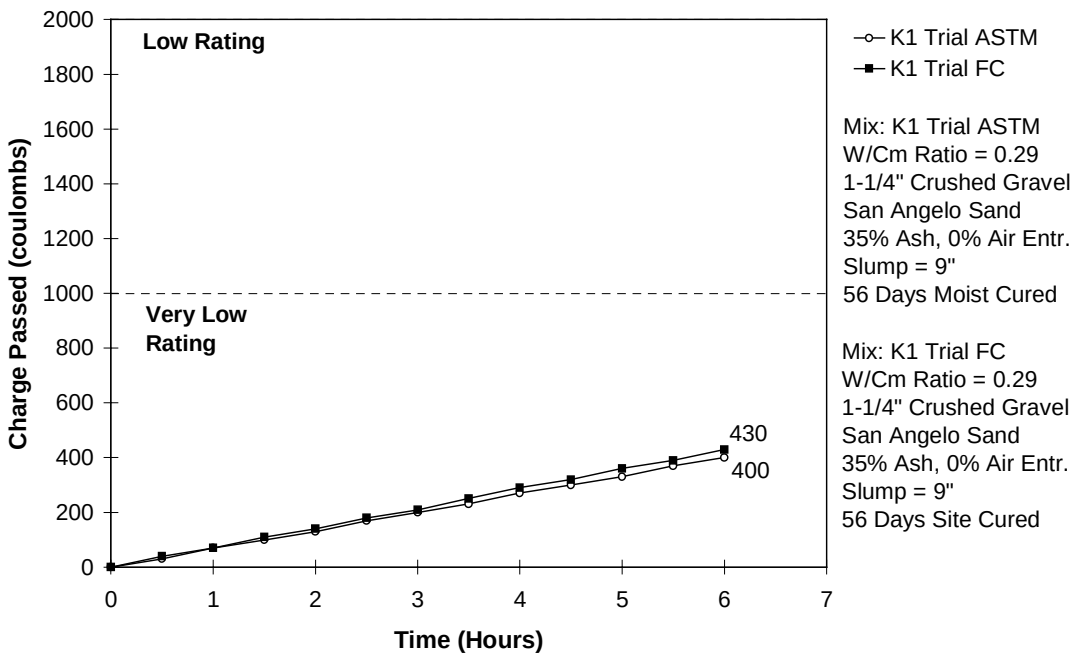


Figure B.64: Rapid Ion Permeability for San Angelo Class K Pier Field Trial Mix 1

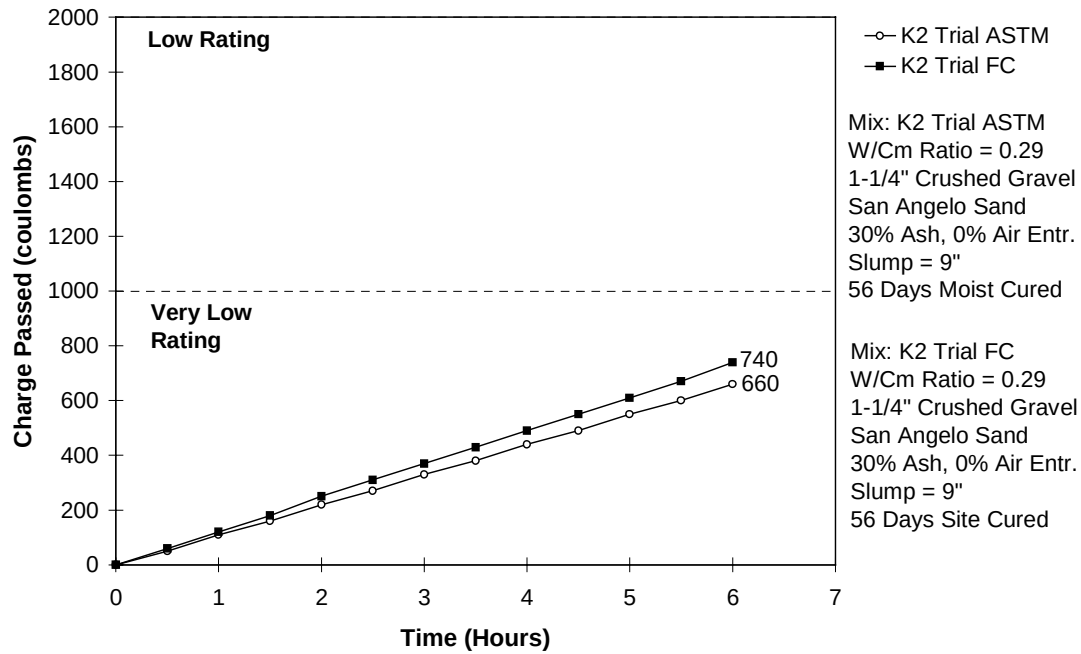


Figure B.65: Rapid Ion Permeability for San Angelo Class K Pier Field Trial Mix 2

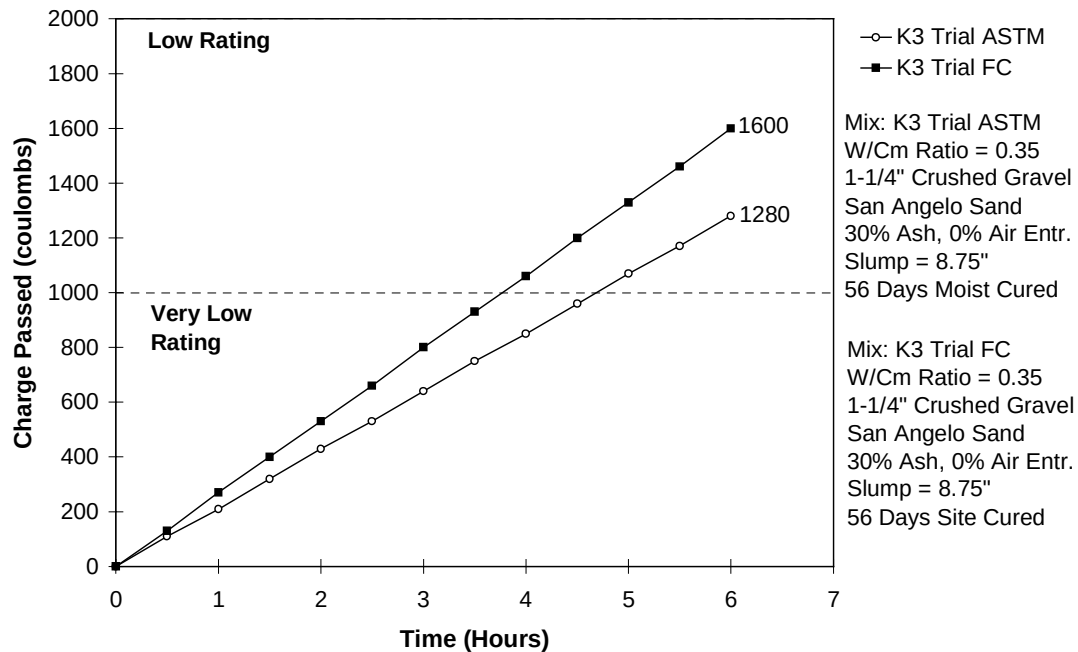


Figure B.66: Rapid Ion Permeability for San Angelo Class K Pier Field Trial Mix 3

