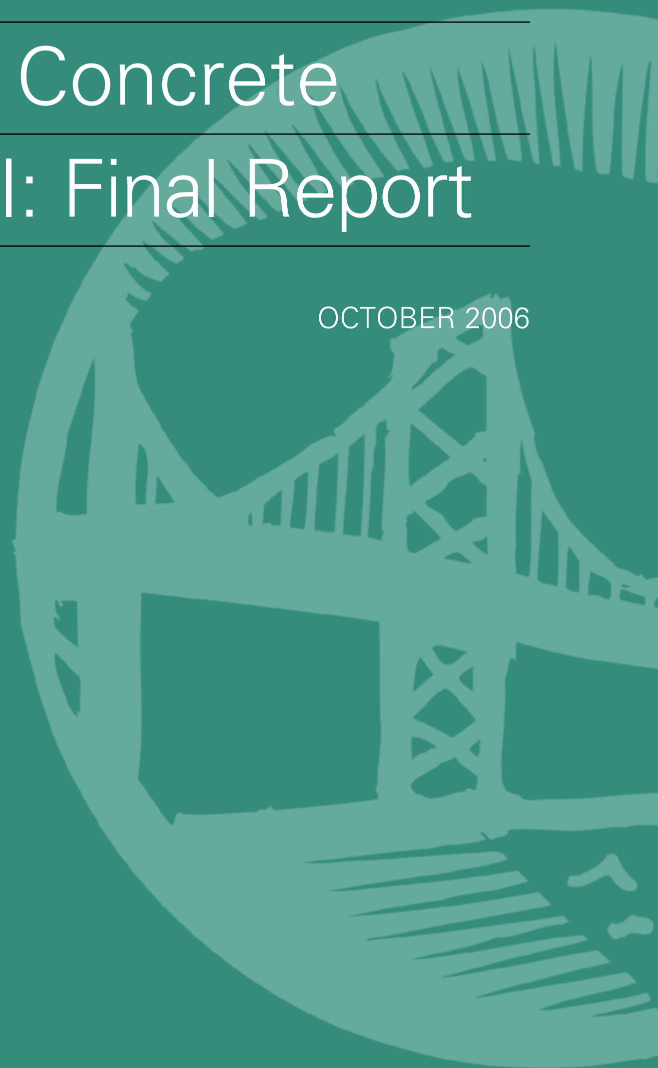

Compilation and Evaluation of Results from High-Performance Concrete Bridge Projects, Volume I: Final Report

PUBLICATION NO. FHWA-HRT-05-056

OCTOBER 2006

U.S. Department of Transportation
Federal Highway Administration

Research, Development, and Technology
Turner-Fairbank Highway Research Center
6300 Georgetown Pike
McLean, VA 22101-2296

FOREWORD

In 1993, the Federal Highway Administration (FHWA) initiated a national program to implement the use of high-performance concrete (HPC) in bridges. The program included the construction of demonstration bridges throughout the United States. In addition, other States have implemented the use of HPC in various bridge elements. The construction of these bridges has provided a large amount of data on the use of HPC.

The first part of this project involved collecting and compiling information from each joint State-FHWA HPC bridge project and other HPC bridge projects. The compilation is available on a CD-ROM and includes information on the benefits of HPC, costs, structural design, specified concrete properties, concrete mix proportions, measured properties, associated research projects, sources of data, and specifications. Information from 19 bridges in 14 States is included. A summary of the compiled information is provided in this final report.

The second part of this project involved a review of the American Association of State Highway and Transportation Officials (AASHTO) *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, the AASHTO *Standard Specifications for Highway Bridges*, the AASHTO *Load and Resistance Factor Design (LRFD) Bridge Design Specifications*, and the AASHTO *LRFD Bridge Construction Specifications* for provisions that directly impact the use of HPC. The detailed review is included in this report.

The third part of the project involved developing proposed revisions to the AASHTO specifications where sufficient research results exist to support the revisions. Proposed revisions to 15 material specifications, 14 test methods, 30 articles of the standard design specifications, 17 articles of the LRFD design specifications, and 16 articles of the LRFD construction specifications are included in this report. These proposed revisions were submitted to the appropriate AASHTO technical committees for consideration for adoption into the relevant specifications.

Also in the third part of this project, a new material specification for combined aggregates and a new test method for slump flow are proposed. In addition, proposed revisions to the FHWA definition of HPC are included.

The fourth part of the project involved developing specific recommendations for needed research where sufficient results do not exist to support needed changes in the specifications. Six research problem statements related to concrete materials and four research problem statements related to structural design are recommended. These research problem statements have been submitted to the appropriate Transportation Research Board technical committees for prioritization and funding recommendations.

Gary L. Henderson
Director, Office of Infrastructure
Research and Development

NOTICE

QUALITY ASSURANCE STATEMENT

Technical Report Documentation Page

1. Report No. FHWA-HRT-05-056	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Compilation and Evaluation of Results From High-Performance Concrete Bridge Projects, Volume I: Final Report		5. Report Date October 2006
		6. Performing Organization Code
7. Author(s) H.G. Russell, R.A. Miller, H.C. Ozyildirim, and M.K. Tadros		8. Performing Organization Report No.
9. Performing Organization's Name and Address Henry G. Russell, Inc. 720 Coronet Road Glenview, IL 60025		10. Work Unit No. (TRAIS)
		11. Contract or Grant No. DTFH61-00-C-00009
12. Sponsoring Agency's Name and Address Federal Highway Administration 6300 Georgetown Pike McLean, VA 22101-2296		13. Type of Report and Period Covered Final Report November 1999–February 2003
		14. Sponsoring Agency's Code

15. Supplementary Notes
Contracting Officer's Technical Representative (COTR): Joseph L. Hartmann, HRDI-06

16. Abstract

In 1993, the Federal Highway Administration (FHWA) initiated a national program to implement the use of high-performance concrete (HPC) in bridges. The program included the construction of demonstration bridges throughout the United States. In addition, other States have implemented the use of HPC in various bridge elements. The construction of these bridges has provided a large amount of data on the use of HPC.

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The appendixes for this volume appears in volume II:
Compilation and Evaluation of Results from High-Performance Concrete Bridge Projects, Volume II: Appendixes (FHWA-HRT-05-057).

17. Key Words Bridges, cast-in-place concrete, high-strength concrete, high-performance concrete, precast concrete, prestressed concrete.	18. Distribution Statement No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22161.

19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages 178	22. Price n/a

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
^oC	Celsius	1.8C+32	Fahrenheit	^oF
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.
(Revised March 2003)

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Tables taken from *AASHTO LRFD Bridge Design Specifications*, Second Edition, 1998, and the 1999, 2000, and 2001 interim revisions

CHAPTER 1. INTRODUCTION

HIGH-PERFORMANCE CONCRETE BRIDGES

In 1993, the Federal Highway Administration (FHWA) initiated a national program to implement the use of high-performance concrete (HPC) in bridges. The program included the construction of demonstration bridges in each of the FHWA regions and the dissemination of the technology and results at showcase workshops. Initially, a total of 18 bridges in 13 States were included in the national program. In addition, other States have implemented the use of HPC in various bridge elements.

The bridges were located in different climatic regions of the United States and used different types of superstructures. The bridges demonstrated practical applications of HPC. In addition, construction of these bridges provided opportunities to learn more about the placement and actual behavior of HPC in bridges. Consequently, many of the bridges were instrumented to monitor their short- and long-term performances. Additionally, concrete material properties were measured for most of the bridges.

SPECIFICATIONS OF THE AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS (AASHTO)

The AASHTO *Standard Specifications for Transportation Materials and Methods of Sampling and Testing* consists of specifications and test methods for materials commonly used in the construction of highway facilities.[1] Part I contains specifications for materials. The relevant specifications for this project are indexed under the three general subject areas of aggregates; concrete, curing materials, and admixtures; and hydraulic cement. Many of the specifications are similar to the equivalent specification published by the American Society for Testing and Materials (ASTM). However, equivalent documents are frequently not identical. When the AASHTO document does not contain a particular specification, bridge owners will reference the ASTM specification. Part II of the AASHTO Standard Specifications contains the test methods. The relevant specifications for this project are indexed under the three general subject areas of aggregates; concrete, curing materials, and admixtures; and hydraulic cement. As with the specifications for materials, many of the test methods are similar, but not identical, to their equivalent ASTM method.

The AASHTO Standard Specifications, like the ASTM specifications are generally based on conventional concrete and have proved to be reliable over the years. However, with the rapidly changing pace of technology, it is difficult for consensus standards to maintain pace with the technology and new information. This is particularly true with the many aspects of HPC. HPC has unique characteristics such as high strength, improved workability, and low permeability. These require closer attention to quality control and quality assurance. HPC is not as forgiving as conventional concrete. It is engineered concrete and must be produced with great care and attention. Performance specifications are highly desirable for HPC. Yet, many specifications still remain prescriptive in their approach and need to be revised to make them more appropriate for use with HPC.

The first edition of the AASHTO *Standard Specifications for Highway Bridges* was published in 1931. Since that time, the AASHTO Standard Specifications have been continuously updated to the 16th edition in 1996, plus interim revisions published in 1997, 1998, 1999, and 2000. See references 2 through 6, respectively. Generally, these updates reflect changes in the state-of-the-art and state-of-the-practice in bridge engineering. However, article 9.15 states that the design of precast, prestressed members ordinarily shall be based on the compressive strength of 34 megapascals (MPa) (5,000 pounds force per square inch (psi)). "An increase to 6000 psi is permissible where, in the Engineer's judgment, it is reasonable to expect that this strength will be obtained consistently. Still higher concrete strengths may be considered on an individual area basis. In such cases, the Engineer shall satisfy himself that the controls over materials and fabrication procedures will provide the required strengths." Article 9.15, which affects all precast, prestressed concrete members, appears to be out of date in light of the consistently higher strengths being achieved in practice and represents a barrier to the use of higher strength concretes.

The design philosophy of the AASHTO Standard Specifications is being slowly replaced by the newer design philosophy of load and resistance factor design (LRFD) as published in the *AASHTO LRFD Bridge Design Specifications*. Since many States are still using the AASHTO Standard Specifications, it is important to address advances in technology that may impact the provisions of the AASHTO Standard Specifications. This includes the use of higher strength concretes.

The first edition of the *AASHTO LRFD Bridge Design Specifications* was published in 1994. The second edition was published in 1998, with interim revisions in 1999, 2000, and 2001. See references 7 through 10, respectively. The AASHTO LRFD Specifications introduced the design philosophy of LRFD for all materials. In this approach, variability in the behavior of structural elements is taken into account in an explicit manner. The AASHTO LRFD Specifications rely on the use of statistical methods, but set forth the results in a readily usable manner. Design of concrete structures in the AASHTO LRFD Specifications is addressed in one section that contains all provisions for design of reinforced, prestressed, and partially prestressed concrete. This is in contrast to the AASHTO Standard Specifications, which has reinforced concrete and prestressed concrete in separate sections.

Article 5.4.2.1 of the *AASHTO LRFD Bridge Design Specifications* limits the applicability of the specifications to a maximum concrete compressive strength of 69 MPa (10,000 psi) unless the physical tests are made to establish the relationship between concrete strength and other properties. Hence, the AASHTO LRFD Specifications have extended the implied limit from 41 MPa (6000 psi) in the AASHTO Standard Specifications to 69 MPa (10,000 psi). With the greater use of higher strength concrete and its economical and technical advantages, consideration needs to be given to raising the limit above 69 MPa (10,000 psi).

The first edition of the *AASHTO LRFD Bridge Construction Specifications* was published in 1998.[11] Interim editions were published in 1999, 2000, and 2001. See references 12 through 14, respectively. Section 8 of the specifications deals with concrete structures and is essentially the same as the AASHTO *Standard Specifications for Highway Bridges*, division II, section 8, "Concrete Structures."

OBJECTIVES

Based on the above introduction, the following objectives for the project were established:

1. Collect and compile information on concrete mixtures, concrete properties, research projects, girder fabrication, bridge construction, live-load tests, and specifications from each of the joint State-FHWA HPC bridge projects and other HPC bridge projects underway or completed. This includes all information related to material properties and structural performance.

2. Analyze and evaluate the compiled information in comparison with existing AASHTO specifications and guidelines for materials, testing, and structural design. For topics where the AASHTO specifications and guidelines need to be revised or do not exist, evaluate the compiled information in comparison with specifications and guidelines from ASTM; the American Concrete Institute (ACI); the Expanded Shale, Clay and Slate Institute (ESCSI); the Precast/Prestressed Concrete Institute (PCI®); the Post-Tensioning Institute (PTI); the Concrete Reinforcing Steel Institute (CRSI); the American Segmental Bridge Institute (ASBI); the Ontario Highway Bridge Design Code (OHBDC); and the Fédération Internationale du Béton (*fib)*.

3. Where sufficient research results exist, recommend equations, specifications with commentary, and guidelines for material and structural properties. These are for use by the AASHTO Highway Subcommittee on Bridges and Structures in the AASHTO *Standard Specifications for Highway Bridges*, the *AASHTO LRFD Bridge Design Specifications*, and the *AASHTO LRFD Bridge Construction Specifications*, and for use by the AASHTO Highway Subcommittee on Materials in the AASHTO *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*.

4. Where insufficient research results exist, produce specific recommendations for needed research.

The above objectives were accomplished through the following tasks:

- Task A: Compilation of Information and Specifications from HPC Bridges.
- Task B: Compilation and Review of AASHTO Specifications and Other Publications.
- Task C: Preparation of an Interim Report.
- Task D: Development of Recommended Equations, Specifications, and Guidelines.
- Task E: Development of Specific Recommendations for Needed Research.
- Task F: Preparation of Final Report and Technical Summary.

This final report constitutes the end product from task F. A summary of the results from task A is given in chapter 2. The detailed compilation from task A is included in a separate CD-ROM. The review of the AASHTO specifications from task B is given in chapter 3. The interim report from task C was completed in May 2001. The proposed revisions to the AASHTO specifications developed in task D are included in appendixes A through E. The proposed changes have been developed for direct use by the AASHTO highway subcommittees. The research problem statements from task E are included in appendix F. The research problem statements have been prepared using the format of the National Cooperative Highway Research Program (NCHRP) so that they may be submitted directly to the appropriate subcommittees of the Transportation Research Board (TRB) and AASHTO.

CHAPTER 2. COMPILATION OF INFORMATION AND SPECIFICATIONS FROM HPC BRIDGES

OBJECTIVE AND SCOPE

The objective of task A was to collect and compile information on concrete mixtures, concrete properties, research projects, girder fabrication, bridge construction, live-load tests, and specifications from each of the joint State-FHWA HPC bridge projects and other HPC bridge projects. This includes all information related to material properties and structural performance.

Information from a total of 19 bridges located in 14 States was collected. The States, abbreviated bridge names, and bridge locations are listed in table 1.

Table 1. HPC bridges included in the compilation.

State	Bridge Name	Location
Alabama	Highway 199	Highway 199 over Uphapee Creek, Macon County
Colorado	Yale Avenue	Interstate 25 (I-25) over Yale Avenue, Denver
Georgia	State Route (S.R.) 920	S.R. 920 (Jonesboro Road) over I-75
Louisiana	Charenton Canal Bridge	LA 87 over Charenton Canal in St. Mary Parish
Nebraska	120th Street	120th Street and Giles Road Bridge, Sarpy County
New Hampshire	Route 104, Bristol	Route 104 over Newfound River, Bristol
New Hampshire	Route 3A, Bristol	Route 3A over Newfound River, Bristol
New Mexico	Rio Puerco	Old Route 66 over the Rio Puerco
North Carolina	U.S. 401	Northbound U.S. 401 over Neuse River, Wake County
Ohio	U.S. Route 22 near Cambridge	U.S. Route 22 over Crooked Creek at Mile Post 6.57 near Cambridge in Guernsey County
South Dakota	I-29 Northbound	I-29 Northbound over Railroad in Minnehana County, Structure No. 50-181-155
South Dakota	I-29 Southbound	I-29 Southbound over Railroad in Minnehana County, Structure No. 50-180-155
Tennessee	Porter Road	Porter Road over S.R. 840, Dickson County
Tennessee	Hickman Road	Hickman Road over S.R. 840, Dickson County
Texas	Louetta Road	Louetta Road Overpass, State Highway (SH) 249, Houston
Texas	San Angelo	U.S. Route 67 over North Concho River, U.S. Route 87, and South Orient Railroad, San Angelo
Virginia	Route 40, Brookneal	Route 40 over Falling River, Brookneal, Lynchburg District
Virginia	Virginia Avenue, Richlands	Virginia Avenue over Clinch River, Richlands
Washington	S.R. 18	Eastbound lanes of S.R. 18 over S.R. 516 in King County

CD-ROM COMPILATION

The information collected to date has been placed on a CD-ROM for easy retrieval and viewing. On the CD-ROM, the information is presented in two formats. The first format consists of an individual compilation for each bridge. Each compilation contains information that is similar to that shown in tables 2 through 11 of this report. However, more details are provided in the compilation.

The compilation is divided into the following sections:

1. Description: This section contains a summary of the overall bridge features.
2. Benefits of HPC and Costs: Highlights why HPC was used in the bridge and provides total cost, cost per square foot, cost per foot, or any other information that was obtained.
3. Structural Design: Lists essential features about the structural design of the bridge.
4. Specified Items: This section includes relevant items that were required by the HPC special provisions. If items were not identified as being specified, the line is left blank.
5. Concrete Materials: This section lists information obtained before actual construction of the bridges. It represents the information that would normally be submitted for approval of concrete mix proportions plus additional data that were available because of the research component.
6. Concrete Material Properties: This section contains information obtained during the actual construction. It is separated into sections on material properties from quality control (QC) tests and material properties from research tests. Separate sections are provided for each HPC element used in the bridge, such as girders and deck.
7. Other Research Data: This section contains research data specifically related to the construction of the showcase bridge. The information varies considerably between compilations depending on the approach and interests of the researchers.
8. Other Related Research: This section contains other research information that was usually obtained prior to construction of the bridge.
9. Sources of Data: References for documents used for the compilation are listed. Some of the data were obtained directly from the States and do not appear in the published data. The names of individuals who supplied the data are listed.
10. Drawings: This section contains miscellaneous details to clarify the written information.
11. HPC Specifications: When available, the special provisions for HPC in the bridge are included.

The compiled information from each bridge may be opened as a Microsoft® Word file from which a hard copy can be printed. For some of the data shown as graphs, the data are available in Microsoft® Excel files. For proper execution of these options, it is necessary to have Word or Excel available on the computer being used to view the CD-ROM. Otherwise, all other software is contained on the CD-ROM.

The second format on the CD-ROM compilation consists of 10 summary tables that can be used to compare data from different States and different bridges. The information contained in the summary tables is not as detailed as the information in the individual bridge compilations. The 10 summary tables reflect the primary information contained in the following main sections of the individual bridge compilations.

- Description (two tables).
- Structural Design (one table).
- Specified Items (two tables).
- Concrete Materials (two tables).
- Concrete Material Properties (three tables).

The summaries of the information in sections 4, 5, and 6 are divided into separate tables for the prestressed concrete girders and the cast-in-place concrete deck. The summary for section 6 is further divided into separate tables related to structural properties and durability properties. The summary information is included in tables 2 through 11 of this report. For comparison purposes, data originally in metric units have been converted to English units.

On the CD-ROM, information on a specific topic can be obtained by using the search option.

The CD-ROM is available in two versions. An interim version contained information collected during the first 15 months of this project. The final version of the CD-ROM contains all information collected for the project.

In the process of collecting the information, some inconsistencies in the data were noted. Attempts were made to resolve as many of these as possible within the time constraints of the project. Nevertheless, some inconsistencies still remain. Also, for most of the bridges, it was not possible to obtain all of the information listed in the compilation. Although the information may exist, obtaining 100 percent of the detailed information proved to be difficult and time-consuming. Blank spaces in tables 2 through 11 indicate that the data may exist, but were not collected or compiled.

Table 2. Major features of the prestressed concrete girders.

STATE	AL	CO	GA	LA	NE	NH	NH	NM	NC	OH
BRIDGE NAME	AL 199	Yale Ave.	S.R. 920	Charenton	120th St.	Route 104	Route 3A	Rio Puerco	U.S. 401	U.S. 22
Girder Type	BT-54	BOX	II, IV	III	NU1100	III	NE 1000	BT1600	IV, III	B42–48
Girder Depth, inches	54	30	36, 54	45	43.3	45	39.4	63	54, 45	42
Maximum Span, feet (ft)	114	112	127.1	72	75	65	60	101.1	91.9	115.5
Maximum Spacing, ft	8.75	Adjacent	7.6	10	12.4	12.5	11.5	12.6	10.2	Adjacent
Maximum No. of Strands	50	64	56	34	30	40	26	42	30	30
Diameter of Strands, inches	0.6	0.6	0.6	0.5	0.5	0.5	0.6	0.5	0.6	0.6
Concrete Strengths										
Specified at Release, psi	8,000	6,500	8,000	7,000	5,500	6,500	5,500	7,000	7,000	6,000
Actual at Release, psi	8,040–9,810	5,600-10,900	10,464	7,618-9,852	8,471	6,700	6,800	7,325	7,700-10,500	6,670–9,210
Age at Release, hours (h)	19–45	—	24	21–40	—	14–17	—	72	27	18
Specified Design, psi	10,000	10,000	10,000	10,000	12,000	8,000	8,000	10,000	10,000	10,000
Actual at Design Age, psi	8,440−11,060	7,800–14,000	13,300	10,502–12,023	13,944	7,755	11,200	10,151	11,800−15,000	9,570–12,920
Design Age, days	28	56	56	56	56	28	28	56	28	56

1 ft = 0.305 m
1 in = 25.4 millimeters (mm)
1 psi = 6.89 MPa

Table 2. Major features of the prestressed concrete girders—*Continued*

STATE	**SD**	**SD**	**TN**	**TN**	**TX (Louetta)[1]**		**TX (San Angelo)[1]**		**VA**	**VA**	**WA**
BRIDGE NAME	I-29 NB	I-29 SB	Porter	Hickman	NB	SB	EB[2]	WB	Route 40	VA Ave.	S.R. 18
Girder Type	II	II	BT-72	BT-72	U 54	U 54	IV	IV	IV	III	W74G
Girder Depth, inches	36	36	72	72	54	54	54	54	54	45	73.5
Maximum Span, ft	61	61	159	151.33	136.5	134.0	157	140.3	80	74	137
Maximum Spacing, ft	11.4	11.4	8.33	8.33	12.94	16.62	11	8.26	10	9.25	8
Maximum No. of Strands	32	32	54	50	87	87	C	64	54	34	40
Diameter of Strands, inches	0.5	0.5	0.6 sp	9/16	0.6	0.6	0.6	0.5	0.5	0.6	0.6
Concrete Strengths											
Specified at Release, psi	8,520	8,520	8,000	8,000	8,800	8,800	8,100	6,600	6,000	6,800	7,400
Actual at Release, psi	—	—	8,635	8,719	9,190	9,680	11,630	8,560	7,340 s 7,820 m	8,840	8,150
Age at Release, h	—	—	24–72	24–72	21	21	46	—	18 s, 72 m	18	18–60
Specified Design, psi	9,900	9,900	10,000	10,000	13,100	13,100	14,000	8,900	8,000	10,000	10,000
Actual at Design Age, psi	15,900	13,250	9,651	10,529	14,440	14,550	15,240	10,130	9,060 s 11,490m	11,200	12,220
Design Age, days	28	28	28	28	28[3]	28[3]	28[3]	28	28	28	56

1 ft = 0.305 m
1 in = 25.4 mm
1 psi = 6.89 MPa

[1] For the Texas bridges, different concrete strengths were specified for different girder span lengths. Listed strengths are the largest values.
[2] Values are for modified design.
[3] Specified at 56 days, generally tested at 28 days.
c = combination of pretensioning and post-tensioning was used, m = moist curing, s = steam curing, sp = special.

Table 3. Major features of the cast-in-place concrete decks.

STATE	AL	CO	GA	LA	NE	NH	NH	NM	NC	OH[1]
BRIDGE NAME	AL 199	Yale Ave.	S.R. 920	Charenton	120th St.	Route 104	Route 3A	Rio Puerco	U.S. 401	U.S. 22
Total Deck Thickness, inches	7	11.5	8	8	7.5	9	9	8.7	8.5	5.5
Curing Type[2]	Wet	Wet[3]	Wet	Wet	Wet	Wet	Wet	Wet	Moist	—
Curing Duration, days	7	5	7	7	8	4	7	14	7	—
Permeability										
Specified, coulombs (C)	—	—	2,000	2,000	1,800	1,000	1,000	—	—	—
Actual, C	2,870	5,597	3,963	1,390	589	753	1,060	—	—	—
Age, days	56	—	56	56	56	56	56	—	—	—
Concrete Strengths										
Specified, psi	6,000	5,076	7,250	4,200	8,000	6,000	6,000	6,000	6,000	—
Actual, psi	7,370	5,310	7,740	5,493	10,433	9,020	9,004	6,160	7,150	—
Age, days	28	28	56	28	56	28	28	28	28	—

1 psi = 6.89 MPa

Table 3. Major features of the cast-in-place concrete decks—*Continued*

STATE	SD	SD	TN	TN	TX (Louetta)		TX (San Angelo)		VA	VA	WA
BRIDGE NAME	I-29 NB	I-29 SB	Porter	Hickman	NB	SB	EB	WB	Route 40	VA Ave.	S.R. 18
Total Deck Thickness, inches	9	9	8.25	8.25	7.25	7.25	7.5	7.5	8.5	8.5	7.5
Curing Type[2]	Wet	Wet	Wet	Wet	Wet	Wet	Wet	Wet	Moist	Moist	Wet
Curing Duration, days	7	7	7	7	10	10	10	10	7	7	14
Permeability											
Specified, C	—	—	1500	1500	—	—	—	—	2500	2500	—
Actual, C	461	1058	—	—	1730	900	—	—	778	1457	2645
Age, days	90	—	28[4]	28[4]	56	56	—	—	28[4]	28[4]	> 210
Concrete Strengths											
Specified, psi	4,500	4,500	5,000	5,000	4,000	8,000	6,000	4,000	4,000	5,000	4,000
Actual, psi	7,070	6,170	8,265	6,460	5,700	9,100	7,345	6,120	6,600	5,400	5,490
Age, days	28	28	28	28	28	28	28	28	28	28	28

1 psi = 6.89 MPa

[1] Ohio bridge used had 5.5-inch-thick (139.7-mm-thick) top flange of the box beam and 3-inch-thick asphalt.
[2] The terminology is that used by the States. In general, wet curing and moist curing represent the same procedures.
[3] May–September only. For November–March, membrane curing with insulated blankets was specified. For April and October, either method was allowed.
[4] Includes 21 days at 100 degrees Fahrenheit (°F) (37.8 degrees Celsius (°C)).

Table 4. Structural design considerations.

STATE	AL	CO	GA	LA	NE	NH	NH	NM	NC	OH
BRIDGE NAME	AL 199	Yale Ave.	S.R. 920	Charenton	120th St.	Route 104	Route 3A	Rio Puerco	U.S. 401	U.S. 22
Specifications	—	STD	STD	STD	STD	STD	STD	STD	STD	STD
Live Loads	—	HS 25	MS 18	HS 20	HS 20	HS 25	HS 25	MS 18	MS 18	HS 25
Prestress Loss, psi	—	65,753	64,500	49,500	—	—	48,800	—	35,585	19.6%
Allowable Tensile Stress										
Top of Girder at Release, $\sqrt{f'_{ci}}$, psi	—	7.5	3.0	7.5	—	7.5	7.5	—	0	7.5
Bottom of Girder after Losses, $\sqrt{f'_{c}}$, psi	—	6.0	6.0	6.0	6.0	3.0	0	—	6.0	6.0
Concrete Cover										
Girder, inches	—	1	1.1–2.1	1	0.8–1.8	1	1.6	1	1.2, 1.5	2, 2.5, 1.25
Top of Deck, inches	2	2.5	2.75	2	2.25	3	3	2.5	2.5	—
Bottom of Deck, inches	1	—	1	1	1	1.25	1.75	1.3	1.2	—

1 psi = 6.89 MPa
1 in = 25.4 mm

Table 4. Structural design considerations—*Continued*

STATE	SD	SD	TN	TN	TX (Louetta)		TX (San Angelo)		VA	VA	WA
BRIDGE NAME	I-29 NB	I-29 SB	Porter	Hickman	NB	SB	EB	WB	Route 40	VA Ave.	S.R. 18
Specifications	STD	STD	STD	STD	STD	STD	STD	STD	STD	STD	LRFD
Live Loads	HS 25	HS 25	HS 20	HS 20	HS 20	HS 20	HS 20	HS 20	HS 20	HS 20	HL 93
Prestress Loss, psi	28%	28%	—	—	55,390	55,390	49,070	47,910	28%	30.81%	41,100
Allowable Tensile Stress											
Top of Girder at Release, $\sqrt{f'_{ci}}$, psi	3	3	—	—	10	10	10	7.5	3	3	200 psi
Bottom of Girder after Losses, $\sqrt{f'_{c}}$, psi	6	6	—	—	8	8	8	6	3	3	0
Concrete Cover											
Girder, inches	1	1	Varies	Varies	1	1	1	—	2, 1	2, 1	1
Top of Deck, inches	2.5	2.5	2.5	2.5	2	2	2	—	2.75[1]	2.75[1]	2.5
Bottom of Deck, inches	1	1	1	1	1.75[2]	1.75[2]	1.75[2]	—	1.5[1]	2.75[1]	1

1 psi = 6.89 MPa
1 in = 25.4 mm

[1] To center of bar.
[2] Cover to 0.375-inch-diameter (9.5 mm-diameter) strand in panels.
LRFD = *AASHTO LRFD Bridge Design Specifications*.
STD = AASHTO *Standard Specifications for Highway Bridges*.

Table 5. Specified concrete properties for prestressed concrete girders.

STATE	AL	CO	GA	LA	NE	NH	NH	NM	NC	OH
BRIDGE NAME	AL 199	Yale Ave.	S.R. 920	Charenton	120th St.	Route 104	Route 3A	Rio Puerco	U.S. 401	U.S. 22
Max. Concrete Temp., °F	160	158	—	160	160	160	160	—	185	—
Slump, inches	≤ 8	—	2–7	≤ 10	—	5–7	5–7	—	≤ 8	6–8
Air Content, %	3.5–6	—	3.5–6.5	—	—	5–8	5	—	3–5	5–7
Girder Curing Method	ps	—	—	sc or h	—	s	—	s	s or h	s
Cylinder Curing Method	mc	—	mc, std	mc	s, std	mc	Mc	—	s or h	s, std
Concrete Strengths										
Release, psi	8,000	6,500	8,000	7,000	5,500	6,500	5,500	7,000	7,000	6,000
Design, psi	10,000	10,000	10,000	10,000	12,000	8000	8000	10,000	10,000	10,000
Design Age, days	28	56	56	≤ 56	56	28	28	28	28	56
Permeability										
Specified, C	—	—	3000	2000	—	1000	1500	—	2000	1000
Age, days	—	—	56	56	—	56	56	—	—	56

1 psi = 6.89 MPa
°C = (°F−32)*5/9

Table 5. Specified concrete properties for prestressed concrete girders—*Continued*

STATE	SD	SD	TN	TN	TX (Louetta)[1]		TX (San Angelo)[1]		VA	VA	WA
BRIDGE NAME	I-29 NB	I-29 SB	Porter	Hickman	NB	SB	EB[2]	WB	Route 40	VA Ave.	S.R. 18
Max. Concrete Temp., °F	—	—	—	—	—	140	—	—	190	190	—
Slump, inches	5	—	2–8	2–8	—	—	—	4–5	0–7	0–7	≤ 7
Air Content, %	5.5–7.5	—	4–8	4–8	—	—	—	—	3–6	3–6	—
Girder Curing Method	s, rh	—	—	—	sc, s	sc, s	sc, s	sc	s, m	s	—
Cylinder Curing Method	—	—	s	s	sc, s	sc, s	sc, s	sc	s, m	s	mc
Concrete Strengths											
Release, psi	8,520	8,520	8,000	8,000	8,800	8,800	8,100	6,600	6,000	6,800	7,400
Design, psi	9,900	9,900	10,000	10,000	13,100	13,100	14,000	8,900	8,000	10,000	10,000
Design Age, days	28	28	28	28	56	56	56	28	28	28	56
Permeability											
Specified, C	—	—	2500	2500	—	—	—	—	1500	1500	1000
Age, days	—	—	28[3]	28[3]	—	—	—	—	28[3]	28[3]	56

1 psi = 6.89 MPa
°C = (°F−32)*5/9

[1] For the Texas bridges, different concrete strengths were specified for different girder span lengths. Listed strengths are the largest values.
[2] Values are for modified design.
[3] Includes 21 days at 100 °F (37.8 °C).

h = heat curing, m = moist curing, mc = match curing, rh = radiant heat curing, ps = partial steam curing, s = steam curing, sc = self-curing (concrete insulated without additional heat), std = AASHTO T 23 standard cure.

Table 6. Specified concrete properties for cast-in-place concrete decks.

STATE	AL	CO	GA	LA	NE	NH	NH	NM	NC	OH[1]
BRIDGE NAME	AL 199	Yale Ave.	S.R. 920	Charenton	120th St.	Route 104	Route 3A	Rio Puerco	U.S. 401	U.S. 22
Maximum Water/ Cementitious Material (W/CM) Ratio	0.40	0.44	0.35	0.40	—	0.38	0.38	—	0.43	0.28
Slump, inches	≤ 5	[2]	2–5	2–8	< 8	2–3	2–3	—	≤ 5	5–7
Air Content, %	3.5–6	5–8	3.5–6.5	4–7	5–7.5	6–9	5–9	—	4.5–7.5	5–7
Curing Type[3]	Wet	Wet[4]	Wet	Wet	Wet	Wet	Wet	Wet	Moist	Wet
Curing Duration, days	7	5	7	7	8	4	7	14	7	7
Concrete Strengths										
Compressive Strength, psi	6,000	5,076	7,250	4,200	8,000	6,000	6,000	6,000	6,000	8,000
Age, days	28	28	56	28	56	28	28	28	28	56
Permeability										
Specified, C	—	—	2,000	2,000	1,800	1,000	1,000	—	—	1,000
Age, days	—	—	56	56	56	56	56	—	—	56

1 psi = 6.89 MPa

Table 6. Specified concrete properties for cast-in-place concrete decks—*Continued*

STATE	SD	SD	TN	TN	TX (Louetta)		TX (San Angelo)		VA	VA	WA
BRIDGE NAME	I-29 NB	I-29 SB	Porter	Hickman	NB	SB	EB	WB	Route 40	VA Ave.	S.R. 18
Maximum W/CM Ratio	0.39	—	0.43	0.43	0.44	—	0.44	0.44	0.45	0.45	0.39
Slump, inches	5–7	—	2–8	2–8	3–4	3–9	3–9	3–4	2–7	2–7	—
Air Content, %	5.5–7.5	—	4–8	4–8	5	0	6	6	5–8	5–8	6
Curing Type[3]	Wet	Wet	Wet	Wet	Wet	Wet	Wet	Wet	Moist	Moist	Wet
Curing Duration, days	7	7	7	7	10	10	10	10	7	7	14
Concrete Strengths											
Compressive Strengths, psi	4,500	4,500	5,000	5,000	4,000	8,000	6,000	4,000	4,000	5,000	4,000
Age, days	28	28	28	28	28	28	28	28	28	28	28
Permeability											
Specified, C	—	—	1500	1500	—	—	—	—	2500	2500	—
Age, days	—	—	28[5]	28[5]	—	—	—	—	28[5]	28[5]	—

1 psi = 6.89 MPa
$^{o}C = (^{o}F-32)*5/9$
1 in = 25.4 millimeters (mm)

[1] Ohio bridge did not have a separate concrete deck. Values are for the abutments.
[2] Not more than 1.5 inches (38.1 mm) greater than the slump of the approved mix design.
[3] The terminology is that used by the States. In general, wet curing and moist curing represent the same procedures.
[4] May–September only. For November–March, membrane curing with insulated blankets was specified. For April and October, either method was allowed.
[5] Includes 21 days at 100 ^{o}F (37.8 °C).

Table 7. Concrete mix proportions for prestressed concrete girders.[1]

STATE	AL	CO	GA	LA	NE	NH[2]	NH[2]	NM	NC	OH
BRIDGE NAME	AL 199	Yale Ave.	S.R. 920	Charenton	120th St.	Route 104	Route 3A	Rio Puerco	U.S. 401	U.S. 22
W/CM Ratio	0.28	0.29	0.25	0.25	0.24	0.33	0.30	0.30	0.30	0.28
Cement Type	III	III	I	III	I	III	II	—	I/II	III
Cement Quantity, pounds per cubic yard (lb/yd^3)	753	730	800	691	750	777	550	846	900	846
Fly Ash Type	C	—	F	C	C	—	—	F	—	—
Fly Ash Quantity, lb/yd^3	133	—	100	296	200	—	—	127	—	—
Silica Fume, lb/yd^3	—	35	75	—	50	50	50	68	50	100
Ground Granulated Blast-Furnace Slag, lb/yd^3	—	—	—	—	—	—	200	—	—	—
Fine Aggregate, lb/yd^3	1,069	1,363	932	1,135	990	1,075	1,200	953	905	927
Coarse Aggregate Maximum Size, inches	0.75[3]	0.375	0.75	0.5	0.5	0.75	0.75	—	0.75	0.375
Coarse Aggregate Quantity, lb/yd^3	1,916	1,775	1,802	1,803	1,860	1,850	1,750	1,446	2,000	1,774
Water, lb/yd^3	248	219	246	247	240	273	242	312	275	262
Air Entrainment, fluid ounces per cubic yard ($fl\ oz/yd^3$)	35	—	4	—	—	10	5	—	6.0	21
Water Reducer, $fl\ oz/yd^3$	—	15–58	27	60	30	—	—	—	—	—
Retarder, $fl\ oz/yd^3$	—	—	—	—	—	14	—	—	36	28
High-Range Water Reducer, $fl\ oz/yd^3$	225	44–131	150	150	225	206	80	—	81	203

1 in = 25.4 mm
1 $fl\ oz/yd^3$ = 38.69 ml/m^3 (38.69 milliliters per cubic meter)
1 lb/yd^3 = 0.593 kg/m^3 (0.593 kilograms per cubic meter)

Table 7. Concrete mix proportions for prestressed concrete girders[1]—*Continued*

STATE	SD	SD	TN	TN	TX (Louetta)		TX (San Angelo)		VA (s)	VA	WA
BRIDGE NAME	I-29 NB	I-29 SB	Porter	Hickman	NB	SB	EB	WB	Route 40	VA Ave.	S.R. 18
W/CM Ratio	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.27	0.32	0.28	0.27
Cement Type	II	II	I	I	III	III	III	III	I	I	III
Cement Quantity, lb/yd^3	680	680	747	747	671	671	671	526	752	752	728
Fly Ash Type	—	—	C	C	C	C	C	C	—	—	C
Fly Ash Quantity, lb/yd^3	—	—	249	249	315	315	312	196	—	—	222
Silica Fume, lb/yd^3	84	84	—	—	—	—	—	—	55	75	50
Ground Granulated Blast-Furnace Slag, lb/yd^3	—	—	—	—	—	—	—	—	—	—	—
Fine Aggregate, lb/yd^3	1200	1200	974	974	1086	1086	1062	1160	1425	1350	890
Coarse Aggregate Maximum Size, inches	0.75	0.75	0.75	0.75	0.5	0.5	0.5	0.75	0.75	0.5	0.5
Coarse Aggregate Quantity, lb/yd^3	1825	1825	1920	1920	1919	1919	1863	1998	1675	1671	1870
Water, lb/yd^3	190	190	248	248	248	248	246	196	255	235	265
Air Entrainment, fl oz/yd^3	6.0	4.0	—	—	—	—	—	—	3–7	7	—
Water Reducer, fl oz/yd^3	46	46	—	—	—	—	—	—	24–30	—	29
Retarder, fl oz/yd^3	—	—	—	—	27	27	28	16	24–30	25–30	—
High-Range Water Reducer, fl oz/yd^3	260	382	5–15	5–15	200	200	200	159	202	207	215

1 in = 25.4 mm
1 fl oz/yd^3 = 38.69 ml/m^3 (38.69 milliliters per cubic meter)
1 lb/yd^3 = 0.593 kg/m^3 (0.593 kilograms per cubic meter)

[1] Based on approved concrete mix proportions.
[2] Also includes 4.0 gallons per cubic yard (gal/yd^3) (4.951 liters per cubic meter (l/m^3)) of corrosion inhibitor.
[3] Later changed to 0.5 inch (12.7 mm).
(s) Steam-cured girders only.

Table 8. Concrete mix proportions for cast-in-place concrete decks.[1]

STATE	AL	CO	GA	LA	NE	NH[2]	NH[2]	NM	NC	OH[3]
BRIDGE NAME	AL 199	Yale Ave.	S.R. 920	Charenton	120th St.	Route 104	Route 3A	Rio Puerco	U.S. 401	U.S. 22
W/CM Ratio	0.37	0.38	0.34	0.39	0.31	0.38	0.38	0.32	0.33	0.22
Cement Type	II	—	I	IS	IP	II	Blended	I/II	I/II	I
Cement Quantity, lb/yd^3	658	705	651	306	750	607	608	687	587	803
Fly Ash Type	C	—	—	—	C	—	—	F	F	F
Fly Ash Quantity, lb/yd^3	165	—	—	—	75	—	—	172	175	68.5
Silica Fume, lb/yd^3	—	—	12	—	—	53	52	—	—	87.5
Ground Granulated Blast-Furnace Slag, lb/yd^3	—	—	—	305	—	—	—	—	—	—
Fine Aggregate, lb/yd^3	1,042	1,384	1,385	1,176	1,400	1,190	1,190	1,290	1,000	868
Coarse Aggregate Maximum Size, inches	1	0.75	0.75	1	0.5	0.75	0.75	0.5	1	0.375
Coarse Aggregate Quantity, lb/yd^3	1,860	1,488	1,700	1,900	1,400	1,815	1,815	1,400	1,825	1,721
Water, lb/yd^3	304	266	225	238	255	253	253	275	250	210
Air Entrainment, fl oz/yd^3	32	3.4	16.2	4.0	5	6	4.5	8.6	—	31.6
Water Reducer, fl oz/yd^3	25	—	19.5	—	30	20	19.8	—	—	—
Retarder, fl oz/yd^3	—	—	—	36.7	—	—	—	—	—	—
High-Range Water Reducer, fl oz/yd^3	98	19	143	—	135	79	105.6	56.3	—	191.8

1 in = 25.4 mm
1 fl oz/yd^3 = 38.69 ml/m^3 (38.69 milliliters per cubic meter)
1 lb/yd^3 = 0.593 kg/m^3 (0.593 kilograms per cubic meter)

Table 8. Concrete mix proportions for cast-in-place concrete decks[1]—*Continued*

STATE	SD	SD	TN	TN	TX (Louetta)		TX (San Angelo)		VA	VA	WA
BRIDGE NAME	I-29 NB	I-29 SB	Porter	Hickman	NB	SB	EB	WB	Route 40	VA Ave.	S.R. 18
W/CM Ratio	0.39	0.36	0.36	0.36	0.43	0.35	0.31	0.42	0.40	0.45	0.39
Cement Type	II	II	I	I	I	I	II	II	II	I	I
Cement Quantity, lb/yd^3	511	590	494	494	383	474	490	427	329	560	660
Fly Ash Type	—	F	C	C	C	C	C	C	—	F	C
Fly Ash Quantity, lb/yd^3	118	124	153	153	148	221	210	184	—	140	75
Silica Fume, lb/yd^3	55	—	50	50	—	—	—	—	—	—	—
Ground Granulated Blast-Furnace Slag, lb/yd^3	—	—	—	—	—	—	—	—	329	—	—
Fine Aggregate, lb/yd^3	1,100	1,222	1,115	1,115	1,243	1,303	1,365	1,240	1,173	1,004	1,100
Coarse Aggregate Maximum Size, inches	—	—	1	1	1.5	1	1–1/4	1–1/4	1	1	0.5
Coarse Aggregate Quantity, lb/yd^3	1,725	1,634	1,810	1,810	1,856	1,811	1,900	1,856	1,773	1,724	1,700
Water, lb/yd^3	264	255	233	233	229	244	219	258	263	315	290
Air Entrainment, fl oz/yd^3	—	—	—	—	2.1	—	3.1	3.1	8.5	5	—
Water Reducer, fl oz/yd^3	41	22	—	—	—	—	—	—	66	—	6
Retarder, fl oz/yd^3	—	—	—	—	45	22	28	26	—	21	—
High-Range Water Reducer, fl oz/yd^3	—	—	—	—	—	122	156	—	13–20	—	—

1 in = 25.4 mm
1 fl oz/yd^3 = 38.69 ml/m^3 (38.69 milliliters per cubic meter)
1 lb/yd^3 = 0.593 kg/m^3 (0.593 kilograms per cubic meter)

[1] Based on approved concrete mix proportions.
[2] Also includes 4.0 gal/yd^3 (4.951 l/m^3) of corrosion inhibitor.
[3] Ohio bridge did not have a separate concrete deck. Values are for the abutments.

Table 9. Measured structural concrete properties for prestressed concrete girders.[1]

STATE	AL	CO	GA	LA	NE	NH	NH	NM	NC	OH
BRIDGE NAME	AL 199	Yale Ave.	S.R. 920	Charenton	120th St.	Route 104	Route 3A	Rio Puerco	U.S. 401	U.S. 22
Max. Concrete Temp., °F	—	158	—	147	132	—	—	—	162	—
Slump, inches	7.7	—	3.5–6	6.6	—	6.3	7	7.5	4–7	6
Air Content, %	4.4	—	3–5	—	—	6.6	3.5	7.2	3–5	6–7
Unit Weight, lb/ft^3	149.7	—	145.0	—	—	147.1	148.6	137.5	—	—
Curing Type	Ps	—	sc	sc, ps	s	s	s	—	—	s
Compressive Strength										
Age at Release, h	19–45	—	24	21–40	—	14–17	—	72	27	18
Release, psi	8,040–9,810	5,600–10,900	10,464	8,130	8,471	6,700	6,800	7,325	9,400	7,810
7 days, psi	—	—	10,360	9,250	10,529	8,000	10,240	—	—	9,060
28 days, psi	9,920	—	12,480	11,018	12,554	7,755	11,200	9,076	13,100	11,290
56 days, psi	9,950	7800–14,000	13,300	11,238	13,944	9,000	11,850	10,151	—	11,810

1 in = 25.4 millimeters (mm)
1 psi = 6.89 MPa
°C = (°F−32)*5/9

Table 9. Measured structural concrete properties for prestressed concrete girders[1]—*Continued*

STATE	SD	SD	TN	TN	TX (Louetta)[2]		TX (San Angelo)[2]		VA	VA	WA
BRIDGE NAME	I-29 NB	I-29 SB	Porter	Hickman	NB	SB	EB	WB	Route 40	VA Ave.	S.R. 18
Max. Concrete Temp., °F	140	140	—	—	—	—	158	155	166	161	—
Slump, inches	8	6.9	—	—	3.8–8.8	3.8–8.5	3.5–9	7–8	6.8	6.6	2.8–6
Air Content, %	4	6.7	—	—	2	2	2	2	5.9	4.4	1
Unit Weight, lb/ft^3	—	—	—	—	154	154	153	149	—	—	159
Curing Type	rh	rh	S	s	Sc	sc	sc	sc	s, m	s	s
Compressive Strength											
Age at Release, h	—	—	24–72	24–72	17–21	19–21	17–46	26	18 s	18	18–60
Release, psi	—	—	8635	8719	8460	8740	9940	8560	7340 s	8840	8150
7 days, psi	11,970	10,400	9484	9903	—	—	—	—	—	—	—
28 days, psi	15,900	13,250	9651	10,529	13,460	13,610	14,510	10,130	9060 s	11,200	11,370
56 days, psi	15,590	—	10,090	10,651	—	—	—	—	9875 s	—	12,220

1 in = 25.4 millimeters (mm)
1 psi = 6.89 MPa
°C = (°F−32)*5/9

[1] Based on production concrete.
[2] For the Texas bridges, different concrete strengths were specified for different girder span lengths. Listed strengths are average values except for San Angelo WB, where the largest values are used.

m = moist curing, rh = radiant heat curing, ps = partial steam curing, s = steam curing, sc = self-curing (concrete insulated without additional heat).

Table 10. Measured structural concrete properties for cast-in-place concrete decks.[1]

STATE	AL	CO	GA	LA	NE	NH	NH	NM	NC	OH[2]
BRIDGE NAME	AL 199	Yale Ave.	S.R. 920	Charenton	120th St.	Route 104	Route 3A	Rio Puerco	U.S. 401	U.S. 22
Slump, inches	5.8	3.8	4–7	—	—	3–5	5.25	2.5–8.5	4–5	4.5
Air Content, %	4.7	5.5	3.2–6.5	—	—	4.0–5.8	6.0	4.5–8.2	5.7–6.8	6.5
Unit Weight, psi	—	143.4	149.2	—	—	144–147	147.4	134–150	—	141
Curing Type	Wet	Wet	Wet	Wet	Wet	Wet	Wet	Wet	Wet	Wet
Curing Duration, days	7	5	7	7	—	5.7	7	14	7	7
Compressive Strength										
7 days, psi	6010	4390	5040	3149	7252	6890	7100	5116	—	—
28 days, psi	7370	5310	6217	5493	9606	9020	9004	6160	7150 NB	8689
56 days, psi	—	5950	7740	5785	10,433	—	9120	7501	—	—

1 in = 25.4 millimeters (mm)
1 psi = 6.89 MPa

Table 10. Measured structural concrete properties for cast-in-place concrete decks[1]—*Continued*

STATE	SD	SD	TN	TN	TX (Louetta)		TX (San Angelo)		VA	VA	WA
BRIDGE NAME	I-29 NB	I-29 SB	Porter	Hickman	NB	SB	EB	WB	Route 40	VA Ave.	S.R. 18
Slump, inches	—	2.5	—	—	4	7	7.4	—	5.7	3.6	3.3
Air Content, %	—	6.7	—	—	3.8	0	6.3	4.7	7.0	5.8	5.6
Unit Weight, psi	—	143	—	—	143	150	—	145	—	—	—
Curing Type	Wet	Wet	Wet	Wet	Wet	Wet	Wet	Wet	Wet	Wet	Wet
Curing Duration, days	7	7	7	7	10	10	10	10	7	7	14
Compressive Strength											
7 days, psi	5120	4920	4945	4287	—	—	6054	—	5388	4260	—
28 days, psi	7070	6170	8265	6460	5700	9100	7345	6120	6600	5400	5490
56 days, psi	—	—	8713	7197	5700	9740	—	—	—	6710	—

1 in = 25.4 mm
1 psi = 6.89 MPa

[1] Based on production concrete.
[2] Ohio bridge did not have a separate concrete deck. Values are for the abutments.

Table 11. Measured durability properties for prestressed concrete girders and cast-in-place concrete decks.[1]

STATE	AL	CO	GA	LA	NE	NH	NH	NM	NC	OH
BRIDGE NAME	AL 199	Yale Ave.	S.R. 920	Charenton	120th St.	Route 104	Route 3A	Rio Puerco	U.S. 401	U.S. 22
Prestressed Concrete Girders										
Air Content, %	4.4	—	3–5	—	—	6.6	3.5	—	3–5	6–7
Chloride Permeability, C	2,720	—	198	1,355	—	1,590	390	—	1,257–3,055	213
Age, days	56	—	56	56	—	56	56	—	90	320
Freeze-Thaw Resistance, %	98	—	—	—	—	107[2]	—	—	—	—
Scaling Resistance	—	—	—	—	—	—	—	—	—	—
Abrasion Resistance, inches	—	—	—	—	—	—	—	—	—	—
Cast-in-Place Concrete Decks										
Air Content, %	4.7	5.5	3.2–6.5	—	—	4.0–5.8	6.0	4.5–8.2	5.7–6.8	—
Chloride Permeability, C	2870	5597	3963	1390	589	753	1060	—	—	—
Age, days	56	—	56	56	—	56	56	—	—	—
Freeze-Thaw Resistance, %	92	—	—	—	—	97[2]	—	—	—	—
Scaling Resistance,	—	—	—	—	—	0–1	—	—	—	—
Abrasion Resistance, inches	—	0.025	—	—	—	—	—	—	—	—

1 in = 25.4 mm

Table 11. Measured durability properties for prestressed concrete girders and cast-in-place concrete decks[1]—*Continued*

STATE	SD	SD	TN	TN	TX (Louetta)		TX (San Angelo)		VA	VA	WA
BRIDGE NAME	I-29 NB	I-29 SB	Porter	Hickman	NB	SB	EB	WB	Route 40	VA Ave.	S.R. 18
Prestressed Concrete Girders											
Air Content, %	4	6.7	—	—	2	2	2	2	5.9	4.4	1
Chloride Permeability, C	65	88	390	496	≤ 1,000	≤ 1,000	≤ 1,000	≤ 1,000	228	125	496
Age, days	90	—	56	56	56	56	56	56	28[3]	28[3]	> 300
Freeze-Thaw Resistance, %	—	—	—	—	—	—	—	—	—	26, 91[2]	—
Scaling Resistance	—	—	—	—	—	—	—	—	—	0–1	—
Abrasion Resistance, inches	—	—	—	—	—	—	—	—	—	—	—
Cast-in-Place Concrete Decks											
Air Content, %	—	6.8	—	—	3.8	0	6.3	4.7	7.0	5.8	5.6
Chloride Permeability, C	461	1058	1297	317[3]	1730[4]	900[4]	703[4]	2573[4]	778	1457	2645
Age, days	90	—	56	28	56	56	56	56	28[3]	28[3]	> 210
Freeze-Thaw Resistance, %	—	—	—	—	—	—	97.9	97.3	—	108[2]	—
Scaling Resistance	—	—	—	—	—	—	2–3	0	—	0–1	—
Abrasion Resistance, inches	—	—	—	—	—	—	0.04	0.07	—	—	[5]

1 in = 25.4 millimeters (mm)

[1] Based on production concrete.
[2] ASTM C 666 procedure A or AASHTO T 161 procedure A.
[3] Includes 21 days at 100 °F.
[4] ASTM standard cure.
[5] Reported as a weight loss of 3.33 grams (gm).

CHAPTER 3. COMPILATION AND REVIEW OF AASHTO SPECIFICATIONS AND PROPOSED REVISIONS

INTRODUCTION

This chapter contains the results of the task B review of the AASHTO *Standard Specifications for Transportation Materials and Methods of Sampling and Testing,* the AASHTO *Standard Specifications for Highway Bridges*, the *AASHTO LRFD Bridge Design Specifications*, and the *AASHTO LRFD Bridge Construction Specifications* to identify provisions that directly impact the use of HPC.

For each provision that is listed, an action item is included to indicate that a proposed revision is included, a research problem statement is proposed, or no action was indicated. Details of the proposed revisions are included in appendixes A through E. Research problem statements are included in appendix F. No action was taken on some provisions because the change would not have a significant impact on the use of HPC.

Methodology

The compilation of the information from the AASHTO Specifications was accomplished by identifying provisions that impact the use of HPC. For those sections of the specifications that relate to structural design, the biggest impact comes from the use of high-strength concrete (HSC). For those sections that relate to materials, the impact is from the use of HPC as a durable concrete, HSC, or a combination of both. Consequently, when a provision impacts the use of high-strength concrete only, the abbreviation HSC is used. When a provision impacts a broader range of performance, the abbreviation HPC is used.

Units

The AASHTO *Standard Specifications for Transportation Materials and Methods of Sampling and Testing* contains some specifications and tests written with metric units as the primary measurement and others with English units as the primary measurement. The AASHTO *Standard Specifications for Highway Bridges* uses only English units. The *AASHTO LRFD Bridge Design Specifications* has two separate versions. One uses only English units. The other uses only metric units. The *AASHTO LRFD Bridge Construction Specifications* uses metric units.

In this compilation, any quotations from a document use the units as they appear in the original document. Subsequent discussion uses metric units first, followed by English units in parentheses.

AASHTO STANDARD SPECIFICATIONS FOR TRANSPORTATION MATERIALS AND METHODS OF SAMPLING AND TESTING, PART I: SPECIFICATIONS

The compilation in this section is based on the AASHTO *Standard Specifications for Transportation Materials and Methods of Sampling and Testing,* 21st edition, 2001, Part I: Specifications.(1) All specifications listed in the table of contents under the headings of Aggregates; Concrete, Curing Materials, and Admixtures; and Hydraulic Cement were reviewed.

For each specification, the title and scope of the specification are shown in italics followed by specific comments about potential changes for use with HPC. References in the comments to specific sections or tables refer to the document being reviewed and not the sections or tables in this report. If no change appears to be needed, this is stated. The end result of the project is stated under the action item. Proposed revisions are included in appendix A. Research problem statements are included in appendix F.

Aggregates

M 6 FINE AGGREGATE FOR PORTLAND CEMENT CONCRETE

This specification covers the quality and grading of fine aggregate for portland cement concrete used in pavements or bases, highway bridges, and incidental structures.

Section 5.3 has limits on percent passing any sieve and retained on the next consecutive sieve as well as limits for the fineness modulus of the fine aggregate. For HPC, the grading and fineness of all combined materials are important. A preferred approach for HPC is to provide limits on the grading of fine and coarse aggregate combined. In prescriptive specifications, narrower limits on the aggregate size retained on each sieve would be appropriate for HPC. However, the goal is to move toward performance-based specifications where the emphasis is on desired concrete properties.

Section 5.4 provides an exception for aggregate failing to meet the sieve analysis and fineness modulus based on the track record of concrete made using the aggregate. For HPC, the track record is unlikely to exist. Alternative wording to allow exceptions for HPC needs to be considered.

Supplementary requirement S1.1 has an upper limit of 0.60 percent for alkalies in cement when reactive aggregate is a concern. The limit of 0.60 percent has not provided satisfactory performance in all cases. A lower limit such as 0.40 percent, a limit on total alkalies in the concrete, or a limit on expansion using AASHTO T 303 could be used for HPC and should be evaluated.

ACTION: A new specification for combined aggregates is proposed. Revisions to the title and requirements for reactive aggregates are proposed.

M 43 SIZES OF AGGREGATES FOR ROAD AND BRIDGE CONSTRUCTION (ASTM DESIGNATION: D 448)

This specification defines aggregate size designations and ranges in mechanical analyses for standard sizes of coarse aggregate and screenings for use in the construction and maintenance of various types of highways and bridges.

The percentages for each sieve size given in table 1 have a wide range. In prescriptive specifications, a narrower limit on the aggregate retained on each sieve size would be appropriate for HPC. Improved gradings enable the use of less water, which is very important for both HSC and durable concrete. For HPC, the grading and fineness of all combined materials are important. A combined aggregate grading that includes both the fine and coarse aggregates is preferred for

HPC and should be considered. However, the goal is to move toward performance-based specifications where the emphasis is on the desired concrete properties.

ACTION: A new specification for combined aggregates is proposed.

M 80 COARSE AGGREGATE FOR PORTLAND CEMENT CONCRETE

This specification covers coarse aggregate, other than lightweight aggregate, for use in concrete. Several classes and gradings of coarse aggregate are described.

Section 5 refers to specification M 43. The percentages for each sieve given in table 1 of M 43 have a wide range. In prescriptive specifications, narrower limits on the aggregate size retained on each sieve size would be appropriate for HPC. However, the goal is to move toward performance-based specifications where the emphasis is on the desired concrete properties.

Section 6.2 has an upper limit of 0.60 percent for alkalies in cement when reactive aggregate is a concern. The limit of 0.60 percent has not provided satisfactory performance in all cases. A lower limit such as 0.40 percent, a limit on total alkalies in the concrete, or a limit on expansion using AASHTO T 303 could be used and should be evaluated.

The appendix describes methods for evaluating potential reactivity of an aggregate. This appendix needs updating to incorporate more recent information. Reference to AASHTO T 303 and ASTM C 1293 would provide current information.

ACTION: A new specification for combined aggregates is proposed. Revisions to the title and requirements for reactive aggregates are proposed.

M 195 LIGHTWEIGHT AGGREGATES FOR STRUCTURAL CONCRETE (ASTM DESIGNATION: C 330)

This specification covers lightweight aggregates intended for use in structural concrete in which the prime consideration is reducing the density while maintaining the compressive strength of the concrete. Procedures covered in this specification are not intended for job control of concrete.

Table 1 provides grading requirements for lightweight aggregates. The percentages have a wide range. In prescriptive specifications, narrower limits on the aggregate size retained on each sieve size would be appropriate for HPC. Also, a combined grading that includes both the fine and coarse aggregates is preferred for HPC and should be considered. However, the goal is to move toward performance-based specifications where the emphasis is on the desired concrete properties.

ACTION: Revisions to include references to the new combined aggregate specifications are proposed.

Concrete, Curing Materials, and Admixtures

M 148 LIQUID MEMBRANE-FORMING COMPOUNDS FOR CURING CONCRETE (ASTM DESIGNATION: C 309)

This specification covers liquid membrane-forming compounds suitable for application to concrete surfaces to reduce the loss of water during the early-hardening period. White-pigmented, membrane-forming compounds serve the additional purpose of reducing the temperature rise in concrete exposed to radiation from the sun. The membrane-forming compounds covered by this specification are suitable for use as curing media for fresh concrete, and may also be used for further curing of concrete after removal of forms or after initial moist curing.

Many States use water curing of bridge decks for improved performance. However, some States still use curing compounds. Section 6 restricts loss of water to 0.55 kilograms per square meter (kg/m^2) (0.11 lb/ft^2) in 72 h. Loss of water during hydration is critical for any concrete, especially for HPC. The loss of water limits the hydration reaction and causes volumetric changes that can result in cracking. Therefore, a stricter requirement for loss of water is essential for HPC. For example: the Virginia Department of Transportation (VDOT) specifications require no more than 0.116 kg/m^2 (0.024 lb/ft^2) moisture loss at 24 h and no more than 0.232 kg/m^2 (0.048 lb/ft^2) at 72 h when tested using Virginia's test methods. However, there are some differences in the procedures that would need to be addressed. Even with a stricter requirement, the use of curing compounds may not be appropriate for HPC with a very low water-cementitious materials ratio. The adequacy of curing compounds for HPC with different water-cementitious materials ratios needs to be evaluated.

Temperature management is important for hydration and volume changes in HPC. Section 7 limits reflectance to not less than 60 percent. This may need to be higher for HPC. For different classifications of HPC, loss of moisture and reflectance limits may need to be different.

ACTION: A research problem statement is proposed to address the effectiveness of curing compounds.

M 154 AIR-ENTRAINING ADMIXTURES FOR CONCRETE (ASTM DESIGNATION: C 260)

This specification covers materials proposed for use as air-entraining admixtures to be added to concrete mixtures in the field.

Section 6.1 lists requirements for bleeding, time of setting, compressive strength, flexural strength, shrinkage, and relative durability. In prescriptive specifications, narrower limits on the properties of test concretes containing the admixture would be appropriate for HPC. However, the goal is to move toward performance-based specifications where the emphasis is on the desired concrete properties.

In section 6.1.1, the dynamic modulus of elasticity at the end of the test calculated as a percentage of the dynamic modulus of elasticity at zero cycles is required to be 60 or greater. The value 60 should be increased to a higher value for HPC. In addition, a relative durability factor of 80 is allowed for an admixture under test compared to the referenced admixture. A higher value should be considered for HPC to ensure satisfactory durability. The selected value will depend on the variability of the test procedure.

Section 6.1.2 lists requirements for length change on the drying of concrete for 14 days. Some HPC appears to dry more slowly than conventional concrete. The provisions of this specification need to be reviewed for use with HPC.

Section 8.1 references ASTM designation C 233 for test methods and recommends that tests be made using the cement proposed for the specific work. Most HPC contains pozzolanic material. Consequently, cementitious material containing the proposed pozzolans should be used in the test.

ACTION: Revisions to section 8.1 and the addition of an optional test age of 56 days are proposed. A research problem statement is proposed to address performance requirements.

M 157 READY-MIXED CONCRETE

This specification covers ready-mixed concrete manufactured and delivered to a purchaser in a freshly mixed and unhardened state as hereinafter specified. Requirements for the quality of the concrete shall be either as hereinafter specified or as specified by the purchaser. In any case, where the requirements of the purchaser differ from those in this specification, the purchaser's specification shall govern. This specification does not cover the placement, consolidation, curing, or protection of the concrete after delivery to the purchaser.

In section 3.1, the definition of ready-mixed concrete needs to include all hydraulic cements.

Section 4.1.1 on cement needs to include cement conforming to ASTM C 1157.

Section 4.1.2 mentions only fly ash. It needs to include other pozzolans and ground granulated blast-furnace slag.

Section 4.1.4.1 allows the use of wash water when approved by the purchaser. Since wash water can vary greatly, its use in HPC needs to be assessed carefully.