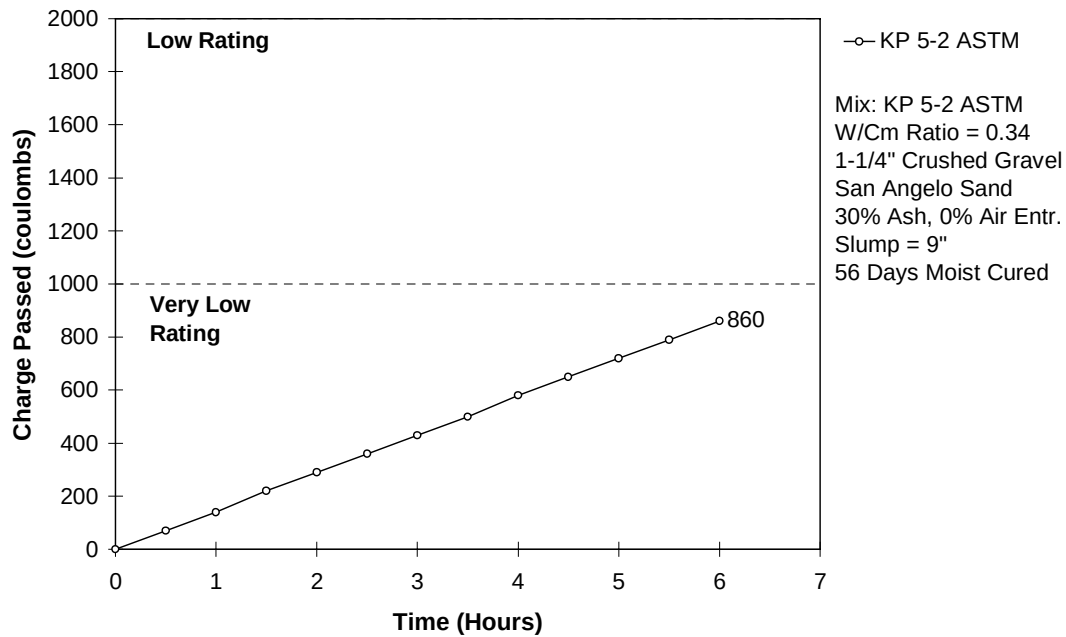


**Figure B.67: Rapid Ion Permeability for San Angelo Class K Pier
Cast May 2, 1996**

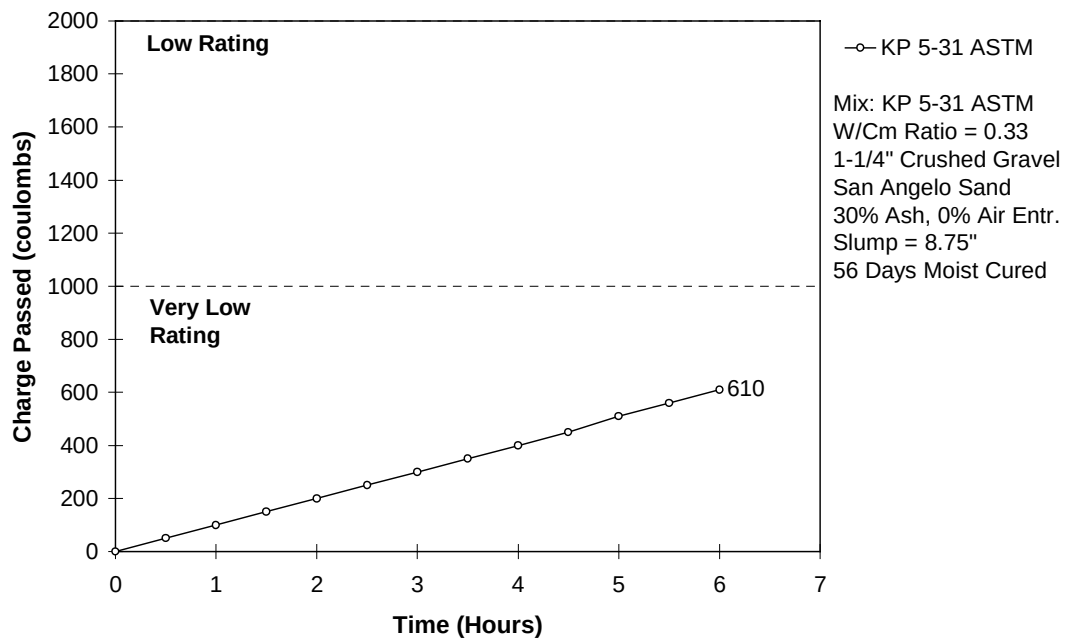


**Figure B.68: Rapid Ion Permeability for San Angelo Class K Pier
Cast May 31, 1996**

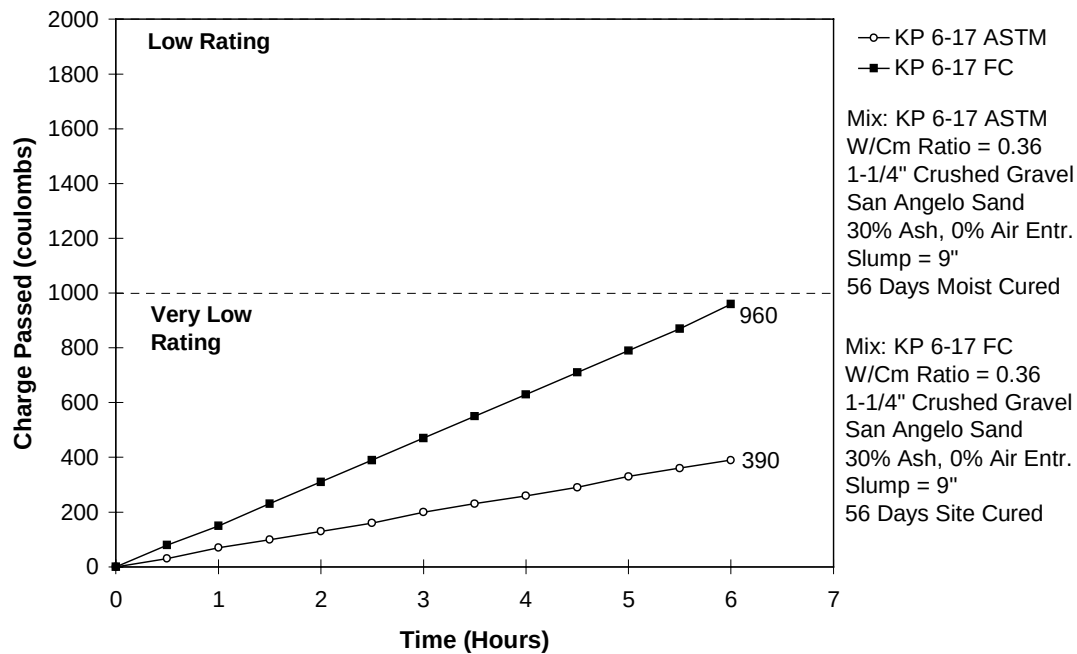


**Figure B.69: Rapid Ion Permeability for San Angelo Class K Pier
Cast June 17, 1996**

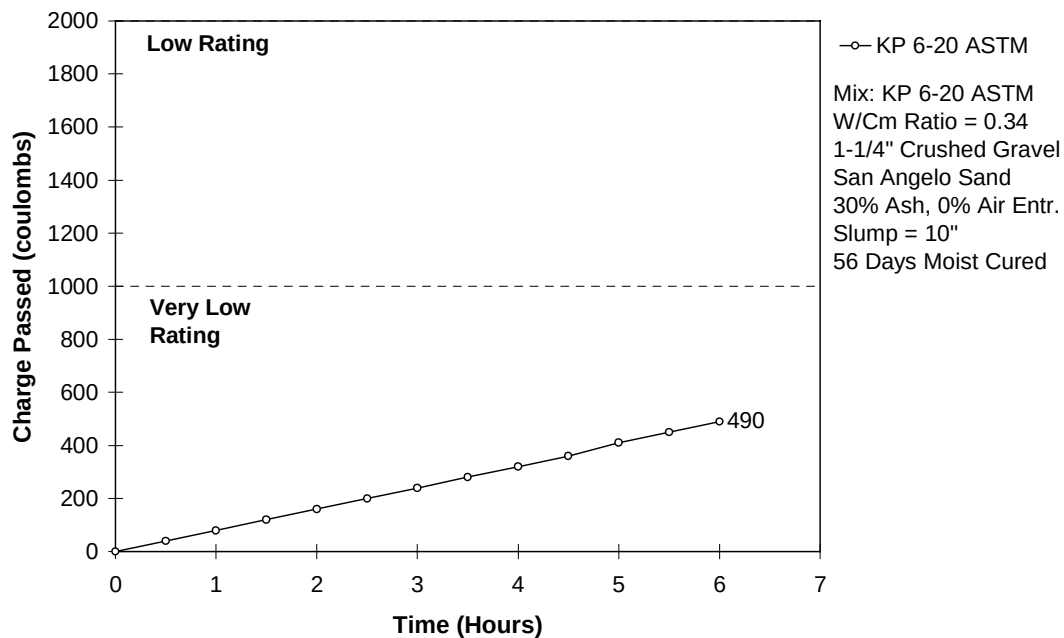


**Figure B.70: Rapid Ion Permeability for San Angelo Class K Pier
Cast June 20, 1996**

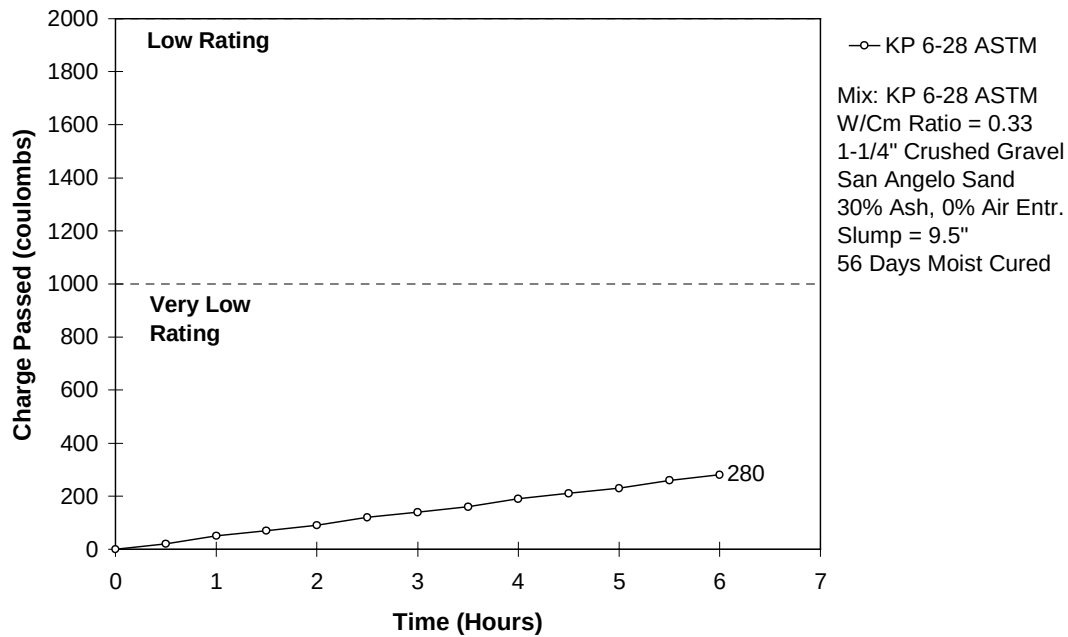


**Figure B.71: Rapid Ion Permeability for San Angelo Class K Pier
Cast June 28, 1996**

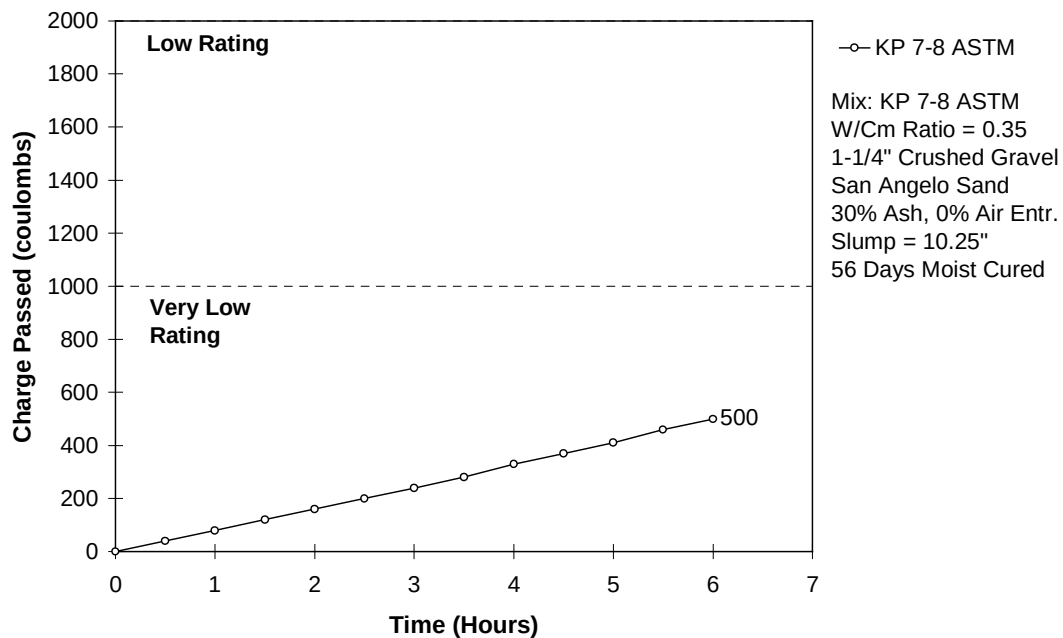


**Figure B.72: Rapid Ion Permeability for San Angelo Class K Cap
Cast July 8, 1996**

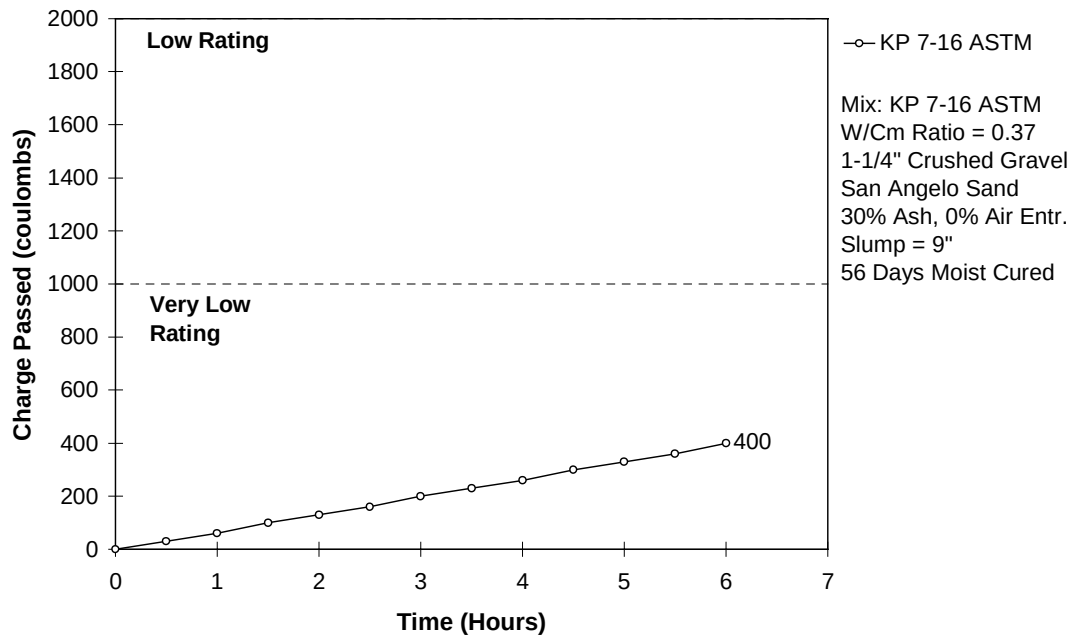


**Figure B.73: Rapid Ion Permeability for San Angelo Class K Cap
Cast July 16, 1996**

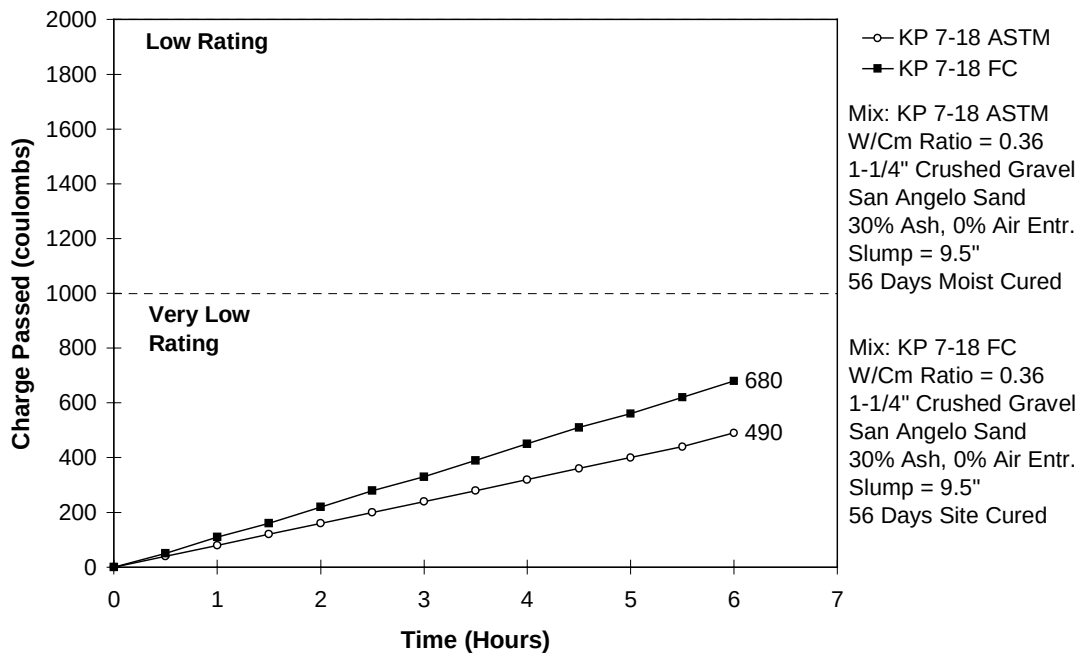


**Figure B.74: Rapid Ion Permeability for San Angelo Class K Pier Cap
Cast July 18, 1996**

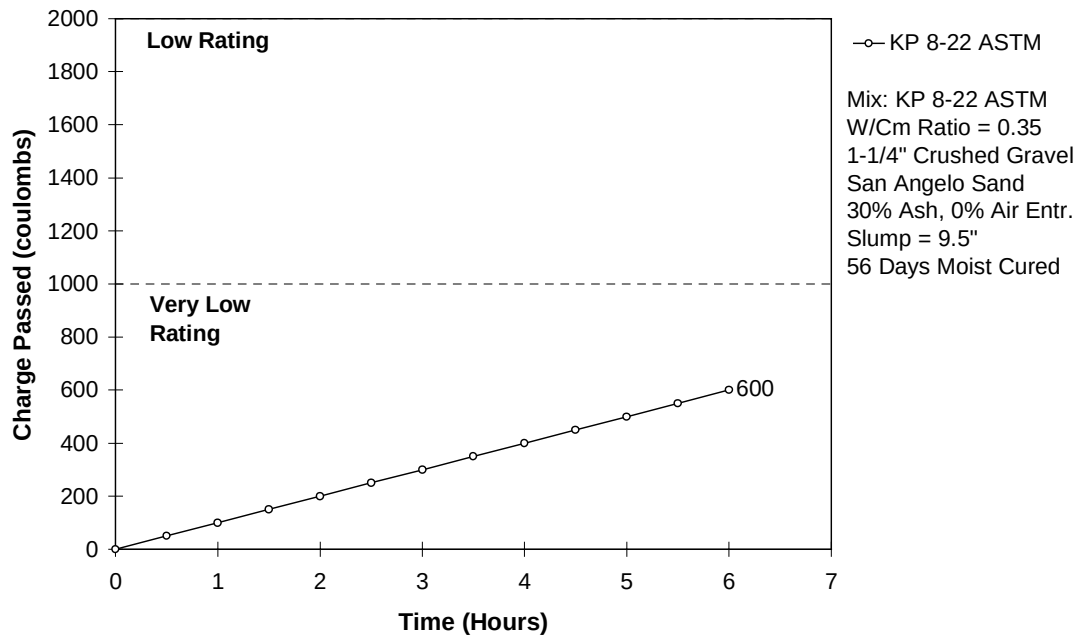


**Figure B.75: Rapid Ion Permeability for San Angelo Class K Pier Cap
Cast August 22, 1996**

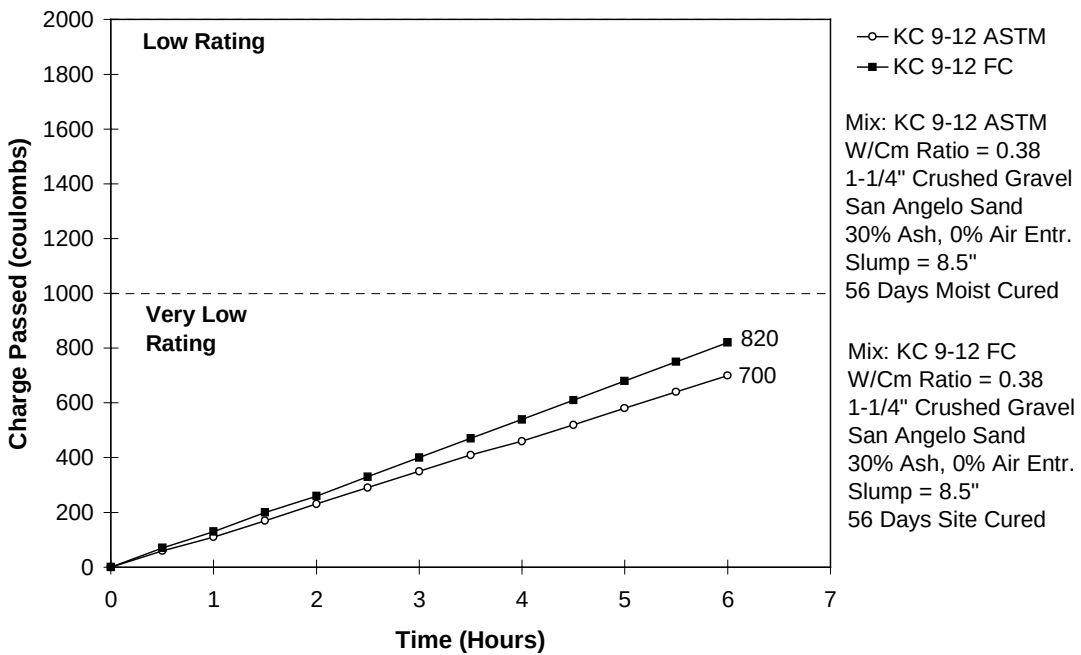


**Figure B.76: Rapid Ion Permeability for San Angelo Class K Pier Cap
Cast September 12, 1996**

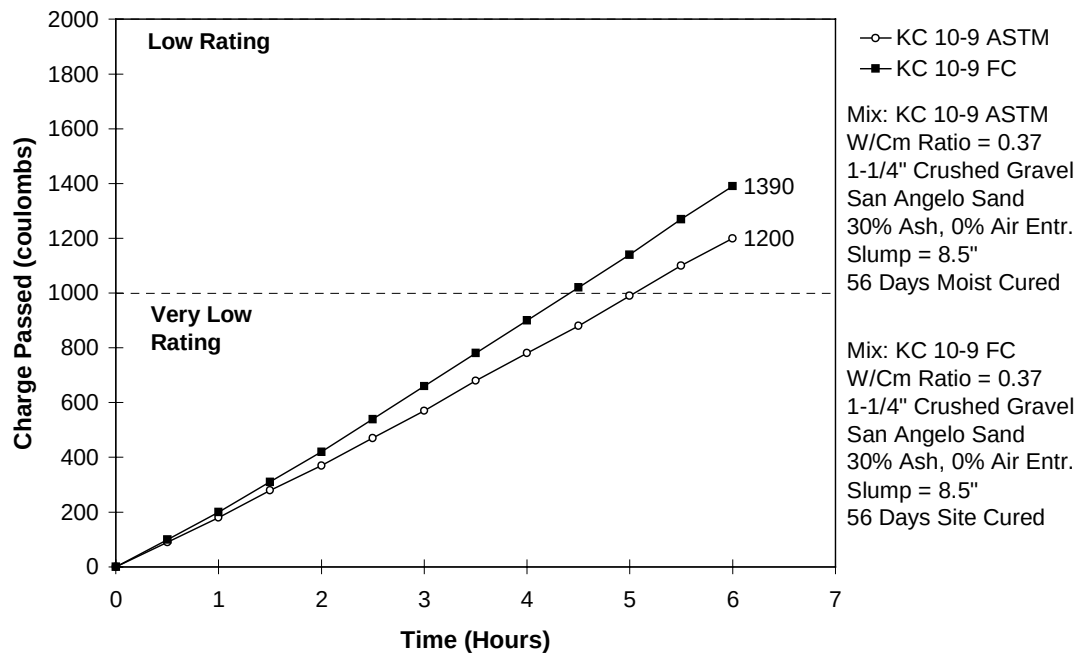


**Figure B.77: Rapid Ion Permeability for San Angelo Class K Pier Cap
Cast October 9, 1996**

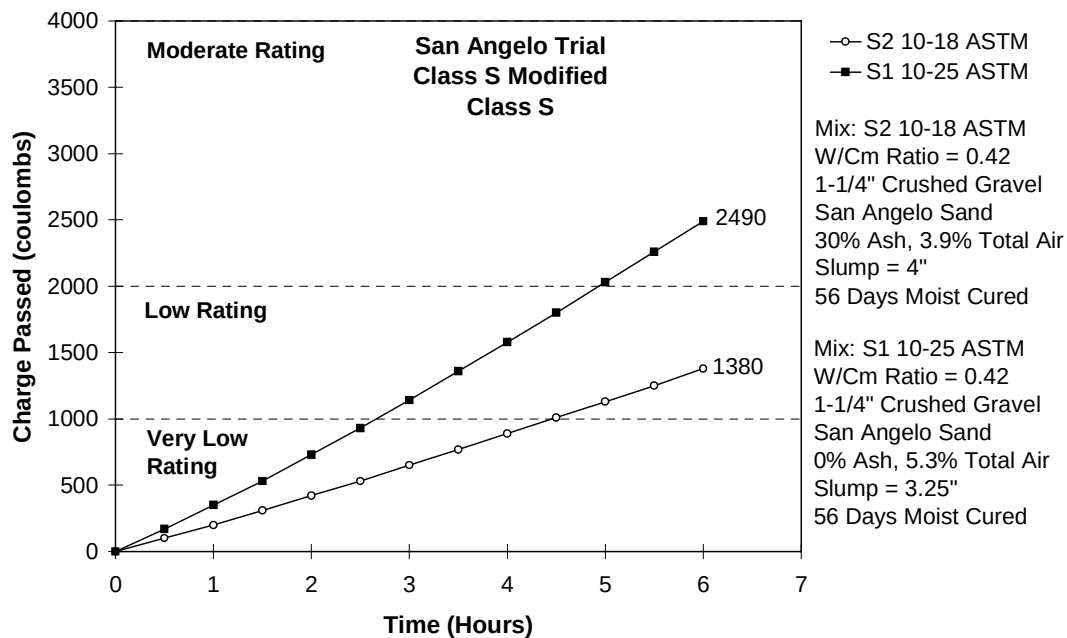


Figure B.78: Rapid Ion Permeability for San Angelo Deck Trial Field Mixes

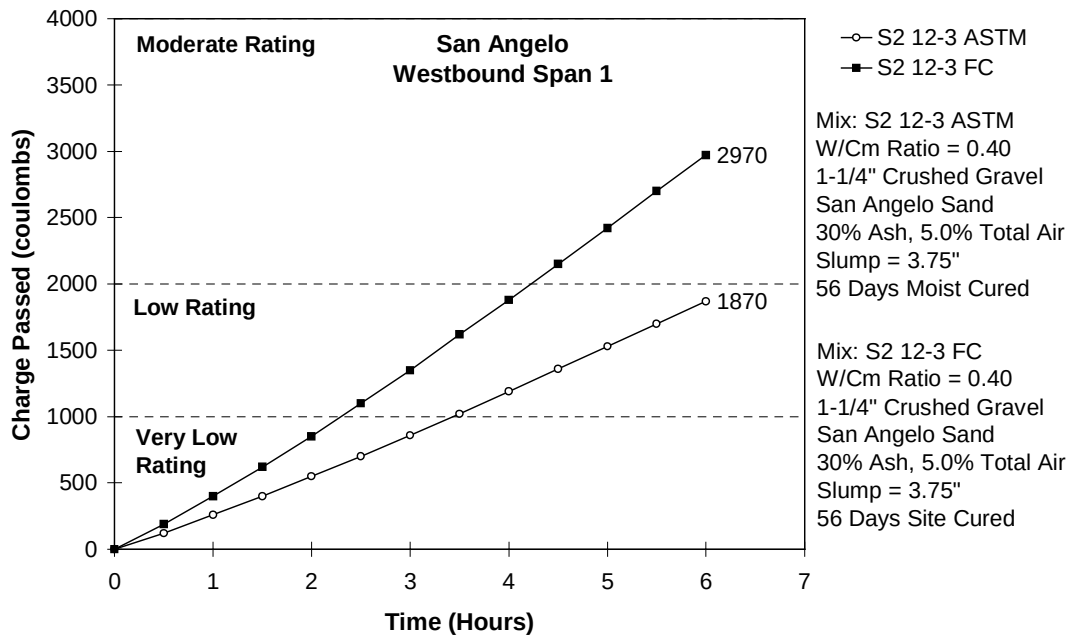


**Figure B.79: Rapid Ion Permeability for San Angelo Class S Mod Deck
Cast December 3, 1996**

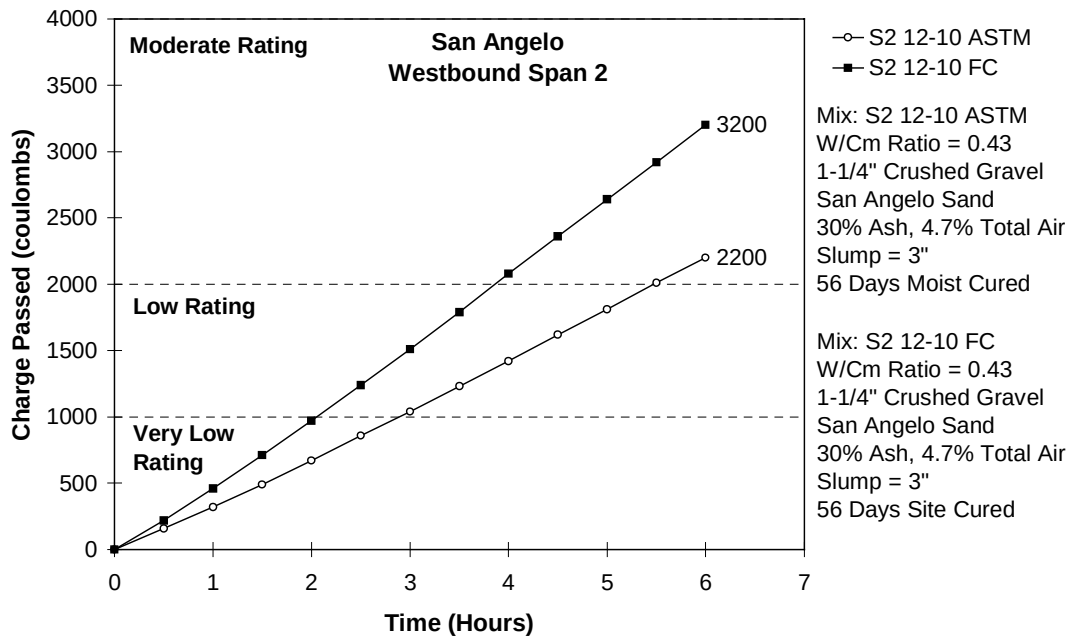


**Figure B.80: Rapid Ion Permeability for San Angelo Class S Mod Deck
Cast December 10, 1996**

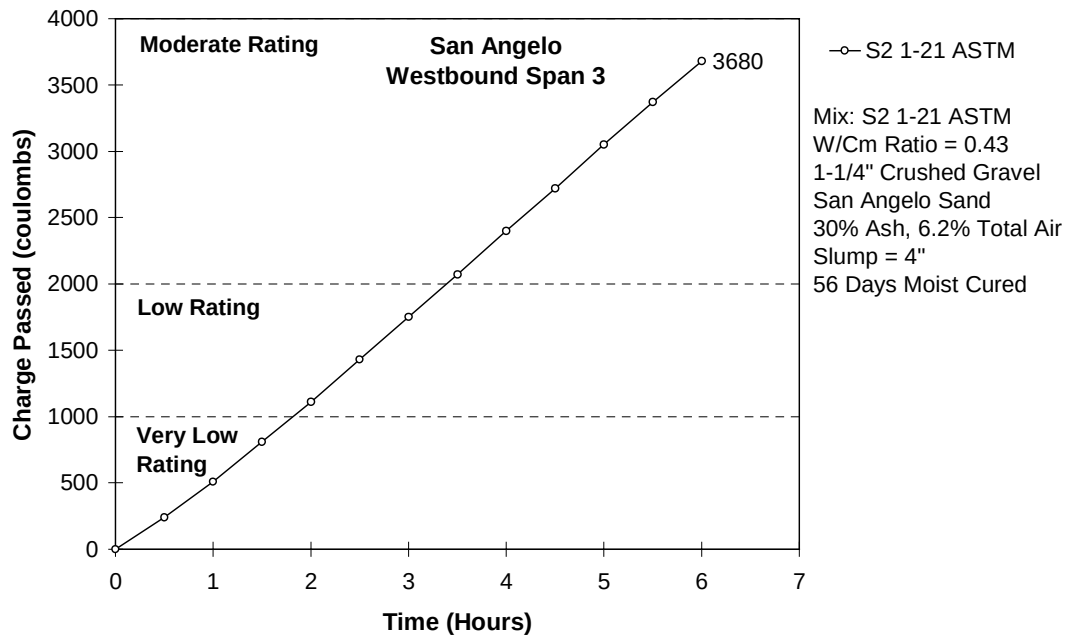


Figure B.81: Rapid Ion Permeability for San Angelo Class S Mod Deck Cast January 21, 1997

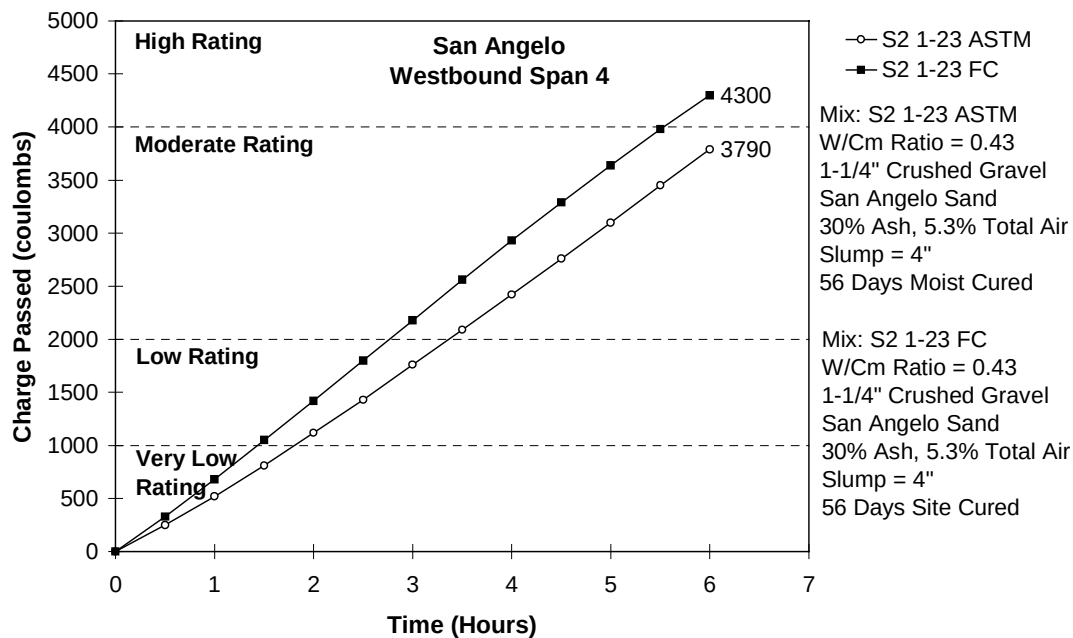


Figure B.82: Rapid Ion Permeability for San Angelo Class S Mod Deck Cast January 23, 1997