Section 5.1 allows the engineer to specify the mix design. Mixture proportions for HPC should only be specified by those familiar with the material. However, the option of allowing the engineer to specify the mix design may not be appropriate for HPC.

Sections 5.1.1 through 5.1.6 and 5.2 are based on prescriptive concrete. For HPC, performance criteria need to be specified, followed by trial batches and then approval by the purchaser. Prescriptive specifications do not allow for the innovation required for HPC.

Section 6 discusses tolerances in slump. The entire issue of specifying slump for HPC needs to be reviewed.

In section 7.2, the point of discharge needs to be defined: ready-mix truck or discharge end of pump line. Concrete should be tested at the point nearest to its final location. Where it is not practical to test at this point, correlations between the properties measured at the end point and the nearest practical location should be established.

In table 5, slumps higher than 152 millimeters (mm) (6 inches) are not mentioned. Slumps higher than 152 mm (6 inches) are common for HPC and need to be included. A difference of 7.5 percent is allowed for variation in compressive strength for two samples from the same truck. This variance may be too large for HPC.

Section 11.3.1 gives the acceptable mixing time as 1 minute for capacities of 0.76 m^3 (1 yd^3) or less and, for greater capacities, the minimum is increased by 15 seconds (s) for each additional cubic yard. HPC requires thorough mixing. The speed of the mixers and the number of revolutions should be revisited.

Section 11.5 gives the minimum and maximum revolutions as 70 and 100, respectively, for truck-mixed concrete. These may not be sufficient for HPC with a stiff consistency.

Section 15 requires slump and air content to be measured. Since M 157 applies to concrete made with lightweight aggregate, unit weight should be a required test and should be included in this section.

ACTION: Revisions to sections 2.1, 3.1, 4.1.1, 4.1.2, 4.1.3, 5.1, 5.1.1, 5.1.3, 7.2, 11.3.1, and 15.2 and the addition of a new section 5.3 are proposed. A research problem statement is proposed to address the use of wash water.

M 171 SHEET MATERIALS FOR CURING CONCRETE (ASTM DESIGNATION: C 171)

This specification covers materials in sheet form used for covering the surfaces of hydraulic cement concrete to inhibit moisture loss during the curing period and, in the case of white reflective-type materials, to also reduce temperature rise in concrete exposed to radiation from the sun.

Section 6 lists performance requirements. The moisture loss is limited to a maximum of 0.55 kg/m^2 (0.11 lb/ft^2) in 72 h when tested according to ASTM C 156. The daylight reflectance of white curing paper is limited to a minimum of 50 percent when measured by ASTM E 1347. These limits need to be assessed for use with HPC.

ACTION: A research problem statement is proposed to assess the limits.

M 182 BURLAP CLOTH MADE FROM JUTE OR KENAF

This specification covers requirements for burlap made from jute or kenaf for use in curing concrete.

Either this specification should include cotton mats that are used in several States or another specification needs to be developed.

ACTION: Revisions to include cotton mats are proposed.

M 194 CHEMICAL ADMIXTURES FOR CONCRETE (ASTM DESIGNATION: C 494)

This specification covers materials for use as chemical admixtures to be added to portland cement concrete mixtures in the field for the purpose or purposes indicated for the seven types.

The scope should be extended to hydraulic cement concrete rather than portland cement concrete.

Note 1 refers to ASTM C 1017, "Specification for Chemical Admixtures for Use in Producing Flowing Concrete." AASHTO should consider adopting ASTM C 1017 or a similar specification.

Table 1 lists physical requirements, including compressive and flexural strengths. The test ages should include 56 days, since this test age is frequently used for HPC.

For air-entrained concrete that may be exposed to freezing and thawing while wet, table 1 states that the relative durability factor shall be at least 80. A higher value should be considered for HPC to ensure satisfactory durability. For shrinkage, a length change of concrete containing an admixture is allowed to be 135 percent of a control concrete without the admixture. The appropriateness of this number needs to be assessed for HPC.

ACTION: Revisions to include hydraulic cement concrete and a test age of 56 days are proposed. A research problem statement is proposed to address the durability factor.

M 205 MOLDS FOR FORMING CONCRETE TEST CYLINDERS VERTICALLY (ASTM DESIGNATION: C 470)

This specification covers molds for use in forming cylindrical concrete specimens. The provisions of this specification include the requirements for both reusable and single-use molds.

Section 5.1 includes the use of paper as a mold material. Several test programs have shown that the dimensional stability of molds is an important factor that can influence compressive strength test results for HSC. The suitability of paper products for cylindrical molds needs to be assessed. ACI Committee 363 reports that even high-quality cardboard molds produced concrete strengths 13 percent lower than when steel molds are used.(15) The committee has also recommended that plastic molds with a wall thickness of less than 6 mm (0.25 inch) should have a cap to maintain a circular shape and that the cap should be domed to provide clearance to the concrete surface.(16) This specification needs to be revised for the above items.

ACTION: A revision to section 5.1 to eliminate paper products and a requirement for top caps are proposed.

M 210 USE OF APPARATUS FOR THE DETERMINATION OF LENGTH CHANGE OF HARDENED CEMENT PASTE, MORTAR, AND CONCRETE (ASTM DESIGNATION: C 490)

This practice covers the requirements for the apparatus and equipment used to prepare specimens for the determination of length change in hardened cement paste, mortar, and concrete; the apparatus and equipment used for the determination of these length changes; and the procedures for its use.

This practice does not require any modification for use with HPC.

ACTION: None.

M 224 USE OF PROTECTIVE SEALERS FOR PORTLAND CEMENT CONCRETE

This guide includes the selection factors for and the use of protective sealers for highway purposes to be applied to hardened concrete for the purpose of protecting new concrete or prolonging the life of sound, in-service concrete. Information in this guide is not applicable to the repair of badly deteriorated concrete.

This guide does not require any modification for use with HPC. However, penetrating sealers may not penetrate as much with HPC because of the lower permeability.

ACTION: None.

M 233 BOILED LINSEED OIL MIXTURE FOR TREATMENT OF PORTLAND CEMENT CONCRETE

This specification covers the boiled linseed oil-petroleum spirits mixture to be applied to hardened portland cement concrete as a protection against damage by deicing chemicals.

This specification does not require any modification for use with HPC.

ACTION: None.

M 241 CONCRETE MADE BY VOLUMETRIC BATCHING AND CONTINUOUS MIXING (ASTM DESIGNATION: C 685)

This specification covers concrete made from materials continuously batched by volume, mixed in a continuous mixer, and delivered to the purchaser in a freshly mixed and unhardened state. Tests and criteria for batching accuracy and mixing efficiency are specified herein.

Cement conforming to ASTM C 1157 needs to be added to section 5.1.1.

Optional chemical limits for wash water need to be tightened for HPC applications.

Silica fume needs to be included in section 5, Materials.

In section 6.1.2, point of delivery needs to be defined: ready-mix truck or discharge end of pump line. Concrete should be tested at the point nearest to its final location. Where it is not practical to test at this point, correlations of properties measured at the end point and the nearest practical location should be established.

Section 6.3 defines ordering information when the purchaser assumes responsibility for the proportioning of the concrete mixture. This option may not be appropriate for HPC.

Notes 7 and 8 need to include reference to ACI 211.4 for HSC with fly ash.[17]

Section 10.1 should require that the fresh unit weight of concrete be measured when lightweight concrete is used.

Section 10.3 lists tolerances in slump. These should be reviewed for appropriateness with HPC.

The overdesign requirements in table 4 need to be reviewed for use with HSC. ACI 318 has revised its overdesign requirements.[18]

Section 11.3 specifies two standard-size cylinders for each strength test. Since M 241 refers to ASTM C 31 for the procedure to be used, standard-size cylinder may be interpreted to mean 152 by 305 mm (6 by 12 inches). ACI 363 recommends three cylinders for HSC and allows the use of 102- by 203-mm (4- by 8-inch) cylinders.[16] The effect of a different number of specimens and specimen sizes need to be evaluated.

ACTION: Revisions to sections 2, 5.1.1, 5.1.6, 6.1.3, 6.1.5, 11.2, 11.3, and 11.5.2; notes 7 and 8; and tables 5 and 6 are proposed. A research problem statement is proposed to address the limits for wash water.

M 295 COAL FLY ASH AND RAW OR CALCINED NATURAL POZZOLAN FOR USE AS A MINERAL ADMIXTURE IN CONCRETE (ASTM DESIGNATION: C 618)

This specification covers coal fly ash and raw or calcined natural pozzolan for use as a mineral admixture in concrete where cementitious or pozzolanic action, or both, is desired, or where other properties normally attributed to finely divided mineral admixtures may be desired or where both objectives are to be achieved.

In table 3, the strength activity index should include a test age of 56 days because it is more appropriate for HSC.

Table 4 on optional physical requirements should include tests for sulfate resistance. Tests have shown that concretes with some fly ashes will deteriorate when exposed to sulfates in soils or water.[19]

Table 4 allows an increase of 0.03 percent in drying shrinkage of mortar bars when fly ash is used. This may not be appropriate for HPC.

ACTION: Revisions to eliminate table 2, Supplementary Optional Chemical Requirements, and to add 56 days in table 3 are proposed. Research problem statements are proposed to address drying shrinkage limits and sulfate resistance.

M 302 GROUND GRANULATED BLAST-FURNACE SLAG FOR USE IN CONCRETE AND MORTARS (ASTM DESIGNATION: C 989)

This specification covers three strength grades of finely ground granulated blast-furnace slag for use as a cementitious material in concrete and mortar.

In table 1, the slag activity index should include a test age of 56 days because it is more appropriate for HSC.

In appendix A3, Effectiveness of Slag in Preventing Excessive Expansion of Concrete Due to Alkali-Aggregate Reaction, a high-alkali content is assumed to be 0.60 percent. A lower limit such as 0.40 percent, a limit on total alkalies in concrete, or a limit on expansion using AASHTO T 303 would be preferred for HPC.

This specification does not include a requirement on shrinkage similar to that in M 295 for fly ash and pozzolans and M 307 for silica fume.

ACTION: Revisions to table 1, section A3.2, and appendix A4 are proposed. A research problem statement is proposed to address the shrinkage requirement.

Hydraulic Cement

M 85 PORTLAND CEMENT (ASTM DESIGNATION: C 150)

This specification covers eight types of portland cement.

In table 2, Optimal Chemical Requirement, alkali content should be lower than 0.60 percent for HPC. A lower limit such as 0.40 percent, a limit on total alkalies in concrete, or a limit on expansion using AASHTO T 303 would be preferred.

In tables 3 and 4 on physical requirements, an age of 56 days should be included for compressive strength tests because 56 days is more appropriate for HSC.

ACTION: A footnote to table 2 about the alkali content is proposed.

M 240 BLENDED HYDRAULIC CEMENT (ASTM DESIGNATION: C 595)

This specification pertains to five classes of blended hydraulic cements for both general use and special applications, using slag or pozzolan, or both, with portland cement or portland cement clinker or slag with lime.

In section 10.1.12, mortar expansion using ASTM C 227 is mentioned. ASTM C 227 covers the determination of the susceptibility of cement-aggregate combinations to expansive reactions involving hydroxyl ions associated with the alkalies. The test takes a long time, with measurements up to 12 months, and, if necessary, every 6 months thereafter. ASTM C 441 includes pozzolans or slag. It determines the effectiveness of mineral admixtures or slag in preventing excessive expansion of concrete because of alkali-silica reactivity. It is much faster because of the use of highly reactive Pyrex® glass for aggregate and should be included in M 240.

In table 2, Physical Requirements, an age of 56 days should be included for strength tests because 56 days is more appropriate for HSC.

In table 3, the slag and pozzolan activity index should include a test age of 56 days because 56 days is more appropriate for HSC.

ACTION: Revisions to tables 2 and 3 are proposed.

M 307 MICROSILICA FOR USE IN CONCRETE AND MORTAR

This specification covers microsilica for use as a mineral admixture in portland cement concrete and mortar to fill small voids and/or where pozzolanic action is desired. Microsilica is a product with a particle size typically two orders of magnitude smaller than portland cement. It is a material often marketed as an aqueous suspension with a typical 50 percent solids content. This specification details requirements and tests to be performed on the dry material before being processed into either the dry compacted form (densified powder) or a slurry.

This specification is listed under Hydraulic Cement Concrete in the table of contents. It should be listed under Concrete, Curing Materials, and Admixtures. The specification uses the term "microsilica." Silica fume is the more generic term. The provisions of M 307 should be compared with ASTM C 1240 for differences that may be appropriate for HPC.

Section 1.1 states that the material is often marketed as an aqueous suspension. For transportation structures, the description may not be appropriate since the dry form is often used.

In table 3, Physical Requirements, the strength activity index is required at 28 days. A test age of 56 days may be more appropriate for HSC and should be evaluated.

In table 4, Optional Physical Requirements, the reduction of mortar expansion with cement alkalies is limited to 80 percent. For HPC, it should be increased from 80 to 100 percent.

Section 8.2.4 refers to ASTM C 311. ASTM C 311 requires that water be added to the test mixture to achieve a flow comparable to that of the control mixture. Use of a constant water/cementitious materials ratio is being considered by ASTM and is more appropriate for concrete containing silica fume.

ACTION: Revisions to the title, sections 1.1, 3.2, 8.2.4, and tables 2, 3, and 4 are proposed. A research problem statement is proposed to address the reduction in mortar expansion.

AASHTO STANDARD SPECIFICATIONS FOR TRANSPORTATION MATERIALS AND METHODS OF SAMPLING AND TESTING, PART II: TESTS

The compilation in this section is based on the AASHTO *Standard Specifications for Transportation Materials and Methods of Sampling and Testing,* 21st edition, 2001, Part II: Tests.[1] All tests listed in the tables of contents of these document under the headings of Aggregates; Concrete, Curing Materials, and Admixtures; and Hydraulic Cement were reviewed. For each test, the title and scope of the test is shown in italics, followed by specific comments about potential changes for use with HPC. Reference in the comments to specific sections or tables refer to the document being reviewed and not the sections or tables in this report. If no changes appear to be needed, this is stated. The end result of the project is stated under the action item. Proposed revisions are included in appendix B. Research problem statements are included in appendix F.

Aggregates

With the exception of methods T 96 and T 304, none of the aggregate tests listed in this section require any modification for use with HPC.

ACTION: No action needed on any of the test methods for aggregates.

T 2 SAMPLING OF AGGREGATES

This practice covers sampling of coarse and fine aggregates.

T 11 MATERIALS FINER THAN 75-μM (NO. 200) SIEVE IN MINERAL AGGREGATES BY WASHING (ASTM DESIGNATION: C 117)

This test method covers determination of the amount of material finer than 75-μm (No. 200) sieve in aggregates by washing.

T 19 BULK DENSITY ("UNIT WEIGHT") AND VOIDS IN AGGREGATE (ASTM DESIGNATION: C 29)

This test method covers determination of the bulk density ("unit weight") of aggregate in a compacted or loose condition, and calculated voids in fine, coarse, or mixed aggregates based on the same determination. This test method is applicable to aggregates not exceeding 125 mm (5 inches) in nominal maximum size.

T 21 ORGANIC IMPURITIES IN FINE AGGREGATES FOR CONCRETE (ASTM DESIGNATION: C 40)

This test method covers the procedure for an approximate determination of the presence of injurious organic compounds in fine aggregates that are to be used in cement mortar or concrete.

T 27 SIEVE ANALYSIS OF FINE AND COARSE AGGREGATES (ASTM DESIGNATION: C 136)

This method covers the determination of the particle size distribution of fine and coarse aggregates by sieving.

This method provides separate analysis for fine and coarse aggregate. For HPC, a test that provides a combined grading for fine and coarse aggregate is needed.

T 30 MECHANICAL ANALYSIS OF EXTRACTED AGGREGATE

This method of test covers a procedure for the determination of the particle size distribution of fine and coarse aggregates extracted from bituminous mixtures, using sieves with square openings.

T 37 SIEVE ANALYSIS OF MINERAL FILLER FOR ROAD AND PAVING MATERIALS (ASTM DESIGNATION: D 546)

This method of test covers the sieve analysis of mineral fillers used in bituminous paving materials.

T 71 EFFECT OF ORGANIC IMPURITIES IN FINE AGGREGATE ON STRENGTH OF MORTAR (ASTM DESIGNATION: C 87)

This test method covers determination of the effect on mortar strength of the organic impurities in fine aggregate, whose presence is indicated by tests with T 21. Comparison is made between compressive strengths of mortar made with washed and unwashed fine aggregate.

T 84 SPECIFIC GRAVITY AND ABSORPTION OF FINE AGGREGATE (ASTM DESIGNATION: C 128)

This method covers determination of bulk and apparent specific gravity, 23/23 ^{o}C (73.4/73.4 ^{o}F), and absorption of fine aggregate.

T 85 SPECIFIC GRAVITY AND ABSORPTION OF COARSE AGGREGATE (ASTM DESIGNATION: C 127)

This method covers the determination of specific gravity and absorption of coarse aggregate. The specific gravity may be expressed as bulk specific gravity, bulk specific gravity (saturated-surface-dry (SSD)), or apparent specific gravity. The bulk specific gravity (SSD) and absorption are based on aggregate after 15 hours soaking in water. This method is not intended to be used with lightweight aggregates.

T 96 RESISTANCE TO DEGRADATION OF SMALL-SIZE COARSE AGGREGATE BY ABRASION AND IMPACT IN THE LOS ANGELES MACHINE (ASTM DESIGNATION: C 131)

This test method covers a procedure for testing sizes of coarse aggregate smaller than 37.5 mm (1½ inches) for resistance to degradation using the Los Angeles testing machine.

The applicability of this test to lightweight aggregate should be evaluated.

T 103 SOUNDNESS OF AGGREGATES BY FREEZING AND THAWING

This method describes three procedures to be followed in testing aggregates to determine their resistance to disintegration by freezing and thawing. It furnishes information helpful in judging the soundness of aggregates subjected to weathering, particularly when adequate information is not available from service records of the behavior of the aggregate.

T 104 SOUNDNESS OF AGGREGATE BY USE OF SODIUM SULFATE OR MAGNESIUM SULFATE

This method covers the procedures to be followed in testing aggregates to determine their resistance to disintegration by saturated solutions of sodium sulfate or magnesium sulfate.

T 112 CLAY LUMPS AND FRIABLE PARTICLES IN AGGREGATE (ASTM DESIGNATION: C 142)

This method covers the approximate determination of clay lumps and friable particles in natural aggregates.

T 113 LIGHTWEIGHT PIECES IN AGGREGATE (ASTM DESIGNATION: C 123)

This method covers the determination of the percentage of lightweight pieces in aggregate by means of sink-float separation in a heavy liquid of suitable specific gravity.

T 210 AGGREGATE DURABILITY INDEX (ASTM DESIGNATION: D 3744)

This method describes the procedure for determining the durability of aggregates. The durability index is a value indicating the relative resistance of an aggregate to produce detrimental claylike fines when subjected to the prescribed mechanical methods of degradation.

T 248 REDUCING SAMPLES OF AGGREGATE TO TESTING SIZE (ASTM DESIGNATION: C 702)

These methods cover the reduction of large samples of aggregate to the appropriate size for testing, employing techniques that are intended to minimize variations in measured characteristics between the test samples so selected and the large sample.

T 255 TOTAL EVAPORABLE MOISTURE CONTENT OF AGGREGATE BY DRYING (ASTM DESIGNATION: C 566)

This method covers determination of the percentage of evaporable moisture in a sample of aggregate by drying both surface moisture and moisture in the pores of the aggregate.

T 279 ACCELERATED POLISHING OF AGGREGATES USING THE BRITISH WHEEL (ASTM DESIGNATION: D 3319)

This method covers a laboratory procedure by which an estimate may be made of the extent to which different coarse aggregates may polish.

T 304 UNCOMPACTED VOID CONTENT OF FINE AGGREGATE

This method describes the determination of the loose uncompacted void content of a sample of fine aggregate.

This test indirectly addresses aggregate shape and texture. However, a more direct test measuring particle shape would be beneficial (e.g., the videograder).[20]

Concrete, Curing Materials, and Admixtures

T 22 COMPRESSIVE STRENGTH OF CYLINDRICAL CONCRETE SPECIMENS (ASTM DESIGNATION: C 39)

This method covers determination of compressive strength of cylindrical concrete specimens such as molded cylinders and drilled cores. It is limited to concrete having a unit weight in excess of 800 kg/m^3 (50 lb/ft^3).

Section 7.3 lists permissible time tolerances for different ages, including 28 and 90 days. A tolerance for 56 days needs to be added since 56 days is frequently used with HSC. Section 7.5 provides a rate of loading of 20 to 50 psi/s (0.14 to 0.34 MPa/s). In HSC, such a rate may take a long time. Loading rate should be evaluated for use with HSC.

A precision statement does not exist. When available, it should include HSC and compare 102- by 203-mm (4- by 8-inch) cylinders with the 152- by 305-mm (6- by 12-inch) cylinders. Comparisons of averages for different sizes and variability for each size are needed. The appendix to this test method describes the procedure for determining the compressive strength of cylindrical concrete specimens using neoprene caps. The scope is limited to the testing of 152- by 305-mm (6- by 12-inch) cylinders. Because of the limited capacity of the testing machines, 102- by 203-mm (4- by 8-inch) cylinders are frequently used for testing HSC. The appendix needs to be revised to include 102- by 203-mm (4- by 8-inch) cylinders.

In A 12.3.1, verification of cap systems is limited to 41.4 MPa (6000 psi). Higher strengths should be included. ASTM C 1231 permits the use of cap systems up to a cylinder compressive strength of 85 MPa (12,000 psi).

ACTION: Revisions to sections 6.2, 7.3, 7.5.1, 10.1, and the appendix are proposed. A research problem statement is proposed to address other issues.

T 23 MAKING AND CURING CONCRETE TEST SPECIMENS IN THE FIELD (ASTM DESIGNATION: C 31)

This method covers procedures for making and curing cylindrical and prismatic specimens using job concrete that can be consolidated by rodding or vibration as described herein.

Section 5.2 requires that beams made in the field shall not have a width or depth less than 152 mm (6 inches). The test method should consider using flexural strength beams that are smaller in cross section rather than 152 by 152 mm (6 by 6 inches) when the maximum size aggregate is 25 mm (1 inch) or less. It is more convenient to use smaller specimens. However, the effect of specimen size on measured flexural strength will need to be evaluated.

Section 8.3.1 requires consolidation either by rodding or vibration, depending on the slump value. This test should consider self-consolidating concretes and flowing concretes, which require less consolidation effort. HPC with high workability, as in self-consolidating concrete, has been successfully used in Japan, Canada, and Europe.

Section 9.2.1 requires that, after molding, the specimens shall be stored between 16 and 27 degrees Celsius (^{o}C) (60 and 80 ^{o}F). A stricter requirement for HPC is needed.

ACTION: Revisions to sections 8.3.1, 8.3.3.1, 9.2.1, and table 1 are proposed. A research problem statement is proposed to address the use of smaller specimens.

T 24 OBTAINING AND TESTING DRILLED CORES AND SAWED BEAMS OF CONCRETE (ASTM DESIGNATION: C 42)

This method covers obtaining, preparing, and testing: (1) cores drilled from concrete for length or compressive or splitting tensile strength determinations, and (2) beams sawed from concrete for flexural strength determinations.

Section 6.4 requires capping of drilled cores in accordance with AASHTO T 231, Capping Cylindrical Concrete Specimens. Test method T 231 specifies high-strength gypsum plaster or sulfur mortar. Test method T 24 should include testing with pads in steel extrusion controllers since it is a very convenient test procedure and is used with HSC. Testing with lapped ends should also be included.

In section 6.9, the precision statement is limited to compressive strengths less than 48.3 MPa (7000 psi). Higher strengths should be included.

ACTION: Revisions to sections 6.2.2, 6.4, and 6.7.2 are proposed.

T 26 QUALITY OF WATER TO BE USED IN CONCRETE

The acidity or alkalinity shall be determined by one of the following methods, A or B.

Section 3.4 mentions other AASHTO tests that may need changes to accommodate HPC. These are addressed in other sections of this compilation.

ACTION: None.

T 97 FLEXURAL STRENGTH OF CONCRETE (USING SIMPLE BEAM WITH THIRD-POINT LOADING) (ASTM DESIGNATION: C 78)

This method covers determination of the flexural strength of concrete by use of a simple beam with third-point loading.

Section 5.2 states that the load is to be applied continuously at a rate that constantly increases the extreme fiber stress within a range of 861 to 1207 kilopascals per minute (kPa/min) (125 to 175 psi/min). For HSC, the test may take too long and the possibility of a faster loading rate should be considered.

ACTION: None.

T 119 SLUMP OF HYDRAULIC CEMENT CONCRETE (ASTM DESIGNATION: C 143)

This method covers the determination of slump of concrete, both in the laboratory and in the field.

After lifting the slump cone, slump is determined by the vertical difference between the top of the mold and the displaced original center of the top surface of the specimen. HPC concretes can have very high slump values limited by the height of the maximum-size aggregate. Slump spread may be a better measure. However, slump or slump spread alone does not address workability. Measurement of rheological properties, including viscosity, filling capacity, or flow time, may provide better understanding of workability, pumpability, and placeability.(21)

ACTION: A new specification for slump flow is proposed.

T 121 MASS PER CUBIC METER (CUBIC FOOT), YIELD, AND AIR CONTENT (GRAVIMETRIC) OF CONCRETE (ASTM DESIGNATION: C 138)

This method covers determination of the mass per cubic meter (or cubic foot) of freshly mixed concrete and gives formulas for calculating the yield, cement content, and the air content of the concrete.

The samples are to be consolidated by either rodding or internal vibration. External vibration should also be allowed. Also, in the case of self-consolidating HPC, no vibration should be permitted.

ACTION: None.

T 126 MAKING AND CURING CONCRETE TEST SPECIMENS IN THE LABORATORY (ASTM DESIGNATION: C 192)

This method covers procedures for making and curing test specimens of concrete in the laboratory under accurate control of materials and test conditions using concrete that can be consolidated by rodding or vibration as described herein.

Section 7.4.1 requires consolidation either by rodding or vibration, depending on the slump value. This test should consider self-consolidating concretes and flowing concretes, which require less consolidation effort.

The precision statement presents the standard deviation for 7-day compressive strengths. A longer test age should be included for HPC. The precision statement is for laboratory trial batches that contain prescribed quantities of materials with a prescribed water-cementitious materials ratio. It is stated that values should be used with caution for air-entrained concrete and concrete with a slump of less than 50 mm (2 inches) or more than 150 mm (6 inches). The limitations are not suitable for HPC and need to be broadened.

ACTION: A revision to section 7.4.1 is proposed. A research problem statement to include the measurement of precision is proposed.

T 140 COMPRESSIVE STRENGTH OF CONCRETE USING PORTIONS OF BEAMS BROKEN IN FLEXURE (ASTM DESIGNATION: C 116)

This method covers the determination of compressive strength of concrete, using portions of beams broken in flexure for test specimens.

This method was withdrawn in February 1999 by ASTM because it was not updated by the end of the eighth year.

In section 6.3, the rate of loading is given as 0.05 inch/min. This rate may be too time-consuming for HSC.

ACTION: None.

T 141 SAMPLING FRESHLY MIXED CONCRETE (ASTM DESIGNATION: C 172)

This method covers the procedures for obtaining representative samples of fresh concrete as delivered to the project site and on which tests are to be performed to determine compliance with the quality requirements of the specifications under which the concrete is furnished.

This test method requires the sampling of two or more portions of the batch and combining them into a composite sample. The use of a single sample should be evaluated.

ACTION: None.

T 148 MEASURING LENGTH OF DRILLED CORES (ASTM DESIGNATION: C 174)

This test method covers determination of the length of a core drilled from a concrete pavement or structure.

This test method does not require any modification for use with HPC.

ACTION: None.

T 152 AIR CONTENT OF FRESHLY MIXED CONCRETE BY THE PRESSURE METHOD (ASTM DESIGNATION: C 231)

This method covers determination of the air content of freshly mixed concrete from observation of the change in volume of concrete with a change in pressure.

This method does not require any modification for use with HPC.

ACTION: None.

T 155 WATER RETENTION BY CONCRETE CURING MATERIALS (ASTM DESIGNATION: C 156)

This test method covers laboratory determination of the efficiency of liquid membrane-forming compounds and sheet materials for curing concrete, as measured by their ability to reduce moisture loss during the early hardening period.

This test method does not require any modification for use with HPC. However, section 17 states that "efforts to establish a more meaningful measure of the precision of this test method continue." This test is known to have high variability and improvements are needed.

ACTION: None.

T 157 AIR-ENTRAINING ADMIXTURES FOR CONCRETE (ASTM DESIGNATION: C 233)

This test method covers the testing of materials proposed for use as air-entraining admixtures in the field.

This test method may require modification for use with HPC. Currently, it does not indicate the behavior of air-entraining admixtures in the presence of other admixtures and when higher consistency mixtures are prepared. Section 4.4 needs to be revised to include the various constituent materials used in HPC.

ACTION: Revisions to sections 4.1, 4.4, 10.1.1, and 13.1.6 to include other materials and a test age of 56 days are proposed.

T 158 BLEEDING OF CONCRETE (ASTM DESIGNATION: C 232)

These test methods cover the determination of the relative quantity of mixing water that will bleed from a sample of freshly mixed concrete. Two test methods that differ primarily in the degree of vibration to which the concrete sample is subjected are included.

These test methods do not require any modification for use with HPC. There is a typographical error in section 5.1. Line 1 mentions T 120, which should be T 126.

ACTION: A revision to section 5.1 is proposed.

T 159 COMPARING CONCRETE ON THE BASIS OF THE BOND DEVELOPED WITH REINFORCING STEEL (ASTM DESIGNATION: C 234)

This test method covers comparison of concretes on the basis of the bond developed with reinforcing steel.

Note 4 recommends internal vibration with low-slump concrete. External vibration should also be permissible. For self-consolidating concretes, no vibration should be permitted.

ACTION: None.

T 160 LENGTH CHANGE OF HARDENED HYDRAULIC CEMENT MORTAR AND CONCRETE (ASTM DESIGNATION: C 157)

This test method covers determination of the length changes of hardened hydraulic cement mortar and concrete due to causes other than externally applied forces and temperature changes.

This test method does not require any modification for use with HPC.

ACTION: None.

T 161 RESISTANCE OF CONCRETE TO RAPID FREEZING AND THAWING (ASTM DESIGNATION: C 666)

This method covers the determination of the resistance of concrete specimens to rapidly repeated cycles of freezing and thawing in the laboratory by two different procedures.

Section 8.1 indicates that unless some other age is specified, specimens are cured in lime-saturated water for 14 days. For HPC, a longer curing period, or a drying period before testing, should be included unless the elements are in contact with water continuously after placement. Subjecting saturated specimens to rapid freezing and thawing after 14 days of moist curing is a very severe environment and may not correlate well with field experience. In many applications, longer curing times are specified for HPC.

Section 8.3 states continuation of the test for 300 cycles or until a relative dynamic modulus of elasticity (RDM) of 60 is reached. In HPC, more cycles and higher RDM are desirable.

ACTION: A revision to section 8.3 for HPC is proposed. A research problem statement to evaluate the test method is proposed.

T 177 FLEXURAL STRENGTH OF CONCRETE (USING SIMPLE BEAM WITH CENTER-POINT LOADING) (ASTM DESIGNATION: C 293)

This method covers the determination of the flexural strength of small-sized concrete specimens by the use of a simple beam with center-point loading.

Section 5.2 indicates that loads will be applied at a rate to increase the extreme fiber stress within a range of 0.86 to 1.21 MPa/min (125 to 175 psi/min). HSC may have high flexural strength and such a rate may take a long time. The possibility of using a faster loading rate should be considered.

ACTION: None.

T 178 CEMENT CONTENT OF HARDENED PORTLAND CEMENT CONCRETE (ASTM DESIGNATION: C 1084)

This method of test for determining the cement content of concrete is applicable to hardened portland cement concretes except those containing certain aggregates or combinations of aggregates or admixtures that yield significant amounts of dissolved calcium oxide (CaO) and dissolved silica (SiO_2) under conditions of the test.

This method only addresses portland cement concrete. Pozzolans and ground granulated blast-furnace slag are widely used in HPC. There is a need to know the amount of cementitious materials (including portland cement) used in hydraulic cement concrete. This test method may need to be modified or a new one developed.

ACTION: A research problem statement is proposed.

T 196 AIR CONTENT OF FRESHLY MIXED CONCRETE BY THE VOLUMETRIC METHOD (ASTM DESIGNATION: C 173)

This test method covers determination of the air content of freshly mixed concrete containing any type of aggregate, whether it be dense, cellular, or lightweight.

This method does not require any modification for use with HPC.

ACTION: None.

T 197 TIME OF SETTING OF CONCRETE MIXTURES BY PENETRATION RESISTANCE (ASTM DESIGNATION: C 1084)

This test method covers the determination of the time of setting of concrete, with slump greater than zero, by means of penetration resistance measurements on mortar sieved from the concrete mixture.

This test method does not require any modification for use with HPC.

ACTION: None.

T 198 SPLITTING TENSILE STRENGTH OF CYLINDRICAL CONCRETE SPECIMENS (ASTM DESIGNATION: C 496)

This method covers the determination of the splitting tensile strength of cylindrical concrete specimens such as molded cylinders and drilled cores.

Section 6.5 specifies a constant rate of loading that increases the splitting tensile stress from 689 to 1380 kPa/min (100 to 200 psi/min). HSC may have a high splitting tensile strength and such a rate may take a long time. The possibility of providing a faster loading rate should be considered.

ACTION: None.

T 199 AIR CONTENT OF FRESHLY MIXED CONCRETE BY THE CHACE INDICATOR

This method of test covers the determination of the air content of freshly mixed concrete by displacing the air with alcohol and observing the change in level of the liquid in a tube.

This method does not require any modification for use with HPC.

ACTION: None.

T 231 CAPPING CYLINDRICAL CONCRETE SPECIMENS (ASTM DESIGNATION: C 617)

This method covers apparatus, materials, and procedures for capping freshly molded concrete cylinders with neat cement and hardened cylinders and drilled concrete cores with high-strength gypsum plaster or sulfur mortar.

This test procedure specifies capping with neat cement for freshly molded specimens and high-strength gypsum plaster or sulfur mortar for hardened specimens. Note 6 states that type I neat cement caps require 6 days to develop acceptable strength, and section 5.2.2 states that sulfur mortar may be used if allowed 2 h to harden. Section 6.2.1 states that caps should be about 3 mm (0.125 inch) thick and in no instance shall any part of a cap be more than 8 mm (0.3125 inch) thick.

National Ready Mixed Concrete Association (NRMCA) Technical Memo 5 recommends that when sulfur mortar is used, cylinders with a compressive strength of 34 MPa (5000 psi) and higher strengths should be capped at least 1 day and preferably 7 days prior to testing.(22) Furthermore, it is stated that cylinder ends should be sawed to ensure that the thickness of the cap is 3 mm (0.125 inch) or less, and preferably 1.5 mm (0.0625 inch). NRMCA also recommends that when the strength of concrete exceeds 69 MPa (10,000 psi), neat cement paste be used to cap the ends. Cylinders should be capped 7 days prior to testing and the cap thickness should be less than 3 mm (0.125 inch). Thus, type and strength of capping material, thickness of caps, and the time to cap prior to testing need to be considered and appropriate changes made to T 231.

ACTION: Revisions to sections 5.1 through 5.2.2 to make AASHTO T 231 consistent with ASTM C 617 are proposed. A research problem statement is proposed to address other issues related to capping.

T 259 RESISTANCE OF CONCRETE TO CHLORIDE ION PENETRATION

This method covers the determination of the resistance of concrete specimens to the penetration of chloride ion.

Section 3.1 specifies abrading the surface if the surface is exposed to vehicular traffic. However, since in most applications where a comparison between different concretes is sought, the surface abrasion can be deleted.

Sections 3.4 and 3.5 specify ponding for 90 days. This time period is not long enough to discern differences between concretes, especially HPC. A longer ponding period should be included for HPC.[23] A specified ponding period of 90 days or an even shorter period may be used if thinner sections than 13 mm (0.5 inch) are prepared for the chloride profile. For example, *Nordtest NT Build 443* requires an exposure of at least 35 days and 1- to 2-mm- (0.04- to 0.08-inch-) thick layers for determination of chloride are common.[24]

Section 3.6 requires the determination of the chloride content at two depths, each approximately 13 mm (0.5 inch) thick. A 13-mm (0.5-inch) increment does not provide enough data points over the depth. Chloride values at different depths are used to determine the diffusion coefficients. Diffusion coefficients are desirable since they can be incorporated into service-life prediction models. To determine diffusion coefficients, at least four (or even six) chloride values from different layers are needed. Consequently, the thickness of the sampled layer needs to be much less and should be chosen to enable the collection of 10 grams (0.4 ounces) of representative material as required by AASHTO T 260. Thicknesses as thin as 1 mm (0.04 inch) have been used.[25]

ACTION: Revisions to sections 2.1 and 3.4 are proposed. A research problem statement is proposed to address other issues.

T 260 SAMPLING AND TESTING FOR CHLORIDE ION IN CONCRETE AND CONCRETE RAW MATERIALS

This method covers procedures for the determination of the acid-soluble chloride ion content or the water-soluble chloride ion content of aggregates, portland cement, mortar or concrete.

This method does not require any modification for use with HPC.

ACTION: None.

T 271 DENSITY OF PLASTIC AND HARDENED PORTLAND CEMENT CONCRETE IN-PLACE BY NUCLEAR METHODS

These methods cover the determination of the density of plastic and hardened concrete in place by gamma radiation.

This method does not require any modification for use with HPC.

ACTION: None.

T 276 DEVELOPING EARLY-AGE COMPRESSION TEST VALUES AND PROJECTING LATER-AGE STRENGTHS (ASTM DESIGNATION: C 918)

This test method covers a procedure for making, curing, and testing specimens of concrete stored under conditions intended to measure the maturity as it relates to strength gain in the concrete.

Section 4.1 states that the test method uses conventional curing. ASTM C 1074 and AASHTO TP52 provide a procedure for the estimation of strength using other temperature histories.

Section 9.1 has the latest test age at 28 days, but requires a later test age if the age for which the projected strength is to be achieved exceeds 28 days. HPC often requires a later age. Ages of 56 and 90 days should be specifically listed.

The use of this test method to predict high early strengths should be evaluated.

ACTION: A research problem statement is proposed.

T 277 ELECTRICAL INDICATION OF CONCRETE'S ABILITY TO RESIST CHLORIDE ION PENETRATION (ASTM DESIGNATION: C 1202)

This test method covers the determination of the electrical conductance of concrete to provide a rapid indication of its resistance to penetration of chloride ions.

The curing procedure and test age are not given in the test method. They are important variables and should be included. Concrete with pozzolans and ground granulated blast-furnace slag usually take a longer time to exhibit reduced permeability. (Silica fume concrete may be an exception.) Accelerated curing at higher temperatures or extended curing ages are needed to observe the benefits of these supplementary cementitious materials. The curing procedure and test age need to be addressed in this method.

ACTION: A revision to section 8.1 to define test age and the option of rapid curing is proposed.

T 285 BEND TEST FOR BARS FOR CONCRETE REINFORCEMENT

This method covers a bend test for evaluating the ductility of bars used for concrete reinforcement.

This method does not require any modification for use with HPC. However, this test should be listed under Metallic Materials for Bridges in the table of contents.

ACTION: A relocation in the index is proposed.

T 299 RAPID IDENTIFICATION OF ALKALI-SILICA REACTION PRODUCTS IN CONCRETE

This test covers the rapid visual detection of the products of alkali-silica reaction in portland cement concrete.

This method does not require any modification for use with HPC.

ACTION: None.

T 303 ACCELERATED DETECTION OF POTENTIALLY DELETERIOUS EXPANSION OF MORTAR BARS DUE TO ALKALI-SILICA REACTION

This test method allows detection within 16 days of the potential for deleterious expansion of mortar bars due to alkali-silica reaction.

This method does not require any modification for use with HPC.

ACTION: None.

Hydraulic Cement

Hydraulic cement tests are applicable to cementitious material used in HPC. However, in many of these test methods, the word "cement" is used and generally addresses only portland cement. In HPC, pozzolans and ground granulated blast-furnace slag are widely used. Precision statements should be based on cements with pozzolans or slag.

T 98 FINENESS OF PORTLAND CEMENT BY THE TURBIDIMETER (ASTM DESIGNATION: C 115)

This test method covers determination of the fineness of portland cement as represented by a calculated measure of specific surface, expressed as square centimeters of total surface area per gram, or square meters of total surface area per kilogram, of cement, using the Wagner Turbidimeter.

This test method does not require any modification for use with HPC.

ACTION: None.

T 105 CHEMICAL ANALYSIS OF HYDRAULIC CEMENT (ASTM DESIGNATION: C 114)

These test methods cover the chemical analyses of hydraulic cements.

These test methods do not require any modification for use with HPC.

ACTION: None.

T 106 COMPRESSIVE STRENGTH OF HYDRAULIC CEMENT MORTAR (USING 50-MM OR 2-INCH CUBE SPECIMENS) (ASTM DESIGNATION: C 109)

This test method covers determination of the compressive strength of hydraulic cement mortar, using 50-mm (or 2-inch) cube specimens.

In section 10.6, Determination of Compressive Strength, an age of 56 days should be included since it is more appropriate for use with HSC.

ACTION: Revisions to tables 3 and 4 to include 56 days are proposed.

T 107 AUTOCLAVE EXPANSION OF PORTLAND CEMENT (ASTM DESIGNATION: C 151)

This test method covers determination of the autoclave expansion of portland cement by means of a test on a neat cement specimen.

This test method does not require any modification for use with HPC.

ACTION: None.

T 127 SAMPLING AND AMOUNT OF TESTING OF HYDRAULIC CEMENT (ASTM DESIGNATION: C 183)

This practice covers procedures for sampling and for the amount of testing of hydraulic cement after it has been manufactured and is ready to be offered for sale.

This test method does not require any modification for use with HPC.

ACTION: None.

T 128 FINENESS OF HYDRAULIC CEMENT BY THE 150-μM (NO. 100) AND 75-μM (NO. 200) SIEVES (ASTM DESIGNATION: C 184)

This test method covers determination of the fineness of hydraulic cement by the 150-μm (No. 100) and 75-μm (No. 200) sieves.

This test method does not require any modification for use with HPC.

ACTION: None.

T 129 NORMAL CONSISTENCY OF HYDRAULIC CEMENT (ASTM DESIGNATION: C 187)

This method covers determination of the normal consistency of hydraulic cement.

This method does not require any modification for use with HPC.

ACTION: None.

T 131 TIME OF SETTING OF HYDRAULIC CEMENT BY VICAT NEEDLE (ASTM DESIGNATION: C 191)

This method covers determination of the time of setting of hydraulic cement by means of the Vicat needle.

This method does not require any modification for use with HPC.

ACTION: None.

T 132 TENSILE STRENGTH OF HYDRAULIC CEMENT MORTARS

This method covers determination of the tensile strength of hydraulic cement mortars employing the briquet specimen. It is primarily for use by those interested in research on methods for determining tensile strength of hydraulic cement.

The oldest test age listed for this test is 28 days. Since HPC is tested at later ages, tests at 56 days need to be included.

The precision and bias statements in section 11 are only applicable to portland cement mortars. They need to be extended to include blended cement mortars.

ACTION: Revisions to include supplementary cementing materials and a test age of 56 days are proposed. A research problem statement is proposed to address precision and bias statements.

T 133 DENSITY OF HYDRAULIC CEMENT (ASTM DESIGNATION: C 188)

This method covers determination of the density of hydraulic cement.
This method does not require any modification for use with HPC.

ACTION: None.

T 137 AIR CONTENT OF HYDRAULIC CEMENT MORTAR (ASTM DESIGNATION: C 185)

This test method covers determination of the air content of hydraulic cement mortar under the conditions hereinafter specified.

This test method does not require any modification for use with HPC.

ACTION: None.

T 153 FINENESS OF HYDRAULIC CEMENT BY AIR PERMEABILITY APPARATUS (ASTM DESIGNATION: C 204)

This test method covers determination of the fineness of hydraulic cement, using the Blaine air permeability apparatus, in terms of the specific surface expressed as total surface area in square centimeters per gram, or square meters per kilogram of cement.

This test method does not require any modification for use with HPC.

ACTION: None.

T 154 TIME OF SETTING OF HYDRAULIC CEMENT BY GILLMORE NEEDLES (ASTM DESIGNATION: C 266)

This test method covers determination of the time of setting of hydraulic-cement paste by means of the Gillmore needles.

This test method does not require any modification for use with HPC.

ACTION: None.

T 162 MECHANICAL MIXING OF HYDRAULIC CEMENT PASTES AND MORTAR OF PLASTIC CONSISTENCY (ASTM DESIGNATION: C 305)

This method covers the mechanical mixing of hydraulic cement pastes and mortars of plastic consistency.

Sections 7 and 8 provide procedures for mixing pastes and mortars, respectively. Pastes and mortars are mixed at low and medium speeds for specified durations. HPC pastes and mortars may be more cohesive than conventional pastes and mortars. Consequently, longer mixing times or higher speeds may be needed.

ACTION: None.

T 185 EARLY STIFFENING OF PORTLAND CEMENT (MORTAR METHOD) (ASTM DESIGNATION: C 359)

This method covers the determination of early stiffening in portland cement mortar.

Section 10 describes the procedure and requires mixing dry and wet materials at specified speeds for specified durations. HPC mortars may be more cohesive than conventional mortars and may require longer mixing times or higher speeds.

ACTION: None.

T 186 EARLY STIFFENING OF HYDRAULIC CEMENT (PASTE METHOD) (ASTM DESIGNATION: C 451)

This test method covers the determination of early stiffening in hydraulic cement paste.

Section 10 describes the procedure and requires mixing materials at specified speeds for specified durations. HPC mortars may be more cohesive than conventional mortars and may require longer mixing times or higher speeds.

ACTION: None.

T 188 EVALUATION BY FREEZING AND THAWING OF AIR-ENTRAINING ADDITIONS TO PORTLAND CEMENT

This method is intended for use in determining the ability of cement containing an air-entraining agent to produce frost-resistant concrete when tested in accordance with the procedures described below.

This method only indicates the performance of an air-entrained addition in a concrete with a fixed cement content, fixed slump, and optimum ratio of fine aggregate to total aggregate. As such, it does not indicate the behavior of air-entraining agents in the presence of other admixtures or when higher consistency mixes are used. Modifications may be needed for HPC.

ACTION: Revisions to the title and a requirement for including other admixtures are proposed.

T 192 FINENESS OF HYDRAULIC CEMENT BY THE 45-μM (NO. 325) SIEVE (ASTM DESIGNATION: C 430)

This test method covers determination of the fineness of hydraulic cement by means of the 45-μm (No. 325) sieve.

This test method does not require any modification for use with HPC.

ACTION: None.

AASHTO STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES

The compilation in this section is based on the *AASHTO Standard Specifications for Highway Bridges*, Sixteenth Edition, 1996, and the 1997, 1998, 1999, and 2000 interim revisions. See references 2 through 6. This section only lists articles affected by HPC. For each listed article, the portion affected by HPC is shown in italics, followed by specific comments in regular font. For long articles, only a synopsis, followed by comments, is included. References in the comments to specific sections, articles, or tables refer to the document being reviewed and not the sections, articles, or tables in this report. The end result of the project is stated under the action item. Proposed revisions are included in appendix C. Research problem statements are included in appendix F.

Division I: Design

Section 3: LOADS

3.3 DEAD LOAD

3.3.6 *The following weights are to be used in computing the dead load:*
Concrete, plain, or reinforced ....150 pcf

HPC may be somewhat denser than conventional concrete. A slightly higher unit weight may be justified and needed.

ACTION: A revision to 3.3.6 is proposed.

Section 8: REINFORCED CONCRETE

8.3 REINFORCEMENT

8.3.3 *Designs shall not use a yield strength, f_y, in excess of 60,000 psi.*

The use of higher yield strengths should be allowed with concretes of all strengths.

ACTION: A revision to 8.3.3 is proposed.

8.5 EXPANSION AND CONTRACTION

8.5.3 *The coefficient of thermal expansion and contraction for normal-weight concrete may be taken as 0.000006 per deg F.*

ACI Committee 363, High-Strength Concrete, reports that the coefficient of thermal expansion and contraction for HSC is approximately the same as that for conventional strength concrete, but cites only two studies.[15] An attempt should be made to find more information on the coefficient of thermal expansion and contraction, and to verify the coefficient of thermal expansion for HPC.

ACTION: A revision and a research problem statement are proposed.

8.5.4 *The coefficient of shrinkage for normal-weight concrete may be taken as 0.0002.*

Data on HPC show a wide variation in measured shrinkage strains. There is no consistent comparison with conventional concrete; some HPC shows less shrinkage, some HPC shows more, and some the same. A single value of shrinkage may not be valid for HPC. The use of 0.0002 for shrinkage of HPC needs to be verified. In addition, the condition under which a shrinkage strain of 0.0002 is valid needs to be defined.

ACTION: Revisions to 8.5.4 and 8.5.5 are proposed.

8.7 MODULUS OF ELASTICITY AND POISSON'S RATIO

8.7.1 *The modulus of elasticity, E_c, for concrete may be taken as $w_c^{1.5}\,33\sqrt{f_c'}$ in psi for values of w_c between 90 and 155 pounds per cubic foot. For normal-weight concrete (w_c = 145 pcf), E_c may be considered as $57{,}000\sqrt{f_c'}$.*

The ACI Committee 363 report shows data indicating that the formula given in this article may not be appropriate for HSC.[15] Other data suggest that the E_c for HSC may be influenced by aggregate stiffness.[26] Furthermore, some HSCs have a unit weight greater than 2.48 megagrams per cubic meter (Mg/m^3) (155 lb/ft^3). Thus, the formula for E_c in this article needs to be evaluated using recent data for HPC from many locations.

ACTION: A revision based on NCHRP project 18-07 is proposed.

8.7.3 *Poisson's ratio may be assumed as 0.2.*

The ACI Committee 363 report shows data that indicate that the statement in this article is valid for HPC.(15) A check of the literature should be made to see if there are any recent data that might contradict the cited report.

ACTION: None.

8.13 COMPUTATION OF DEFLECTIONS

8.13.4 *Unless values are obtained by a more comprehensive analysis, the long-time deflection for both normal-weight and lightweight concrete flexural members shall be the immediate deflection caused by the sustained load considered, computed in accordance with Article 8.13.3, multiplied by one of the following factors:*

(a) Where the immediate deflection has been based on I_g, the multiplication factor for the long-time deflection shall be taken as 4.

(b) Where the immediate deflection has been based on I_e, the multiplication factor for the long-time deflection shall be taken as $3 - 1.2(A_s'/A_s) \geq 1.6$.

HSC usually has lower creep than conventional strength concrete, so long-time deflection multipliers may be less. Long-time deflection may need to be verified for use with HSC. Consideration should also be given to using the factors in the ACI Building Code.(18)

ACTION: A research problem statement is proposed.

8.15 SERVICE LOAD DESIGN METHOD (ALLOWABLE STRESS DESIGN)

8.15.2 ALLOWABLE STRESSES

8.15.2.1 CONCRETE

Stresses in concrete shall not exceed the following:

8.15.2.1.1 Flexure

Extreme fiber stress in compression, f_c..$0.40 f_c'$

Extreme fiber stress in tension for plain concrete, f_t.................................... $0.21 f_r$

Modulus of rupture, f_r, from tests, or, if data are not available:

Normal-weight concrete .... $7.5\sqrt{f_c'}$

Sand-lightweight concrete .... $6.3\sqrt{f_c'}$

All-lightweight concrete... $5.5\sqrt{f_c'}$

The modulus of rupture for normal-weight HSC is higher than given in this article. The multipliers need to be evaluated for use with HSC.

ACTION: A revision for normal-weight concrete is proposed. A research problem statement is proposed for other weights of concrete.

8.15.2.1.2 Shear

For detailed summary of allowable shear stress, v_c, see Article 8.15.5.2.

See comments on article 8.15.5.2.

ACTION: None. Further research is being conducted under NCHRP project 12-56.

8.15.2.1.3 Bearing Stress

The bearing stress, f_b, on loaded area shall not exceed $0.30\, f_c'$.

When the supporting surface is wider on all sides than the loaded area, the allowable bearing stress on the loaded area may be multiplied by $\sqrt{A_2 / A_1}$, but not by more than 2.

When the supporting surface is sloped or stepped, A_2 may be taken as the area of the lower base of the largest frustrum of the right pyramid or cone contained wholly within the support and having for its upper base the loaded area, and having side slopes of 1 vertical to 2 horizontal.

When the loaded area is subjected to high-edge stresses due to deflection or eccentric loading, the allowable bearing stress on the loaded area, including any increase due to the supporting surface being larger than the loaded area, shall be multiplied by a factor of 0.75.

The upper limit for the bearing stress, f_b, needs to be verified for HSC.

ACTION: A research problem statement is proposed.

8.15.5 SHEAR

8.15.5.2 SHEAR STRESS CARRIED BY CONCRETE

8.15.5.2.1 Shear in Beams and One-Way Slabs and Footings

For members subject to shear and flexure only, the allowable shear stress carried by the concrete, v_c, may be taken as 0.95 $\sqrt{f_c'}$. A more detailed calculation of the allowable shear stress can be made using:

$$v_c = 0.9\sqrt{f_c'} + 1{,}100\rho_w\left(\frac{Vd}{M}\right) \leq 1.6\sqrt{f_c'}$$

(8-4) [Equation 1]

Note:

(a) M is the design moment occurring simultaneously with V at the section being considered.

(b) The quantity Vd/M shall not be taken greater than 1.0.

Test results have indicated that the shear stress carried by the concrete can be proportionally lower for HSC because of the smoother crack surfaces.(15) Consequently, the constants in this article need to be verified for HSC.

ACTION: None. Further research is being conducted under NCHRP project 12-56.

8.15.5.2.2 Shear in Compression Members

For members subject to axial compression, the allowable shear stress carried by the concrete, v_c, may be taken as 0.95 $\sqrt{f_c'}$. A more detailed calculation can be made using:

$$v_c = 0.9\left(1 + 0.0006\frac{N}{A_g}\right)\sqrt{f_c'}$$

(8-5) [Equation 2]

The quantity N/A_g shall be expressed in pounds per square inch.

The constants in this article need to be verified for HSC.

ACTION: A research problem statement is proposed.

8.15.5.2.3 Shear in Tension Members

For members subject to axial tension, shear reinforcement shall be designed to carry total shear, unless a more detailed calculation is made using:

$$v_c = 0.9\left(1 + 0.004\frac{N}{A_g}\right)\sqrt{f_c'}$$

(8-6) [Equation 3]

Note:

(a) N is negative for tension.

(b) *The quantity N/A_g shall be expressed in pounds per square inch.*

The constants in this article need to be verified for HSC.

ACTION: A research problem statement is proposed.

8.15.5.2.4 Shear in Lightweight Concrete

The provisions for shear stress, v_c, carried by the concrete, apply to normal-weight concrete. When lightweight aggregate concretes are used, one of the following modifications shall apply:

(a) *When f_{ct} is specified, the shear stress, v_c, shall be modified by substituting $f_{ct}/6.7$ for $\sqrt{f_c'}$, but the value of $f_{ct}/6.7$ used shall not exceed $\sqrt{f_c'}$.*

(b) *When f_{ct} is not specified, the shear stress, v_c, shall be multiplied by 0.75 for all-lightweight concrete, and 0.85 for sand-lightweight concrete. Linear interpolation may be used when partial sand replacement is used.*

HPC can be made with lightweight aggregate. Consequently, the constants need to be verified for use with lightweight HPC.

ACTION: A research problem statement is proposed.

8.15.5.3 SHEAR STRESS CARRIED BY SHEAR REINFORCEMENT

8.15.5.3.2 When shear reinforcement perpendicular to the axis of the member is used:

$$A_v = \frac{(v - v_c)\, b_w\, s}{f_s} \qquad (8\text{-}7) \qquad \text{[Equation 4]}$$

8.15.5.3.3 When inclined stirrups are used:

$$A_v = \frac{(v - v_c)\, b_w\, s}{f_s\,(\sin\alpha + \cos\alpha)} \qquad (8\text{-}8) \qquad \text{[Equation 5]}$$

8.15.5.3.4 When shear reinforcement consists of a single bar or a single group of parallel bars all bent up at the same distance from the support:

$$A_v = \frac{(v - v_c)\, b_w\, d}{f_s \sin\alpha} \qquad (8\text{-}9) \qquad \text{[Equation 6]}$$

where $(v - v_c)$ shall not exceed $1.5\sqrt{f_c'}$.

Limited test results have indicated that shear stress carried by the reinforcement is apparently higher with HSC. Therefore, the limit on $(v - v_c)$ needs to be verified for HSC.

ACTION: None. Further research is being conducted under NCHRP project 12-56.

8.15.5.3.8 When $(v - v_c)$ exceeds $2\sqrt{f_c'}$, the maximum spacings given in Article 8.19 shall be reduced by one-half.

The limit on $(v - v_c)$ needs to be verified for HSC.

ACTION: None. Further research is being conducted under NCHRP project 12-56.

8.15.5.3.9 The value of (vv_c) shall not exceed $4\sqrt{f_c'}$.

The upper limit on $(v - v_c)$ needs to be verified for HPC.

ACTION: None. Further research is being conducted under NCHRP project 12-56.

8.15.5.4 SHEAR FRICTION

8.15.5.4.3 Shear-Friction Design Method

(a) When shear-friction reinforcement is perpendicular to shear plane, the area of shear-friction reinforcement, A_{vf}, shall be computed by:

$$A_{vf} = \frac{V}{f_s \mu} \qquad \text{(8-10)} \qquad \text{[Equation 7]}$$

where μ is the coefficient of friction in accordance with Art. 8.15.5.4.3(c).

(b) When shear-friction reinforcement is inclined to the shear plane such that the shear force produces tension in shear-friction reinforcement, the area of shear-friction reinforcement, A_{vf} , shall be computed by:

$$A_{vf} = \frac{V}{f_s\,(\mu \sin\alpha_f + \cos\alpha_f)} \qquad \text{(8-11)} \qquad \text{[Equation 8]}$$

where α_f is the angle between the shear-friction reinforcement and the shear plane.

(c) Coefficient of friction μ in Eq. (8-10) and Eq. (8-11) shall be:

concrete placed monolithically *1.4λ*

concrete placed against hardened concrete with surface intentionally roughened as specified in Art. 8.15.5.4.7. .... *1.0λ*

concrete placed against hardened concrete not intentionally roughened ... *0.6λ*

concrete anchored to as-rolled structural steel by headed studs or by reinforcing bars (see Art. 8.15.5.4.8) *0.7λ*

where $\lambda = 1.0$ for normal-weight concrete, 0.85 for sand-lightweight concrete, and 0.75 for all-lightweight concrete. Linear interpolation may be applied when partial sand replacement is used.

Tests have indicated that a smoother crack plane occurs with HSC.(15) Consequently, the values of μ and λ need to be verified for HSC.

ACTION: A research problem statement is proposed.

8.15.5.4.4 Shear stress, v, shall not exceed $0.09f_c'$ nor 360 psi.

This article imposes a limit of 28 MPa (4000 psi) on the compressive strength of concrete that can be used in design and is a barrier to the effective use of HSC. This limit needs to be evaluated based on recent test data.

ACTION: A research problem statement is proposed.

8.15.5.5 HORIZONTAL SHEAR DESIGN FOR COMPOSITE CONCRETE FLEXURAL MEMBERS

8.15.5.5.3 Design horizontal shear stress v_{dh} at any cross section may be computed by:

$$v_{dh} = \frac{V}{b_v\, d} \qquad \textit{(8-11A)} \qquad \text{[Equation 9]}$$

where V is the design shear force at the section considered and d is for the entire composite section. Horizontal shear v_{dh} shall not exceed permissible horizontal shear v_h in accordance with the following:

(a) When contact surface is clean, free of laitance, and intentionally roughened, shear stress v_h shall not exceed 36 psi.

(b) When minimum ties are provided in accordance with paragraph 8.15.5.5.5, and the contact surface is clean and free of laitance, but not intentionally roughened, shear stress v_h shall not exceed 36 psi.

(c) When minimum ties are provided in accordance with paragraph 8.15.5.5.5, and the contact surface is clean, free of laitance, and intentionally roughened to a full magnitude of approximately ¼ inch, shear stress v_h shall not exceed 160 psi.

(d) For each percent of tie reinforcement crossing the contact surface in excess of the minimum required by 8.15.5.5.5, permissible v_h may be increased by $72f_y/40{,}000$ psi.

The horizontal shear values, v_{dh}, need to be evaluated for HPC.

ACTION: A research problem statement is proposed.

8.15.5.6 SPECIAL PROVISIONS FOR SLABS AND FOOTINGS

8.15.5.6.3 Design shear stress, v, shall not exceed v_c given by Equation (8-13) unless shear reinforcement is provided in accordance with Article 8.15.5.6.4.

$$v_c = \left(0.8 + \frac{2}{\beta_c}\right)\sqrt{f_c'} \le 1.8\sqrt{f_c'} \qquad (8\text{-}13) \qquad \text{[Equation 10]}$$

β_c is the ratio of long side to short side of concentrated load or reaction area.

The constants used in equation 8-13 need to be verified for HSC.

ACTION: A research problem statement is proposed.

8.15.5.6.4 Shear reinforcement consisting of bars or wires may be used in slabs and footings in accordance with the following provisions:

(a) Shear stresses computed by Equation (8-12) shall be investigated at the critical section defined in 8.15.5.6.1(b) and at successive sections more distant from the support.

(b) Shear stress v_c at any section shall not exceed $0.9\sqrt{f_c'}$ and v shall not exceed $3\sqrt{f_c'}$.

(c) Where v exceeds $0.9\sqrt{f_c'}$, shear reinforcement shall be provided in accordance with Article 8.15.5.3.

The limiting values of v and v_c need to be verified for HSC.

ACTION: A research problem statement is proposed.

8.15.5.7 SPECIAL PROVISIONS FOR SLABS OF BOX CULVERTS

For slabs of box culverts under 2 feet or more fill, shear stress v_c may be computed by:

$$v_c = \sqrt{f_c'} + 2{,}200\rho\left(\frac{Vd}{M}\right)$$

(8-14) [Equation 11]

but v_c shall not exceed $1.8\sqrt{f_c'}$. For single cell box culverts only, v_c for slabs monolithic with walls need not be taken less than $1.4\sqrt{f_c'}$, and v_c for slabs simply supported need not be taken less than $1.2\sqrt{f_c'}$. The quantity Vd/M shall not be taken greater than 1.0 where M is the moment occurring simultaneously with V at the section considered. For slabs of box culverts under less than 2 feet of fill, applicable provisions of Articles 3.24 and 6.4 should be used.

Although HSC may not be used in slabs of box culverts, the constants in equation 8-14 and limiting values of v_c should be verified.

ACTION: A research problem statement is proposed.

*8.15.5.8 SPECIAL PROVISIONS FOR BRACKETS AND CORBELS**

8.15.5.8.3 The section at the face of support shall be designed to resist simultaneously a shear V, a moment $[Va_v + N_c(h - d)]$, and a horizontal tensile force N_c. Distance h shall be measured at the face of support.

(a) Design of shear-friction reinforcement, A_{vf}, to resist shear, V, shall be in accordance with Article 8.15.5.4. For normal-weight concrete, shear stress v shall not exceed 0.09 f_c' nor 360 psi. For all-lightweight or sand-lightweight concrete, shear stress v shall not exceed $(0.09 - 0.03a_v/d)\, f_c'$ nor $(360 - 126a_v/d)$ psi.

(b) Reinforcement A_f to resist moment $[Va_v + N_c(h - d)]$ shall be computed in accordance with Articles 8.15.2 and 8.15.3.

(c) Reinforcement A_n to resist tensile force N_c shall be computed by $A_n = N_c/f_s$. Tensile force N_c shall not be taken less than 0.2V unless special provisions are made to avoid tensile forces.

(d) Area of primary tension reinforcement, A_s, shall be made equal to the greater of $(A_f + A_n)$ or $(2A_{vf}/3 + A_n)$.

**These provisions do not apply to beam ledges. The PCA publication, "Notes on ACI 318-83," contains an example design of beam ledges, Part 16, example 16-3.*

Article (a) imposes a limit of 28 MPa (4000 psi) on the compressive strength on concrete that can be used in design and is a barrier to the effective use of HSC. The limits and factors need to be evaluated.

ACTION: A research problem statement is proposed.

8.15.5.8.5 Ratio $\rho = A_s/bd$ shall not be taken less than $0.04(f_c'/f_y)$.

Since the minimum value of ρ increases as concrete strength increases, values of ρ can be high with HSC. It needs to be determined if this minimum applies to HSC.

ACTION: A research problem statement is proposed.