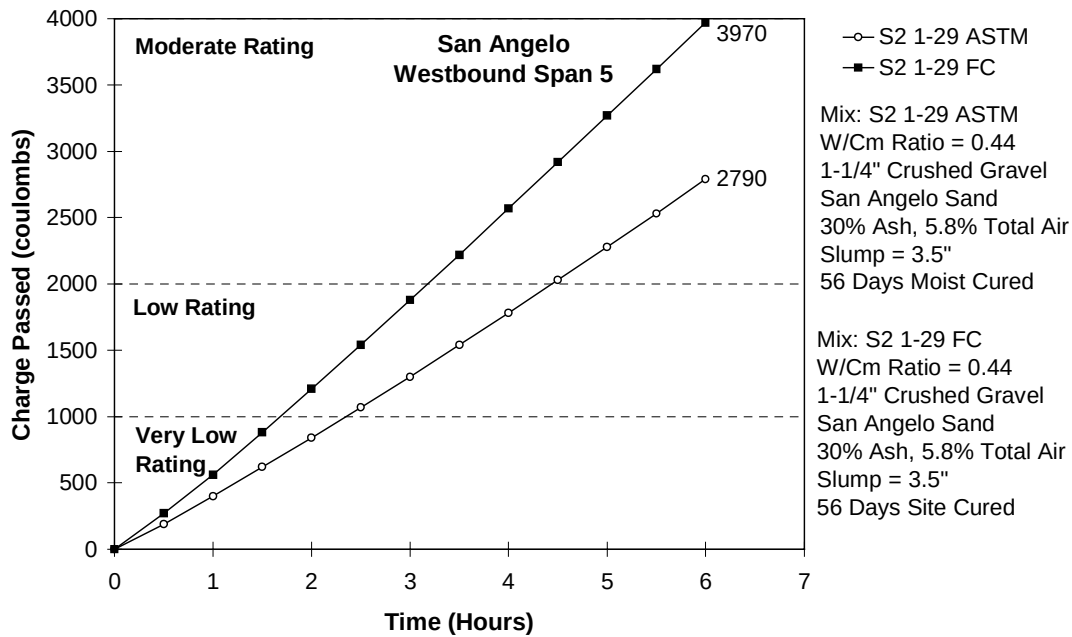


**Figure B.83: Rapid Ion Permeability for San Angelo Class S Mod Deck
Cast January 29, 1997**

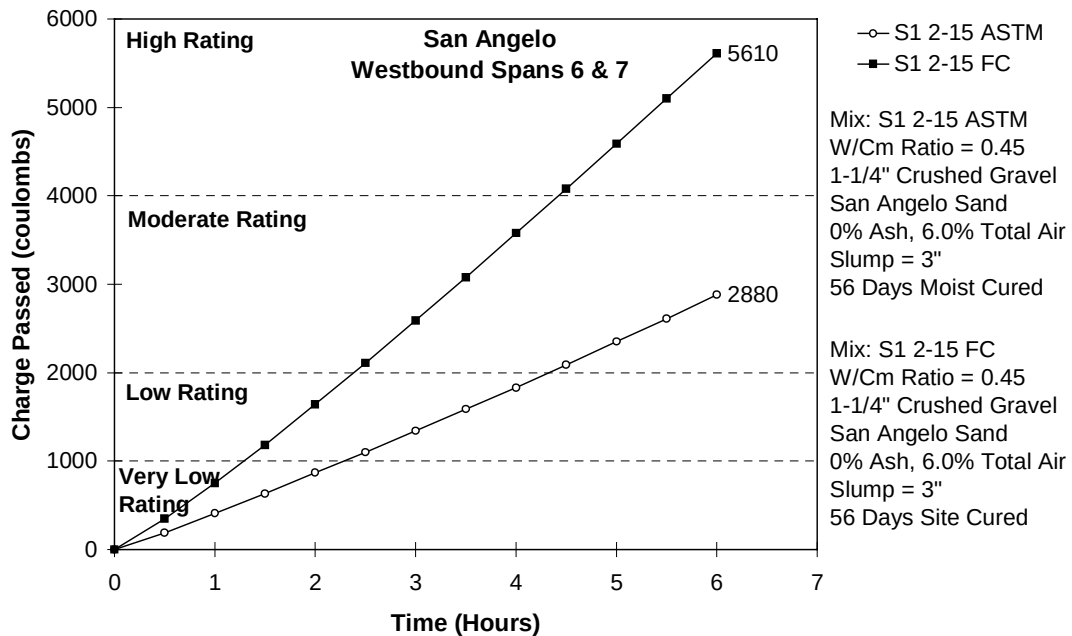


**Figure B.84: Rapid Ion Permeability for San Angelo Class S Deck
Cast February 15, 1997**

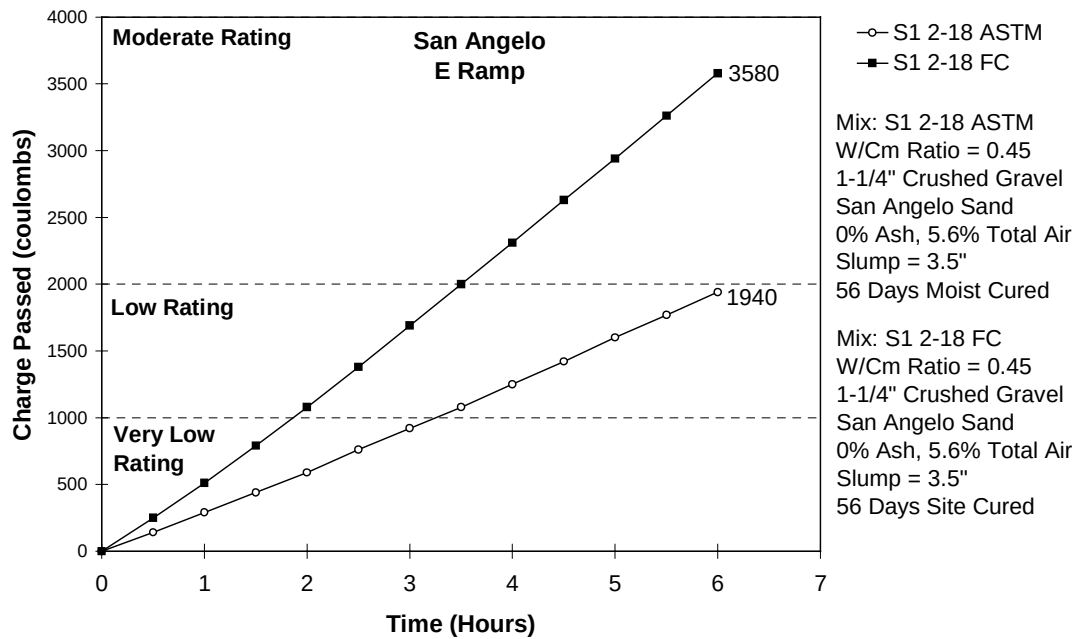


**Figure B.85: Rapid Ion Permeability for San Angelo Class S Ramp
Cast February 18, 1997**

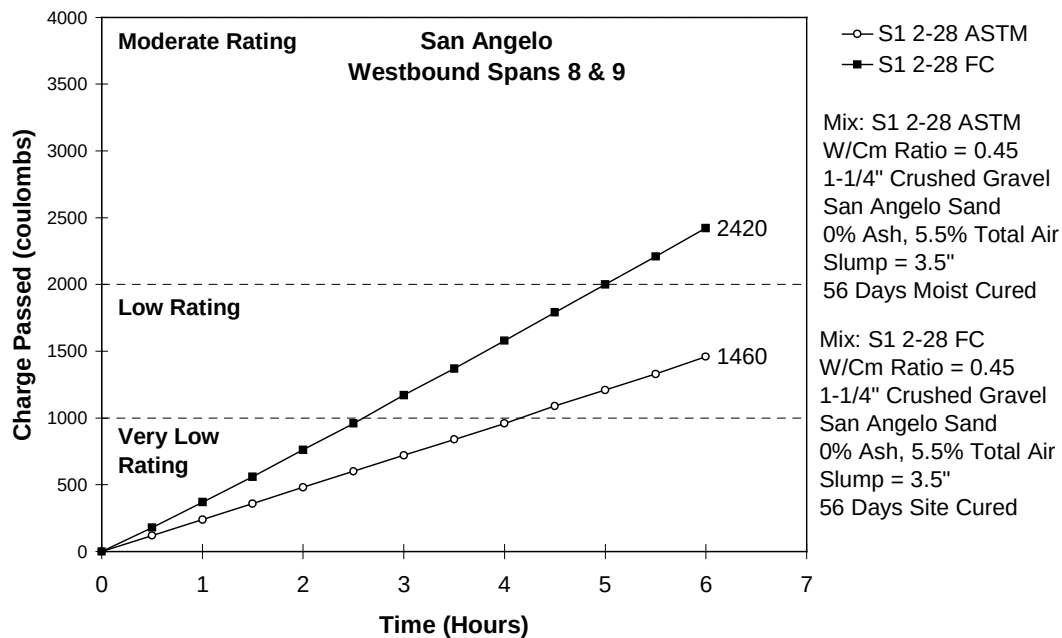


**Figure B.86: Rapid Ion Permeability for San Angelo Class S Deck
Cast February 28, 1997**

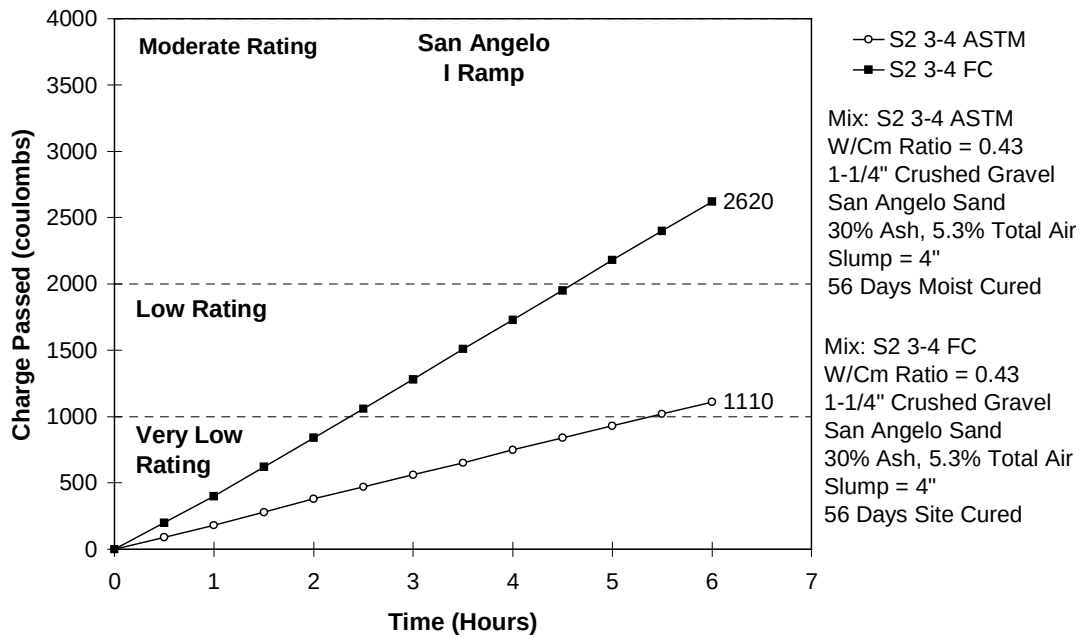


**Figure B.87: Rapid Ion Permeability for San Angelo Class S Ramp
Cast March 4, 1997**

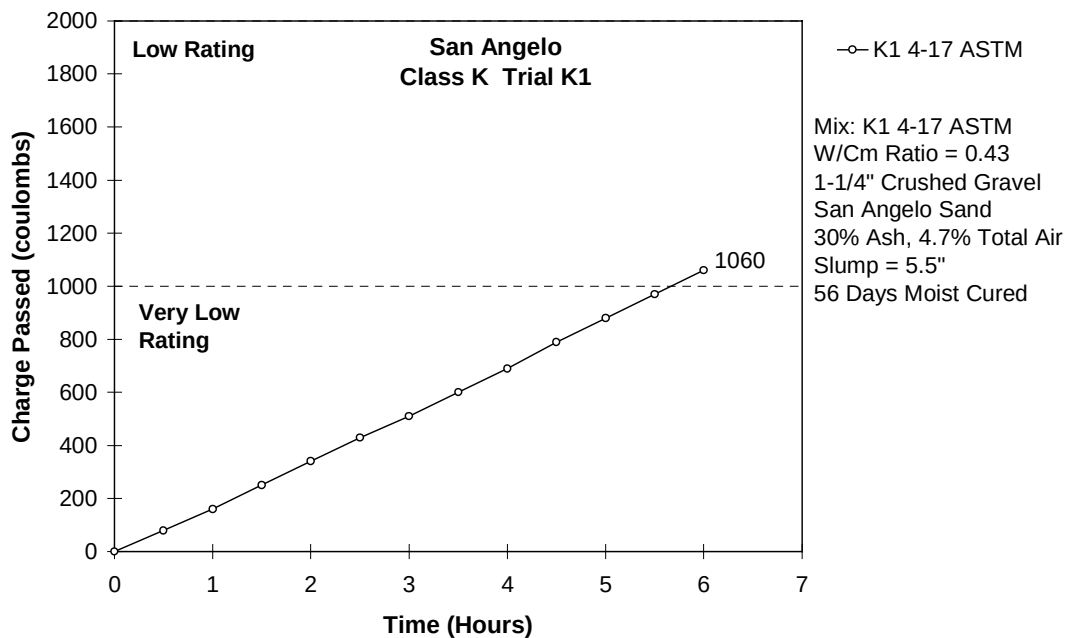


Figure B.88: Rapid Ion Permeability for San Angelo Class K Deck Trial Mix 1

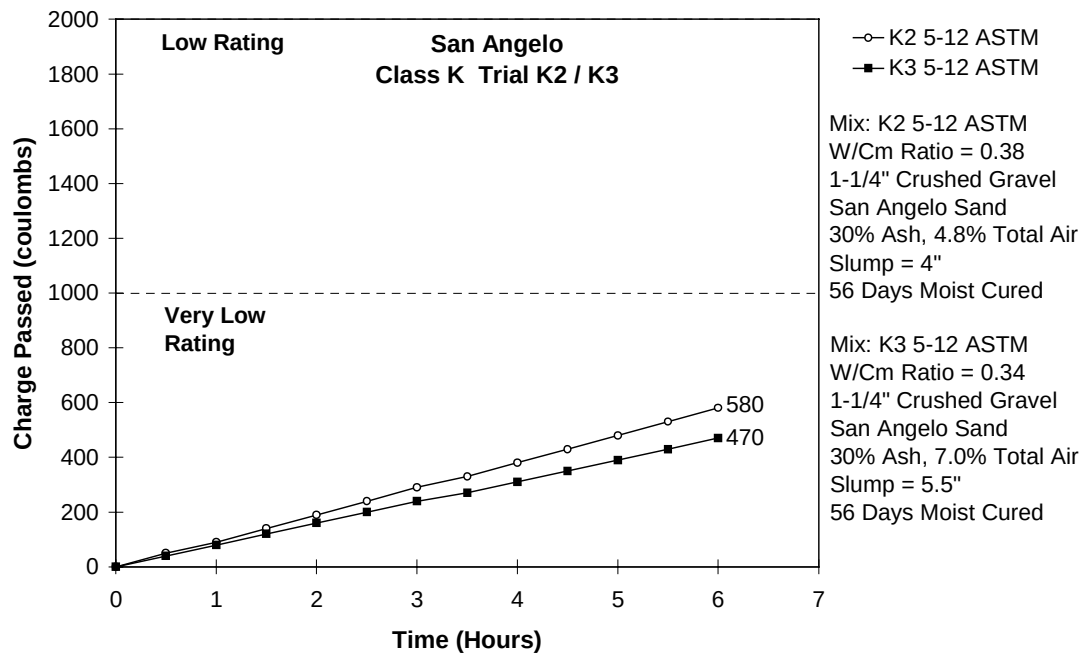


Figure B.89: Rapid Ion Permeability for San Angelo Class K Deck Trial Mix 2 & 3

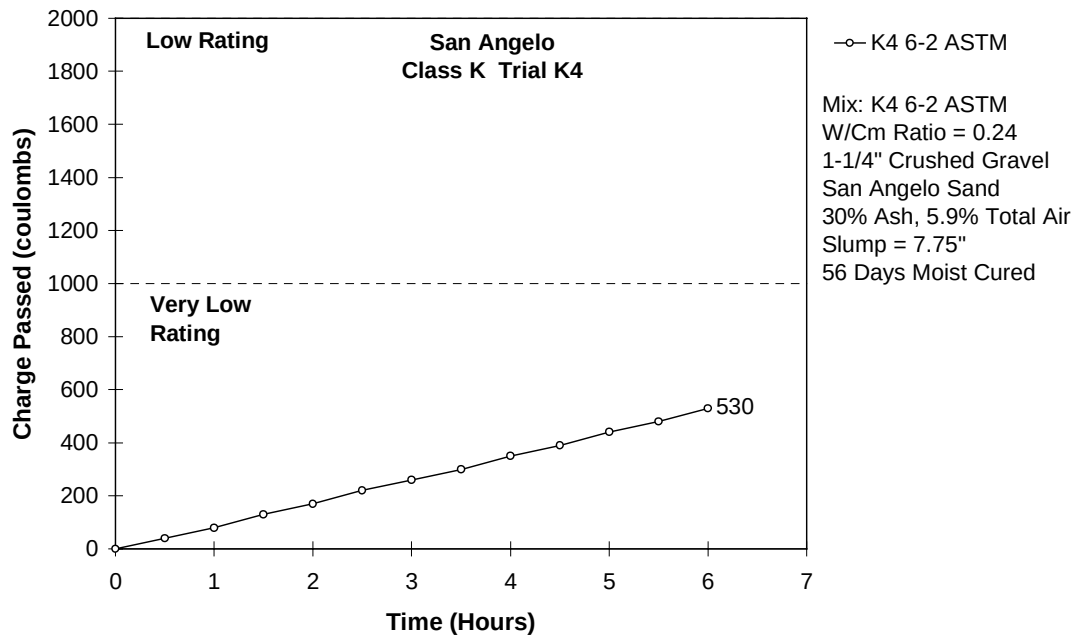


Figure B.90: Rapid Ion Permeability for San Angelo Class K Deck Trial Mix 4

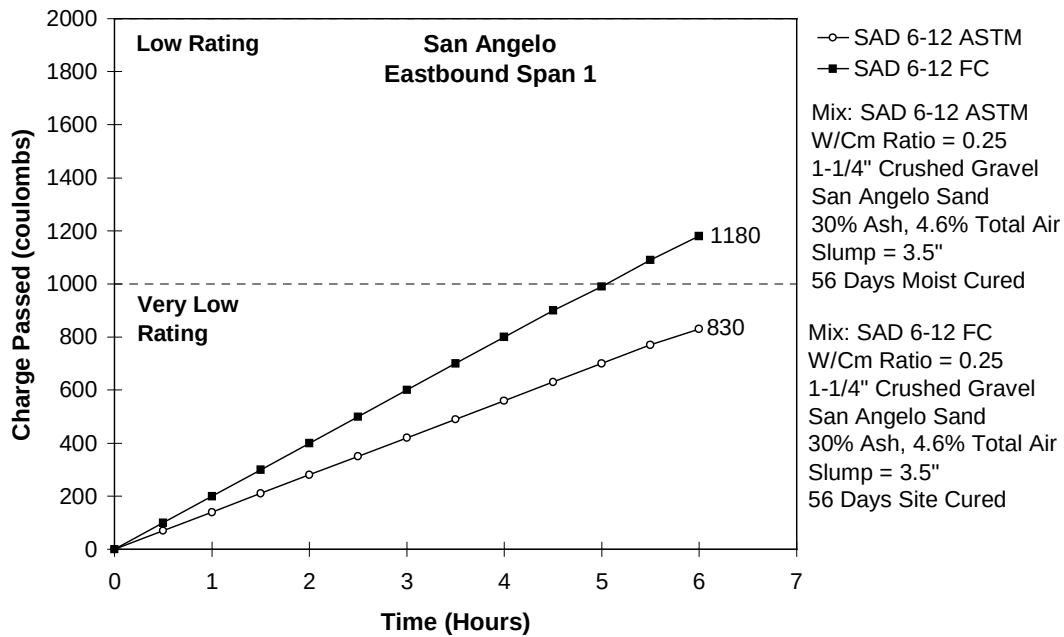


**Figure B.91: Rapid Ion Permeability for San Angelo Class K Deck
Cast June 12, 1997**

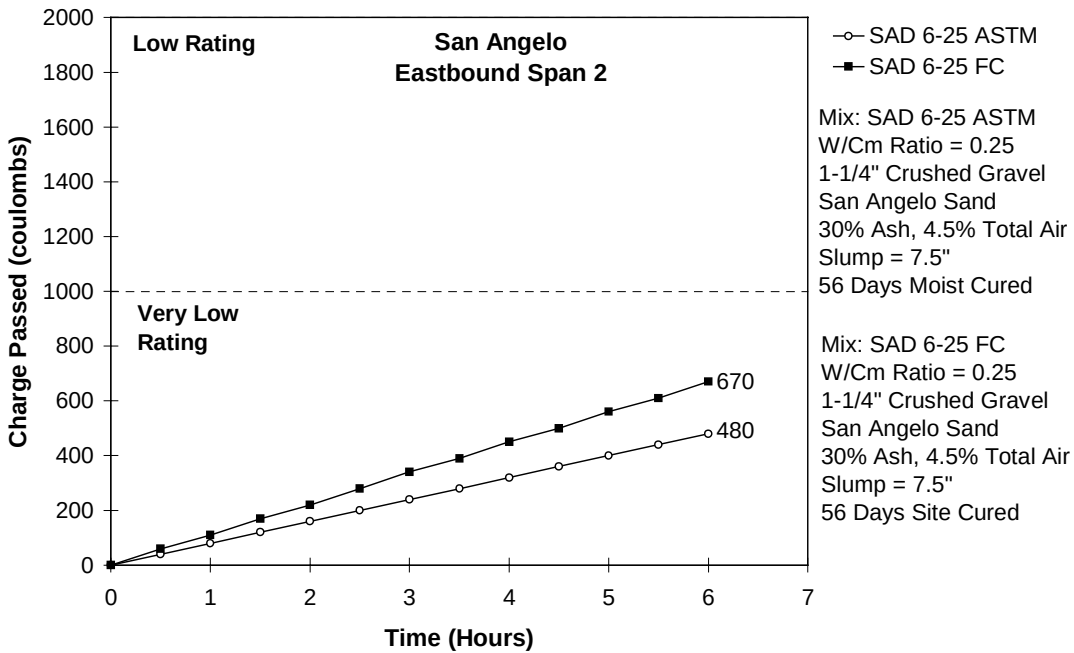


**Figure B.92: Rapid Ion Permeability for San Angelo Class K Deck
Cast June 25, 1997**

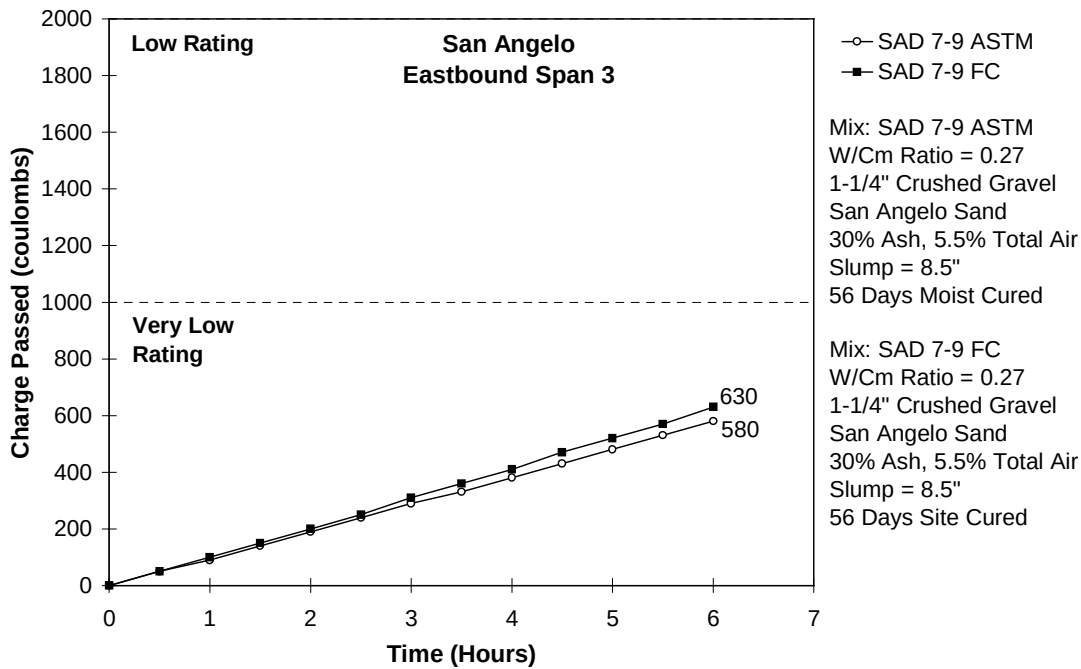