8.16 STRENGTH DESIGN METHOD (LOAD FACTOR DESIGN)

8.16.1 Strength Requirements

8.16.1.1 REQUIRED STRENGTH

The required strength of a section is the strength necessary to resist the factored loads and forces applied to the structure in the combinations stipulated in Article 3.22. All sections of structures and structural member shall have design strengths at least equal to the required strength.

8.16.1.2 DESIGN STRENGTH

8.16.1.2.1 The design strength provided by a member or cross section in terms of load, moment, shear, or stress shall be the nominal strength calculated in accordance with the requirements and assumptions of the strength-design method, multiplied by a strength-reduction factor ϕ. The strength-reduction factors, ϕ, shall be as follows:*

(a) Flexure .... $\phi = 0.90$

(b) Shear .... $\phi = 0.85$

(c) Axial compression with:
Spirals .... $\phi = 0.75$
Ties .... $\phi = 0.70$

(d) Bearing on concrete .... $\phi = 0.70$

The value of ϕ may be increased linearly from the value for compression members to the value for flexure as the design axial load strength, ϕP_n, decreases from $0.10\ f_c' A_g$ or ϕP_b, whichever is smaller, to zero.

** The coefficient ϕ provides for the possibility that small adverse variations in material strengths, workmanship, and dimensions, while individually within acceptable tolerances and limits of good practice, may combine to result in understrength.*

HPC tends to be very sensitive to water content and constitutive materials. The chance of understrength concrete may increase, especially at very high compressive-strength levels. On the other hand, HPC is produced with stricter quality control and a lower coefficient of variation than conventional concrete. Also, HSC has less lateral expansion than conventional strength concrete, so the effect of confinement is less. This affects column behavior. There is, therefore, a need to verify the suitability of the given strength-reduction factors for HPC, especially HSC.

ACTION: A research problem statement is proposed to address strength-reduction factors.

8.16.2 Design Assumptions

8.16.2.7 *A compressive stress/strain distribution, which assumes a concrete stress of 0.85* $f_c^{'}$ *uniformly distributed over an equivalent compression zone bounded by the edges of the cross section and a line parallel to the neutral axis at a distance* $a = \beta_1 c$ *from the fiber of maximum compressive strain, may be considered to satisfy the requirements of Article 8.16.2.6. The distance c from the fiber of maximum strain to the neutral axis shall be measured in a direction perpendicular to that axis. The factor* β_1 *shall be taken as 0.85 for concrete strengths,* $f_c^{'}$*, up to and including 4,000 psi. For strengths above 4,000 psi,* β_1 *shall be reduced continuously at a rate of 0.05 for each 1,000 psi of strength in excess of 4,000 psi, but* β_1 *shall not be taken less than 0.65.*

The stress/strain curve for HSC is more linear than for conventional strength concrete. However, the stress block factors are generally considered to be still valid for members where flexure predominates. For members where axial compression predominates, the concrete stress of 0.85 $f_c^{'}$ may need to be reduced as concrete strength increases.(27) In the *Canadian Standard for Design of Concrete Structures*, the 0.85 factor is replaced by $(0.85 - 0.0015 f_c^{'}) \geq 0.67$, in which $f_c^{'}$ is in megapascals.(28) A review is needed to determine if the rectangular stress block and factors are valid with HSC.

ACTION: Revisions to the maximum usable strain and the β factor are proposed.

8.16.3 Flexure

8.16.3.2 RECTANGULAR SECTIONS WITH TENSION REINFORCEMENT ONLY

8.16.3.3 FLANGED SECTIONS WITH TENSION REINFORCEMENT ONLY

8.16.3.4 RECTANGULAR SECTIONS WITH COMPRESSION REINFORCEMENT ONLY

These three articles provide equations for the calculation of design moment strength and balanced reinforcement ratio based on a rectangular stress block and an assumed limiting concrete strain of 0.003. If the rectangular stress block and the limiting strains are not appropriate for HSC, the equations will need to be revised or their application restricted to lower concrete strengths.

ACTION: None. Further research is the objective of NCHRP project 12-64.

8.16.4 Compression Members

8.16.4.1 GENERAL REQUIREMENTS

8.16.4.1.2 Members subject to compressive axial load combined with bending shall be designed for the maximum moment that can accompany the axial load. The factored axial load, P_u, at a given eccentricity shall not exceed the design axial load strength $\phi P_{n(max)}$, where:

(a) For members with spiral reinforcement conforming to Article 8.18.2.2

$$P_{n(max)} = 0.85[0.85 f_c' (A_g - A_{st}) + f_y A_{st}] \qquad (8\text{-}29) \quad \text{[Equation 12]}$$

$$\phi = 0.75$$

(b) For members with tie reinforcement conforming to Article 8.18.2.3

$$P_{n(max)} = 0.80[0.85 f_c' (A_g - A_{st}) + f_y A_{st}] \qquad (8\text{-}30) \quad \text{[Equation 13]}$$

$$\phi = 0.70$$

The maximum factored moment, M_u, shall be magnified for slenderness effects in accordance with Article 8.16.5.

HSC has less lateral expansion than conventional strength concrete, so the confinement effect is less. This affects column behavior. The constants used in equations 8-29 and 8-30 need to be evaluated for use with HSC.

ACTION: None. Further research is the objective of NCHRP project 12-64.

8.16.4.2 COMPRESSION MEMBER STRENGTHS

8.16.4.2.1 Pure Compression

The design axial load strength at zero eccentricity, ϕP_o, may be computed by:

$$\phi P_o = \phi [0.85 f_c' (A_g - A_{st}) + A_{st} f_y] \qquad (8\text{-}31) \quad \text{[Equation 14]}$$

For design, pure compressive strength is a hypothetical condition since Article 8.16.4.1.2 limits the axial load strength of compression members to 85 and 80 percent of the axial load at zero eccentricity.

See comments on articles 8.16.2.7 and 8.16.4.1.2.

ACTION: None. Further research is the objective of NCHRP project 12-64.

8.16.6 Shear

8.16.6.2 SHEAR STRENGTH PROVIDED BY CONCRETE

8.16.6.2.1 Shear in Beams and One-Way Slabs and Footings

For members subject to shear and flexure only, V_c shall be computed by,

$$V_c = \left(1.9\sqrt{f_c^{'}} + 2{,}500\rho_w \frac{V_u\, d}{M_u} \right) b_w\, d \qquad \textit{(8-48)} \qquad \text{[Equation 15]}$$

or

$$V_c = 2\sqrt{f_c^{'}}\; b_w\; d \qquad \textit{(8-49)} \qquad \text{[Equation 16]}$$

where b_w is the width of web and d is the distance from the extreme compression fiber to the centroid of the longitudinal tension reinforcement. Whenever applicable, effects of torsion shall be included. For a circular section, b_w shall be the diameter and d need not be less than the distance from the extreme compression fiber to the centroid of the longitudinal reinforcement in the opposite half of the member. For tapered webs, b_w shall be the average width or 1.2 times the minimum width, whichever is smaller.

Note:

(a) V_c shall not exceed $3.5\sqrt{f_c^{'}}\, b_w d$ when using more detailed calculations.

(b) The quality $V_u d/M_u$ shall not be greater than 1.0 where M_u is the factored moment occurring simultaneously with V_u at the section being considered.

At higher compressive strengths, HSC is more brittle and the shear cracks are smoother. As a result, there is less friction along the shear cracks. Since this friction carries some of the shear load, shear provided by the concrete may be less. Consequently, the constants used in the equations need to be investigated. The ACI Building Code Requirements for Structural Concrete (ACI 318-99) limits the term $\sqrt{f_c^{'}}$ to a maximum value of 690 kPa (100 psi) for most shear provisions.[18] Recent research data should be evaluated to determine if a similar limit is needed in the AASHTO Standard Specifications.

ACTION: None. Further research is being conducted under NCHRP project 12-56.

8.16.6.2.2 Shear in Compression Members

For members subject to axial compression, V_c may be computed by:

$$V_c = 2\left(1 + \frac{N_u}{2{,}000\ A_g}\right)\sqrt{f_c'}\left(b_w d\right) \qquad \textit{(8-50)} \qquad \text{[Equation 17]}$$

or

$$V_c = 2\sqrt{f_c'}\ b_w d \qquad \textit{(8-51)} \qquad \text{[Equation 18]}$$

Note:

The quantity N_u/A_g shall be expressed in pounds per square inch.

See comments on 8.16.6.2.1.

ACTION: A research problem statement is proposed.

8.16.6.2.3 Shear in Tension Members

For members subject to axial tension, shear reinforcement shall be designed to carry total shear, unless a more detailed calculation is made using:

$$v_c = 2\left(1 + \frac{N_u}{500\ A_g}\right)\sqrt{f_c'}\left(b_w d\right) \qquad \textit{(8-52)} \qquad \text{[Equation 19]}$$

Note:

(a) *N_u is negative for tension.*

(b) *The quantity N_u/A_g shall be expressed in pounds per square inch.*

In equation 8-52, v_c should be V_c. See comments on 8.16.6.2.1.

ACTION: A research problem statement is proposed.

8.16.6.2.4 Shear in Lightweight Concrete

The provisions for shear stress, v_c, carried by the concrete, apply to normal-weight concrete. When lightweight aggregate concretes are used, one of the following modifications shall apply:

(a) *When f_{ct} is specified, the shear strength, V_c, shall be modified by substituting $f_{ct}/6.7$ for $\sqrt{f_c'}$, but the value of $f_{ct}/6.7$ used shall not exceed $\sqrt{f_c'}$.*
(b) *When f_{ct} is not specified, V_c shall be multiplied by 0.75 for all-lightweight concrete, and 0.85 for sand-lightweight concrete. Linear interpolation may be used when partial sand replacement is used.*

HPC can be all-lightweight or sand-lightweight concrete. The constants used in the article need to be evaluated for lightweight and sand-lightweight HPC.

ACTION: A research problem statement is proposed.

8.16.6.3 SHEAR STRENGTH PROVIDED BY SHEAR REINFORCEMENT

8.16.6.3.8 When shear strength V_s exceeds $4\sqrt{f_c'}\, b_w d$, spacing of shear reinforcement shall not exceed one-half the maximum spacing given in Article 8.19.3.

There is a need to verify spacing requirements for HSC.

ACTION: None. Further research is being conducted under NCHRP project 12-56.

8.16.6.3.9 Shear strength V_s shall not be taken great than $8\sqrt{f_c'}\, b_w d$.

The upper limit for V_s needs to be verified for HSC. The AASHTO LRFD Specifications allow a substantially higher value for the maximum shear strength.

ACTION: None. Further research is being conducted under NCHRP project 12-56.

8.16.6.4 SHEAR FRICTION

8.16.6.4.4 Shear-Friction Design Method

(a) When the shear-friction reinforcement is perpendicular to the shear plane, shear strength V_n shall be computed by:

$$V_n = A_{vf} f_y \mu \qquad (8\text{-}56) \qquad \text{[Equation 20]}$$

where μ is the coefficient of friction in accordance with Article (c).

(b) When the shear-friction reinforcement is inclined to the shear plane, such that the shear force produces tension in shear-friction reinforcement, shear strength V_n shall be computed by:

$$V_n = A_{vf} f_y (\mu \sin \alpha_f + \cos \alpha_f) \qquad (8\text{-}56A) \qquad \text{[Equation 21]}$$

where α_f is the angle between the shear-friction reinforcement and shear plane.

(c) Coefficient of friction μ in Eq. (8-56) and Eq. (8-56A) shall be

Concrete placed monolithically 1.4λ

Concrete placed against hardened concrete with surface intentionally roughened as specified in Article 8.16.6.4.8 .... 1.0λ

Concrete placed against hardened concrete not intentionally roughened 0.6λ

Concrete anchored to as-rolled structural steel by headed studs or by reinforcing bars (see Article 8.16.6.4.9) 0.7λ

where $\lambda = 1.0$ for normal-weight concrete, 0.85 for sand-lightweight concrete, and 0.75 for all-lightweight concrete. Linear interpolation may be applied when using partial sand replacement.

Tests have indicated that a smoother crack plane occurs with HSC.(15) Consequently, the values of μ and λ need to be evaluated for HSC.

ACTION: A research problem statement is proposed.

8.16.6.4.5 Shear strength V_n shall not be taken greater than $0.2 f_c' A_{cv}$ nor $800 A_{cv}$ in pounds, where A_{cv} is area of the concrete section resisting shear transfer.

This article imposes a limit of 28 MPa (4000 psi) on the compressive strength of concrete that can be used in design and is a barrier to the effective use of HSC. The limits of $0.2 f_c' A_{cv}$ and 5.5 MPa (800 psi) need to be evaluated based on recent test data.

ACTION: A research problem statement is proposed.

8.16.6.5 HORIZONTAL SHEAR STRENGTH FOR COMPOSITE CONCRETE FLEXURAL MEMBERS

8.16.6.5.3 Design of cross sections subject to horizontal shear may be based on:

$$V_u \le \phi V_{nh} \qquad (8\text{-}57) \qquad \text{[Equation 22]}$$

where V_u is the factored shear force at the section considered, V_{nh} is the nominal horizontal shear strength in accordance with the following, and d is for the entire composite section.

(a) When contact surface is clean, free of laitance, and intentionally roughened, shear strength V_{nh} shall not be taken greater than $80\, b_v d$, in pounds.

(b) When minimum ties are provided in accordance with paragraph 8.16.6.5.5, and contact surface is clean and free of laitance, but not intentionally roughened, shear strength V_{nh} shall not be taken greater than $80\, b_v d$, in pounds.

(c) When minimum ties are provided in accordance with paragraph 8.16.6.5.5, and contract surface is clean, free of laitance, and intentionally roughened to a full amplitude of approximately ¼ inch, shear strength V_{nh} shall not be taken greater than $350\, b_v d$, in pounds.

(d) For each percent of tie reinforcement crossing the contact surface in excess of the minimum required by 8.16.6.5.5, shear strength V_{nh} may be increased by $(160f_y/40{,}000)b_v d$, in pounds.

The horizontal shear resistance needs to be evaluated for HPC.

ACTION: A research problem statement is proposed.

8.16.6.6 SPECIAL PROVISIONS FOR SLABS AND FOOTINGS

8.16.6.6.2 Design of slab or footing for two-way action shall be based on Equation (8-46), where shear strength V_n shall not be taken greater than shear strength V_c given by Equation (8-58), unless shear reinforcement is provided in accordance with Article 8.16.6.6.3.

$$V_c = \left(2 + \frac{4}{\beta_c}\right)\sqrt{f_c'}\, b_o d \le 4\sqrt{f_c'}\, b_o d \qquad (8\text{-}58) \qquad \text{[Equation 23]}$$

β_c is the ratio of long side to short side of concentrated load or reaction area, and b_o is the perimeter of the critical section defined in Article 8.16.6.6.1(b).

The constants used in equation 8-58 need to be verified for HSC.

ACTION: A research problem statement is proposed.

8.16.6.6.3 Shear reinforcement consisting of bars or wires may be used in slabs and footings in accordance with the following provisions:

(a) Shear strength V_n shall be computed by Equation (8-47), where shear strength V_c shall be in accordance with paragraph (d) and shear strength V_s shall be in accordance with paragraph (e).

(b) Shear strength shall be investigated at the critical section defined in 8.16.6.6.1(b), and at successive sections more distant from the support.

(c) Shear strength V_n shall not be taken greater than $6\sqrt{f_c'}\, b_o d$, where b_o is the perimeter of the critical section defined in paragraph (b).

(d) Shear strength V_c at any section shall not be taken greater than $2\sqrt{f_c'}\, b_o d$, where b_o is the perimeter of the critical section defined in paragraph (b).

(e) Where the factored shear force V_u exceeds the shear strength ϕV_c as given in paragraph (d), the required area A_v and shear strength V_s of shear reinforcement shall be calculated in accordance with Article 8.16.6.3.

The limiting values of V_n and V_c need to be verified for HSC.

ACTION: A research problem statement is proposed.

8.16.6.7 SPECIAL PROVISIONS FOR SLABS OF BOX CULVERTS

8.16.6.7.1 For slabs of box culverts under 2 feet or more fill, shear strength V_c may be computed by:

$$V_c = \left(2.14\sqrt{f_c'} + 4{,}600\rho \frac{V_u d}{M_u} \right) bd \qquad (8\text{-}59) \qquad \text{[Equation 24]}$$

but V_c shall not exceed $4\sqrt{f_c'}$ bd. For single-cell box culverts only, V_c for slabs monolithic with walls need not be taken less than $3\sqrt{f_c'}$ bd, and V_c for slabs simply supported need not be taken less than $2.5\sqrt{f_c'}$ bd. The quantity $V_u d/M_u$ shall not be taken greater than 1.0 where M_u is the factored moment occurring simultaneously with V_u at the section considered. For slabs of box culverts under less than 2 feet of fill, applicable provisions of Articles 3.24 and 6.4 should be used.

Although HSC may not be used in slabs of box culverts, the constants in equation 8-59 and the limiting values of V_c should be verified.

ACTION: A research problem statement is proposed.

*8.16.6.8 SPECIAL PROVISIONS FOR BRACKETS AND CORBELS**

8.16.6.8.3 The section at the face of the support shall be designed to resist simultaneously a shear V_u, a moment $(V_u a_v + N_{uc} (h - d))$, and a horizontal tensile force N_{uc}. Distance h shall be measured at the face of support.

(a) In all design calculations in accordance with Article 8.16.6.8, the strength-reduction factor ϕ shall be taken equal to 0.85.

(b) Design of shear-friction reinforcement A_{vf} to resist shear V_u shall be in accordance with Article 8.16.6.4. For normal-weight concrete, shear strength V_n shall not be taken greater than $0.2 f_c' b_w d$ nor $800\, b_w d$ in pounds. For all-lightweight or sand-lightweight concrete, shear strength V_n shall not be taken greater than $(0.2 - 0.07\, a_v/d)\ f_c' b_w d$ nor $(800 - 280\, a_v/d) b_w d$ in pounds.

(c) Reinforcement A_f to resist moment $(V_u a_v + N_{uc} (h - d))$ shall be computed in accordance with Articles 8.16.2 and 8.16.3.

(d) Reinforcement A_n to resist tensile force N_{uc} shall be determined from $N_{uc} \le \phi A_n f_y$. Tensile force N_{uc} shall not be taken less than $0.2V_u$ unless special provisions are made to avoid tensile forces. Tensile force N_{uc} shall be regarded as a live load even when tension results from creep, shrinkage, or temperature change.

(e) *Area of primary tension reinforcement A_s shall be made equal to the greater of (A_f +A_n) or:*

$$\frac{2A_{vf}}{3} + A_n \qquad \text{[Equation 25]}$$

**These provisions do not apply to beam ledges. The PCA publication, "Notes on ACI 318-83," contains an example design of beam ledges, Part 16, example 16-3.*

Article (b) imposes a limit of 28 MPa (4000 psi) on the compressive strength of concrete that can be used in design and is a barrier to the effective use of HSC. The limits and factors need to be evaluated.

ACTION: A research problem statement is proposed.

8.16.7 Bearing Strength

8.16.7.1 *The bearing stress, f_b, on concrete shall not exceed $0.85\phi\ f_c'$ except as provided in Articles 8.16.7.2, 8.16.7.3, and 8.16.7.4.*

8.16.7.2 *When the supporting surface is wider on all sides than the loaded area, the allowable bearing stress on the loaded area may be multiplied by $\sqrt{A_2 / A_1}$ but not by more than 2.*

8.16.7.3 *When the supporting surface is sloped or stepped, A_2 may be taken as the area of the lower base of the largest frustum of a right pyramid or cone contained wholly within the support and having for its upper base the loaded area, and having side slopes of 1 vertical to 2 horizontal.*

8.16.7.4 *When the loaded area is subjected to high edge stresses due to deflection or eccentric loading, the allowable bearing stress on the loaded area, including any increase due to the supporting surface being larger than the loaded area, shall be multiplied by a factor of 0.75.*

The upper limit for the bearing stress, f_b, needs to be verified for HSC.

ACTION: A research problem statement is proposed.

8.17 REINFORCEMENT OF FLEXURAL MEMBERS

8.17.1 Minimum Reinforcement

8.17.1.1 *At any section of a flexural member where tension reinforcement is required by analysis, the reinforcement provided shall be adequate to develop a moment at least 1.2 times the cracking moment calculated on the basis of the modulus of rupture for normal-weight concrete specified in Article 8.15.2.1.1.*

$$\phi M_n \geq 1.2 M_{cr} \qquad (8\text{-}62) \qquad \text{[Equation 26]}$$

The purpose of this article is to ensure that the section does not go to its ultimate strength state as soon as it cracks. HSC is known to have proportionately higher tensile strength than conventional strength concrete. This means that the actual value of M_{cr} is higher than that calculated using $7.5\sqrt{f_c'}$ for the modulus of rupture. Therefore, any factor of safety provided by this article would be lost. Revision to the equation for the modulus of rupture and/or the 1.2 factor may be needed.

ACTION: Revisions to article 8.15.2.1.1 are proposed.

8.18 REINFORCEMENT OF COMPRESSION MEMBERS

8.18.2 Lateral Reinforcement

8.18.2.2 SPIRALS

Spiral reinforcement for compression members shall conform to the following:

8.18.2.2.1 Spirals shall consist of evenly spaced continuous bar or wire, with a minimum diameter of ⅜ inch.

8.18.2.2.2 The ratio of spiral reinforcement to total volume of core, ρ_s, shall not be less than the value given by:

$$\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f_c'}{f_y} \qquad (8\text{-}63) \qquad \text{[Equation 27]}$$

where f_y is the specified yield strength of spiral reinforcement, but not more than 60,000 psi.

Spirals are less effective for confinement in HSC. Another formula is reported by ACI Committee 363 and should be considered.[15] In addition, the ratio of reinforcement required by equation 8-63 may be too high to be practical with HSC. The concept for providing spiral reinforcement to strengthen the core to offset the loss of strength when the concrete shell is lost may not be appropriate for HSC.

ACTION: A research problem statement is proposed.

8.18.2.3 TIES

Tie reinforcement for compression members shall conform to the following:
8.18.2.3.1 All bars shall be enclosed by lateral ties which shall be at least No. 3 in size for longitudinal bars that are No. 10 or smaller, and at least No. 4 in size for No. 11, No. 14, No. 18, and bundled longitudinal bars. Deformed wire or welded wire fabric of equivalent area may be used instead of bars.

8.18.2.3.2 The spacing of ties shall not exceed the least dimension of the compression member or 12 inches. When two or more bars larger than No. 10 are bundled together, tie spacing shall be one-half that specified above.

8.18.2.3.3 Ties shall be located not more than half a tie spacing from the face of a footing or from the nearest longitudinal reinforcement of a cross-framing member.

8.18.2.3.4 No longitudinal bar shall be more than 2 feet, measured along the tie, from a restrained bar on either side. A restrained bar is one which has lateral support provided by the corner of a tie having an included angle of not more than 135 degrees. Where longitudinal bars are located around the perimeter of a circle, a complete circular tie may be used.

Since ties may be less effective with HSC, these provisions need to be evaluated.

ACTION: A research problem statement is proposed.

8.19 LIMITS FOR SHEAR REINFORCEMENT

8.19.1 Minimum Shear Reinforcement

8.19.1.1 *A minimum area of shear reinforcement shall be provided in all flexural members, except slabs and footings, where:*

(a) For design by Strength Design, factored shear force V_u exceeds one-half the shear strength provided by concrete ϕV_c.

(b) For design by Service Load Design, design shear stress v exceeds one-half the permissible shear stress carried by concrete v_c.

8.19.1.2 *Where shear reinforcement is required by Article 8.19.1.1, or by analysis, the area provided shall not be less than:*

$$A_v = \frac{50 b_w s}{f_y}$$

(8-64) [Equation 28]

where b_w and s are in inches.

The ACI 318 Building Code requires that the minimum area of shear reinforcement increase as concrete strength increases, but shall not be less than the value of A_v calculated by equation 8-64.[18] A similar modification to equation 8-64 should be considered.

ACTION: A revision for minimum reinforcement is proposed.

8.22 PROTECTION AGAINST CORROSION

8.22.1 *The following minimum concrete cover shall be provided for reinforcement:*

	Minimum Cover (inches)
Concrete cast against and permanently exposed to earth	*3*
Concrete exposed to earth or weather:	
Primary reinforcement	*2*
Stirrups, ties, and spirals	*1½*
Concrete deck slabs in mild climates:	
Top reinforcement	*2*
Bottom reinforcement	*1*
Concrete deck slabs that have no positive corrosion protection and are frequently exposed to deicing salts:	
Top reinforcement	*2½*
Bottom reinforcement	*1*
Concrete not exposed to weather or in contact with ground:	
Primary reinforcement	*1½*
Stirrups, ties, and spirals	*1*
Concrete piles cast against and/or permanently exposed to earth	*2*

HPC is less permeable than conventional concrete, and offers the possibility of reducing the minimum concrete cover. The advantages and disadvantages of reducing the minimum cover with HPC should be evaluated.

ACTION: None.

8.25 DEVELOPMENT OF DEFORMED BARS AND DEFORMED WIRE IN TENSION

The development length, l_d, in inches shall be computed as the product of the basic development length defined in Article 8.25.1 and the applicable modification factor or factors defined in Article 8.25.2 and 8.25.3, but l_d shall be not less than that specified in Article 8.25.4.

8.25.1 *The basic development length shall be:*

No. 11 bar and smaller .... $\dfrac{0.04 A_b f_y}{\sqrt{f'_c}}$

but not less than .... $0.0004\, d_b f_y$

No. 14 bars .... $\dfrac{0.085 f_y}{\sqrt{f_c'}}$

No. 18 bars .... $\dfrac{0.11 f_y}{\sqrt{f_c'}}$

deformed wire .... $\dfrac{0.03 d_b f_y}{\sqrt{f_c'}}$

The ACI 318 Building Code limits the value of $\sqrt{\mathrm{f_c'}}$ to a maximum of 690 kPa (100 psi) in the calculation of development length.(18) The need for a similar limit in the AASHTO Standard Specifications should be assessed. Limited tests have indicated that the development lengths calculated using the above provisions are applicable to HSC.(29) However, the tests indicate a more sudden failure than occurs with conventional strength concrete. The development lengths need to be verified for HSC.

ACTION: None. Further research is the objective of NCHRP project 12-60.

8.25.2 *The basic development length shall be multiplied by the following applicable factor or factors:*

8.25.2.1 *Top reinforcement so placed that more than 12 inches of concrete is cast below the reinforcement* *1.4*

8.25.2.2 *Lightweight aggregate concrete when f_{ct} is specified* .... $\dfrac{6.7\sqrt{f_c'}}{f_{ct}}$

but not less than 1.0.

When f_{ct} is not specified

all-lightweight concrete *1.33*

sand-lightweight concrete *1.18*

Linear interpolation may be applied when partial sand replacement is used.

8.25.2.3 *Bars coated with epoxy with cover less than $3d_b$ or clear spacing between bars*

less than 6 d_b *1.5*

all other cases *1.15*

The product obtained when combining the factor for top reinforcement with the applicable factor for epoxy-coated reinforcement need not be taken greater than 1.7.

8.25.3 *The basic development length, modified by the appropriate factors of Article 8.25.2, may be multiplied by the following factors when:*

8.25.3.1 *Reinforcement being developed in the length under consideration is spaced laterally at least 6 inches on center with at least 3 inches clear cover measured in the direction of the spacing...0.8*

8.25.3.2 *Anchorage or development for reinforcement strength is not specifically required or reinforcement in flexural members is in excess of that required by analysis*

$$(A_s \text{ required})/(A_s \text{ provided})$$

8.25.3.3 *Reinforcement is enclosed within a spiral of not less than 1/4 inch in diameter and not more than 4-inch pitch 0.75*

8.25.4 *The development length, l_d, shall not be less than 12 inches except in the computation of lap splices by Article 8.32.3 and development of shear reinforcement by Article 8.27.*

The factors need to be verified for HPC.

ACTION: None. Further research is the objective of NCHRP project 12-60.

8.26 DEVELOPMENT OF DEFORMED BARS IN COMPRESSION

The development length, l_d, in inches, for deformed bars in compression shall be computed as the product of the basic development length of Article 8.26.1 and applicable modification factors of 8.26.2, but l_d shall not be less than 8 inches.

8.26.1 *The basic development length shall be ....$0.02\, d_b f_y / \sqrt{f_c'}$*

but not less than ....$0.0003\, d_b f_y$

8.26.2 *The basic development length may be multiplied by applicable factors when:*

8.26.2.1 *Anchorage or development for reinforcement strength is not specifically required, or reinforcement is in excess of that required by analysis............. (A_s required)/(A_s provided)*

8.26.2.2 *Reinforcement is enclosed in a spiral of not less than 1/4 inch in diameter and not more than 4-inch pitch 0.75*

The factors need to be verified for HPC.

ACTION: None. Further research is the objective of NCHRP project 12-60.

8.28 DEVELOPMENT OF BUNDLED BARS

The development length of individual bars within a bundle, in tension or compression, shall be that for the individual bar, increased by 20 percent for a three-bar bundle, and 33 percent for a four-bar bundle.

The factors need to be verified for HPC.

ACTION: None. Further research is the objective of NCHRP project 12-60.

8.29 DEVELOPMENT OF STANDARD HOOKS IN TENSION

8.29.1 *Development length l_{dh} in inches, for deformed bars in tension terminating in a standard hook (Article 8.23.1) shall be computed as the product of the basic development length l_{hb} of Article 8.29.2 and the applicable modification factor or factors of Article 8.29.3, but l_{dh} shall not be less than $8d_b$ or 6 inches, whichever is greater.*

8.29.2 *Basic development length l_{hb} for a hooked bar with f_y equal to 60,000 psi shall be*

.. $1{,}200\ d_b/\sqrt{f_c'}$

8.29.3 *Basic development length l_{hb} shall be multiplied by applicable modification factor or factors for:*

8.29.3.1 *Bar yield strength:*

Bars with f_y other than 60,000 psi..$f_y/60{,}000$

8.29.3.2 *Concrete cover:*

For No. 11 bar and smaller, side cover (normal to plane of hook) not less than 2½ inches, and for 90-deg hook, cover on bar extension beyond hook not less than 2 inches...............0.7

8.29.3.3 *Ties or stirrups:*

For No. 11 bar and smaller, hook enclosed vertically or horizontally within ties or stirrup-ties spaced along the full development length l_{dh} not greater than $3d_b$, where d_b is diameter of hooked bar...0.8

8.29.3.4 *Excess reinforcement:*

Where anchorage or development for f_y is not specifically required, reinforcement in excess of that required by analysis... (A_s required)/(A_s provided)

8.29.3.5 *Lightweight aggregate concrete 1.3*

8.29.4 *For bars being developed by a standard hook at discontinuous ends of members with both side cover and top (or bottom) cover over hook less than 2½ inches, hooked bar shall be enclosed within ties or stirrups spaced along the full development length* l_{dh}, *not greater than 3* d_b, *where* d_b *is the diameter of the hooked bar. For this case, the factor of Article 8.29.3.3 shall not apply.*

8.29.5 *Hooks shall not be considered effective in developing bars in compression.*
All the provisions of article 8.29 need to be verified for HPC.

ACTION: None. Further research is the objective of NCHRP project 12-60.

8.30 DEVELOPMENT OF WELDED WIRE FABRIC IN TENSION

8.30.1 Deformed Wire Fabric

8.30.1.1 *The development length,* l_d, *in inches of welded deformed wire fabric measured from the point of critical section to the end of wire shall be computed as the product of the basic development length of Article 8.30.1.2 or 8.30.1.3 and the applicable modification factor or factors of Articles 8.25.2 and 8.25.3, but* l_d *shall not be less than 8 inches except in computation of lap splices by Article 8.32.5 and development of shear reinforcement by Article 8.27.*

8.30.1.2 *The basic development length of welded deformed wire fabric, with at least one cross wire within the development length not less than 2 inches from the point of critical section, shall be:*

$$0.03 d_b \left(f_y - 20{,}000\right)/\sqrt{f_c'}\ * \qquad (8\text{-}66) \qquad \text{[Equation 29]}$$

but not less than

$$0.20 \frac{A_w}{s_w} \cdot \frac{f_y}{\sqrt{f_c'}} \qquad (8\text{-}67) \qquad \text{[Equation 30]}$$

**The units for 20,000 are psi.*

8.30.1.3 *The basic development length of welded deformed wire fabric, with no cross wires within the development length, shall be determined as for deformed wire in accordance with Article 8.25.*

8.30.2 Smooth Wire Fabric

The yield strength of welded smooth wire fabric shall be considered developed by embedment of two cross wires, with the closer cross wire not less than 2 inches from the point of critical section. However, development length l_d *measured from the point of critical section to outermost cross wire shall not be less than:*

$$0.27 \frac{A_w}{s_w} \cdot \frac{f_y}{\sqrt{f_c'}} \qquad (8\text{-}68) \qquad \text{[Equation 31]}$$

modified by (A_s required)/(A_s provided) for reinforcement in excess of that required by analysis and by factor of Article 8.25.2 for lightweight aggregate concrete, but l_d shall not be less than 6 inches except in computation of lap splices by Article 8.32.6.