**Figure B.93: Rapid Ion Permeability for San Angelo Class K Deck
Cast July 9, 1997**

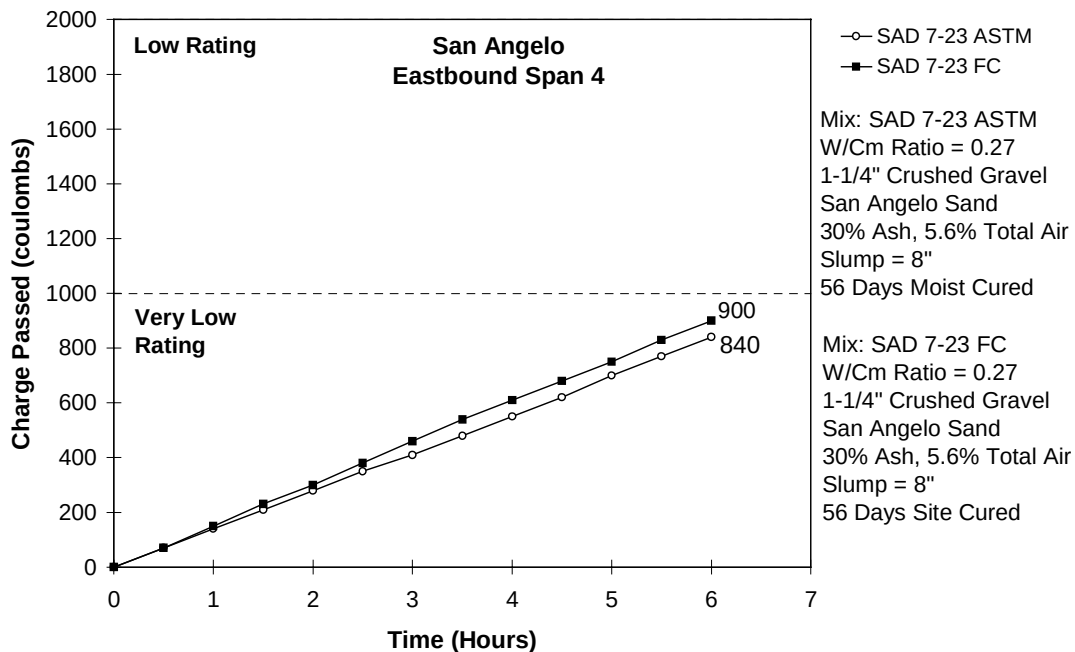


**Figure B.94: Rapid Ion Permeability for San Angelo Class K Deck
Cast July 23, 1997**

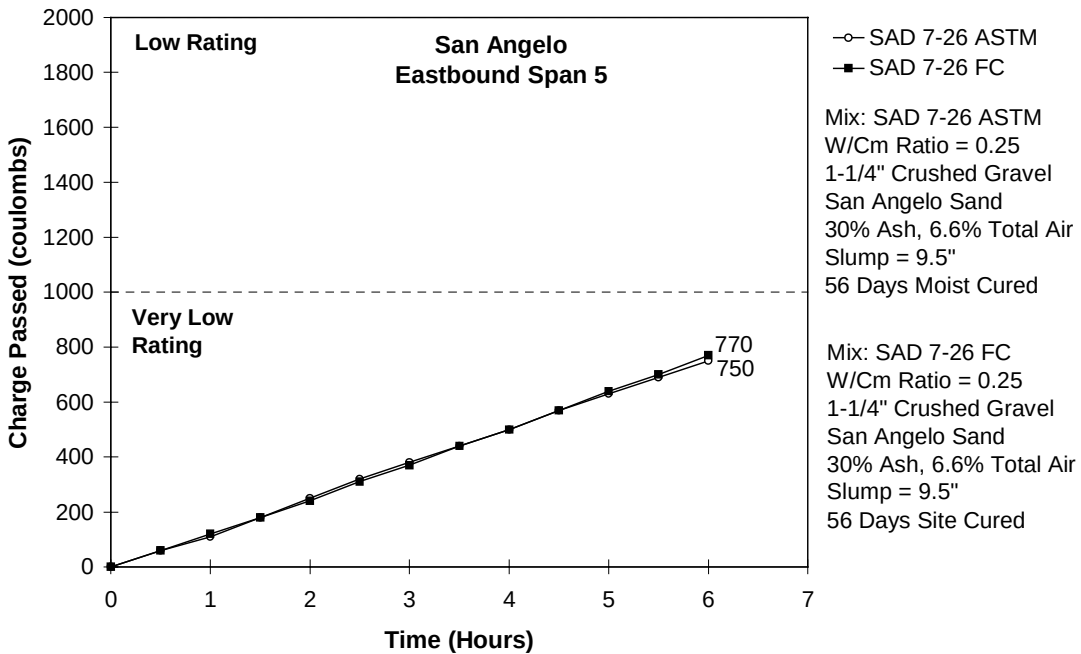


**Figure B.95: Rapid Ion Permeability for San Angelo Class K Deck
Cast July 26, 1997**

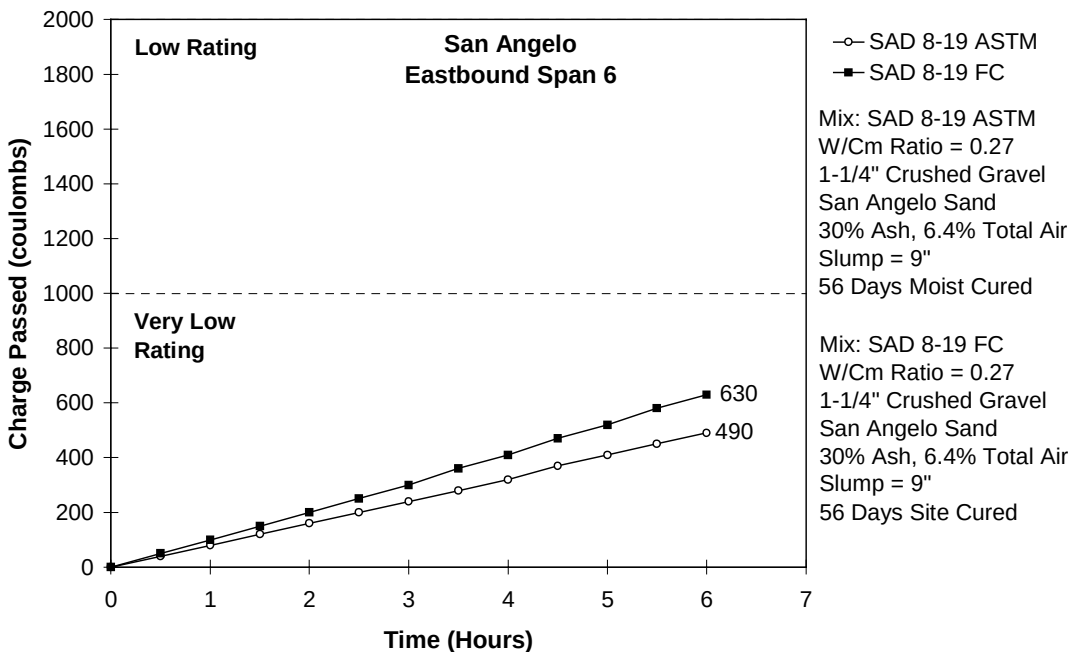


**Figure B.96: Rapid Ion Permeability for San Angelo Class K Deck
Cast August 19, 1997**

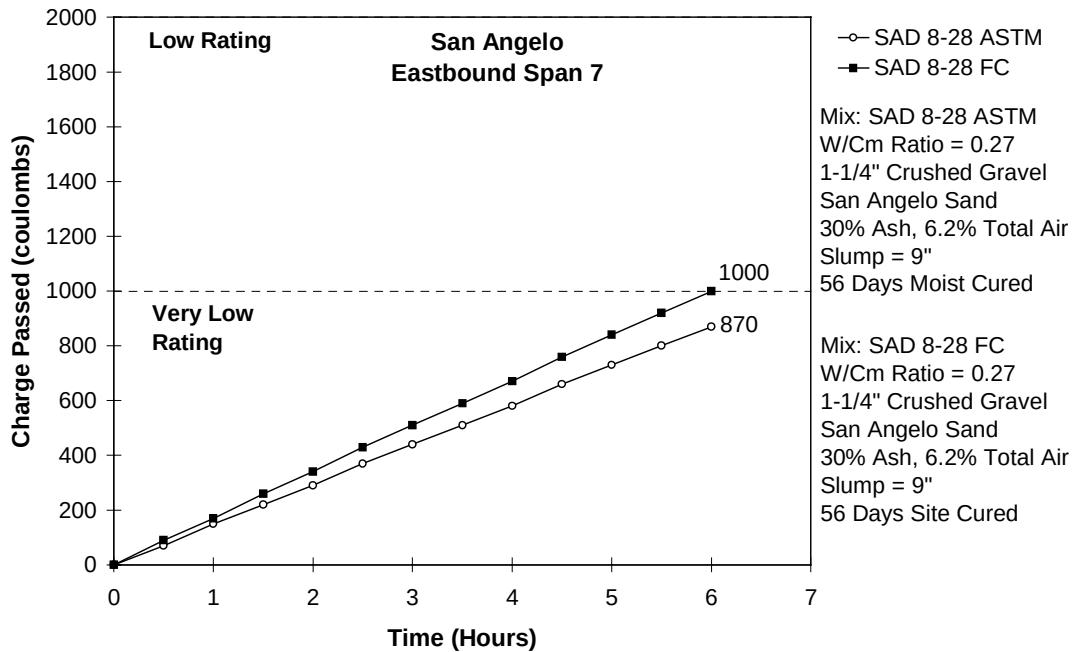


**Figure B.97: Rapid Ion Permeability for San Angelo Class K Deck
Cast August 28, 1997**

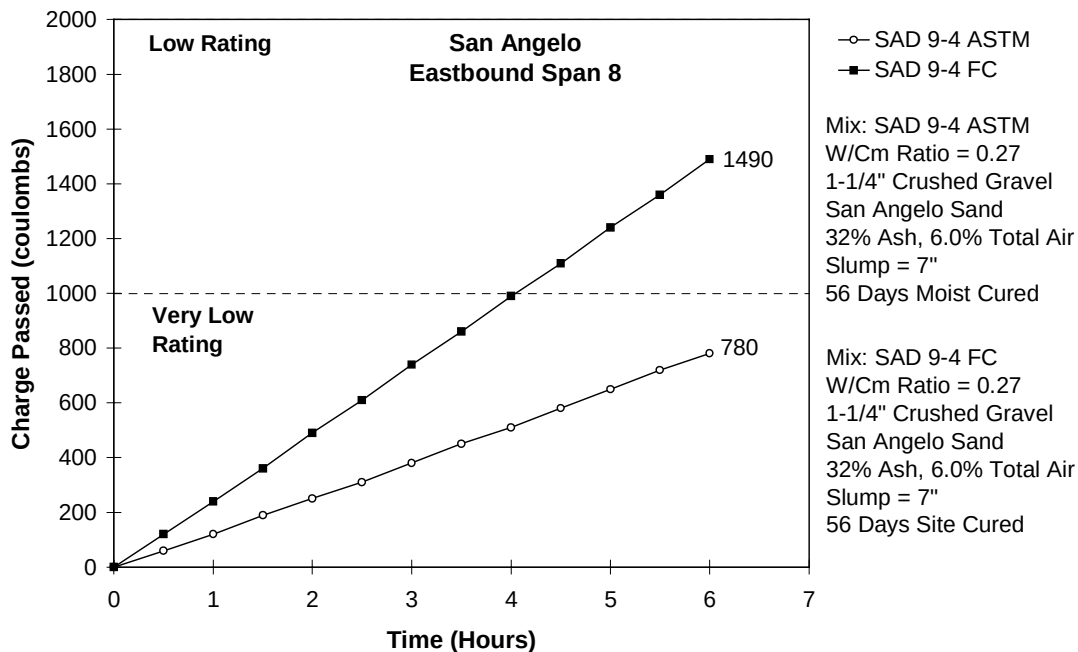


**Figure B.98: Rapid Ion Permeability for San Angelo Class K Deck
Cast September 4, 1997**

Appendix C: Chloride Ion Penetration Cast-in-Place Deck Test Results

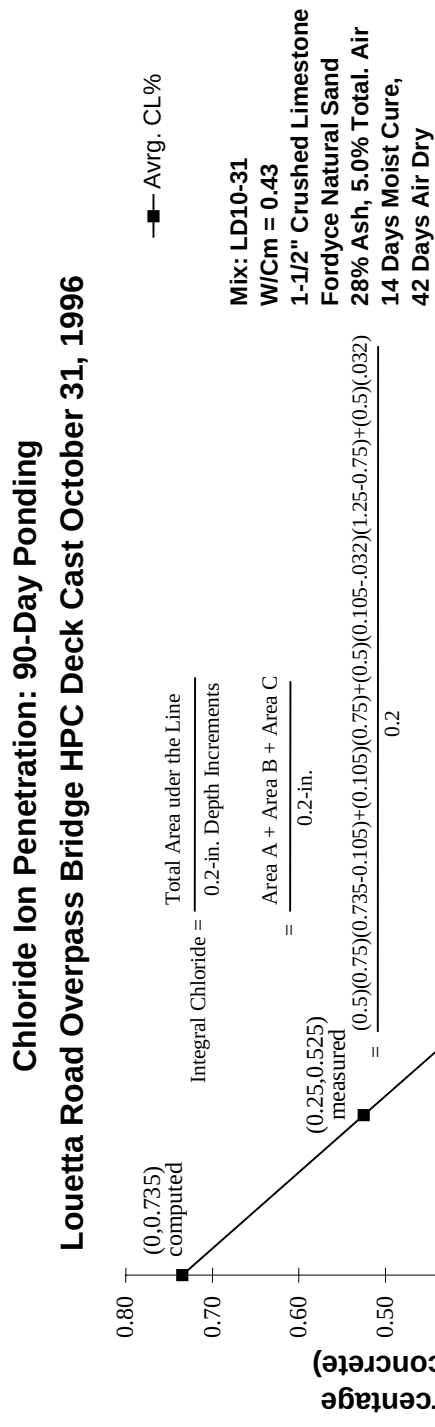


**Figure C.1: Chloride Ion Penetration for Louetta Road Overpass Bridge Northbound HPC Deck
 Cast October 31, 1996**

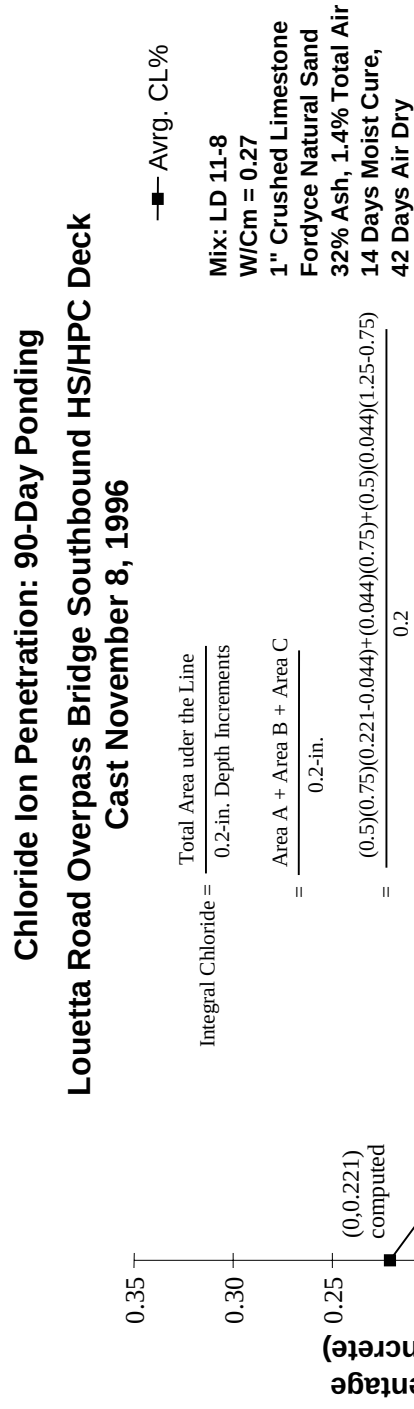


Figure C.2: Chloride Ion Penetration for Louetta Road Overpass Bridge Southbound HS/HPC Deck
Cast November 8, 1996

Chloride Ion Penetration: 90-Day Ponding San Angelo Class S Modified HPC Deck Cast December 3, 1996

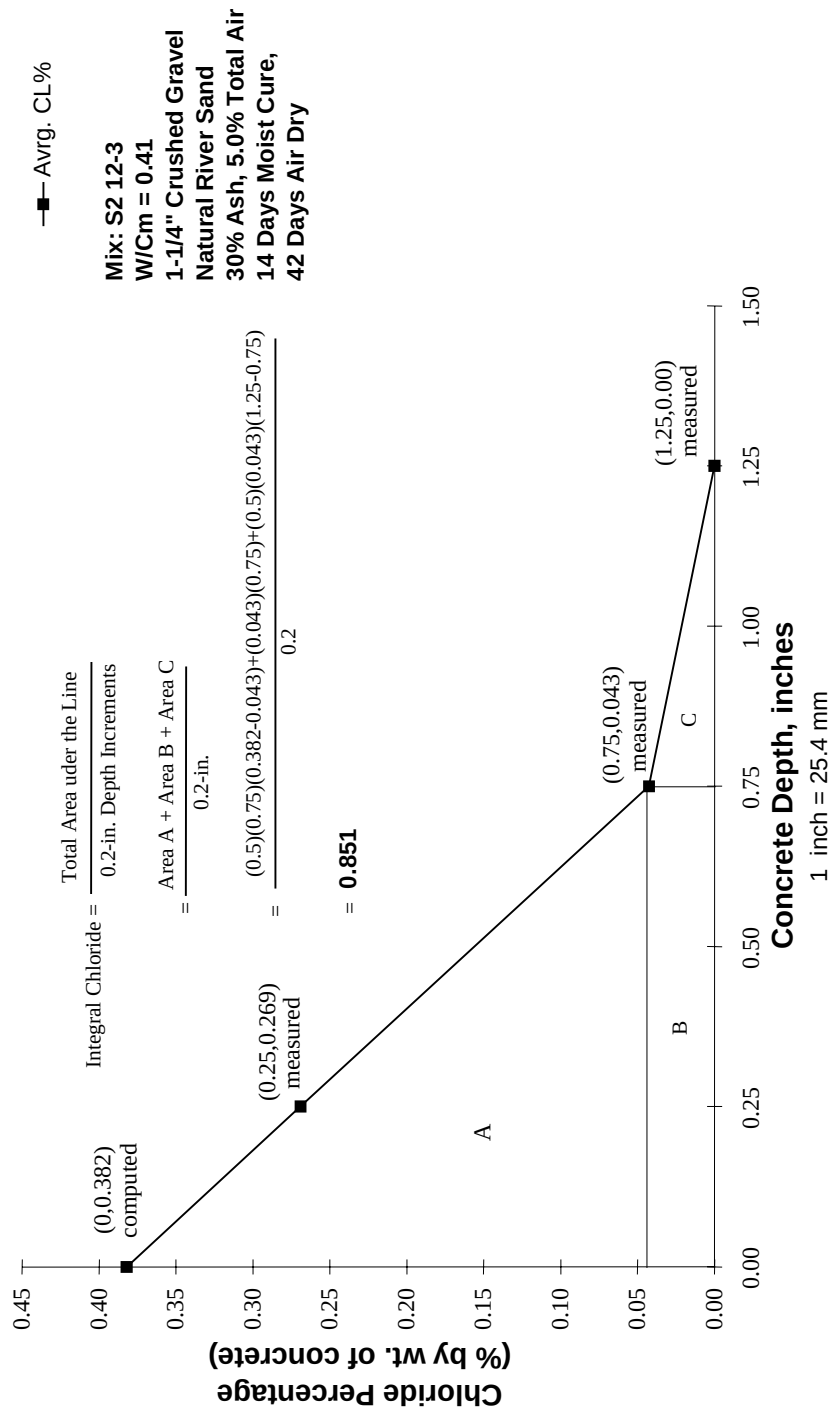


Figure C.3: Chloride Ion Penetration for San Angelo Class S Modified HPC Deck Cast December 3, 1996

Chloride Ion Penetration: 90-Day Ponding San Angelo Class S NSC Deck Cast February 15, 1997

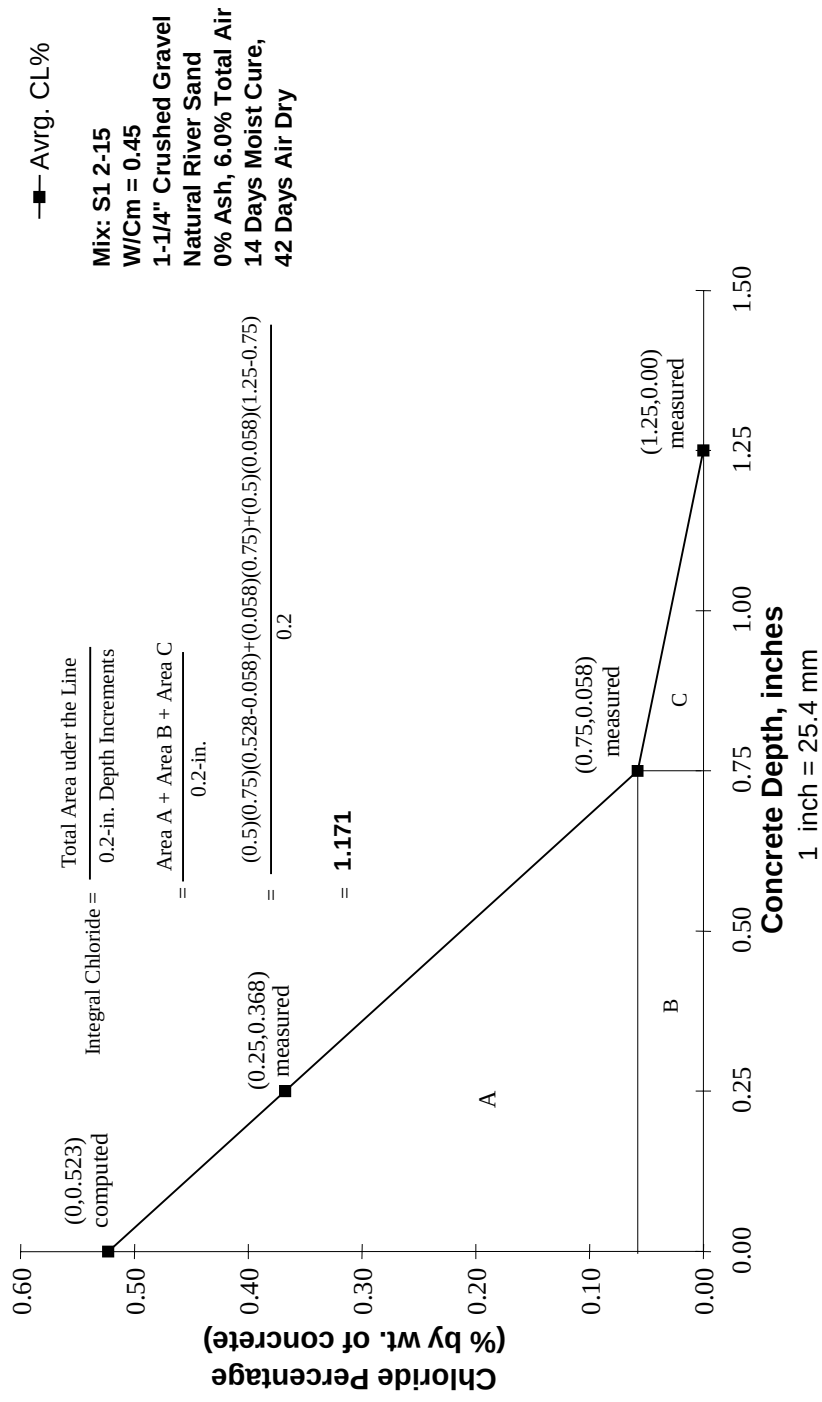


Figure C.4: Chloride Ion Penetration for San Angelo Class S NSC Deck
Cast February 15, 1997

Chloride Ion Penetration: 90-Day Ponding San Angelo Class K HS/HPC Deck Cast June 12, 1997

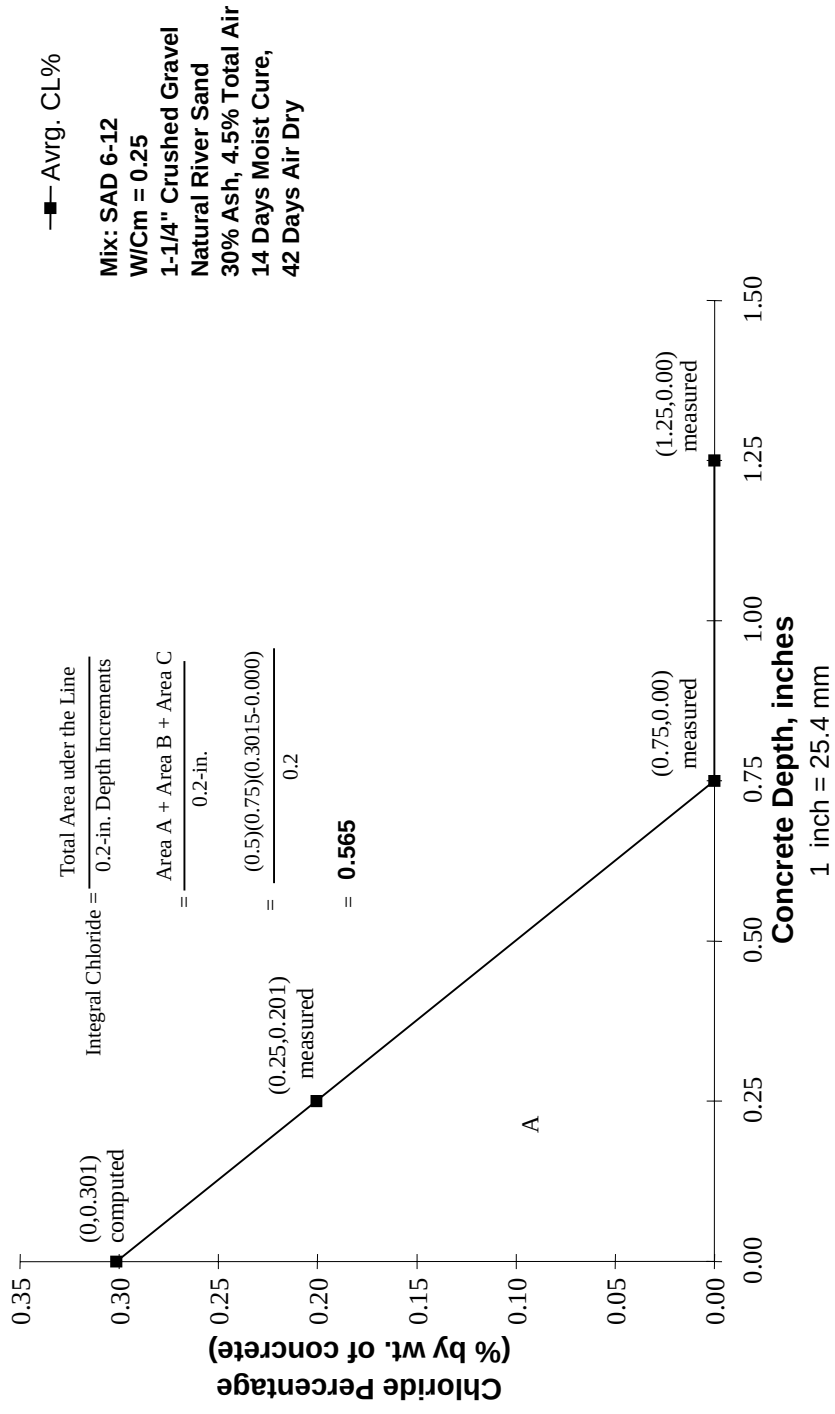


Figure C.5: Chloride Ion Penetration for San Angelo Class K HS/HPC Deck
Cast June 12, 1997

Chloride Ion Penetration: 90-Day Ponding San Angelo Class K HS/HPC Deck Cast August 19, 1997

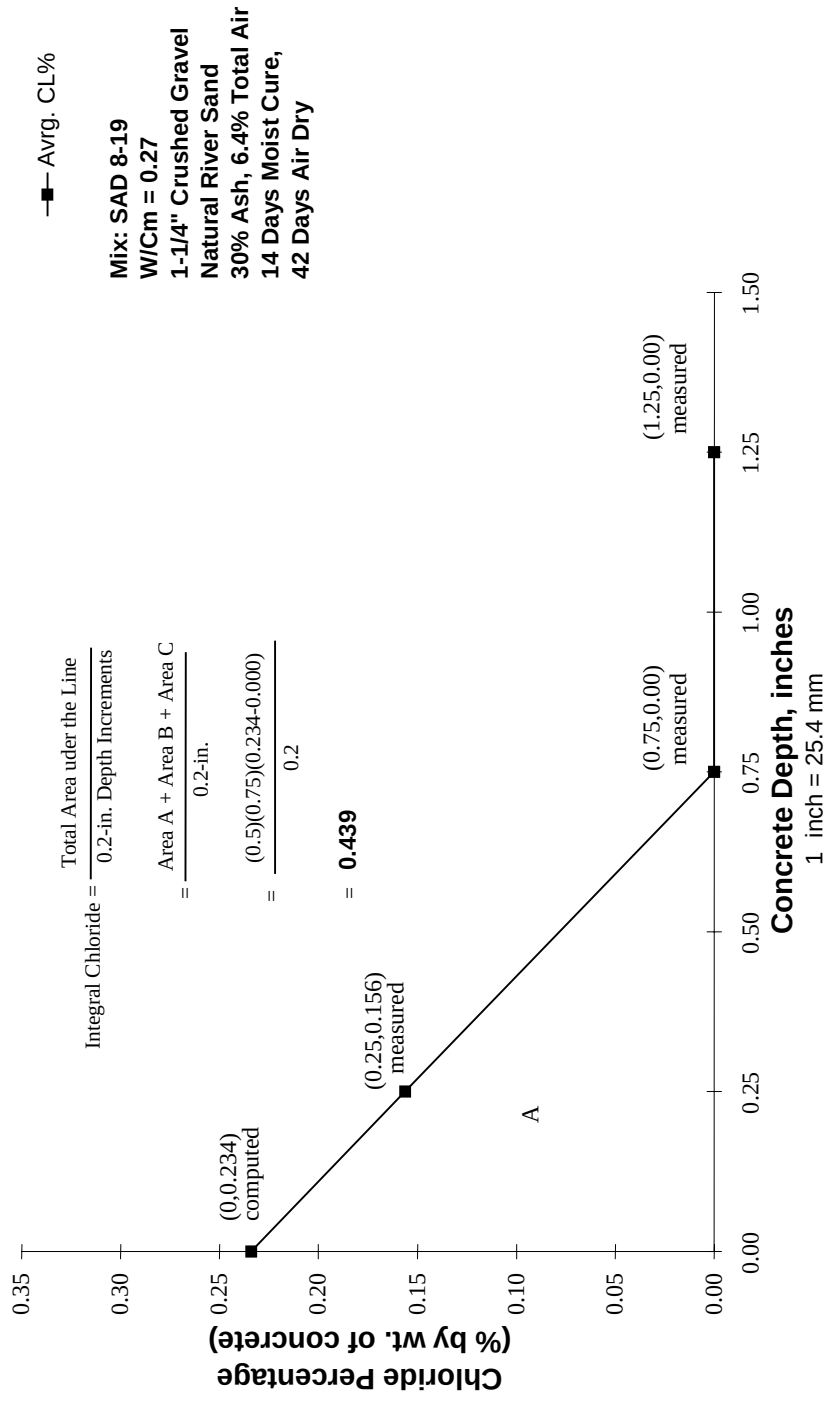


Figure C.6: Chloride Ion Penetration for San Angelo Class K HS/HPC Deck Cast August 19, 1997

Chloride Ion Penetration: 90-Day Ponding HS/HPC AASHTO Type IV Precast Beam Cast February 19, 1996

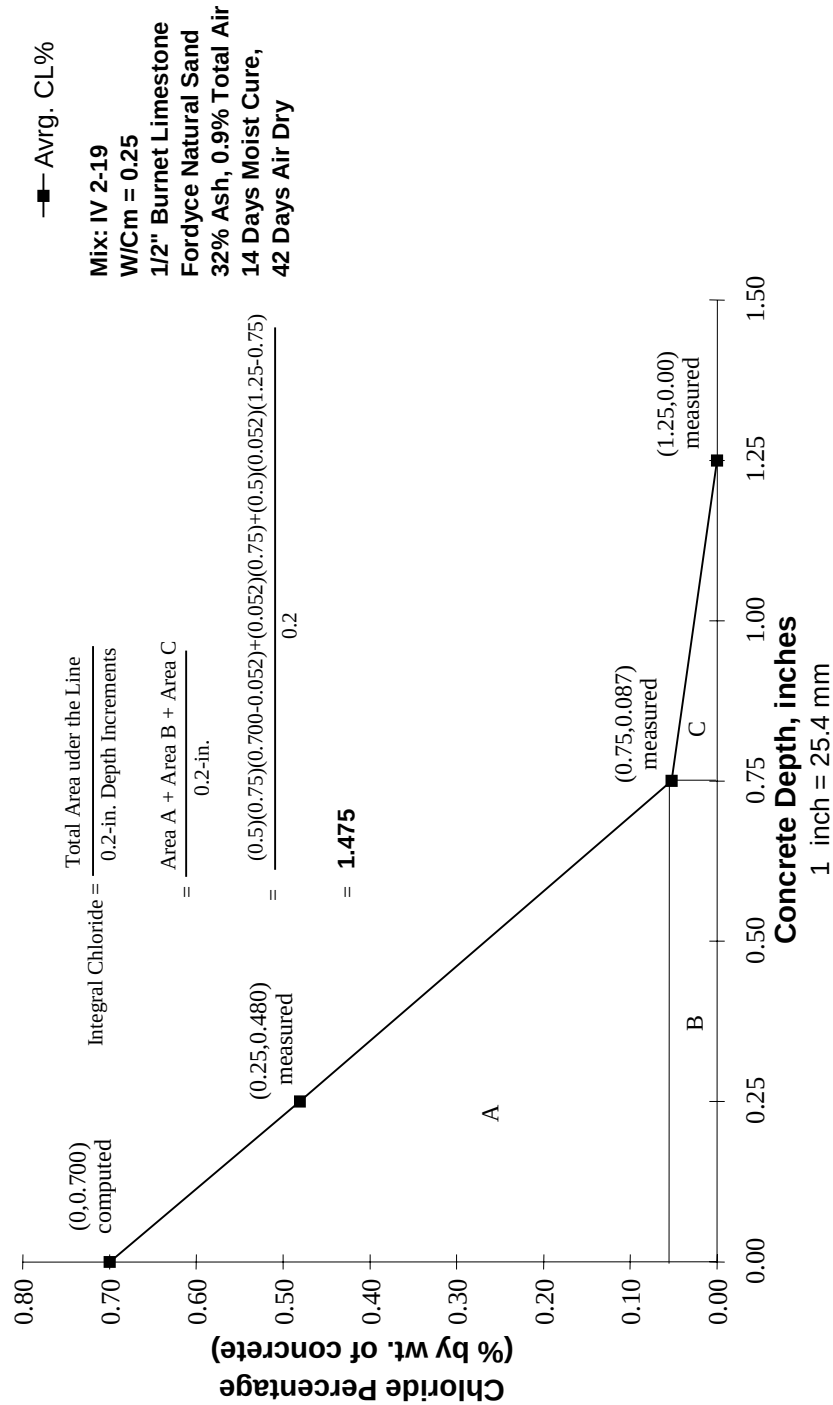


Figure C.7: Chloride Ion Penetration for San Angelo HS/HPC AASHTO Type IV Precast Beam Cast February 19, 1997

Appendix D: Chloride Ion Penetration Correlation Study Test Results

Table D.1: Results of the Rapid Ion Chloride Permeability Test AASTHO T277 ❶

Mixture Designation	W/C Ratio	Charge Passed (Coulombs) AASTHO T277		
		14-Day Moist Cured & 42-Day Air Dried	56-Day Moist Cured	14-Day Moist Cured, 42-Day Air Dried, & 365-Day Ponding
AERG1	0.33	559	1956	442
AERG2	0.32	927	1061	399
AERG3	0.28	772	1282	205
AERG4	0.29	780	1417	207
AERG5	0.30	559	574	149
AERG6	0.30	927	803	247
AERG7	0.26	772	607	332
AERG8	0.26	780	576	336
AERG9	0.27	614	604	264
AELS1	0.33	1163	1083	1109
AELS2	0.33	1227	1178	758
AELS3	0.32	1426	5963	1852
AELS4	0.27	892	444	411
AELS5	0.27	1003	307	129
AELS6	0.27	837	359	639
AELS7	0.26	655	461	365
AELS8	0.27	624	523	297
AELS9	0.27	827	512	214
AETR1	0.33	1559	1343	1147
AETR2	0.33	1638	1484	1253
AETR3	0.33	1904	1614	1594
AETR4	0.25	453	366	161
AETR5	0.25	867	491	79
AETR6	0.25	762	448	215
AETR7	0.28	1306	937	598
AETR8	0.28	938	770	454
AETR9	0.28	1063	773	315
HP11-3	0.28	646	326	N.A.
HP11-32	0.22	N.A.	302	N.A.
SSP-3	0.28	443	323	N.A.
SSP-32	0.22	384	313	N.A.

❶ Average of four 2-in. thick 4-in. diameter slices.

Table D.2: Chloride Ion Content Results after the 90-Day and 365-Day Ponding Cycles ❶

Mixture Designation	90-Day Ponding Cycle (Percent of Concrete Content) Sample Depth (in.)			365-Day Ponding Cycle (Percent of Concrete Content) Sample Depth (in.)			Integral Chloride /	
	0.06-0.5	0.5-1.0	1.0-1.5	0.06-0.5	0.5-1.0	1.0-1.5	90-day	365-day
AERG1	0.200	0.000	0.000	0.243	0.036	0.002	0.562	0.800
AERG2	0.254	0.000	0.000	0.265	0.001	0.000	0.714	0.746
AERG3	0.225	0.000	0.000	0.233	0.000	0.000	0.509	0.656
AERG4	0.129	0.000	0.000	0.346	0.002	0.000	0.363	0.981
AERG5	0.138	0.000	0.000	0.206	0.000	0.000	0.388	0.578
AERG6	0.164	0.000	0.000	0.180	0.000	0.000	0.461	0.507
AERG7	0.168	0.000	0.000	0.283	0.003	0.000	0.472	0.803
AERG8	0.170	0.000	0.000	0.226	0.000	0.000	0.478	0.635
AERG9	0.120	0.000	0.000	0.255	0.000	0.000	0.338	0.716
AELS1	0.210	0.000	0.000	0.249	0.000	0.000	0.591	0.701
AELS2	0.209	0.018	0.000	0.347	0.018	0.003	0.644	1.041
AELS3	0.304	0.038	0.000	0.386	0.047	0.005	0.934	1.246
AELS4	0.188	0.015	0.000	0.323	0.023	0.007	0.576	0.998
AELS5	0.190	0.017	0.000	0.326	0.052	0.004	0.588	1.091
AELS6	0.210	0.007	0.000	0.308	0.014	0.000	0.613	0.909
AELS7	0.178	0.000	0.000	0.216	0.000	0.000	0.501	0.606
AELS8	0.184	0.000	0.000	0.274	0.000	0.000	0.518	0.771
AELS9	0.175	0.000	0.000	0.200	0.016	0.001	0.492	0.614
AETR1	0.254	0.000	0.000	0.490	0.000	0.000	0.714	1.377
AETR2	0.277	0.000	0.000	0.530	0.270	0.000	0.779	1.547
AETR3	0.400	0.000	0.000	0.416	0.002	0.000	1.125	1.177
AETR4	0.089	0.000	0.000	0.635	0.003	0.000	0.250	1.794
AETR5	0.210	0.007	0.000	0.467	0.009	0.000	0.612	1.341
AETR6	0.180	0.000	0.000	0.270	0.000	0.000	0.506	0.759
AETR7	0.311	0.026	0.000	0.371	0.072	0.001	0.506	1.272
AETR8	0.291	0.009	0.000	0.327	0.017	0.000	0.847	0.975
AETR9	0.250	0.002	0.000	0.380	0.027	0.000	0.709	1.168
HP11-3	0.075	0.023	0.000	N.A.	N.A.	N.A.	0.261	N.A.
HP11-32	0.121	0.000	0.000	N.A.	N.A.	N.A.	0.340	N.A.
SSP-3	0.165	0.000	0.000	N.A.	N.A.	N.A.	0.463	N.A.
SSP-32	0.169	0.006	0.000	N.A.	N.A.	N.A.	0.490	N.A.