All the provisions of article 8.30 need to be verified for HPC.

ACTION: None. Further research is the objective of NCHRP project 12-60.

8.32 SPLICES OF REINFORCEMENT

8.32.1 Lap Splices

8.32.1.1 *Lap splices shall not be used for bars larger than No. 11, except as provided in Articles 8.32.4.1 and 4.4.11.4.1*

8.32.1.2 *Lap splices of bundled bars shall be based on the lap splice length required for individual bars within a bundle. The length of lap, as prescribed in Article 8.32.3 or 8.32.4 shall be increased 20 percent for a three-bar bundle and 33 percent for a four-bar bundle. Individual bar splices within the bundle shall not overlap.*

8.32.1.3 *Bars spliced by noncontact lap splices in flexural members shall not be spaced transversely farther apart than $1/5$ the required length of lap or 6 inches.*

The above provisions of article 8.32.1 need to be verified for HPC.

ACTION: None. Further research is the objective of NCHRP project 12-60.

8.32.3 Splices of Deformed Bars and Deformed Wire in Tension

8.32.3.1 *The minimum length of lap for tension lap splices shall be as required for Class A, B, or C splice, but not less than 12 inches.*

Class A splice.. 1.0 l_d

Class B splice.. 1.3 l_d

Class C splice.. 1.7 l_d

8.32.3.2 *Lap splices of deformed bars and deformed wire in tension shall conform to table 8.32.3.2*

Table 8.32.3.2. Tension lap splices.

	Maximum Percent of A_s Spliced Within Required Lap Length		
(A_s provided)/(A_s required)*	50	75	100
Equal to or Greater Than 2 Less Than 2	Class A Class B	Class A Class C	Class B Class C

**Ratio of area reinforcement provided to area of reinforcement required by analysis at splice location.*

The above provisions of article 8.32.3 need to be verified for HPC.

ACTION: None. Further research is the objective of NCHRP project 12-60.

Section 9: PRESTRESSED CONCRETE

9.1 APPLICATION

9.1.2 Notations

$f_c^{'}$ = compressive strength of concrete at 28 days

Since HSC is often specified at ages other than 28 days, consideration should be given to rewording this definition.

ACTION: A revision to the notation is proposed.

9.2 CONCRETE

The specified compressive strength, $f_c^{'}$, of concrete for each part of the structure shall be shown on the plans. The requirements for $f_c^{'}$ shall be based on tests of cylinders made and tested in accordance with Division II, Section 8, "Concrete Structures."

Consideration should be given to including $f_{ci}^{'}$ in this article since making and testing cylinders for $f_{ci}^{'}$ needs to be addressed in division II, section 8, for HSC.

ACTION: A revision to include $f_{ci}^{'}$ is proposed.

9.14 LOAD FACTORS

The computed strength capacity shall not be less than the largest value from load factor design in Article 3.22. For the design of post-tensioned anchorage zones, a load factor of 1.2 shall be applied to the maximum tendon jacking force.
The following strength capacity reduction factors shall be used:

For factory-produced precast, prestressed concrete members, $\phi = 1.0$.

For post-tensioned cast-in-place concrete members, $\phi = 0.95$.

For shear, $\phi = 0.90$.

For anchorage zones, $\phi = 0.85$ *for normal-weight concrete and* $\phi = 0.70$ *for lightweight concrete.*

HSC is known to be more brittle than conventional strength concrete. Also, HPC requires much higher levels of quality control. Strength capacity reduction factors may need to be revised to reflect this.

ACTION: A research problem statement is proposed.

9.15 ALLOWABLE STRESSES

The design of precast prestressed members ordinarily shall be based on $f_c' = 5{,}000$ *psi. An increase to 6,000 psi is permissible where, in the Engineer's judgment, it is reasonable to expect that this strength will be obtained consistently. Still higher concrete strengths may be considered on an individual area basis. In such cases, the Engineer shall satisfy himself completely that the controls over materials and fabrication procedures will provide the required strengths. The provisions of this section are equally applicable to prestressed concrete structures and components designed with lower concrete strengths.*

A survey by the Precast/Prestressed Concrete Institute indicates that its members are consistently producing concrete members with f_c' equal to or greater than 41 MPa (6000 psi). The wording of this article needs to be revised to reflect the current practice and remove a barrier to the use of HSC. This limitation is also inconsistent with the limitation of 70 MPa (10,000 psi) in the AASHTO LRFD Specifications.

ACTION: A revision to raise the limit to 70 MPa (10,000 psi) is proposed.

9.15.2 Concrete

9.15.2.1 TEMPORARY STRESSES BEFORE LOSSES DUE TO CREEP AND SHRINKAGE

Compression:

Pretensioned members .... $0.60\,f'_{ci}$

Post-tensioned members .... $0.55\,f'_{ci}$

Tension:

Precompressed tensile zone..................................*No temporary allowable stresses are specified. See Article 9.15.2.2 for allowable stresses after losses.*

Other Areas:

In tension areas with no bonded reinforcement............................ *200 psi or* $3\sqrt{f'_{ci}}$

Where the calculated tensile stress exceeds this value, bonded reinforcement shall be provided to resist the total tension force in the concrete computed on the assumption of an uncracked section. The maximum tensile stress shall not exceed.................... $7.5\sqrt{f'_{ci}}$

HSC is known to have a proportionally higher tensile strength. It may be possible to allow higher tensile stresses in HSC.

ACTION: A research problem statement is proposed.

9.15.2.2 STRESS AT SERVICE LOAD AFTER LOSSES HAVE OCCURRED

Compression:

(a) The compressive stresses under all load combinations, except as stated in (b) and (c), shall not exceed $0.60\,f'_c$.

(b) The compressive stresses due to effective prestress plus permanent (dead) loads shall not exceed $0.40\,f'_c$.

(c) The compressive stress due to live loads plus one-half of the sum of compressive stresses due to prestress and permanent (dead) loads shall not exceed $0.40\,f'_c$.

Tension in the precompressed tensile zone:

*(a) For members with bonded reinforcement** .... $6\sqrt{f'_c}$

For severe corrosive exposure conditions, such as coastal areas..................... $3\sqrt{f'_c}$

(b) For members without bonded reinforcement..*0*

Tension in other areas is limited by allowable temporary stresses specified in Article 9.15.2.1.

HSC is known to have proportionally higher tensile strength. It may be possible to allow higher tensile stresses in HSC.

ACTION: A research problem statement is proposed.

*9.15.2.3 CRACKING STRESS**

Modulus of rupture from tests or if not available:

For normal-weight concrete .... $7.5\sqrt{f_c'}$

For sand-lightweight concrete... $6.3\sqrt{f_c'}$

For all other lightweight concrete .... $5.5\sqrt{f_c'}$

HSC is known to have proportionally higher tensile strength. It may be possible to allow higher tensile stresses in HSC.

ACTION: A revision for normal-weight concrete is proposed. A research problem statement is proposed for other weights of concrete.

9.15.2.4 ANCHORAGE BEARING STRESS

Post-tensioned anchorage at service load ....*3,000 psi*

This limit is clearly intended for conventional strength concrete. A higher limit for HSC may be justified.

ACTION: A research problem statement is proposed.

9.16 LOSS OF PRESTRESS

9.16.2 Prestress Losses

9.16.2.1 GENERAL

Loss of prestress due to all causes, excluding friction, may be determined by the following method. The method is based on normal-weight concrete and one of the following types of prestressing steel: 250 or 270 ksi, seven-wire, stress-relieved or low-relaxation strand; 240 ksi stress-relieved wires; or 145 to 160 ksi smooth or deformed bars. Refer to documented tests for data regarding the properties and the effects of lightweight aggregate concrete on prestress losses.*

TOTAL LOSS

$$\Delta f_S = SH + ES + CR_c + CR_s \qquad \text{[Equation 32]}$$

where

Δf_S = *total loss excluding friction in pounds per square inch;*

SH = *loss due to concrete shrinkage in pounds per square inch;*

ES = *loss due to elastic shortening in pounds per square inch;*

CR_C = *loss due to creep of concrete in pounds per square inch;*

CR_s = *loss due to relaxation of prestressing steel in pounds per square inch.*

Equations are then provided for calculation of individual components of prestress losses.

ACTION: Proposed revisions to the AASHTO LRFD Specifications for calculations of prestress losses have been developed in NCHRP project 18-07. A revision to adopt the same method in the AASHTO Standard Specifications is proposed.

9.16.2.2 ESTIMATED LOSSES

In lieu of the preceding method, the following estimates of total losses may be used for prestressed members or structures of the usual design. These loss values are based on use of normal-weight concrete, normal prestress levels, and average exposure conditions. For exceptionally long spans, or for unusual designs, the method in Article 9.16.2.1 or a more exact method shall be used.

Table 9.16.2.2. Estimate of prestress losses.

Type of Prestressing Steel	***Total Loss***	
	f_c' **= 4,000 psi**	f_c' **= 5,000 psi**
Pretensioning Strand	—	*45,000 psi*
Post-Tensioning[1]		
Wire or Strand	*32,000 psi*	*33,000 psi*
Bars	*22,000 psi*	*23,000 psi*

[1] Losses due to friction are excluded. Friction losses should be computed according to Article 9.16.1.

Recent research has indicated that these articles, developed for calculating prestress losses in conventional strength concretes, may not always provide reliable estimates for HSC bridge girders. For this reason, NCHRP project 18-07 was initiated with the objective of developing design guidelines for estimating prestress losses in pretensioned HSC bridge girders. Table 9.16.2.2 needs to be extended to include higher concrete strengths if this is feasible. Results of the NCHRP project will need to be incorporated into this article.

ACTION: Proposed revisions to the AASHTO LRFD Specifications for calculations of prestress losses have been developed in NCHRP project 18-07. A revision to adopt the same method in the AASHTO Standard Specifications is proposed.

9.17 FLEXURAL STRENGTH

9.17.2 Rectangular Sections

9.17.3 Flanged Sections

These two articles provide equations for calculation of design flexural strength based on a rectangular stress block. If the rectangular stress block is not appropriate for HSC, the equations will need to be revised or their application restricted to conventional strength concretes.

ACTION: None. Further research is the objective of NCHRP project 12-64.

9.18 DUCTILITY LIMITS

9.18.1 Maximum Prestressing Steel

Prestressed concrete members shall be designed so that the steel is yielding as ultimate capacity is approached. In general, the reinforcement index shall be such that

$$\left(p^* f_{su}^*\right)/ f_c' \text{ for rectangular sections} \qquad (9\text{-}20) \qquad \text{[Equation 33]}$$

and

$$A_{sr} f_{su}^* /\left(b' d f_c'\right) \text{ for flanged sections} \qquad (9\text{-}21) \qquad \text{[Equation 34]}$$

does not exceed $0.36\beta_1$*. (See Article 9.19 for reinforcement indices of sections with non-prestressed reinforcement.)*

For members with reinforcement indices greater than $0.36\beta_1$*, the design flexural strength shall be assumed not greater than:*

For rectangular sections:

$$\phi M_n = \phi\left[\left(0.36\,\beta_1 - 0.08\,\beta_1^2\right) f_c' b d^2\right] \qquad (9\text{-}22) \qquad \text{[Equation 35]}$$

For flanged sections:

$$\phi M_n = \phi\,[(0.36\,\beta_1 - 0.08\,\beta_1^2)\, f_c' b' d^2 + 0.85 f_c'\,(b - b')t(d - 0.5t)] \qquad (9\text{-}23) \qquad \text{[Equation 36]}$$

Unreinforced HSC is more brittle than conventional strength concrete. The equivalent rectangular stress block may require adjustment to reflect the brittleness. This, in turn, would result in a more realistic maximum limit for flexural reinforcement. Equations 9-22 and 9-23 imply that over-reinforced cross sections are allowed in prestressed concrete flexural members, whereas they are prohibited in non-prestressed flexural members. In prestressed concrete members, some sections away from the maximum moment sections may have the same amount of reinforcement as that at the maximum moment section, but with a smaller effective depth and thus a larger steel area index. Since these sections are non-critical sections, they are allowed provided equations 9-22 and 9-23 are used to calculate the moment strength. Other alternative rational approaches should be considered to eliminate the need for a maximum reinforcement index and strength equations for over-reinforced sections.

ACTION: None. Further research is the objective of NCHRP project 12-64.

9.18.2 Minimum Steel

9.18.2.1 *The total amount of prestressed and nonprestressed reinforcement shall be adequate to develop an ultimate moment at the critical section at least 1.2 times the cracking moment* M^*_{cr}

$$\phi M_n \geq 1.2 M^*_{cr}$$ [Equation 37]

where

$$M^*_{cr} = (f_r + f_{pe})S_c - M_{d/nc}(S_c / S_b - 1)$$ [Equation 38]

Appropriate values for $M_{d/nc}$ *and* S_b *shall be used for any intermediate composite sections. Where beams are designed to be noncomposite, substitute* S_b *for* S_c *in the above equation for the calculation of* M^*_{cr}.

The purpose of this article is to ensure that the section does not go to the ultimate strength state as soon as it cracks. HSC is known to have proportionally higher tensile strength than conventional strength concrete. This means that the actual value of M_{cr} would be higher than that calculated using $7.5\sqrt{f'_c}$ for the modulus of rupture. Therefore, any factor of safety provided by this article would be lost. Revisions to both the equation for the modulus of rupture and/or the 1.2 factor may be needed.

ACTION: Revisions to article 9.15.2.3 are proposed.

9.18.2.2 *The minimum amount of non-prestressed longitudinal reinforcement provided in the cast-in-place portion of slabs utilizing precast, prestressed deck panels shall be 0.25 square inch per foot of slab width.*

The purpose of this reinforcement is to help distribute wheel loads in the longitudinal direction when precast deck panels are not connected for longitudinal continuity. The appropriateness of requiring a fixed amount of reinforcement that is independent of concrete strength and the bending strength of the system should be evaluated.

ACTION: A research problem statement is proposed.

9.20 SHEAR

9.20.2 Shear–Strength Provided by Concrete

9.20.2.1 *The shear strength provided by concrete, V_c, shall be taken as the lesser of the values V_{ci} or V_{cw}.*

9.20.2.2 *The shear strength, V_{ci}, shall be computed by:*

$$V_{ci} = 0.6\sqrt{f_c'}\,b'd + V_d + \frac{V_i M_{cr}}{M_{max}} \qquad (9\text{–}27) \qquad \text{[Equation 39]}$$

but need not be less than $1.7\sqrt{f_c'}\,b'd$ and d need not be taken less than 0.8 h.

The moment causing flexural cracking at the section due to external applied loads, M_{cr}, shall be computed by:

$$M_{cr} = \frac{I}{Y_t}\left(6\sqrt{f_c'} + f_{pe} - f_d\right) \qquad (9\text{–}28) \qquad \text{[Equation 40]}$$

The maximum factored moment and factored shear at the section due to externally applied loads, M_{max} and V_i, shall be computed from the load combination causing maximum moment at the section.

9.20.2.3 *The shear strength, V_{cw}, shall be computed by:*

$$V_{cw} = \left(3.5\sqrt{f_c'} + 0.3 f_{pc}\right)b'd + V_p \qquad (9\text{–}29) \qquad \text{[Equation 41]}$$

but d need not be taken less than 0.8h.

9.20.2.4 *For a pretensioned member in which the section at a distance h/2 from the face of support is closer to the end of the member than the transfer length of the prestressing tendons, the reduced prestress shall be considered when computing V_{cw}. The prestress force may be assumed to vary linearly from zero at the end of the tendon to a maximum at a distance from the end of the tendon equal to the transfer length, assumed to be 50 diameters for strand and 100 diameters for single wire.*

9.20.2.5 *The provisions for computing the shear strength provided by concrete, V_{ci} and V_{cw}, apply to normal-weight concrete. When lightweight aggregate concretes are used (see definition, concrete, structural lightweight, Article 8.1.3), one of the following modifications shall apply:*

(a) When f_{ct} is specified, the shear strength, V_{ci} and V_{cw}, shall be modified by substituting $f_{ct}/6.7$ for $\sqrt{f'_{ci}}$, but the value $f_{ct}/6.7$ used shall not exceed $\sqrt{f'_c}$.
(b) When f_{ct} is not specified, V_{ci} and V_{cw} shall be modified by multiplying each term containing $\sqrt{f'_c}$ by 0.75 for all-lightweight concrete, and 0.85 for sand-lightweight concrete. Linear interpolation may be used when partial sand replacement is used.

At higher compressive strengths, HSC is more brittle and shear cracks are smoother. As a result, there is less friction along the shear cracks and the concrete contribution to shear may be less. Consequently, the constants used in the equations need to be investigated.

The above article specifies a transfer length of 50 diameters for strands, whereas, the AASHTO LRFD Specifications specifies 60 diameters. Research data from the FHWA showcase bridges and FHWA research can be used to determine the appropriate number that will include HSC.[30] This is particularly important for 15.2-mm- (0.6-inch-) diameter strands used in most HSC beams.

ACTION: None. Further research is being conducted under NCHRP project 12-56.

9.20.3 Shear Strength Provided by Web Reinforcement

9.20.3.3 *The minimum area of web reinforcement shall be:*

$$A_v = \frac{50b's}{f_{sy}} \qquad \textit{(9-31)} \qquad \text{[Equation 42]}$$

where b' and s are in inches and f_{sy} is in psi.

The ACI 318 Building Code requires that the minimum area of shear reinforcement increase as concrete strength increases, but shall not be less than the value of A_v calculated by equation 9-31.[18] A similar modification to equation 9-31 should be considered.

ACTION: A revision for minimum reinforcement is proposed.

9.20.3.4 *The design yield strength of web reinforcement, f_{sy}, shall not exceed 60,000 psi.*

The use of a design yield strength higher than 414 MPa (60,000 psi) should be considered for both HSC and conventional concrete.

ACTION: A revision to allow higher design yield strengths is proposed.

9.21 POST-TENSIONED ANCHORAGE ZONES

9.21.7 Design of the Local Zone

9.21.7.2 BEARING STRENGTH

9.21.7.2.1 Anchorage devices may be either basic anchorage devices meeting the bearing compressive strength limits of Articles 9.21.7.2.2 through 9.21.7.2.4 or special anchorage devices meeting the requirements of section 9.21.7.3.

9.21.7.2.2 The effective concrete-bearing compressive strength f_b used for design shall not exceed that of Equations (9-39) or (9-40).

$$f_b \leq 0.7\,\phi\, f'_{ci}\,\sqrt{A / A_g} \qquad \textit{(9-39)} \qquad \text{[Equation 43]}$$

but

$$f_b \leq 2.25\,\phi\, f'_{ci} \qquad \textit{(9-40)} \qquad \text{[Equation 44]}$$

where

f_b = *the maximum factored tendon load, P_u, divided by the effective bearing area A_b;*

f'_{ci} = *the concrete compressive strength at stressing;*

A = *the maximum area of the portion of the supporting surface that is geometrically similar to the loaded area and concentric with it;*

A_g = *the gross area of the bearing plate if the requirements or Article 9.21.7.2.3 are met, or is the area calculated in accordance with Article 9.21.7.2.4;*

A_b = *the effective net area of the bearing plate calculated as the area A_g minus the area of openings in the bearing plate.*

Equations (9-39) and (9-40) are only valid if general zone reinforcement satisfying Article 9.21.3.4 is provided and if the extent of the concrete along the tendon axis ahead of the anchorage device is at least twice the length of the local zone as defined in Article 9.21.7.1.3

HSC is proportionally stronger in tension than conventional strength concrete (bearing failures are often splitting failures), but also more brittle. The suitability of the bearing equations needs to be verified for HSC.

ACTION: A research problem statement is proposed.

9.23 CONCRETE STRENGTH AT STRESS TRANSFER

Unless otherwise specified, stress shall not be transferred to concrete until the compressive strength of the concrete as indicated by test cylinders, cured by methods identical with the curing of the members, is at least 4,000 psi for pretensioned members (other than piles) and 3,500 psi for post-tensioned members and pretensioned piles.

Since HSC generates more heat of hydration than conventional strength concrete, it is important that test cylinders be cured at the same temperature as the member. Enclosing the cylinders under the same cover as the member does not ensure this, particularly when external steam or heat curing is not used. This article needs to be revised to be more specific.

ACTION: A revision referencing division II procedures is proposed.

9.26 COVER AND SPACING OF STEEL

9.26.1 Minimum Cover

The following minimum concrete cover shall be provided for prestressing and conventional steel:

9.26.1.1 *Prestressing Steel and Main Reinforcement.............................. 1½ inches*

9.26.1.2 *Slab Reinforcement*

9.26.1.2.1 Top of Slab .... 1½ inches
When deicers are used.. 2 inches

9.26.1.2.2 Bottom of Slab...1 inch

9.26.1.3 *Stirrups and Ties ....1 inch*

9.26.1.4 *When deicer chemicals are used, drainage details shall dispose of deicer solutions without constant contact with the prestressed girders. Where such contact cannot be avoided, or in locations where members are exposed to salt water, salt spray, or chemical vapor, additional cover should be provided.*

HPC is usually more impermeable than conventional concrete and a longer service life is expected.

ACTION: None.

9.28 EMBEDMENT OF PRESTRESSED STRAND

9.28.1 *Three- or seven-wire pretensioning strand shall be bonded beyond the critical section for a development length in inches not less than*

$$\left(f_{su}^{*} - \frac{2}{3} f_{se}\right) D \qquad (9\text{–}42) \qquad \text{[Equation 45]}$$

where D is the nominal diameter in inches, f_{su}^{*} *and* f_{se} *are in kips per square inch, and the parenthetical expression is considered to be without units.*

9.28.3 *Where strand is debonded at the end of a member and tension at service load is allowed in the precompressed tensile zone, the development length required above shall be doubled.*

Development lengths for the combination of 15.2-mm- (0.6-inch-) diameter strand used with HSC need to be evaluated based on FHWA and other research.(30)

ACTION: None. Further research is the objective of NCHRP project 12-60.

Section 17: SOIL-REINFORCED CONCRETE STRUCTURE INTERACTION SYSTEMS

This section covers buried reinforced concrete structures such as pipes and culverts. However, any articles that include concrete materials, stresses, or design refer back to section 8.

ACTION: None.

Division II: Construction

Section 8: CONCRETE STRUCTURES

8.2 CLASSES OF CONCRETE

This article provides definitions for normal-weight and lightweight concrete. A definition of HPC may need to be added as a means of identifying when special provisions for HPC apply.

ACTION: Revisions to add two classes of HPC are proposed.

8.3 MATERIALS

8.3.1 Cements

Portland Cements shall conform to the requirements of AASHTO M 85 (ASTM C 150) and Blended Hydraulic Cements shall conform to the requirements of AASHTO M 240 (ASTM C 595). For Type IP portland-pozzolan cement, the pozzolan constituent shall not exceed 20 percent of the weight of the blend and the loss on ignition of the pozzolan shall not exceed 5 percent.

Unless otherwise specified, only Types I, II, or III Portland Cement; Types IA, IIA, or IIIA Air-Entrained Portland Cement or Types IP or IS Blended Hydraulic Cements shall be used. Types IA, IIA, and IIIA cement may be used only in concrete where air entrainment is required.

Low-alkali cements conforming to the requirements of AASHTO M 85 for low-alkali cement shall be used when specified or when ordered by the Engineer as a condition of use for aggregates of limited alkali-silica reactivity.

Unless otherwise permitted, the product of only one mill of any one brand and type of cement shall be used for like elements of a structure that are exposed to view, except when cements must be blended for reduction of any excessive air entrainment where air-entraining cement is used.

By definition, HPC is a concrete where certain properties have been modified to increase performance. Restricting the cement to types I, II, III, IA, IIA, IIIA, IP, or IS may prevent the designer from using different types of cement to enhance concrete performance. For example, the use of type IV cement reduces the heat of hydration in high cement content HPC, and type V provides higher sulfate resistance. Cements conforming to ASTM C 1157, Standard Performance Specification for Blended Hydraulic Cement, should be included.

HPC is very sensitive to the brand, type, and mill of origin of the cement. Studies have shown that changing the brand of cement can cause great differences in the final hardened properties of HPC.[15] The final paragraph should be modified to include special restrictions for HPC.

Consideration should be given to addressing interaction effects between cement components and mineral or chemical admixtures.

ACTION: Revisions to add ASTM C 1157 and reference the two classes of HPC are proposed.

8.3.2 Water

Mixing water for concrete in which steel is embedded shall not contain a chloride ion concentration in excess of 1,000 ppm or sulfates such as SO_4 in excess of 1,300 ppm.

These limits should be evaluated for use with HPC and prestressed concrete.

ACTION: A research problem statement is proposed.

8.3.3 Fine Aggregates

Fine aggregate for concrete shall conform to the requirements of AASHTO M 6.

The coarse and fine aggregates in HPC should be blended together to obtain the maximum density. This will increase strength, decrease permeability, and lower the required cementitious materials content. The requirements of AASHTO M 6 may be too broad for HPC.

ACTION: A new specification for combined aggregates is proposed.

8.3.4 Coarse Aggregate

Coarse aggregate for concrete shall conform to the requirements of AASHTO M 80.

The coarse and fine aggregates in HPC should be blended together to obtain the maximum density. This will increase strength, decrease permeability, and lower the required cementitious materials content. The requirements of AASHTO M 80 may be too broad for HPC.

ACTION: A new specification for combined aggregates is proposed.

8.3.7 Mineral Admixtures

Fly ash pozzolans and calcined natural pozzolans for use as mineral admixtures in concrete shall conform to AASHTO M 295 (ASTM C 618).

The use of fly ash as produced by plants that utilize the limestone injection process or compounds of sodium, ammonium, or sulfur, such as soda ash, to control stack emissions shall not be used in concrete.

A Certificate of Compliance, based on test results and signed by the producer of the mineral admixture certifying that the material conforms to the above specifications, shall be furnished for each shipment used in the work.

This article should be modified to include other pozzolans and ground granulated blast-furnace slag (AASHTO M 307). Available research should also be checked to determine whether pozzolans from nontraditional sources, such as fly ash from petroleum coke or bark ash, can be used in HPC.

ACTION: A revision to include other materials is proposed.

8.4 PROPORTIONING OF CONCRETE

8.4.1 Mix Design

8.4.1.1 RESPONSIBILITY AND CRITERIA

The contractor shall design and be responsible for the performance of all concrete mixes used in structures. The mix proportions selected shall produce concrete that is sufficiently workable and finishable for all uses intended and shall conform to the requirements in table 8.2 and all other requirements of this section.

For normal-weight concrete, the absolute volume method, as described in American Concrete Institute Publication 211.1, shall be used in selecting the mix proportions. For structural lightweight concrete, the mix proportions shall be selected on the basis of trial mixes with the cement factor rather than the water/cement ratio being determined by the specified strength using methods such as those described in American Concrete Institute Publication 211.2.

The mix design shall be based upon obtaining an average concrete strength sufficiently above the specified strength so that, considering the expected variability of the concrete and test procedures, no more than 1 in 10 strength tests will be expected to fall below the specified strength. Mix designs shall be modified during the course of the work when necessary to ensure compliance with strength and consistency requirements.

HPC is often designed to have enhanced properties other than strength, thus a strength-based specification will not apply to many forms of HPC. This article should be modified to permit other properties to control the concrete mix design.

Table 8.2 needs to be extended to incorporate values for HPC.

This article requires the use of the absolute volume method for normal-weight concrete. For HPC and especially HSC, type, size, grading, and shape of aggregate require more attention than they do with conventional strength concrete mixtures. For HSC, a large amount and smaller nominal maximum size of coarse aggregate are generally required. HSC mixtures usually have a high cementitious materials content and a low water-cementitious materials ratio. For a given set of constituent materials, a decrease in the ratio results in an increase in compressive strength. However, when different materials are used, similar strengths can be achieved at different ratios. Consequently, HPC and HSC mix proportions should always be determined from trial batch tests.

Finally, HPC is very sensitive to changes in the constitutive materials. The mix design of HPC should NOT be changed during the job unless tests show that the material is failing to meet specifications or there is a change in any constitutive material. If changes to the mix design are required, trial batch tests should be required.

ACTION: Revisions to include the two classes of HPC are proposed.

8.4.1.2 TRIAL BATCH TESTS

For classes A, A(AE), and P concrete, for lightweight concrete, and for other classes of concrete when specified or ordered by the Engineer, satisfactory performance of the proposed mix design shall be verified by laboratory tests on trial batches. The results of such tests shall be furnished to the Engineer by the contractor or the manufacturer of the precast elements at the time the proposed mix design is submitted. For mix design approval, the strengths of a minimum of five test cylinders taken from a trial batch shall average at least 800 psi greater than the specified strength.

If materials and a mix design identical to those proposed for use have been used on other work within the previous year, certified copies of concrete test results from this work that indicate full compliance with these specifications may be substituted for such laboratory tests. If the results of more than 10 such strength tests are available from historical records for the past year, the average strength for these tests shall be at least 1.28 standard deviations above the specified strength.

HPC may be designed with an emphasis on properties other than strength. This article should be revised to allow acceptance of a mix based on properties other than strength.

For HSC, the requirement that the average strength be 5.5 MPa (800 psi) above the average may be too low. For example, 5.5 MPa (800 psi) is 20 percent over strength for 28-MPa (4000-psi) concrete, 10 percent over strength for 55-MPa (8000-psi) concrete, but only 7 percent over strength for 83-MPa (12,000-psi) concrete. Consideration should be given to adopting the revisions from ACI Committee 318-02, which have different requirements for conventional and high-strength concrete.[18]

ACTION: Revisions to the over-strength requirement are proposed.

8.4.3 Cement Content

The minimum cement content shall be as listed in table 8.2 or otherwise specified. The maximum cement or cement plus mineral admixture content shall not exceed 800 pounds per cubic yard of concrete. The actual cement content used shall be within these limits and shall be sufficient to produce concrete of the required strength and consistency.

HPC for use in massive bridge foundations may require less cement than the values indicated in table 8.2 to reduce the heat of hydration.

HSC often requires a total cementitious materials content greater than 475 kg/m^3 (800 lb/yd^3). Often, a maximum of 593 kg/m^3 (1000 lb/yd^3) is used as the limit in HSC. Revisions to this provision are required to accommodate HPC. Also, the cement content affects properties such as creep, shrinkage, permeability, etc. The last sentence should be modified to include a broader range of properties.

ACTION: A revision to increase the maximum cementitious materials content for HPC is proposed.

8.4.4 Mineral Admixtures

Mineral admixtures shall be used in the amounts specified. In addition, when either Types I, II, IV, or V (AASHTO M 85) cements are used and mineral admixtures are neither specified nor prohibited, the Contractor will be permitted to replace up to 20 percent of the required Portland cement with a mineral admixture. The weight of the mineral admixture used shall be equal to or greater than the weight of the Portland cement replaced. In calculating the water/cement ratio of the mix, the weight of the cement shall be considered to be the sum of the weights of the Portland cement and the mineral admixture.

HPC is very sensitive to the constitutive materials. Adding mineral admixtures when they are not specifically called for can adversely affect the concrete properties. This article should prohibit the use of mineral admixtures in HPC unless they are specified in the mix design or should require additional trial mixes whenever changes in the mix proportions are proposed. In addition, the maximum replacement percentage needs to be higher when fly ash and ground granulated blast-furnace slag are used and lower for silica fume. For HPC, it is more appropriate to consider total cementitious materials content and water-cementitious materials ratio rather than cement replacement. Changes in the wording should be considered.

ACTION: Revisions to permit larger percentages and other cementitious materials are proposed.

8.5 MANUFACTURE OF CONCRETE

8.5.4 Batching and Mixing Concrete

8.5.4.2 MIXING

The minimum drum revolutions for transit mixers at the mixing speed recommended by the manufacturer shall not be less than 70 and not less than that recommended by the manufacturer.

For HPC, a larger number of revolutions than 70 may be needed to ensure proper mixing of all constituent materials.

ACTION: None.

8.5.7 Evaluation of Concrete Strength

8.5.7.1 TESTS

A strength test shall consist of the average strength of two compressive strength test cylinders fabricated from material taken from a single randomly selected batch of concrete, except that if any cylinder should show evidence of improper sampling, molding, or testing, said cylinder shall be discarded and the strength test shall consist of the strength of the remaining cylinder.

ACI Committee 363 recommends three specimens for each strength test for HSC.[16]

Consideration should be given to revising this provision.

ACTION: A revision to require three specimens for HSC is proposed.

8.5.7.2 FOR CONTROLLING CONSTRUCTION OPERATIONS

For determining adequacy of cure and protection, and for determining when loads or stresses can be applied to concrete structures, test cylinders shall be cured at the structure site under conditions that are not more favorable than the most unfavorable conditions for the portions of the structure that they represent as described in Article 9.4 of AASHTO T 23. Sufficient test cylinders shall be made and tested at the appropriate ages to determine when operations such as release of falsework, application of prestressing forces, or placing the structure in service can occur.

Curing specimens according to the most unfavorable conditions may not be appropriate for high-strength precast, prestressed concrete. A lower temperature for the cylinder will produce a lower compressive strength for the cylinder at release age than is achieved by higher temperatures in the precast member. However, this can result in the cylinder having a higher strength than that of the precast member at later ages. The use of the match-curing technique should be considered for precast, prestressed concrete. Generally, this requires the use of 102- by 203-mm (4- by 8-inch) cylinders. The use of 102- by 203-mm (4- by 8-inch) cylinders should be evaluated. In addition, specifications should be developed for the match-curing system.

ACTION: Revisions to section 8.5.7.5 are proposed.

8.5.7.3 FOR ACCEPTANCE OF CONCRETE

For determining compliance of concrete with a specified 28-day strength, test cylinders shall be cured under controlled conditions as described in Article 9.3 of AASHTO T 23 and tested at the age of 28 days. Samples for acceptance tests for each class of concrete shall be taken not less than once a day nor less than once for each 150 cubic yards of concrete or once for each major placement.

Any concrete represented by a test which indicates a strength which is less than the specified 28-day compressive strength by more than 500 psi will be rejected and shall be removed and replaced with acceptable concrete. Such rejection shall prevail unless either:

(1) The Contractor, at his or her expense, obtains and submits evidence of a type acceptable to the Engineer that the strength and quality of the rejected concrete is acceptable. If such evidence consists of cores taken from the work, the cores shall be obtained and tested in accordance with the standard methods of AASHTO T 24 (ASTM C 42) or,

(2) The Engineer determines that said concrete is located where it will not create an intolerable detrimental effect on the structure and the Contractor agrees to a reduced payment to compensate the Department for loss of durability and other lost benefits.

HPC often contains pozzolans, which hydrate more slowly. In this case, requirements for testing at 28 days may not be appropriate.

The rejection criteria of 3.5 MPa (500 psi) should be re-examined for HSC; 3.5 MPa (500 psi) is 13 percent of the strength for a 28-MPa (4000-psi) concrete, but only 5 percent of the strength for a 69-MPa (10,000-psi) mix. Since a premium price is often paid for HPC, consideration should be given to rejecting concrete that does not meet the specification. A reduced payment cannot compensate for a loss of durability and possible reduced service life.

ACTION: Revisions to include the two classes of HPC are proposed.

8.5.7.5 STEAM AND RADIANT HEAT-CURED CONCRETE

When a precast concrete member is steam or radiant heat cured, the compressive strength test cylinders made for any of the above purposes shall be cured under conditions similar to the member. Such concrete will be considered to be acceptable whenever a test indicates that the concrete has reached the specified 28-day compressive strength provided such strength is reached not more than 28 days after the member is cast.

Consideration needs to be given for strengths specified at ages other than 28 days and to the use of match-cured cylinders. For HPC, curing cylinders under similar conditions may not be sufficient. Match curing provides a more realistic condition. This, generally, requires the use of 102- by 203-mm (4- by 8-inch) cylinders. The use of 102- by 203-mm (4- by 8-inch) cylinders should be evaluated. In addition, specifications should be developed for the match-curing system.

ACTION: Revisions to include match curing are proposed.

8.6 PROTECTION OF CONCRETE FROM ENVIRONMENTAL CONDITIONS

8.6.4 Cold Weather Protection

8.6.4.1 PROTECTION DURING CURE

When there is a probability of air temperatures below 35 ^{o}F during the cure period, the Contractor shall submit for approval by the Engineer prior to concrete placement, a cold weather concreting and curing plan detailing the methods and equipment which will be used to assure that the required concrete temperatures are maintained. The concrete shall be maintained at a temperature of not less than 45 ^{o}F for the first six days after placement, except when pozzolan cement or fly ash cement is used, this period shall be as follows:

Percentage of Cement Replaced, by Weight With Pozzolans	***Required Period of Controlled Temperature***
10%	*8 days*
11–15%	*9 days*
16–20%	*10 days*

The above requirement for an extended period of controlled temperature may be waived if a compressive strength of 65 percent of the specified 28-day design strength is achieved.

This provision only addresses pozzolans. It needs to include other mineral admixtures and higher percentages.

The above table requires a controlled temperature period of 8 days or until 65 percent of the specified strength is achieved when 10 percent pozzolans are used. Article 8.11.1 requires a curing period of 10 days or until 70 percent of the specified strength is achieved when pozzolans in excess of 10 percent are used. These two articles need to be consistent.

In lieu of fixed periods of controlled temperature, the match-curing method or maturity method could be used. The potential for using these methods needs to be evaluated.

ACTION: Revisions to include slag and higher percentages of cement replacement are proposed.

8.6.6 Concrete Exposed to Salt Water

8.6.7 Concrete Exposed to Sulfate Soils or Water

These two articles provide special considerations for concrete exposed to specific environmental conditions. These are ideal applications for HPC and the provisions need to be revised accordingly.

ACTION: Revisions to require HPC are proposed.

8.11 CURING OF CONCRETE

8.11.1 General

All newly placed concrete shall be cured so as to prevent the loss of water by use of one or more of the methods specified herein. Curing shall commence immediately after the free water has left the surface and finishing operations are completed. If the surface of the concrete begins to dry before the selected cure method can be applied, the surface shall be kept moist by a fog spray applied so as not to damage the surface.

Curing by other than steam or radiant heat methods shall continue uninterrupted for 7 days except that when pozzolans in excess of 10 percent, by weight, of the Portland cement are used in the mix. When such pozzolans are used, the curing period shall be 10 days. For other than top slabs of structures serving as finished pavements, the above curing periods may be reduced and curing terminated when test cylinders cured under the same conditions as the structure indicate that concrete strengths of at least 70 percent of that specified have been reached.

When deemed necessary by the Engineer during periods of hot weather, water shall be applied to concrete surfaces being cured by the liquid membrane method or by the forms-in-place method until the Engineer determines that a cooling effect is no longer required. Such application of water will be paid as extra work.

HPC tends to have very little bleed water, especially when a low water-cementitious materials ratio is used with mineral admixtures. As a result, the evaporation protection of the bleed water on the fresh concrete is lost. To prevent plastic shrinkage cracking, this article should require methods to retard or prevent evaporation of bleed water from HPC during placement and finishing.

The lack of bleed water makes the forms-in-place method (8.11.3.1), the liquid membrane curing compound method (8.11.3.3), and the waterproof cover method (8.11.3.4) of curing ineffective for HPC. Also, the low water-cementitious materials ratios in some HPC mixes make the addition of external water desirable. For HPC, only the water method (8.11.3.2) or steam curing (8.11.3.5) should be allowed. In addition, another article needs to be added for concrete that is heat cured from its own heat of hydration.

This article specifies curing times and allows curing to be discontinued in certain members when the concrete achieves a compressive strength equal to 70 percent of the specified strength. This approach was developed for conventional concretes, but may not be appropriate for HPC. For HPC with low water-cementitious materials ratios, external water may be needed to keep the hydration process going. The curing times and percentages need to be evaluated for use with HPC. The curing period of 10 days or until 70 percent of the specified strength is reached needs to be consistent with the requirements of 8.6.4.1.

HPC tends to have larger quantities of cement and, therefore, higher heat of hydration. The use of curing water to cool the concrete may be advisable even in cooler weather. In addition, concrete temperatures during placement and curing should be monitored to avoid excessive temperatures and excessive temperature gradients. Limits on concrete temperatures may need to be specified.

ACTION: Revisions to add curing procedures for HPC are proposed.

8.11.3 Methods

8.11.3.3 LIQUID MEMBRANE CURING COMPOUND METHOD

If the solution is applied in two increments, the second application shall follow the first application within 30 minutes.

The second application should be applied in a direction perpendicular to the direction of the first application.

ACTION: None.

8.11.3.5 STEAM OR RADIANT HEAT CURING METHOD

This method may be used only for precast concrete members manufactured in established plants.

Steam curing or radiant heat curing shall be done under a suitable enclosure to contain the live steam or the heat. Steam shall be low pressure and saturated. Temperature recording devices shall be employed as necessary to verify that temperatures are uniform throughout the enclosure and are within the limits specified.

Since HSC generates significantly more heat than conventional strength concrete, it is important that concrete temperatures be monitored rather than temperatures throughout the enclosure. Otherwise, the concrete temperatures could exceed 100 ^{o}C (212 ^{o}F), while the enclosure temperature is at 71 ^{o}C (160 ^{o}F). In addition, it may be desirable to specify a maximum internal concrete temperature and maximum temperature differentials.

ACTION: Revisions to require measurement of concrete temperatures are proposed.

Unless the ambient temperature is maintained above 60 ^{o}F, for prestressed members the transfer of the stressing force to the concrete shall be accomplished immediately after the steam curing or heat curing has been discontinued.

A temperature of 16 ^{o}C (60 ^{o}F) may be too low for HSC. It would seem that transfer has to occur immediately for all ambient temperatures with HSC.

ACTION: A revision to require immediate transfer is proposed.

8.11.4 Bridge Decks

The top surfaces of bridge decks shall be cured by a combination of the liquid membrane curing compound method and the water method. The liquid membrane shall be Type 2, white pigmented, and shall be applied from finishing bridges progressively and immediately after finishing operations are complete on each portion on the deck. The water cure shall be applied not later than 4 hours after completion of deck finishing or, for portions of the decks on which finishing is completed after normal working hours, the water cure shall be applied not later than the following morning.

This article requires the use of both the liquid membrane method and the water methods for bridge decks. For HPC, it is essential that water cure begin as soon as the concrete finishing is complete and that the water be able to reach the concrete. Delaying the application of water until the following morning is not acceptable.

This provision should also allow the use of membrane curing after the water-curing period.

ACTION: A revision to require water curing with HPC is proposed.

8.13 PRECAST CONCRETE MEMBERS

8.13.4 Curing

Unless otherwise permitted, precast members shall be cured by either the water method or the steam or radiant heat method.

For HSC, there needs to be a method for curing by covering the members to retain heat and moisture, with or without the use of insulating blankets.

ACTION: A revision to include the waterproof cover method is proposed.

AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS

The compilation in this section is based on the *AASHTO LRFD Bridge Design Specifications*, Second Edition, 1998, and the 1999, 2000, and 2001 interim revisions.[7-10] This section only lists articles affected by HPC. For each listed article, the portion affected by HPC is shown in italics, followed by specific comments in regular font. For long articles, only a synopsis, followed by comments, is included. References in the comments to specific sections, articles, or tables refer to the document being reviewed and not the sections, articles, or tables of this report. The end result of the project is stated under the action item. Proposed revisions are included in appendix D. Research problem statements are included in appendix F.

Section 5: CONCRETE STRUCTURES

5.1 SCOPE

The provisions in this section apply to the design of bridge and retaining wall components constructed of normal density or lightweight concrete and reinforced with steel bars and/or prestressing strands or bars. The provisions are based on concrete strengths varying from 2.4 ksi to 10.0 ksi.

The scope should be extended to concrete strengths higher than 70 MPa (10.0 kips/inch^2 (ksi)). It should be made clear in the scope that welded wire reinforcement is included in this section. The scope should be expanded to include prestressing wire. Both welded wire reinforcement and prestressing wire have use in HPC.

ACTION: Revisions to include welded wire reinforcement and design for strengths above 70 MPa (10.0 ksi) are proposed.

5.2 DEFINITIONS

A definition of HPC should be included in this article to facilitate the introduction of provisions about HPC.

ACTION: HPC concretes are proposed for the *AASHTO LRFD Bridge Construction Specifications*.

Normal-Weight Concrete: *Concrete having a weight between 0.135 and 0.155 kcf.*

The definition should be expanded for unit weights greater than 2.48 Mg/m^3 (0.155 kips per cubic foot (kcf)), which can occur with HPC.

ACTION: None. Existing data do not justify a revision.

5.3 NOTATION

f_c' = *specified compressive strength of concrete at 28 days, unless another age is specified (ksi) (5.4.2.1).*

Since strengths are frequently specified at ages other than 28 days for HSC, rewording of this article should be considered.

ACTION: A revision to delete 28 days is proposed.

5.4 MATERIAL PROPERTIES

5.4.1 General

C5.4.1

Occasionally, it may be appropriate to use materials other than those included in the AASHTO LRFD Bridge Construction Specifications; for example, when concretes are modified to obtain very high strengths through the introduction of special admixtures, such as:

- *Silica fume,*
- *Cements other than Portland or blended hydraulic cements,*
- *Proprietary high early-strength cements, and*
- *Other types of reinforcing materials.*

In these cases, the properties of such materials should be established by a specified testing program.

Fly ash, ground granulated blast-furnace slag, and metakaolin should be added to the list of materials.

The different test programs to achieve HPC should be defined.

ACTION: A revision to include slag is proposed.

5.4.2 Normal and Structural Lightweight Concrete

5.4.2.1 COMPRESSION STRENGTH

Concrete strengths above 10.0 ksi shall be used only when physical tests are made to establish the relationships between concrete strength and other properties.

The upper limit of 70 MPa (10.0 ksi) needs to be removed to the extent possible to permit the greater use of HSC.

ACTION: A revision to remove the 70-MPa (10.0-ksi) restriction in specific articles is proposed.

The sum of Portland cement and other cementitious materials shall be specified not to exceed 800 pcy.

Although an upper limit may be appropriate for conventional strength concrete, it should be removed for HSC, which frequently contains more than 475 kg/m^3 (800 lb/yd^3) of cementitious materials. At the same time, excessive use of cementitious materials should be avoided.

ACTION: A revision to increase the maximum cementitious materials content is proposed.

Table C5.4.2.1-1 Concrete Mix Characteristics by Class

This table needs to be extended to incorporate values for HPC.

ACTION: Revisions to add two classes of HPC are proposed.

5.4.2.3 SHRINKAGE AND CREEP

5.4.2.3.1 General

In the absence of more accurate data, the shrinkage coefficients may be assumed to be 0.0002 after 28 days and 0.0005 after 1 year of drying.

The final shrinkage strain differs significantly from the 28-day shrinkage strain. However, the final value, 0.0005, may not be appropriate for HPC. The AASHTO Standard Specifications uses a single number, 0.0002, for shrinkage strain. The conditions under which the stated shrinkage strains are applicable need to be defined. The appropriate values for HPC need to be determined.

ACTION: Revisions based on NCHRP project 18-07 are proposed.

5.4.2.3.2 Creep and *5.4.2.3.3 Shrinkage*

These articles provide equations for the calculation of creep and shrinkage that are based on the recommendations of ACI Committee 209 as modified by additional published data.

Depending on the constituents used to make HPC, the creep and shrinkage strain can be different from the values given by the equations. The equations need to be modified to include creep and shrinkage of HPC with its different constituent materials.

Depending on the curing conditions for the concrete, the creep and shrinkage strain can vary. High early compressive strength is important for HSC to achieve early release of the pretensioning force. In most cases, this is achieved by applying heat or steam curing. This affects the creep of the concrete and needs to be included in the equation for creep strain. It is anticipated that information about creep will be developed as part of NCHRP project 18-07. This information will need to be incorporated into this article.

ACTION: Revisions based on NCHRP project 18-07 are proposed.

5.4.2.4 MODULUS OF ELASTICITY

In the absence of more precise data, the modulus of elasticity, E_c, for concretes with unit weights between 0.090 and 0.155 kcf may be taken as:

$$E_c = 33{,}000\ w_c^{1.5}\sqrt{f_c'} \qquad (5.4.2.4\text{-}1) \qquad \text{[Equation 46]}$$

where

w_c = unit weight of concrete (kcf)

f_c' = specified strength of concrete (ksi)

Equation 5.4.2.4-1 for the modulus of elasticity may not be appropriate for HSC.(15) The stress-strain behavior of HPC is different than that for conventional strength concrete. There are data that suggest that the E_c for HSC may be influenced by aggregate stiffness.(26) Furthermore, some HSCs have a unit weight greater than 2.48 Mg/m^3 (155 lb/ft^3). Thus, the equation for E_c in this article needs to be evaluated using recent data.

ACTION: A revision based on NCHRP project 18-07 is proposed.

5.4.2.6 MODULUS OF RUPTURE

Unless determined by physical tests, modulus of rupture, f_r, in ksi, may be taken as:

- *For normal-weight concrete .... $0.24\sqrt{f_c'}$*
- *For sand-lightweight concrete.. $0.20\sqrt{f_c'}$*
- *For all-lightweight concrete .... $0.17\sqrt{f_c'}$*

The factor in front of $\sqrt{f_c'}$ should be verified for HPC. A higher coefficient than 0.24 is possible for HSC. However, the coefficient seems to depend on the specific materials in the concrete. Information on the specific materials for a given project may not be available at the design stage. Consequently, the limit needs to be a conservative value.

ACTION: Revisions for normal-weight concrete are proposed. A research problem statement is proposed for other weights of concrete.

5.4.2.7 TENSILE STRENGTH

C5.4.2.7

For most regular concretes, the direct tensile strength may be estimated as $f_r = 0.23\sqrt{f_c'}$.

For HSC, a coefficient greater than 0.23 may be possible.

ACTION: A research problem statement is proposed.

5.4.6 Ducts

5.4.6.2 SIZE OF DUCTS

The size of ducts shall not exceed 0.4 times the least gross concrete thickness at the duct.

With HSC precast concrete I-girders, thinner webs are often used to maximize section efficiency. Consideration should be given to allowing larger ducts in the webs of HSC members.

ACTION: A revision to eliminate this provision for precast, pretensioned beams is proposed.

5.5 LIMIT STATES

5.5.3 Fatigue Limit State

5.5.3.1 GENERAL

Fatigue need not be investigated for concrete deck slabs in multigirder applications.

With HSC girders and wider girder spacing, it may be necessary to investigate concrete deck slabs for fatigue.

ACTION: None.

The section properties for fatigue investigations shall be based on cracked sections where the sum of stresses, due to unfactored permanent loads and prestress, and 1.5 times the fatigue load is tensile and exceeds $0.095\sqrt{f_c'}$.

The tensile stress limit in the last paragraph should be investigated for its applicability with HPC.

ACTION: None.

5.5.4 Strength Limit State

5.5.4.2 RESISTANCE TO FACTORS

5.5.4.2.1 Conventional Construction

Resistance factor φ *shall be taken as:*

- *For flexure and tension of reinforced concrete 0.90*

- *For flexure and tension of prestressed concrete....................................1.00*

- *For shear and torsion:*
  - *normal-weight concrete 0.90*
  - *lightweight concrete...0.70*

- *For axial compression with spirals or ties, except as specified in Article 5.10.11.4.1b for Seismic Zones 3 and 4 at the extreme event limit state..........................0.75*

- *For bearing on concrete 0.70*

- *For compression in strut-and-tie models...0.70*

- *For compression in anchorage zones:*
  - *normal-weight concrete 0.80*
  - *lightweight concrete...0.65*

- *For tension in steel anchorage zones...1.00*

- *For resistance during pile driving...1.00*

These resistance factors have been developed for conventional concrete. HPC tends to be very sensitive to water contents and constitutive materials. The chance of understrength concrete may increase, especially at very high compressive strength levels. On the other hand, HPC is produced with stricter quality control and a lower coefficient of variation than conventional concrete. Also, HSC has less lateral expansion than conventional strength concrete, so the effect of confinement is less. This affects column behavior. Therefore, there is a need to verify the suitability of the given resistance factors for HPC, especially HSC.

ACTION: Revisions to include strength design for pretensioned concrete members at release are proposed. A research problem statement is proposed to address resistance factors.

5.6 DESIGN CONSIDERATIONS

5.6.3 Strut-and-Tie Model

5.6.3.3 PROPORTIONING OF COMPRESSIVE STRUTS

5.6.3.3.3 Limiting Compressive Stress in Strut

The limiting compressive stress, f_{cu}, shall be taken as:

$$f_{cu} = \frac{f'_c}{0.8 + 170\varepsilon_1} \leq 0.85 f'_c \qquad (5.6.3.3.3\text{-}1) \qquad \text{[Equation 47]}$$

for which

$$\varepsilon_l = \varepsilon_s + (\varepsilon_s + 0.002) \cot^2 \alpha_s \qquad (5.6.3.3.3\text{-}2) \qquad \text{[Equation 48]}$$

where

α_s = the smallest angle between the compressive strut and adjoining tension ties (deg.)

ε_s = the tensile strain in the concrete in the direction of the tension tie (inches/inch)

f'_c = specified compressive strength (ksi)

The appropriateness of the limiting compressive stress should be verified for HSC.

ACTION: A research problem statement is proposed.

5.6.3.5 PROPORTIONING OF NODE REGIONS

Unless confining reinforcement is provided and its effect is supported by analysis or experimentation, the concrete compressive stress in the node regions of the strut shall not exceed:

- *For node regions bounded by compressive struts and bearing areas: $0.85\varphi f'_c$*
- *For node regions anchoring a one-direction tension tie: $0.75\varphi f'_c$*
- *For node regions anchoring tension ties in more than one direction: $0.65\varphi f'_c$*

The appropriateness of the maximum values of the concrete compressive stress in the node regions should be verified for HSC.

ACTION: A research problem statement is proposed.

5.6.3.6 CRACK CONTROL REINFORCEMENT

Structures and components or regions thereof, except for slabs and footings, which have been designed in accordance with the provisions of Article 5.6.3, shall contain an orthogonal grid of reinforcing bars near each face. The spacing of the bars in these grids shall not exceed 12.0 inches.

The ratio of reinforcement area to gross concrete area shall not be less than 0.003 in each direction.

Since HSC has a higher tensile strength than conventional strength concretes, the minimum reinforcement ratio needs to increase as the concrete compressive strength increases. A revision to this article should be considered.

ACTION: A research problem statement is proposed.

5.7 DESIGN FOR FLEXURAL AND AXIAL FORCE EFFECTS

5.7.1 Assumptions for Service and Fatigue Limit States

The following assumptions may be used in the design of reinforced, prestressed, and partially prestressed concrete components:

- *The modular ratio is not less than 6.0.*

An effective modular ratio of 2n is applicable to permanent loads and prestress.

The modular ratio of concrete is a function of modulus of elasticity of concrete, which is a function of the concrete compressive strength. HSC will frequently result in a modular ratio that is less than 6.0. Therefore, the validity of limiting the modular ratio, n, to 6.0 needs to be evaluated.

Traditional methods of prestressed concrete design do not use an effective modular ratio of $2n$ for permanent loads and prestress. The validity of this article needs to be verified for HSC.

ACTION: A revision to use the actual value of n is proposed.

5.7.2 Assumptions for Strength and Extreme Event Limit States

5.7.2.1 GENERAL

Factored resistance of concrete components shall be based on the conditions of equilibrium and strain compatibility, the resistance factors as specified in Article 5.5.4.2, and the following assumptions:

- *If the concrete is confined, a maximum usable strain exceeding 0.003 may be utilized if verified.*

- *The concrete compressive stress-strain distribution is assumed to be rectangular, parabolic, or any other shape that results in a prediction of strength in substantial agreement with the test results.*

This article defines the assumption for calculating flexural resistance of concrete members. An assessment should be made to determine if the maximum useable strain of 0.003 is applicable for HSC and if it is appropriate to assume any shape for the compressive stress-strain distribution.

ACTION: None. Further research is the objective of NCHRP project 12-64.

5.7.2.2 RECTANGULAR STRESS DISTRIBUTION

The natural relationship between concrete stress and strain may be considered satisfied by an equivalent rectangular concrete compressive stress block of 0.85 $f_c^{'}$ over a zone bounded by the edges of the cross-section and a straight line located parallel to the neutral axis at the distance $a = \beta_1 c$ from the extreme compression fiber. The distance c shall be measured perpendicular to the neutral axis. The factor β_1 shall be taken as 0.85 for concrete strengths not exceeding 4.0 ksi. For concrete strengths exceeding 4.0 ksi, β_1 shall be reduced at a rate of 0.05 for each 1.0 ksi of strength in excess of 4.0 ksi, except that β_1 shall not be taken less than 0.65.

The stress-strain curve for HSC is more linear than for conventional strength concrete. However, the stress block factors are generally considered to be still valid for members where flexure predominates. For members where axial compression predominates, the concrete stress of 0.85 $f_c^{'}$ may need to be reduced as concrete strength increases.[27] In the *Canadian Standard for Design of Concrete Structures*, the 0.85 factor is replaced by $(0.85 - 0.0015 f_c^{'}) \geq 0.067$, in which $f_c^{'}$ is in megapascals.[28] A review is needed to determine if the rectangular stress block and factors are valid with HSC.

ACTION: None. Further research is the objective of NCHRP project 12-64.

5.7.3 Flexural Members

5.7.3.1 STRESS IN PRESTRESSING STEEL AT NOMINAL FLEXURAL RESISTANCE

5.7.3.1.1 Components With Bonded Tendons

For rectangular or flanged sections subjected to flexure about one axis where the approximate stress distribution specified in Article 5.7.2.2 is used and for which f_{pe} is not less than 0.5 f_{pu}, the average stress in prestressing steel, f_{ps}, may be taken as:

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right) \qquad (5.7.3.1.1\text{-}1) \qquad \text{[Equation 49]}$$

for which

$$k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right) \qquad (5.7.3.1.1\text{-}2) \qquad \text{[Equation 50]}$$

for T-section behavior

$$c = \frac{A_{ps} f_{pu} + A_s f_y - A_s^{'} f_y^{'} - 0.85 \beta_1 f_c^{'} (b - b_w) h_f}{0.85 f_c^{'} \beta_1 b_w + k A_{ps} \dfrac{f_{pu}}{d_p}} \qquad (5.7.3.1.1\text{-}3) \qquad \text{[Equation 51]}$$

for rectangular section behavior

$$c = \frac{A_{ps} f_{pu} + A_s f_y - A_s^{'} f_y^{'}}{0.85 f_c^{'} \beta_1 b + k A_{ps} \frac{f_{pu}}{d_p}} \qquad \textit{(5.7.3.1.1-4)} \quad \text{[Equation 52]}$$

The equations in this article are based on the assumption of a rectangular stress block as defined in article 5.7.2.2. If a different stress distribution is required for HSC, these equations may need to be revised or their application restricted to lower concrete strengths.

The LRFD method relates β_1 to the concrete area in compression rather than the neutral axis depth. The results are consistent only if the compression zone has a uniform width, as in a rectangular section. Significant differences may arise for sections with non-rectangular geometry. This can be seen from equation 5.7.3.1.1-3.

Also, equation 5.7.3.1.1-1 of the LRFD provides prestressing steel stress at ultimate as a function of c. This requires iteration, since c is a function of f_{ps}. Combining equation 5.7.3.1.1-1 and equation 5.7.3.1.1-3, and by carrying on further mathematical manipulation, the equations can be written as follows:

$$\beta_1 \left(0.85 f_c^{'}\right)\left[b_w c + (b - b_w) h_f\right] = A_{ps} f_{ps} + A_s f_y - A_s^{'} f_y \qquad \text{[Equation 53]}$$

or

β_1 (0.85 $f_c^{'}$) (total compression area bounded by neutral axis) = (total tensile force in reinforcement)

These equations may considerably overestimate the neutral axis depth, c, and need to be evaluated for use with HSC.

ACTION: None. Further research is the objective of NCHRP project 12-64.

5.7.3.2 FLEXURAL RESISTANCE

5.7.3.2.2 Flanged Sections

For flanged sections subjected to flexure about one axis and for biaxial flexure with axial load as specified in Article 5.7.4.5, where the approximate stress distribution specified in Article 5.7.2.2 is used and the tendons are bonded, and where the compression flange depth is less than c, as determined in accordance with Equation 5.7.3.1.1-3, the nominal flexural resistance may be taken as:

$$M_n = A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right) + A_s f_y\left(d_s - \frac{a}{2}\right) - A_s^{'} f_y^{'}\left(d_s^{'} - \frac{a}{2}\right) + 0.85 f_c^{'}(b - b_w)\beta_1 h_f\left(\frac{a}{2} - \frac{h_f}{2}\right) \qquad \textit{(5.7.3.2.2-1)} \quad \text{[Equation 54]}$$

LRFD specifies a T-section behavior if $c > h_f$. This is inconsistent with the traditional definition in the ACI 318 Building Code and the AASHTO Standard Specifications where a section is considered a T-section if $a > h_f$. The impact of this article needs to be evaluated for use with HSC.

ACTION: None. Further research is the objective of NCHRP project 12-64.

5.7.3.3 LIMITS FOR REINFORCEMENT

5.7.3.3.1 Maximum Reinforcement

The maximum amount of prestressed and nonprestressed reinforcement shall be such that:

$$\frac{c}{d_e} \leq 0.42 \qquad (5.7.3.3.1\text{-}1) \qquad \text{[Equation 55]}$$

for which

$$d_e = \frac{A_{ps} f_{ps} d_p + A_s f_y d_s}{A_{ps} f_{ps} + A_s f_y} \qquad (5.7.3.3.1\text{-}2) \qquad \text{[Equation 56]}$$

The appropriateness of these equations for use with HSC needs to be evaluated.

ACTION: None. Further research is the objective of NCHRP project 12-64.

5.7.3.3.2 Minimum Reinforcement

Unless otherwise specified, at any section of a flexural component, the amount of prestressed and nonprestressed tensile reinforcement shall be adequate to develop a factored flexural resistance, M_r, at least equal to the lesser of:

- *1.2 times the cracking strength determined on the basis of elastic stress distribution and the modulus of rupture, f_r, of the concrete as specified in Article 5.4.2.6, or*

- *1.33 times the factored moment required by the applicable strength load combinations specified in table 3.4.1-1.*

The provisions of Article 5.10.8 shall apply.

The purpose of this article is to ensure that the section does not go to the ultimate strength state as soon as it cracks. HSC is known to have proportionately higher tensile strength than conventional strength concrete. This means that the actual value of cracking strength is higher that that calculated using $0.24\sqrt{f_c^{'}}$ for the modulus of rupture. Therefore, any factor of safety provided by this article would be lost. Revision to the equation for the modulus of rupture and/or the 1.2 factor may be needed. With high-strength, post-tensioned concrete I-girder bridges, the requirement of 1.2 times the cracking moment may result in excessive minimum reinforcement. The cracking moment is relatively large in this application because of the large total prestressing force. It is conceivable that 1.2 times the cracking moment may be very close to or even higher than the required factored load moment M_u. Thus, the section may not even crack under factored load. The problem is further complicated by the fact that in some applications, the section is reinforced to its maximum limit. Thus, 1.33 times the factored moment would make the section significantly over-reinforced. Consequently, the extra reinforcement required to satisfy the minimum reinforcement limit would be largely ineffective. A revision is needed to address this situation.

ACTION: Revisions to article 5.4.2.6 are proposed.

5.7.3.6 DEFORMATIONS

5.7.3.6.2 Deflection and Camber

Unless a more exact determination is made, the long-time deflection may be taken as the instantaneous deflection multiplied by the following factor:

- *If the instantaneous deflection is based on I_g: 4.0*
- *If the instantaneous deflection is based on I_e: $3.0-1.2(A_s'/A_s) > 1.6$*

HSC usually has lower creep than conventional strength concrete, so long-term deflection multipliers may be less. Long-term deflection factors were developed for conventional strength concrete and need to be verified for use with HSC.

An approach similar to that of ACI 318 could be adopted.[18] However, the ACI factors may also need to be modified for use with HSC.

ACTION: A research problem statement is proposed.

5.7.4 Compression Members

5.7.4.2 LIMITS FOR REINFORCEMENT

The maximum area of prestressed and nonprestressed longitudinal reinforcement for noncomposite compression components shall be such that:

$$\frac{A_s}{A_g} + \frac{A_{ps} f_{pu}}{A_g f_y} \leq 0.08 \qquad (5.7.4.2\text{-}1) \qquad \text{[Equation 57]}$$

and

$$\frac{A_{ps} f_{pe}}{A_g f_c^{'}} \leq 0.30 \qquad \textit{(5.7.4.2-2)} \qquad \text{[Equation 58]}$$

The minimum area of prestressed and nonprestressed longitudinal reinforcement for noncomposite compression components shall be such that:

$$\frac{A_s f_y}{A_g f_c^{'}} + \frac{A_{ps} f_{pu}}{A_g f_c^{'}} \geq 0.135 \qquad \textit{(5.7.4.2-3)} \qquad \text{[Equation 59]}$$

This article provides maximum and minimum reinforcement limits for compression members. These equations are different than those that appear in the AASHTO Standard Specifications. The limits given by these equations should be evaluated for use with HSC.