❶ Average of 6 powder samples at each depth of each mixture.
1 inch = 25.4 mm

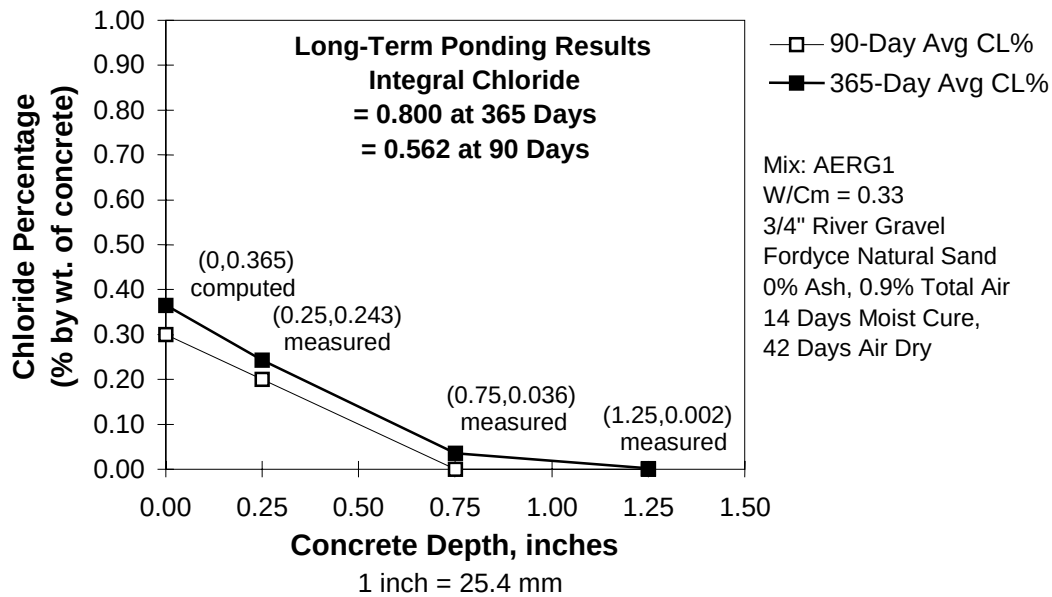


Figure D.1: Chloride Ion Penetration for Laboratory Trial Mix AERG1

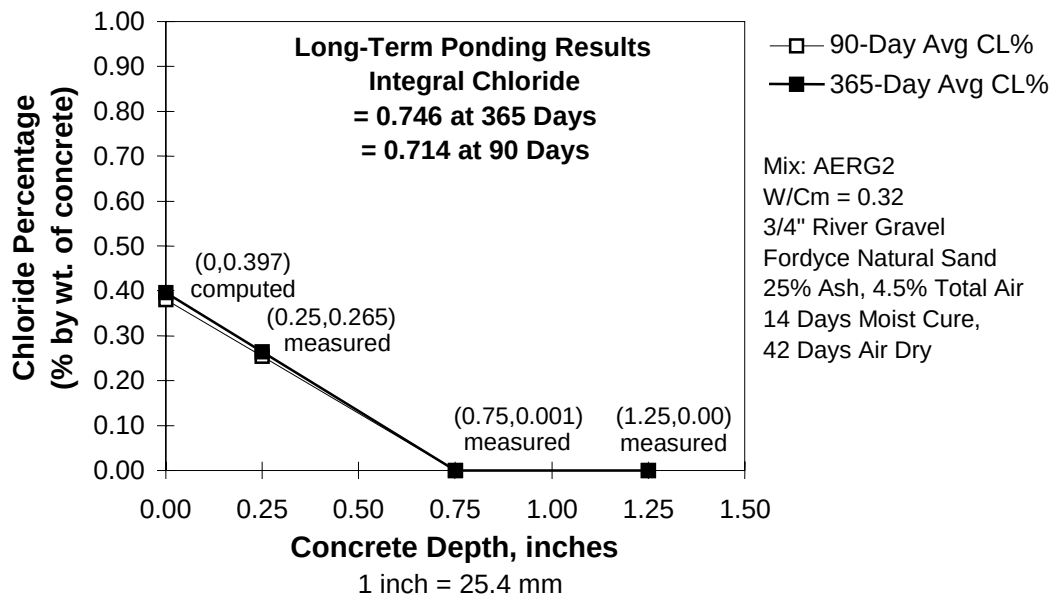


Figure D.2: Chloride Ion Penetration for Laboratory Trial Mix AERG2

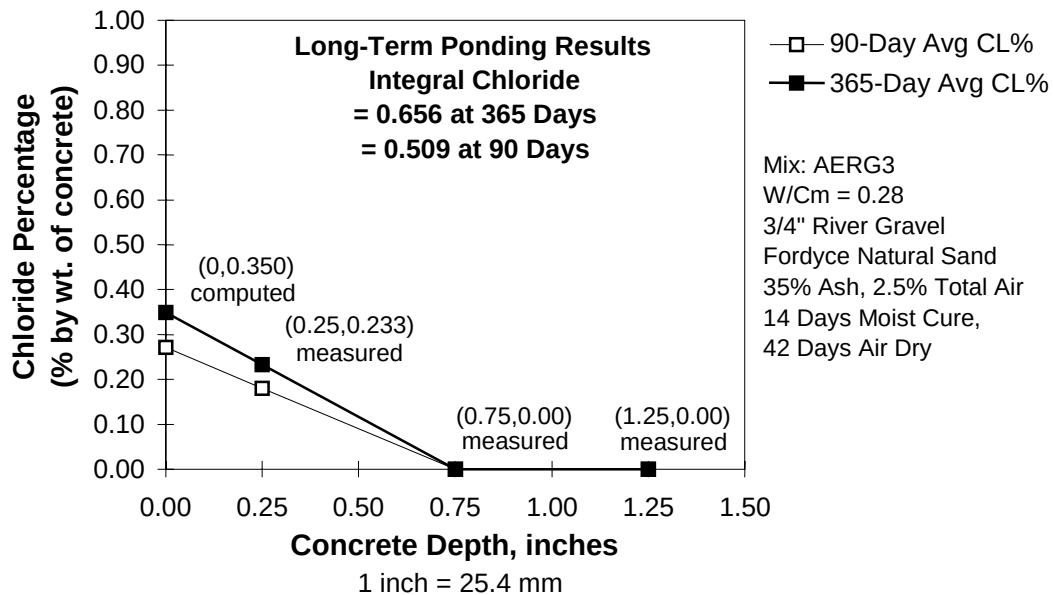


Figure D.3: Chloride Ion Penetration for Laboratory Trial Mix AERG3

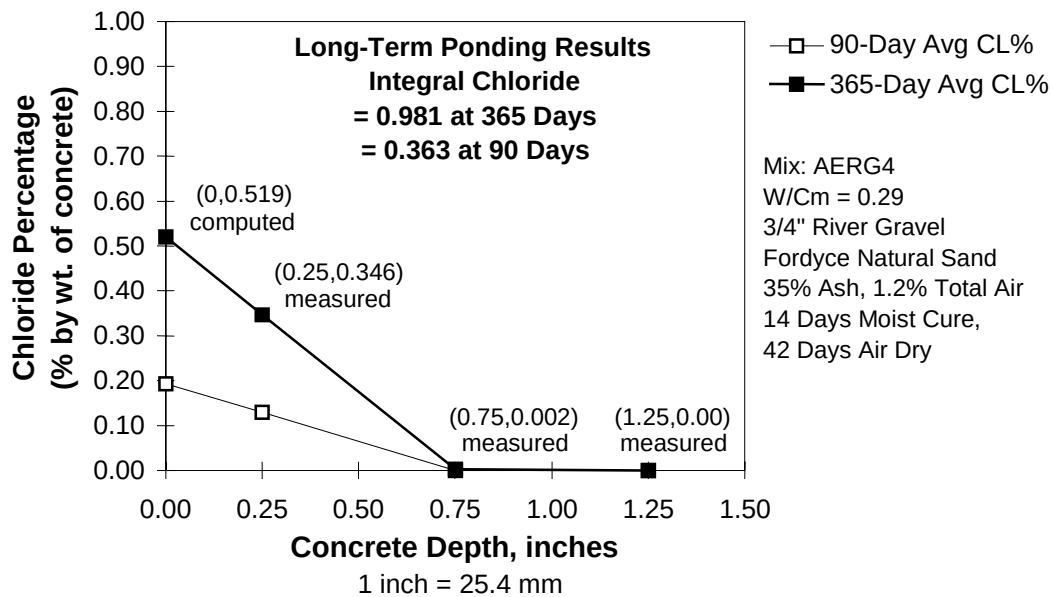


Figure D.4: Chloride Ion Penetration for Laboratory Trial Mix AERG4

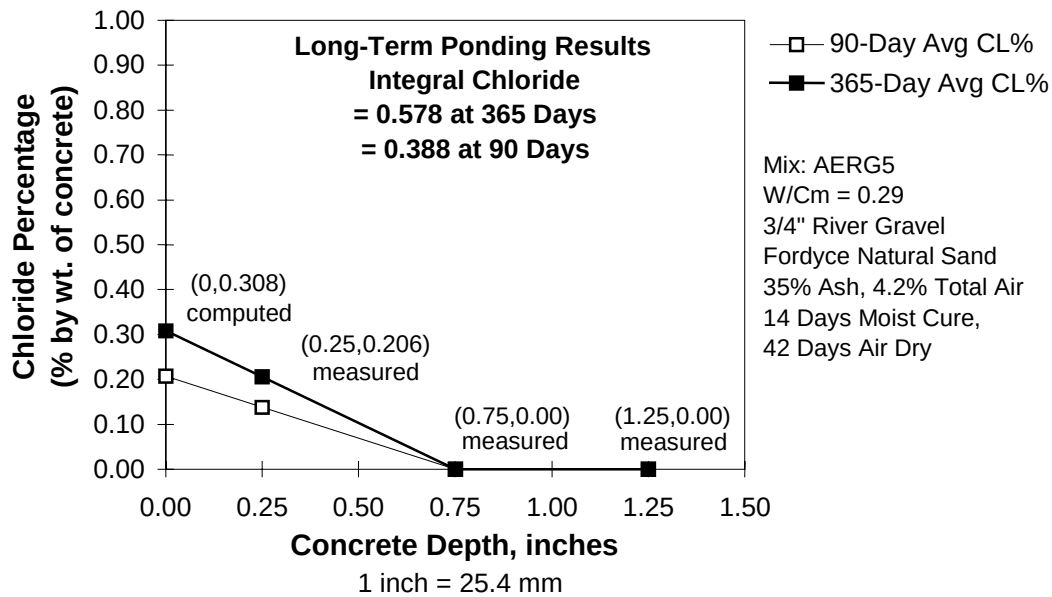


Figure D.5: Chloride Ion Penetration for Laboratory Trial Mix AERG5

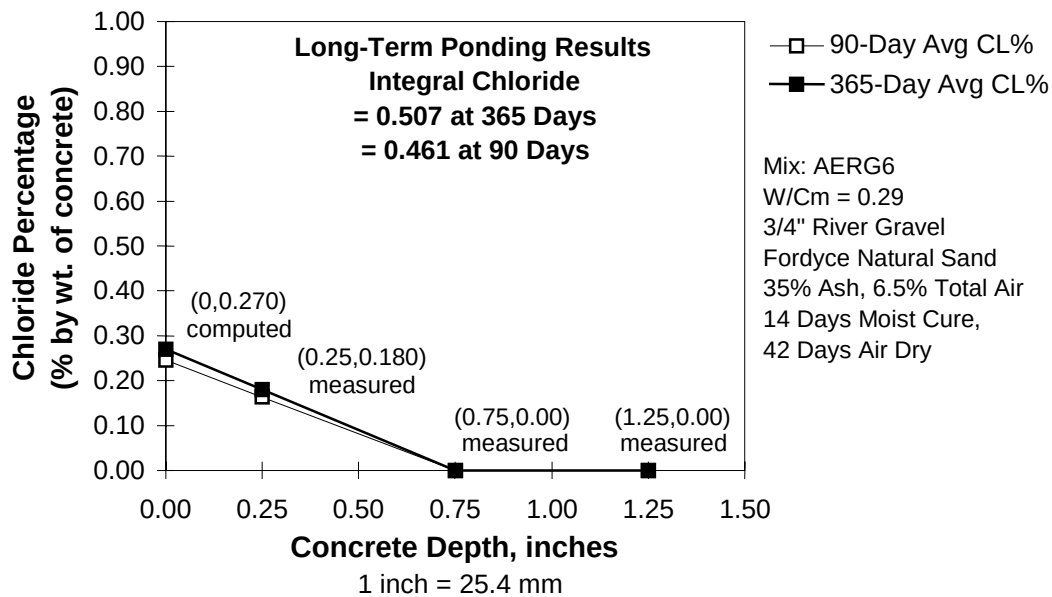


Figure D.6: Chloride Ion Penetration for Laboratory Trial Mix AERG6

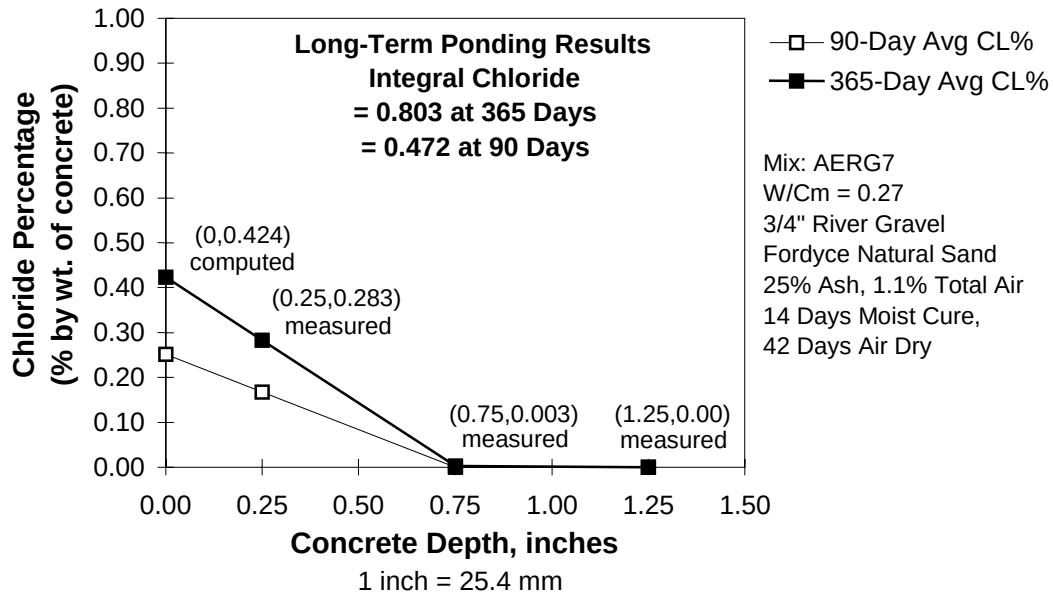


Figure D.7: Chloride Ion Penetration for Laboratory Trial Mix AERG7

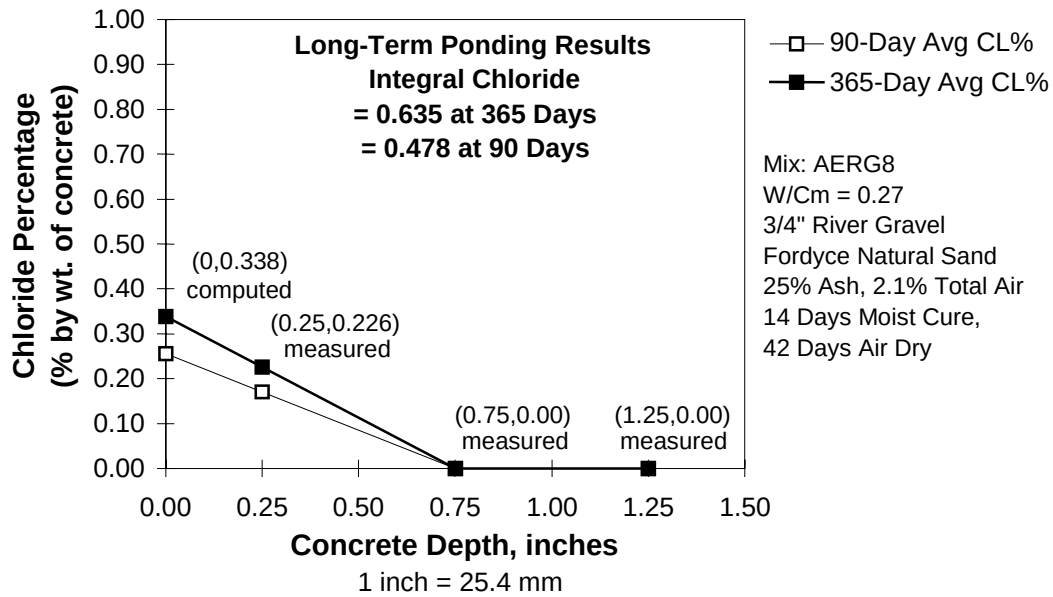


Figure D.8: Chloride Ion Penetration for Laboratory Trial Mix AERG8

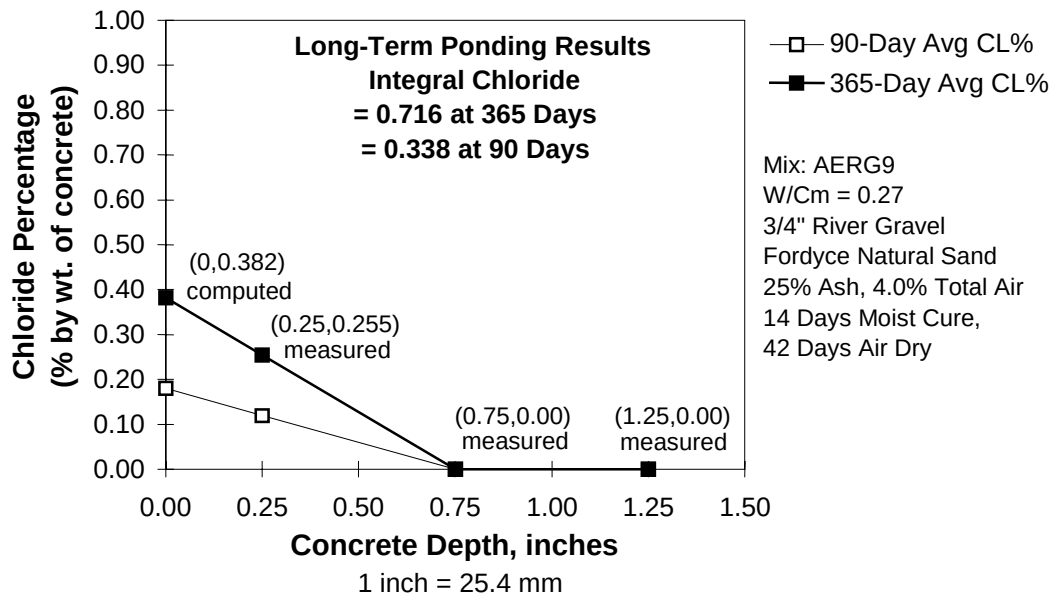


Figure D.9: Chloride Ion Penetration for Laboratory Trial Mix AERG9

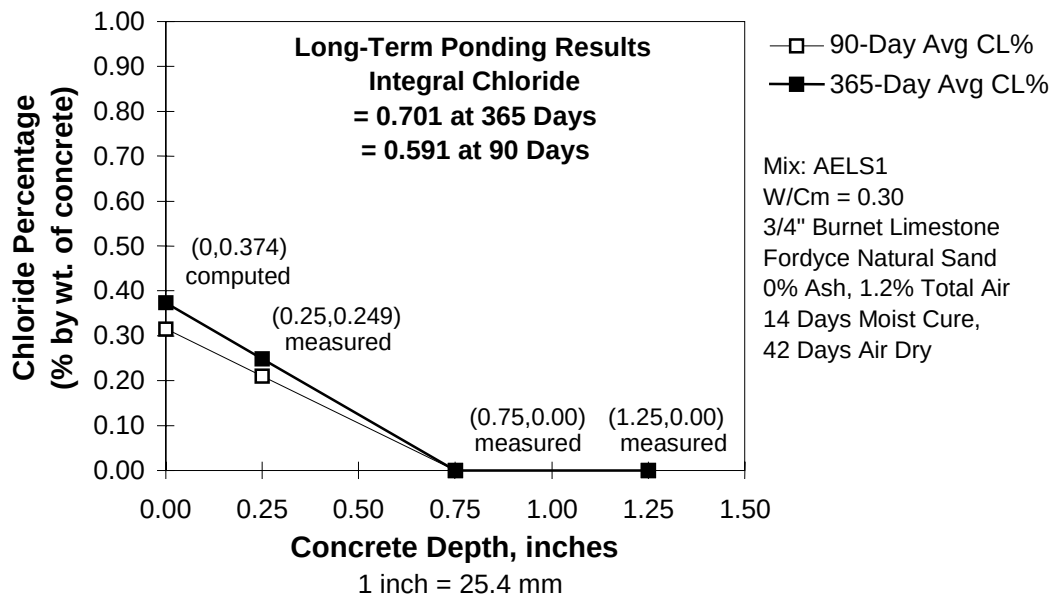


Figure D.10: Chloride Ion Penetration for Laboratory Trial Mix AELS1

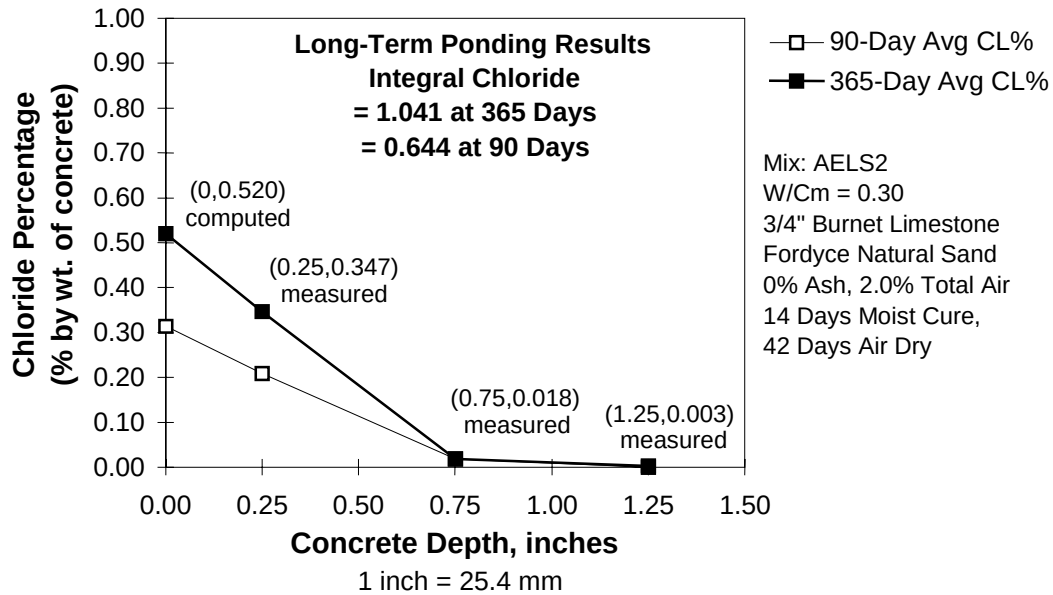


Figure D.11: Chloride Ion Penetration for Laboratory Trial Mix AELS2

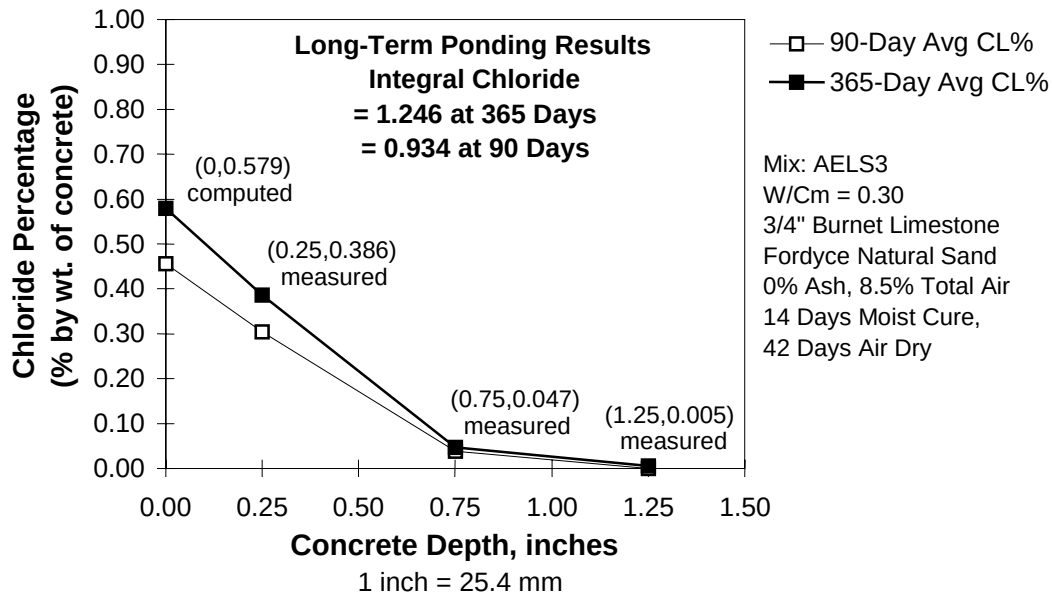


Figure D.12: Chloride Ion Penetration for Laboratory Trial Mix AELS3

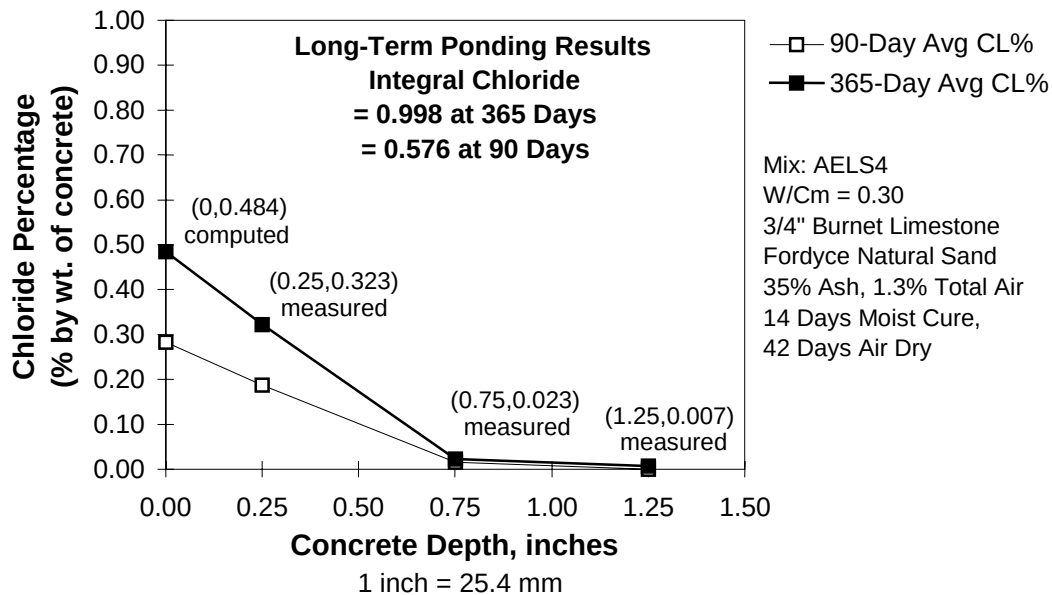


Figure D.13: Chloride Ion Penetration for Laboratory Trial Mix AELS4

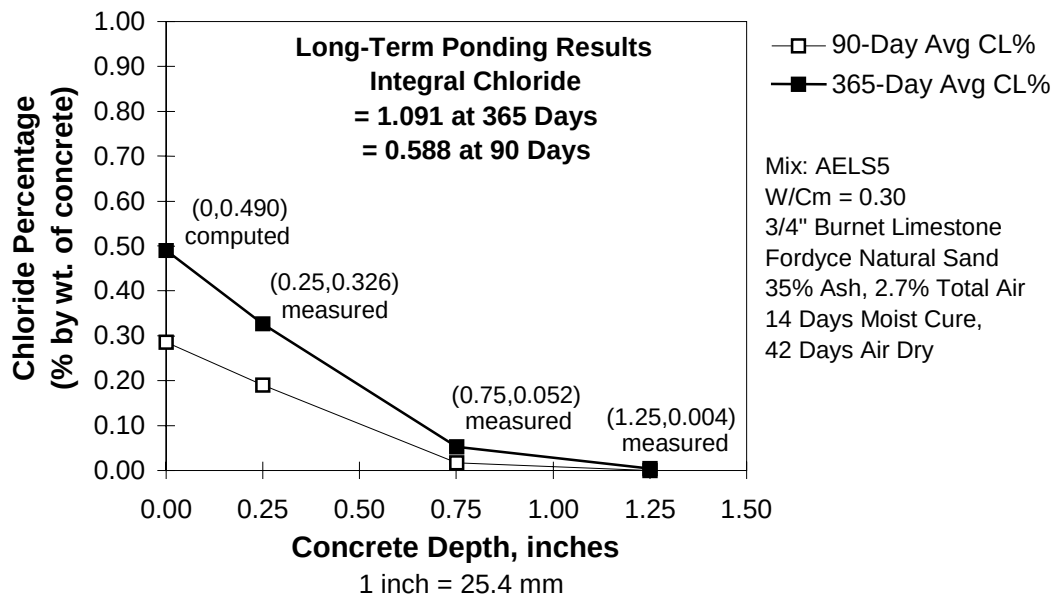


Figure D.14: Chloride Ion Penetration for Laboratory Trial Mix AELS5

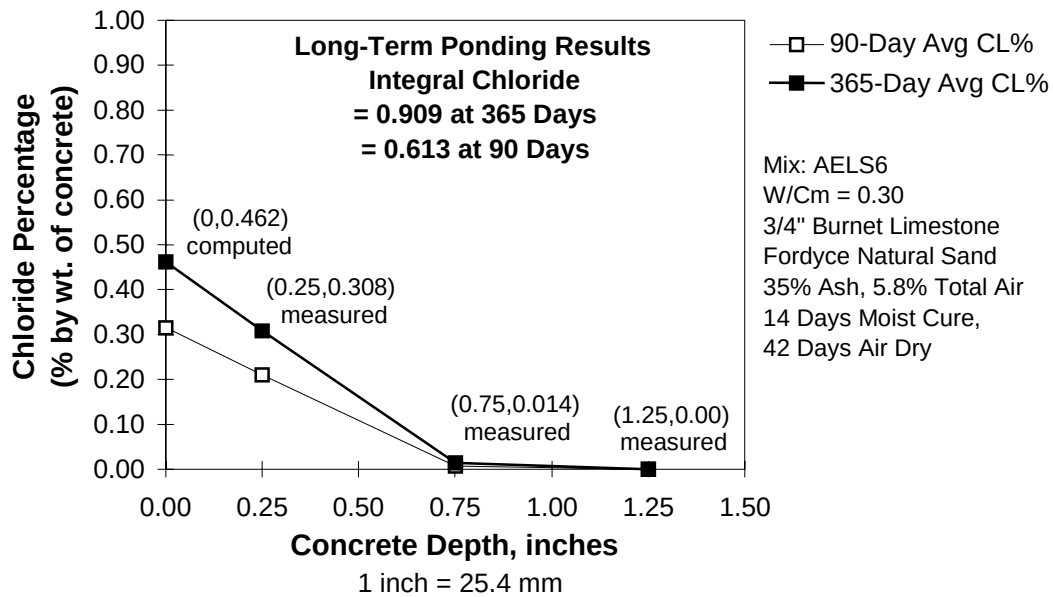


Figure D.15: Chloride Ion Penetration for Laboratory Trial Mix AELS6

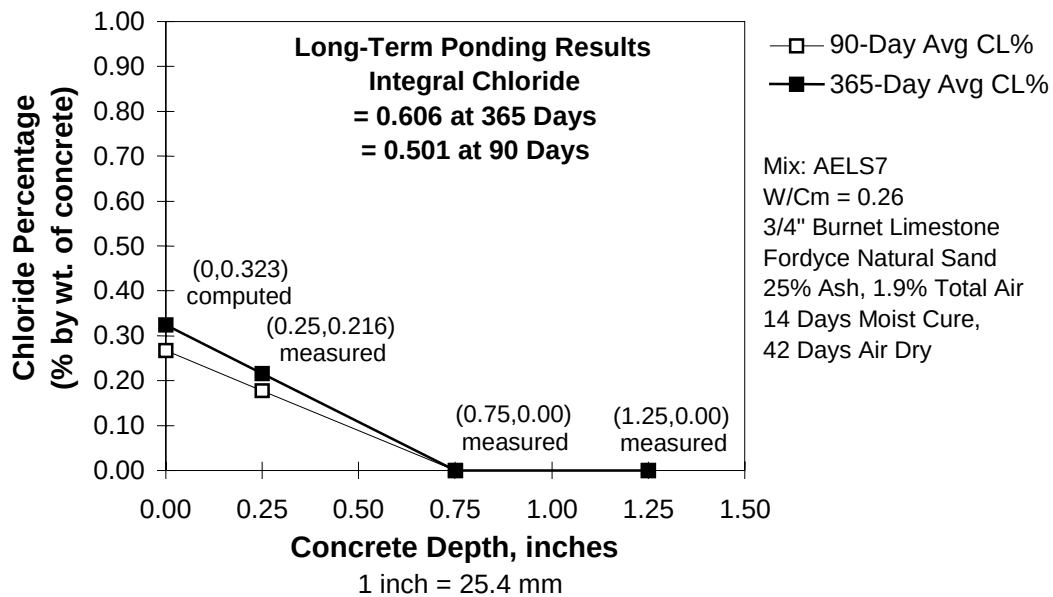


Figure D.16: Chloride Ion Penetration for Laboratory Trial Mix AELS7

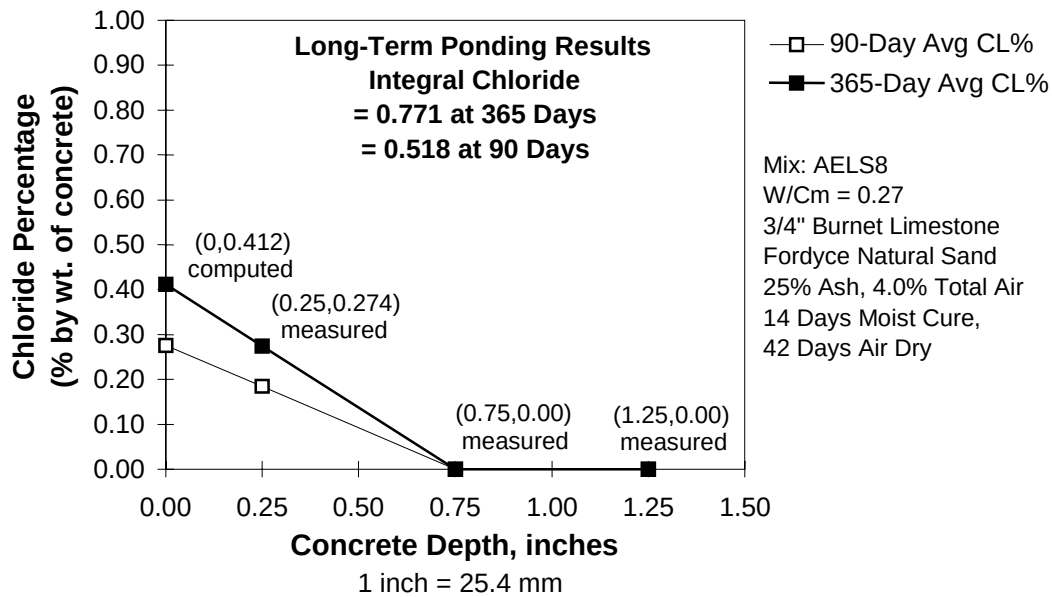


Figure D.17: Chloride Ion Penetration for Laboratory Trial Mix AELS8

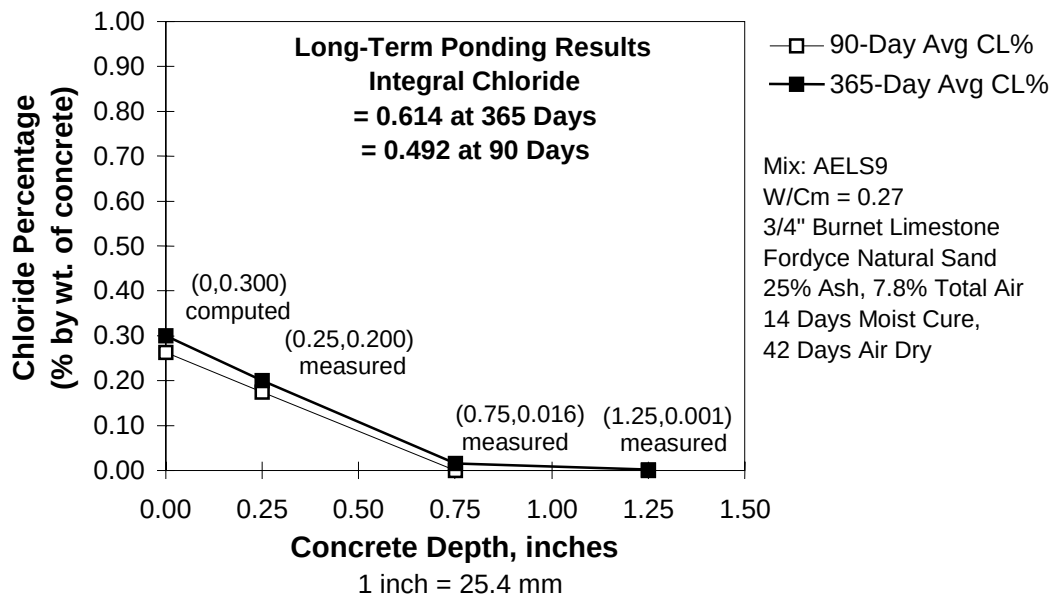


Figure D.18: Chloride Ion Penetration for Laboratory Trial Mix AELS9

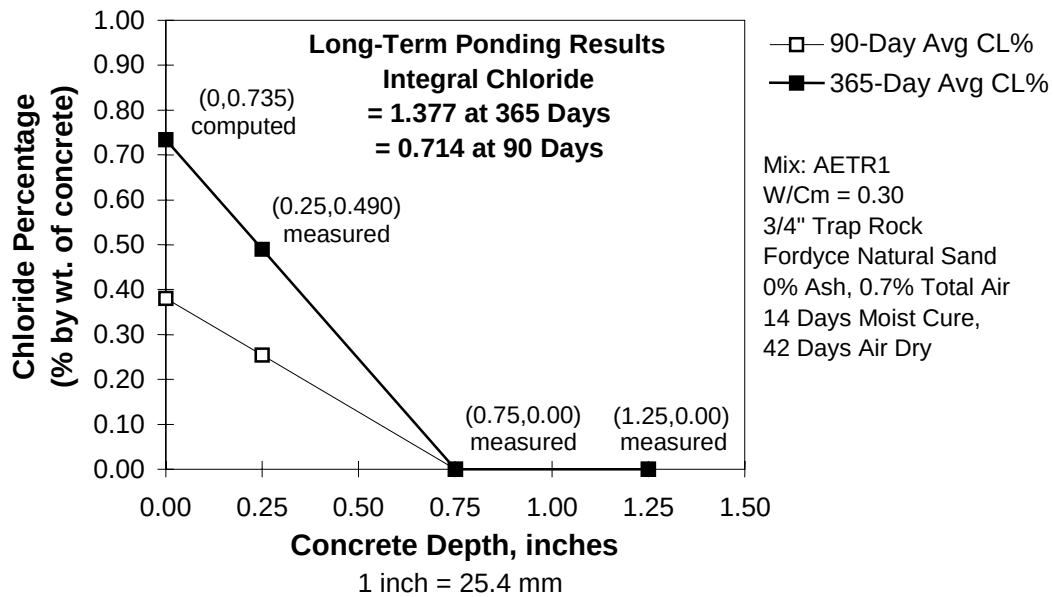


Figure D.19: Chloride Ion Penetration for Laboratory Trial Mix AETR1

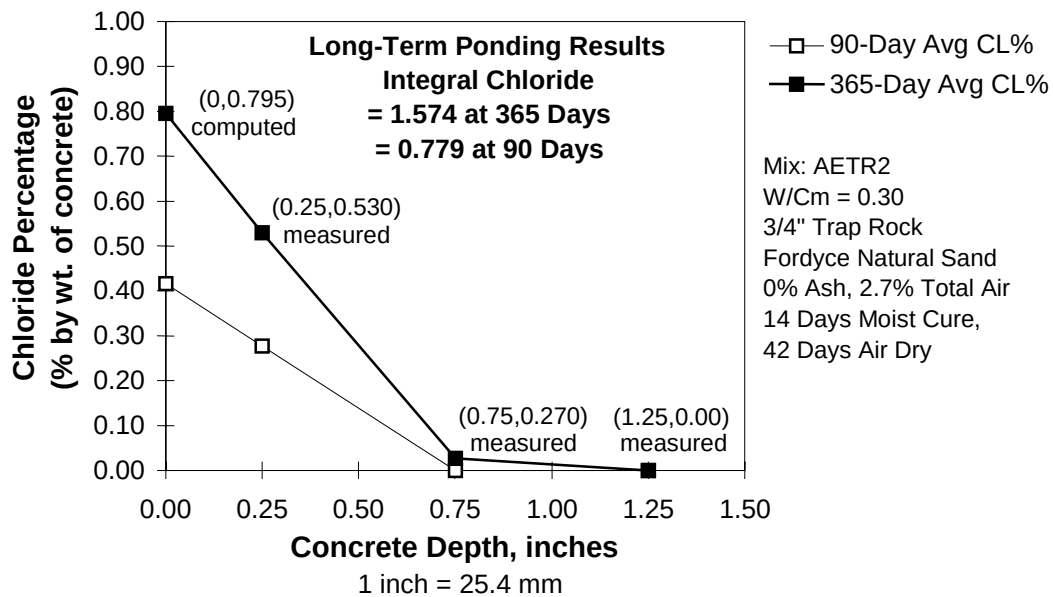


Figure D.20: Chloride Ion Penetration for Laboratory Trial Mix AETR2

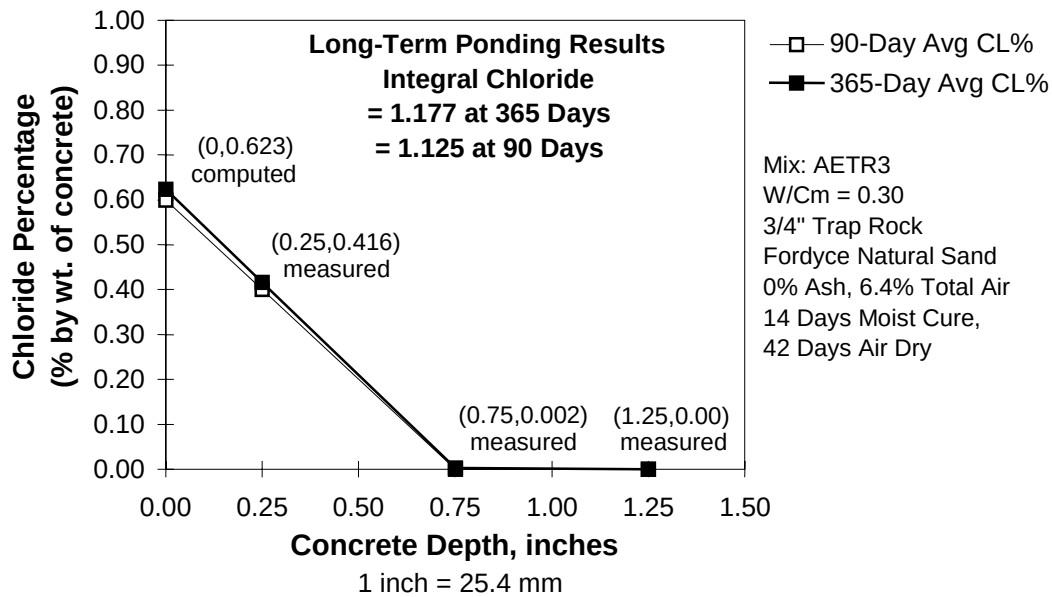


Figure D.21: Chloride Ion Penetration for Laboratory Trial Mix AETR3

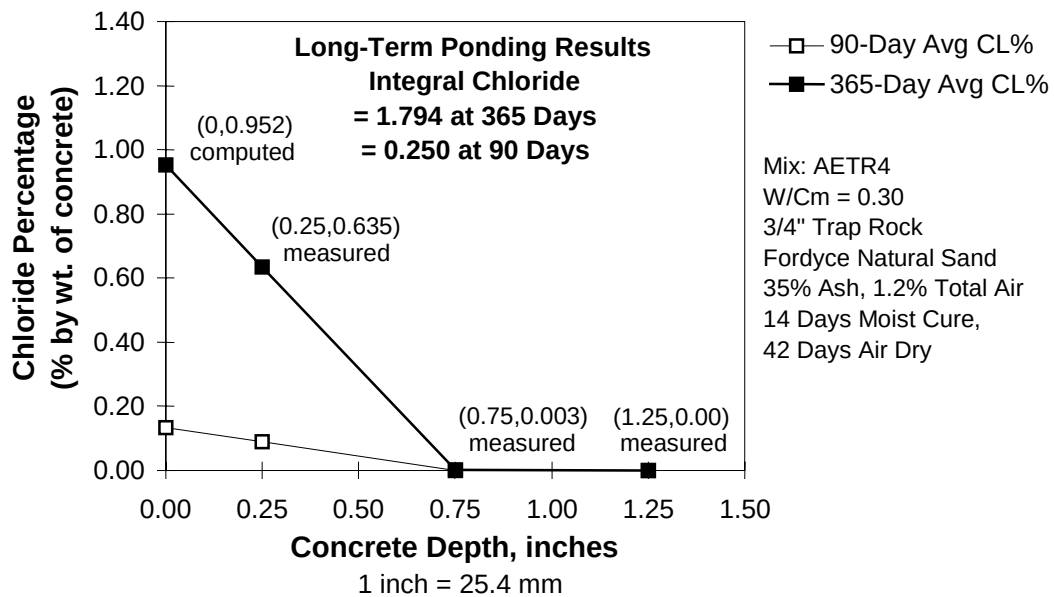


Figure D.22: Chloride Ion Penetration for Laboratory Trial Mix AETR4

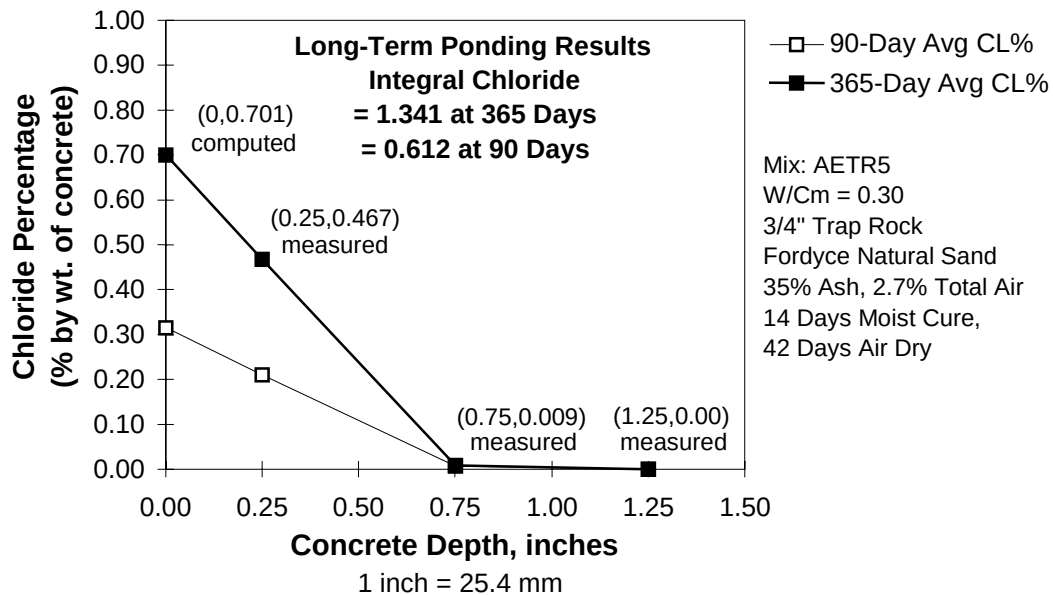


Figure D.23: Chloride Ion Penetration for Laboratory Trial Mix AETR5

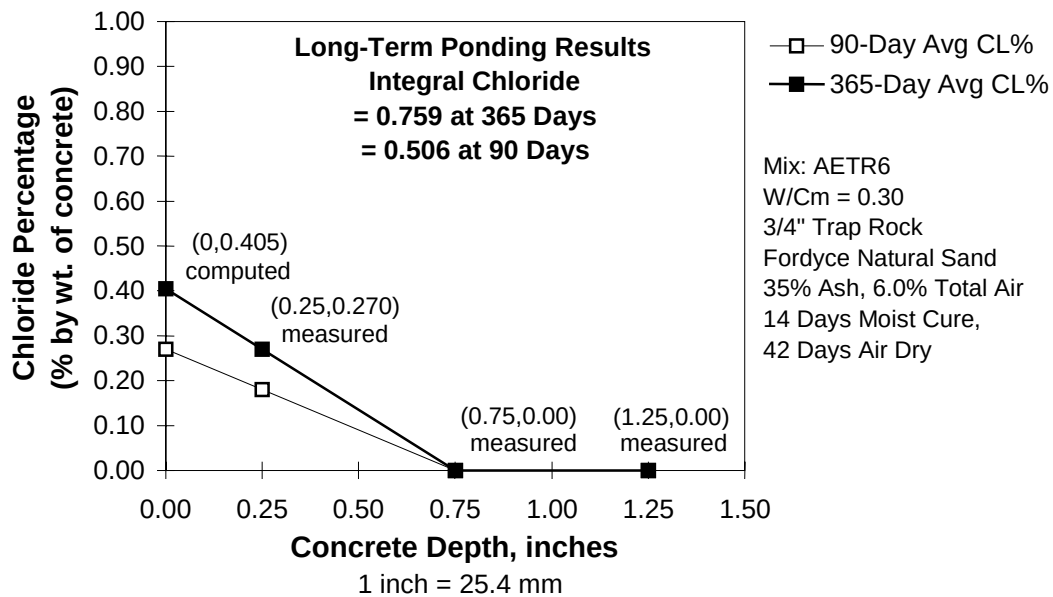


Figure D.24: Chloride Ion Penetration for Laboratory Trial Mix AETR6

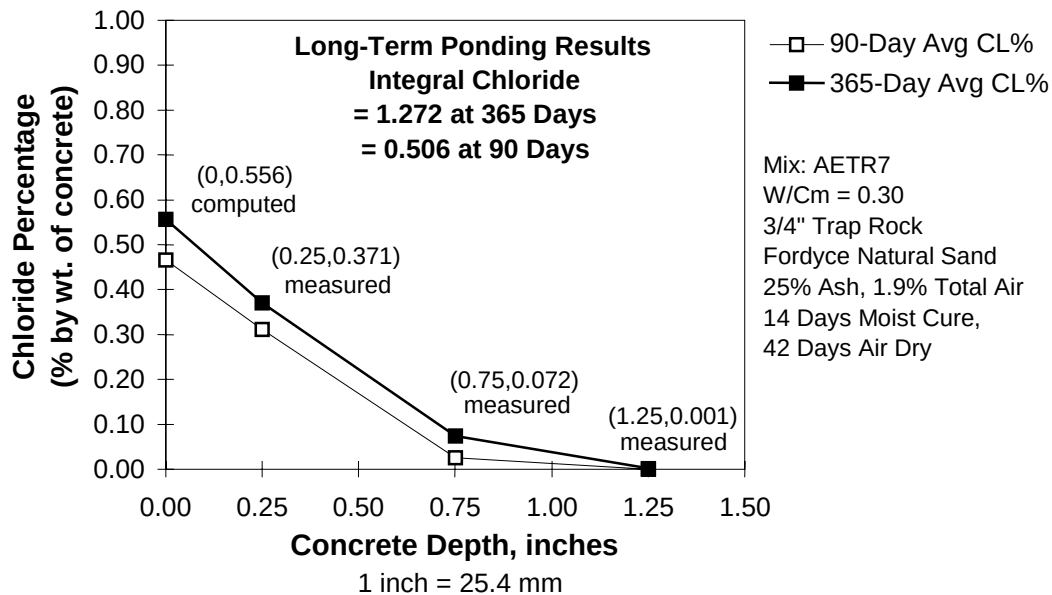


Figure D.25: Chloride Ion Penetration for Laboratory Trial Mix AETR7

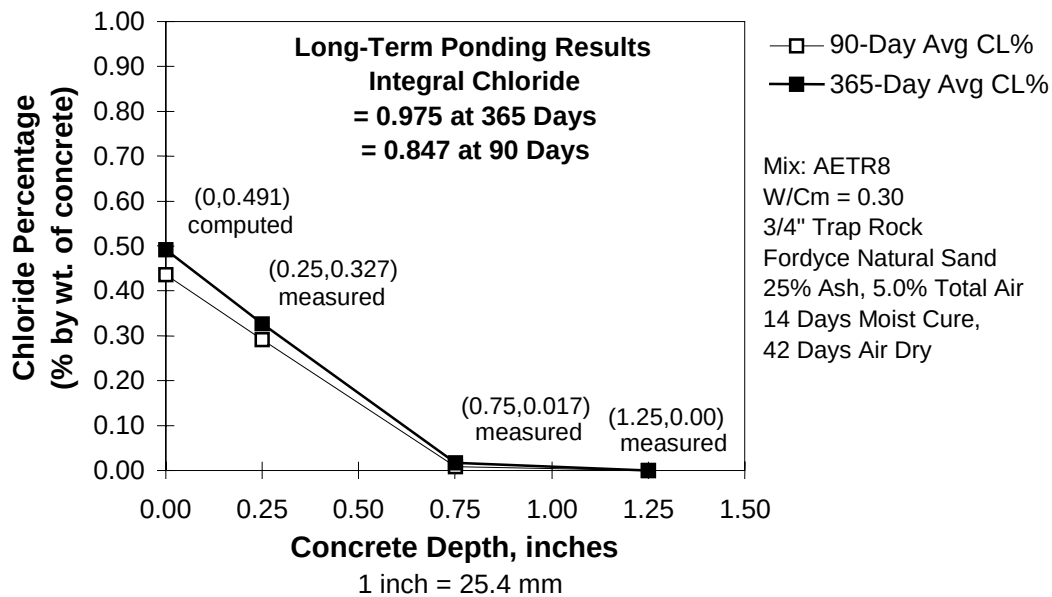


Figure D.26: Chloride Ion Penetration for Laboratory Trial Mix AETR8

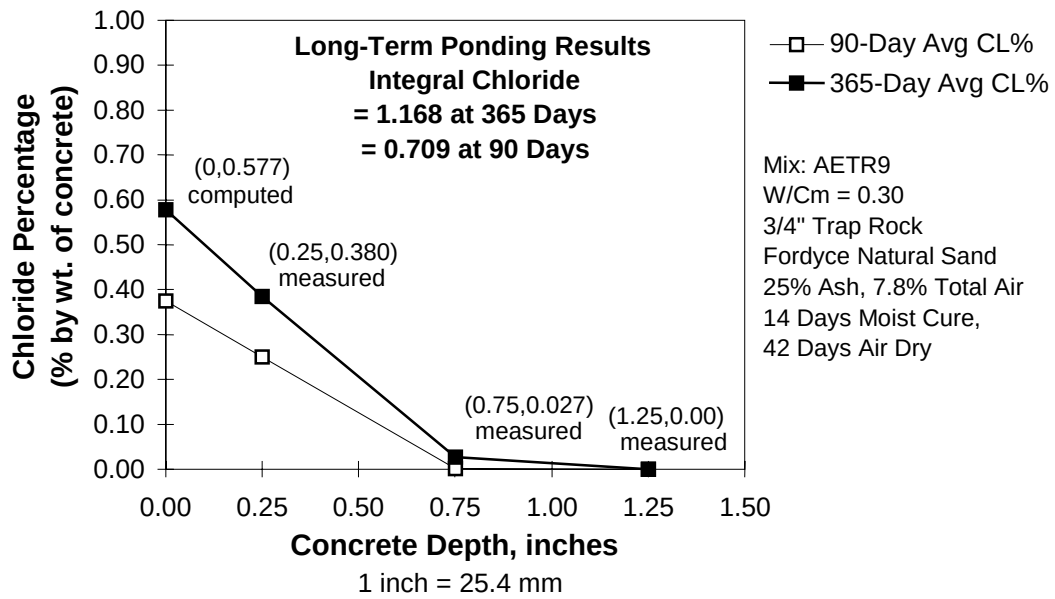


Figure D.27: Chloride Ion Penetration for Laboratory Trial Mix AETR9

Appendix E: Comparative Precast and Cast-In-Place Mix Design Material Costs*

* Does not include transportation or concrete placement costs. Material costs only based on current material cost data as presented in Section 6.1.1.

Table E.1: HS/HPC Precast/Prestressed Beams

material	unit cost per gal or ton	unit cost per oz or lb	material per cy	cost per cy
aggregate	\$07.50	\$0.0038	1918	\$07.19
sand	\$05.50	\$0.0028	1029	\$02.83
cement	\$80.00	\$0.0375	671	\$25.16
fly ash	\$22.00	\$0.0110	316	\$03.48
hrwr	\$08.00	\$0.0645	200	\$12.90
retarder	\$02.75	\$0.0248	27	\$00.67
air entr	\$01.75	\$0.0137	0	\$00.00
water	\$00.00	\$0.0000	247	\$00.00
			total	\$52.24 per CY

Table E.2: Louetta Road Overpass Bridge HS/HPC Precast Column Elements and Precast Deck Panels

material	unit cost per gal or ton	unit cost per oz or lb	material per cy	cost per cy
aggregate	\$07.50	\$0.0038	1983	\$07.44
sand	\$05.50	\$0.0028	1110	\$03.05
cement	\$75.00	\$0.0375	564	\$21.15
fly ash	\$22.00	\$0.0110	163	\$01.79
hrwr	\$08.00	\$0.0645	170.1	\$10.97
retarder	\$02.75	\$0.0248	23.2	\$00.58
air entr	\$01.75	\$0.0137	0	\$00.00
water	\$00.00	\$0.0000	227	\$00.00
			total	\$44.98 per CY

Table E.3: Louetta Class S Modified HPC Deck

material	unit cost per gal or ton	unit cost per oz or lb	material per cy	cost per cy
aggregate	\$07.50	\$0.0038	1856	\$06.96
sand	\$05.50	\$0.0028	1243	\$03.42
cement	\$75.00	\$0.0375	383	\$14.36
fly ash	\$22.00	\$0.0110	148	\$01.63
hrwr	\$08.00	\$0.0645	0	\$00.00
retarder	\$02.75	\$0.0248	45	\$01.12
air entr	\$01.75	\$0.0137	2	\$00.03
water	\$00.00	\$0.0000	230	\$00.00
			total	\$27.51 per CY

Table E.4: Louetta Class K HS/HPC Deck

material	unit cost per gal or ton	unit cost per oz or lb	material per cy	cost per cy
aggregate	\$07.50	\$0.0038	1811	\$06.79
sand	\$05.50	\$0.0028	1304	\$03.59
cement	\$75.00	\$0.0375	473	\$17.74
fly ash	\$22.00	\$0.0110	221	\$02.43
hrwr	\$08.00	\$0.0645	122	\$07.87
retarder	\$02.75	\$0.0248	22	\$00.55
air entr	\$01.75	\$0.0137	0	\$00.00
water	\$00.00	\$0.0000	245	\$00.00
			total	\$38.97 per CY

Table E.5: San Angelo Class C NSC Piers

material	unit cost per gal or ton	unit cost per oz or lb	material per cy	cost per cy
aggregate	\$07.50	\$0.0038	2050	\$07.69
sand	\$05.50	\$0.0028	1200	\$03.30
cement	\$75.00	\$0.0375	564	\$21.15
fly ash	\$22.00	\$0.0110	0	\$00.00
hrwr	\$08.00	\$0.0645	45	\$02.90
retarder	\$02.75	\$0.0248	24	\$00.60
air entr	\$01.75	\$0.0137	3	\$00.04
water	\$00.00	\$0.0000	250	\$00.00
			total	\$35.68 per CY

Table E.6: San Angelo Class F HPC Pier Caps

material	unit cost per gal or ton	unit cost per oz or lb	material per cy	cost per cy
aggregate	\$07.50	\$0.0038	2000	\$07.50
sand	\$05.50	\$0.0028	980	\$02.70
cement	\$75.00	\$0.0375	530	\$19.88
fly ash	\$22.00	\$0.0110	227	\$02.50
hrwr	\$08.00	\$0.0645	83	\$05.36
retarder	\$02.75	\$0.0248	30	\$00.74
air entr	\$01.75	\$0.0137	6	\$00.08
water	\$00.00	\$0.0000	246	\$00.00
			total	\$38.75 per CY

Table E.7: San Angelo Class K HS/HPC Piers & Caps

material	unit cost per gal or ton	unit cost per oz or lb	material per cy	cost per cy
aggregate	7.50	0.0038	1900	\$7.13
sand	5.50	0.0028	1400	\$3.85
cement	75.00	0.0375	491	\$18.41
fly ash	22.00	0.0110	210	\$2.31
hrwr	8.00	0.0645	98	\$6.32
retarder	2.75	0.0248	23	\$0.57
air entr	1.75	0.0137	0	\$0.00
water	0.00	0.0000	262	\$0.00
total				\$38.59 per CY

Table E.8: San Angelo WB NSC Precast Deck Panels

material	unit cost per gal or ton	unit cost per oz or lb	material per cy	cost per cy
aggregate	\$07.50	\$0.0038	1889	\$07.08
sand	\$05.50	\$0.0028	1457	\$04.01
cement	\$75.00	\$0.0375	611	\$22.91
fly ash	\$22.00	\$0.0110	0	\$00.00
hrwr	\$08.00	\$0.0645	257	\$16.58
retarder	\$02.75	\$0.0248	49	\$01.22
air entr	\$01.75	\$0.0137	0	\$00.00
water	\$00.00	\$0.0000	275	\$00.00
total				\$51.80 per CY

Table E.9: San Angelo EB HPC Precast Deck Panels

material	unit cost per gal or ton	unit cost per oz or lb	material per cy	cost per cy
aggregate	\$07.50	\$0.0038	1844	\$06.92
sand	\$05.50	\$0.0028	1355	\$03.73
cement	\$75.00	\$0.0375	250.8	\$09.41
fly ash	\$22.00	\$0.0110	0	\$00.00
hrwr	\$08.00	\$0.0645	300	\$19.36
retarder	\$02.75	\$0.0248	79	\$01.96
air entr	\$01.75	\$0.0137	0	\$00.00
water	\$00.00	\$0.0000	250.8	\$00.00
total				\$41.36 per CY

Table E.10: San Angelo Class S NSC Deck

material	unit cost per gal or ton	unit cost per oz or lb	material per cy	cost per cy
aggregate	\$07.50	\$0.0038	1856	\$06.96
sand	\$05.50	\$0.0028	1243	\$03.42
cement	\$75.00	\$0.0375	611	\$22.91
fly ash	\$22.00	\$0.0110	0	\$00.00
hrwr	\$08.00	\$0.0645	0	\$00.00
retarder	\$02.75	\$0.0248	26	\$00.64
air entr	\$01.75	\$0.0137	3	\$00.04
water	\$00.00	\$0.0000	258	\$00.00
total				\$33.98 per CY

Table E.11: San Angelo Class S Modified HPC Deck

material	unit cost per gal or ton	unit cost per oz or lb	material per cy	cost per cy
aggregate	\$07.50	\$0.0038	1856	\$06.96
sand	\$05.50	\$0.0028	1240	\$03.41
cement	\$75.00	\$0.0375	427	\$16.01
fly ash	\$22.00	\$0.0110	184	\$02.02
hrwr	\$08.00	\$0.0645	0	\$00.00
retarder	\$02.75	\$0.0248	26	\$00.64
air entr	\$01.75	\$0.0137	3	\$00.04
water	\$00.00	\$0.0000	258	\$00.00
total				\$29.09 per CY

Table E.12: San Angelo Class K HS/HPC Deck

material	unit cost per gal or ton	unit cost per oz or lb	material per cy	cost per cy
aggregate	\$07.50	\$00.00	1900	\$07.13
sand	\$05.50	\$00.00	1365	\$03.75
cement	\$75.00	\$00.04	490	\$18.38
fly ash	\$22.00	\$00.01	210	\$02.31
hrwr	\$08.00	\$00.06	156	\$10.07
retarder	\$02.75	\$00.02	28	\$00.69
air entr	\$01.75	\$00.01	3	\$00.04
water	\$00.00	\$00.00	219	\$00.00
total				\$42.37 per CY