ACTION: None. Further research is the objective of NCHRP project 12-64.

5.7.4.4 FACTORED AXIAL RESISTANCE

The factored axial resistance of reinforced concrete compressive components, symmetrical about both principal axes, shall be taken as:

$$P_r = \varphi P_n \qquad \textit{(5.7.4.4-1)} \qquad \text{[Equation 60]}$$

for which

- *For members with spiral reinforcement:*

$$P_n = 0.85[0.85\, f_c^{'}\, (A_g - A_{st}) + f_y\, A_{st}] \qquad \textit{(5.7.4.4-2)} \qquad \text{[Equation 61]}$$

- *For members with tie reinforcement:*

$$P_n = 0.80[0.85\, f_c^{'}\, (A_g - A_{st}) + f_y\, A_{st}] \qquad \textit{(5.7.4.4-3)} \qquad \text{[Equation 62]}$$

HSC has less lateral expansion than conventional strength concrete, so the confinement effect is less. This affects column behavior. The constants used in equations 5.7.4.4-2 and 5.7.4.4-3 need to be evaluated for use with HSC.

ACTION: None. Further research is the objective of NCHRP project 12-64.

5.7.4.6 SPIRALS AND TIES

Where the area of spiral and tie reinforcement is not controlled by:

- *Seismic requirements,*

- *Shear or torsion as specified in Article 5.8, or*

- *Minimum requirements as specified in Article 5.10.6,*

the ratio of spiral reinforcement to total volume of concrete core, measured out-to-out of spirals, shall not be less than:

$$\rho_s = 0.45 \left(\frac{A_g}{A_c} - 1 \right) \frac{f_c^{'}}{f_{yh}} \qquad \textit{(5.7.4.6-1)} \qquad \text{[Equation 63]}$$

Spirals are less effective for confinement in HSC. Another formula is reported by ACI Committee 363 and should be considered.(15) In addition, the ratio of reinforcement required by equation 5.7.4.6-1 may be too high to be practical with HSC. The concept for providing spiral reinforcement to strengthen the core to offset the loss of strength when the concrete shell is lost may not be appropriate for HSC.

ACTION: A research problem statement is proposed.

5.7.5 Bearing

In the absence of confinement reinforcement in the concrete supporting the bearing device, the factored bearing resistance shall be taken as:

$$P_r = \varphi P_n \qquad \textit{(5.7.5-1)} \qquad \text{[Equation 64]}$$

for which

$$P_n = 0.85\, f_c^{'} A_1 m \qquad \textit{(5.7.5-2)} \qquad \text{[Equation 65]}$$

The coefficient of 0.85 needs to be verified for HSC.

ACTION: A research problem statement is proposed.

5.8 SHEAR AND TORSION

5.8.2 General Requirements

5.8.2.2 MODIFICATIONS FOR LIGHTWEIGHT CONCRETE

Where lightweight aggregate concretes are used, the following modifications shall apply in determining resistance to torsion and shear:

- *Where the average splitting tensile strength of lightweight concrete, f_{ct}, is specified, the term $\sqrt{f_c'}$ in the expressions given in Articles 5.8.2 and 5.8.3 shall be replaced by:*

$$4.7 f_{ct} \leq \sqrt{f_c'}$$ [Equation 66]

- *Where f_{ct}, is specified, the term $0.75\sqrt{f_c'}$ for all-lightweight concrete, and $0.85\sqrt{f_c'}$ for sand-lightweight concrete shall be substituted for $\sqrt{f_c'}$ in the expressions given in Articles 5.8.2 and 5.8.3.*

Linear interpolation may be employed when partial sand replacement is used.

The coefficients in front of $\sqrt{f_c'}$ need to be verified for both all-lightweight concrete and sand-lightweight HPC.

ACTION: A research problem statement is proposed.

5.8.2.5 MINIMUM TRANSVERSE REINFORCEMENT

Where transverse reinforcement is required, as specified in Article 5.8.2.4, the area of steel shall not be less than:

$$A_v = 0.0316\sqrt{f_c'}\frac{b_v S}{f_y}$$ *(5.8.2.5-1)* [Equation 67]

Equation 5.8.2.5-1 is similar to that developed for ACI 318 to allow for an increase in the minimum amount of shear reinforcement as concrete strength increases.(18) However, the coefficient in the ACI equation is 0.24. The appropriate coefficient for use with HSC needs to be determined.

ACTION: None. Further research is being conducted under NCHRP project 12-56.

5.8.2.8 DESIGN AND DETAILING REQUIREMENTS

The design yield strength of nonprestressed transverse reinforcement shall not exceed 60.0 ksi.

The use of a design yield strength higher than 60.0 kips/inch^2 should be considered for both HSC and conventional concretes.

ACTION: Revisions to allow higher design yield strengths are proposed.

5.8.3. Sectional Design Model

5.8.3.3 NOMINAL SHEAR RESISTANCE

The nominal shear resistance, V_n, shall be determined as the lesser of:

$$V_n = V_c + V_s + V_p \qquad (5.8.3.3\text{-}1) \qquad \text{[Equation 68]}$$

$$V_n = 0.25 f_c' b_v d_v + V_p \qquad (5.8.3.3\text{-}2) \qquad \text{[Equation 69]}$$

for which

$$V_c = 0.0316 \beta \sqrt{f_c'} b_v d_v \qquad (5.8.3.3\text{-}3) \qquad \text{[Equation 70]}$$

$$V_s = \frac{A_v f_y d_v (\cot\theta + \cot\alpha)\sin\alpha}{s} \qquad (5.8.3.3\text{-}4) \qquad \text{[Equation 71]}$$

This article provides a maximum limit on the nominal shear V_n. In equation 5.8.3.3-2, the presence of f_c' allows much higher shear forces than the equivalent limits in the AASHTO Standard Specifications, which are stated in terms of limiting V_s. The nominal shear resistance of a member increases with an increase in concrete strength. The factor 0.25 needs to be verified for HSC.

At higher compressive strengths, HSC is more brittle and the shear cracks are smoother. As a result, there is less friction along the shear cracks. Since this friction carries some of the shear load, shear provided by the concrete may be less. Consequently, the constants 0.0316 and β used in equation 5.8.3.3-3 need to be investigated.

ACTION: None. Further research is being conducted under NCHRP project 12-56.

5.8.3.4 DETERMINATION OF β AND θ

5.8.3.4.2 General Procedure

This article provides a procedure for determining β and θ using equations, tables, and figures. The equations, tables, and figures were developed considering $f_c^{'}$ no higher than 10 ksi. These tables and curves need to be revisited to be sure that they are applicable to HSC.

ACTION: None. Further research is being conducted under NCHRP project 12-56.

5.8.4 Interface Shear Transfer–Shear Friction

5.8.4.1 GENERAL

Interface shear transfer shall be considered across a given plane at:

- *An existing or potential crack,*
- *An interface between dissimilar materials, or*
- *An interface between two concretes cast at different times.*

The nominal shear resistance of the interface plane shall be taken as:

$$V_n = cA_{cv} + \mu[A_{vf} f_y + P_c] \quad (5.8.4.1\text{-}1) \qquad \text{[Equation 72]}$$

The nominal shear resistance used in the design shall not exceed:

$$V_n \leq 0.2\, f_c^{'} A_{cv} \quad (5.8.4.1\text{-}2) \qquad \text{[Equation 73]}$$

or

$$V_n \leq 0.8\, A_{cv} \quad (5.8.4.1\text{-}3) \qquad \text{[Equation 74]}$$

Reinforcement for interface shear between concretes of slab and beams or girders may consist of single bars, multiple-leg stirrups, or the vertical legs of welded wire fabric. The cross-sectional area, A_{vf}, of the reinforcement per unit length of the beam or girder should not be less than either that required by Equation 1 or:

$$A_{vf} \geq \frac{0.05 b_v}{f_y} \quad (5.8.4.1\text{-}4) \qquad \text{[Equation 75]}$$

Equation 5.8.4.1-3 imposes a limit of 28 MPa (4000 psi) on the compressive strength of concrete that can be used in design. This limit needs to be evaluated based on recent test data.

Equation 5.8.4.1-4 should be changed so that the minimum reinforcement is a function of the concrete compressive strength.

ACTION: A research problem statement is proposed.

5.8.4.2 COHESION AND FRICTION

The following values shall be taken for cohesion factor, c, and friction factor, μ:

- *For concrete placed monolithically:*

$$c = 0.150 \text{ ksi} \quad \text{[Equation 76]}$$

$$\mu = 1.4\lambda \quad \text{[Equation 77]}$$

- *For concrete placed against clean, hardened concrete with surface intentionally roughened to an amplitude of 0.25 inch:*

$$c = 0.100 \text{ ksi} \quad \text{[Equation 78]}$$

$$\mu = 1.0\lambda \quad \text{[Equation 79]}$$

- *For concrete placed against hardened concrete clean and free of laitance, but not intentionally roughened:*

$$c = 0.075 \text{ ksi} \quad \text{[Equation 80]}$$

$$\mu = 0.6\lambda \quad \text{[Equation 81]}$$

- *For concrete anchored to as-rolled structural steel by headed studs or by reinforcing bars where all steel in contact with concrete is clean and free of paint:*

$$c = 0.025 \text{ ksi} \quad \text{[Equation 82]}$$

$$\mu = 0.7\lambda \quad \text{[Equation 83]}$$

The following values shall be taken for λ:

- *For normal-weight concrete 1.00*
- *For sand-lightweight concrete 0.85*
- *For all other lightweight concrete 0.75*

Tests have indicated that a smoother crack plane occurs with HSC.(15) Consequently, the values of c, μ, and λ need to be evaluated for HSC.

ACTION: A research problem statement is proposed.

5.9 PRESTRESSING AND PARTIAL PRESTRESSING

5.9.4 Stress Limits for Concrete

5.9.4.1 FOR TEMPORARY STRESSES BEFORE LOSSES—FULLY PRESTRESSED COMPONENTS

5.9.4.1.1 Compression Stresses

The compressive stress limit for pretensioned and post-tensioned concrete components, including segmentally constructed bridges, shall be 0.60 f'_{ci} *(ksi).*

In article 9.15.2.1 of the AASHTO Standard Specifications, the compressive stress limit for concrete at release for post-tensioned members is $0.55\,f'_{ci}$, compared to $0.60\,f'_{ci}$ in this article. The use of $0.55\,f'_{ci}$ or $0.60\,f'_{ci}$ for HSC should be assessed.

ACTION: Revisions to include strength design for pretensioned concrete members at release are proposed.

5.9.4.1.2 Tension Stresses

The limits in table 1 shall apply for tensile stresses.

Table 5.9.4.1.2-1. Temporary tensile stress limits in prestressed concrete before losses, fully prestressed components.

Bridge Type	***Location***	***Stress Limit***
Other Than Segmentally Constructed Bridges	• *In precompressed tensile zone without bonded reinforcement.*	*N/A*
	• *In areas other than the precompressed tensile zone and without bonded auxiliary reinforcement.*	$0.0948\sqrt{f'_{ci}} \le 0.2$ *(ksi)*
	• *In areas with bonded reinforcement sufficient to resist 120% of the tension force in the cracked concrete computed on the basis of an uncracked section.*	$0.22\sqrt{f'_{ci}}$ *(ksi)*
	• *For handling stresses in prestressed piles.*	$0.158\sqrt{f'_{ci}}$ *(ksi)*
Segmentally Constructed Bridges	*Longitudinal Stresses Through Joints in Precompressed Tensile Zone*	
	• *Type A joints with minimum bonded auxiliary reinforcement through the joints, which is sufficient to carry the calculated tensile force at a stress of* $0.5 f_y$*, with internal tendons or external tendons.*	$0.0948\sqrt{f'_{ci}}$ *maximum tension (ksi)*
	• *Type A joints without the minimum bonded auxiliary reinforcement through the joints.*	*No tension*
	• *Type B joints with external tendons.*	*0.100 ksi minimum compression*
	Transverse Stresses Through Joints	
	• *For any type of joint.*	$0.0948\sqrt{f'_{ci}}$ *(ksi)*
	Stresses in Other Areas	
	• *For areas without bonded, nonprestressed reinforcement.*	*No tension*
	• *Bonded reinforcement sufficient to carry the calculated tensile force in the concrete computed on the assumption of an uncracked section at a stress of* $0.5 f_{sy}$.	$0.19\sqrt{f'_{c}}$ *(ksi)*

HSC is known to have a proportionally higher tensile strength than conventional concrete. It may be possible to have higher stress limits in table 5.9.4.1.2-1.

ACTION: A research problem statement is proposed.

5.9.4.2 FOR STRESSES AT SERVICE LIMIT STATE AFTER LOSSES-FULLY STRESSED COMPONENTS

5.9.4.2.2 Tension Stresses

For service load combinations that involve traffic loading, tension stresses in members with bonded or unbonded prestressing tendons should be investigated using Load Combination Service III specified in table 3.4.1-1. The limits in table 1 shall apply.

Table 5.9.4.2.2-1. Tensile stress limits in prestressed concrete at service limit state after losses, fully prestressed components.

Bridge Type	***Location***	***Stress Limit***
Other Than Segmentally Constructed Bridges	*Tension in the Precompressed Tensile Zone Bridges, Assuming Uncracked Sections*	
	▪ *For components with bonded prestressing tendons or reinforcement that are subjected to not worse than moderate corrosion conditions.*	$0.19\sqrt{f'_c}$ *(ksi)*
	▪ *For components with bonded prestressing tendons or reinforcement that are subjected to severe corrosion conditions.*	$0.0948\sqrt{f'_c}$ *(ksi)*
	▪ *For components with unbonded prestressing tendons.*	*No tension*
Segmentally Constructed Bridges	*Longitudinal Stresses Through Joints in the Precompressed Tensile Zone*	
	▪ *Type A joints with minimum bonded auxiliary reinforcement through the joints sufficient to carry the calculated longitudinal tensile force at a stress of 0.5* f_y, *internal tendons.*	$0.0948\sqrt{f'_c}$ *(ksi)*
	▪ *Type A joint without the minimum bonded auxiliary reinforcement through joints.*	*No tension*
	▪ *Type B joints, external tendons.*	*0.100 ksi minimum compression*
	Transverse Stresses Through Joints	
	▪ *Tension in the transverse direction in precompressed tensile zone.*	$0.0948\sqrt{f'_c}$ *(ksi)*
	Stresses in Other Areas	
	▪ *For areas without bonded reinforcement.*	*No tension*
	▪ *Bonded reinforcement sufficient to carry the calculated tensile force in the concrete computed on the assumption of an uncracked section at a stress of 0.5* f_{sy}.	$0.19\sqrt{f'_c}$ *(ksi)*

HSC is known to have a proportionally higher tensile strength than conventional concrete. It may be possible to have higher stress limits in table 5.9.4.2.2-1.

ACTION: A research problem statement is proposed.

5.9.5 Loss of Prestress

5.9.5.3 APPROXIMATE LUMP SUM ESTIMATE OF TIME-DEPENDENT LOSSES

An approximate lump sum estimate of time-dependent prestress losses resulting from creep and shrinkage of concrete and relaxation of steel in prestressed and partially prestressed members may be taken as specified in table 1 for:

- *Post-tensioned, nonsegmental members with spans up to 160 ft and stressed at a concrete age of 10 to 30 days, and*

Pretensioned members stressed after attaining a compressive strength f'_{ci} *= 3.5 ksi, provided that*

- *Members are made from normal-weight concrete,*
- *The concrete is either steam or moist cured,*
- *Prestressing is by bars or strands with normal and low relaxation properties, and*
- *Average exposure conditions and temperatures characterize the site.*

Table 5.9.5.3-1. Time-dependent losses in ksi.

Type of Beam Section	***Level***	***For Wires and Strands With*** f_{pu} ***= 235, 250, or 270 ksi***	***For Bars With*** f_{pu} ***= 145 or 160 ksi***
Rectangular Beams, Solid Slab	*Upper Bound* *Average*	*29.0 + 4.0 PPR* *26.0 + 4.0 PPR*	*19.0 + 6.0 PPR*
Box Girder	*Upper Bound* *Average*	*21.0 + 4.0 PPR* *19.0 + 4.0 PPR*	*15.0*
I-Girder	*Average*	$33.0\left[1.0-0.15\frac{f'_c-6.0}{6.0}\right]+6.0\ PPR$	*19.0 + 6.0 PPR*
Single-T, Double-T, Hollow Core, and Voided Slab	*Upper Bound* *Average*	$39.0\left[1.0-0.15\frac{f'_c-6.0}{6.0}\right]+6.0\ PPR$ $33.0\left[1.0-0.15\frac{f'_c-6.0}{6.0}\right]+6.0\ PPR$	$31.0\left[1.0-0.15\frac{f'_c-6.0}{6.0}\right]+6.0\ PPR$

The equations in this article are based on parametric studies for a limited range of ultimate creep and shrinkage coefficients. Generally, HSC has lower creep and similar shrinkage values relative to conventional strength concrete. In table 5.9.5.3-1, the equations do not reflect the higher prestress levels used with HSC. The lump-sum equations may need to be evaluated for use with HSC.

NCHRP project 18-07 has the objective of developing design guidelines for estimating prestress losses in pretensioned HSC bridge girders. Results of the NCHRP project need to be incorporated into this article.

ACTION: Revisions to articles 5.9.5.1, 5.9.5.2, and 5.9.5.3 based on NCHRP project 18-07 are proposed.

5.9.5.4 REFINED ESTIMATES OF TIME-DEPENDENT LOSSES

5.9.5.4.1 General

More accurate values of creep-, shrinkage-, and relaxation-related losses than those specified in Article 5.9.5.3 may be determined in accordance with the provisions of either Article 5.4.2.3 or this article for prestressed members with:

- *Spans not greater than 250 ft,*
- *Normal density concrete, and*
- *Strength in excess of 3.50 ksi at the time of prestress.*

For lightweight concrete, loss of prestress shall be based on the representative properties of the concrete to be used.

Equations are then provided for calculation of individual components of prestress losses.

Results of NCHRP project 18-07 will need to be incorporated into this article.

ACTION: Revisions based on NCHRP project 18-07 are proposed.

5.10 DETAILS OF REINFORCEMENT

5.10.11 Provisions for Seismic Design

5.10.11.4 SEISMIC ZONES 3 AND 4

5.10.11.4.1d Transverse Reinforcement for Confinement at Plastic Hinges

The cores of columns and pile bents shall be confined by transverse reinforcement in the expected plastic hinge regions. The transverse reinforcement for confinement shall have a yield strength not more than that of the longitudinal reinforcement, and the spacing shall be taken as specified in Article 5.10.11.4.1e.

For a circular column, the volumetric ratio of spiral reinforcement, ρ_s, shall not be less than either that required in Article 5.7.4.6 or:

$$\rho_s = 0.12\frac{f_c'}{f_y} \qquad (5.10.11.4.1d\text{-}1) \qquad \text{[Equation 84]}$$

For a rectangular column, the total gross sectional area, A_{sh}, of rectangular hoop reinforcement shall not be less than either:

$$A_{sh} = 0.30 s h_c \frac{f_c'}{f_y}\left[\frac{A_g}{A_s} - 1\right] \qquad (5.10.11.4.1d\text{-}2) \qquad \text{[Equation 85]}$$

or

$$A_{sh} = 0.12 s h_c \frac{f_c'}{f_y} \qquad (5.10.11.4.1d\text{-}3) \qquad \text{[Equation 86]}$$

The volumetric ratio of spiral reinforcement and the total gross sectional area of rectangular hoop reinforcement required by equations 5.10.11.4.1d-1 and 5.10.11.4.1d-2, respectively, may be too high to be practical with HSC. The concept of providing reinforcement to strengthen the core to offset the loss of strength when the concrete shell is lost may not be appropriate for HSC.

ACTION: A research problem statement is proposed.

5.10.11.4.2 Requirements for Wall-Type Piers

The factored shear resistance, V_r, in the pier shall be taken as the lesser of:

$$V_r = 0.253\sqrt{f_c^{'}}bd \qquad (5.10.11.4.2\text{-}1) \qquad \text{[Equation 87]}$$

and

$$V_r = \varphi V_n \qquad (5.10.11.4.2\text{-}2) \qquad \text{[Equation 88]}$$

for which

$$V_n = \left[0.063\sqrt{f_c^{'}} + \rho_h f_y\right]bd \qquad (5.10.11.4.2\text{-}3) \qquad \text{[Equation 89]}$$

Equations 5.10.11.4.2-1 and 5.10.11.4.2-3 should be evaluated for use with HSC.

ACTION: A research problem statement is proposed.

5.10.11.4.3 Column Connections

The nominal shear resistance, provided by the concrete in the joint of a frame or bent in the direction under consideration, shall not exceed:

- *For normal-weight aggregate concrete:*

$$V_n \le 0.380bd\sqrt{f_c^{'}} \qquad (5.10.11.4.3\text{-}1) \qquad \text{[Equation 90]}$$

- *For lightweight aggregate concrete:*

$$V_n \le 0.285bd\sqrt{f_c^{'}} \qquad (5.10.11.4.3\text{-}2) \qquad \text{[Equation 91]}$$

Equations 5.10.11.4.3-1 and 5.10.11.4.3-2 should be evaluated for use with HSC.

ACTION: A research problem statement is proposed.

5.11 DEVELOPMENT AND SPLICES OF REINFORCEMENT

5.11.2 Development of Reinforcement

5.11.2.1 DEFORMED BARS AND DEFORMED WIRE IN TENSION

5.11.2.1.1 Tension Development Length

The tension development length, l_d, shall not be less than the product of the basic tension development length, l_{db}, specified herein and the modification factor or factors specified in Articles 5.11.2.1.2 and 5.11.2.1.3. The tension development length shall not be less than 12.0 inches, except for lap splices specified in Article 5.11.5.3.1 and development of shear reinforcement as specified in Article 5.11.2.6

The basic tension development length, l_{db}, in inches, shall be taken as:

- *For No. 11 bar and smaller* .... $\dfrac{1.25 A_b f_y}{\sqrt{f_c'}}$

  but not less than .... $0.4\, d_b f_y$
- *For No. 14 bars* .... $\dfrac{2.70 f_y}{\sqrt{f_c'}}$
- *For No. 18 bars* .... $\dfrac{3.5 f_y}{\sqrt{f_c'}}$
- *For deformed wire* .... $\dfrac{0.95 d_b f_y}{\sqrt{f_c'}}$

This article contains provisions for the development length of reinforcement based on the provisions of ACI 318-89. These were extensively modified in the ACI 318-95 provisions, with a view toward formulating a more user-friendly format while maintaining the same general agreement with research results and with professional judgment. Limited tests have indicated that the development lengths calculated using the above provisions are applicable to HSC.(29) However, the tests resulted in a more sudden failure than occurs with conventional strength concrete. Consideration should be given to adopting similar provisions as ACI 318-95 after the development lengths have been verified for HSC.

ACTION: None. Further work is the objective of NCHRP project 12-60.

5.11.2.1.2 Modification Factors That Increase l_d

The basic development length, l_{db}, shall be multiplied by the following factor or factors, as applicable:

- *For top horizontal or nearly horizontal reinforcement, so placed that more than 12.0 inches of fresh concrete is cast below the reinforcement* 1.4
- *For bars with a cover of d_b or less, or with a clear spacing of $2d_b$ or less*........2.0

- *For lightweight aggregate concrete, where f_{ct} (ksi) is specified....* $\frac{0.22\sqrt{f_c'}}{f_{ct}} \geq 1.0$
- *For all-lightweight concrete, where f_{ct} is not specified 1.3*
- *For sand-lightweight concrete, where f_{ct} is not specified 1.2*

Linear interpolation may be used between all-lightweight and sand-lightweight provisions when partial sand replacement is used.

- *For epoxy-coated bars with a cover less than $3d_b$ or with clear spacing between bars less than $6d_b$..1.5*
- *For epoxy-coated bars not covered above..1.2*

The product obtained when combining the factor for top reinforcement with the applicable factor for epoxy-coated bars need not be taken to be greater than 1.7.

5.11.2.1.3 Modification Factors That Decrease l_d

The basic development length, l_{db}, modified by the factors as specified in Article 5.11.2.1.2, may be multiplied by the following factors, where:

- *Reinforcement being developed in the length under consideration is spaced laterally not less than 6.0 inches center to center, with not less than 3.0 inches clear cover measured in the direction of the spacing...0.8*
- *Anchorage or development for the full yield strength of reinforcement is not required, or where reinforcement in flexural members is in excess of that required by analysis ....* $\left(\frac{A_s required}{A_s provided}\right)$
- *Reinforcement is enclosed within a spiral composed of bars not less than 0.25 inch in diameter and spaced at not more than a 4.0-inch pitch 0.75*

The above factors need to be verified for HPC.

ACTION: None. Further work is the objective of NCHRP project 12-60.

5.11.2.2 DEFORMED BARS IN COMPRESSION

5.11.2.2.1 Compressive Development Length

The development length, l_d, for deformed bars in compression shall not be less than either the product of the basic development length specified herein and the applicable modification factors specified in Article 5.11.2.2.2 or 8 inches.

The basic development length, l_{db}, for deformed bars in compression shall not be less than:

$$l_{db} = \frac{0.63 d_b f_y}{\sqrt{f_c'}} \qquad (5.11.2.2.1\text{-}1) \qquad \text{[Equation 92]}$$

or

$$l_{db} = 0.3\ d_b f_y \qquad (5.11.2.2.1\text{-}2) \qquad \text{[Equation 93]}$$

5.11.2.2.2 Modification Factors

The basic development length, l_{db}, may be multiplied by applicable factors, where:

- *Anchorage or development for the full yield strength of reinforcement is not required, or where reinforcement is provided in excess of that required by analysis* $\left(\dfrac{A_s required}{A_s provided}\right)$
- *Reinforcement is enclosed within a spiral composed of a bar not less than 0.25 inch in diameter and spaced at not more than a 4.0-inch pitch 0.75*

The above factors need to be verified for HPC.

ACTION: None. Further work is the objective of NCHRP project 12-60.

5.11.2.3 BUNDLED BARS

The development length of individual bars within a bundle, in tension, or compression shall be that for the individual bar, increased by 20 percent for a three-bar bundle and by 33 percent for a four-bar bundle.

For determining the factors specified in Articles 5.11.2.1.2 and 5.11.2.1.3, a unit of bundled bars shall be treated as a single bar of a diameter determined from the equivalent total area.

The above factors need to be verified for HPC.

ACTION: None. Further work is the objective of NCHRP project 12-60.

5.11.2.4 STANDARD HOOKS IN TENSION

5.11.2.4.1 Basic Hook Development Length

The development length, l_{dh}, in inches, for deformed bars in tension terminating in a standard hook specified in Article 5.10.2.1 shall not be less than:

- *The product of the basic development length, l_{hb}, as specified in Equation 1, and the applicable modification factor or factors, as specified in Article 5.11.2.4.2;*
- *8.0 bar diameters; or*
- *6.0 inches.*

Basic development length, l_{hb}, for a hooked bar with yield strength, f_y, not exceeding 60.0 ksi shall be taken as:

$$l_{hb} = \frac{38.0 d_b}{\sqrt{f_c^{'}}} \qquad (5.11.2.4.1\text{-}1) \qquad \text{[Equation 94]}$$

The above factors need to be verified for HPC.

ACTION: None. Further work is the objective of NCHRP project 12-60.

5.11.2.4.2 Modification Factors

Basic hook development length, l_{hb}, shall be multiplied by the following factor or factors, as applicable, where:

- *Reinforcement has a yield strength exceeding 60.0 ksi* .... $\frac{f_y}{60.0}$
- *Side cover for No. 11 bar and smaller, normal to plane of hook, is not less than 2.5 inches, and for 90° hook, cover on bar extension beyond hook is not less than 2.0 inches* *0.7*
- *Hooks for No. 11 bar and smaller, enclosed vertically or horizontally within ties or stirrup ties that are spaced along the full development length, l_{dh}, at a spacing not exceeding $3d_b$* *0.8*
- *Anchorage or development of the full yield strength of reinforcement is not required, or where reinforcement is provided in excess of that required by analysis* $\left(\frac{A_s required}{A_s provided}\right)$
- *Lightweight aggregate concrete is used* *0.75*
- *Epoxy-coated reinforcement is used* *1.2*

All provisions of article 5.11.2.4 need to be verified for HPC.

ACTION: None. Further work is the objective of NCHRP project 12-60.

5.11.2.5 WELDED WIRE FABRIC

5.11.2.5.1 Deformed Wire Fabric

For applications other than shear reinforcement, the development length, l_{hd}, in inches, of welded deformed wire fabric, measured from the point of critical section to the end of wire, shall not be less than either:

- *The product of the basic development length and the applicable modification factor or factors, as specified in Article 5.11.2.2.2, or*
- *8.0 inches, except for lap splices, as specified in Article 5.11.6.1.*

5.11.2.5.2 Plain Wire Fabric

The yield strength of welded plain wire fabric shall be considered developed by embedment of two cross wires with the closer cross wire not less than 2.0 inches from the point of critical section. Otherwise, the development length, l_d, measured from the point of critical section to outermost cross wire shall be taken as:

$$l_d = 8.50\frac{A_w f_y}{s_w\sqrt{f_c'}} \qquad (5.11.2.5.2\text{-}1) \qquad \text{[Equation 95]}$$

The development length shall be modified for reinforcement in excess of that required by analysis as specified in Article 5.11.2.4.2, and by the factor for lightweight concrete specified in Article 5.11.2.1.2, where applicable. However, l_d shall not be taken to be less than 6.0 inches, except for lap splices, as specified in Article 5.11.6.2.

All provisions of article 5.11.2.5 need to be verified for HPC.

ACTION: None. Further work is the objective of NCHRP project 12-60.

5.11.4 Development of Prestressing Strand

5.11.4.1 GENERAL

For the purposes of this article, the transfer length may be taken as 60 strand diameters and the development length shall be taken as specified in Article 5.11.4.2.

5.11.4.2 BONDED STRAND

Pretensioning strand shall be bonded beyond the critical section for development length, in inches, taken as:

$$l_d \geq \left(f_{ps} - \frac{2}{3} f_{pe} \right) d_b \qquad (5.11.4.2\text{-}1) \qquad \text{[Equation 96]}$$

This article contains general requirements for the transfer and development lengths of prestressing strand. It is defined in the article that the transfer length for pretensioned prestressing strand may be taken as 60 strand diameters. This value is increased from the 50 diameters used in the AASHTO Standard Specifications, article 9.20.2.4. A lower transfer length is possible for HPC. These provisions should be evaluated for HSC. Researchers at FHWA have suggested new equations for transfer and development lengths of bonded prestressing strand.(30) Those equations should be reviewed for their appropriateness with HSC.

ACTION: None. Further work is the objective of NCHRP project 12-60.

5.11.5 Splices of Bar Reinforcement

5.11.5.3 SPLICES OF REINFORCEMENT IN TENSION

5.11.5.3.1 Lap Splices in Tension

The length of lap for tension lap splices shall not be less than either 12.0 inches or the following for Class A, B, or C splices:
Class A splice.. 1.0 l_d
Class B splice.. 1.3 l_d
Class C splice.. 1.7 l_d

The tension development length, l_d, for the specified yield strength shall be taken in accordance with Article 5.11.2.

The class of lap splice required for deformed bars and deformed wire in tension shall be as specified in table 1.

Table 5.11.5.3.1-1. Classes of tension lap splices.

Ratio of $\left(\frac{A_s \text{ as provided}}{A_s \text{ as required}}\right)$	***Percent of A_s Spliced With Required Lap Length***		
	50	*75*	*100*
≥ 2	*A*	*A*	*B*
< 2	*B*	*C*	*C*

All provisions of article 5.11.5 need to be verified for HPC.

ACTION: None. Further work is the objective of NCHRP project 12-60.

5.12 DURABILITY

5.12.2 Alkali-Silica Reactive Aggregates

The contract documents shall prohibit the use of aggregates from sources that are known to be excessively alkali-silica reactive.

If aggregate of limited reactivity is used, the contract documents shall require the use of either low-alkali-type cements or a blend of regular cement and pozzolanic materials, provided that their use has been proven to produce concrete of satisfactory durability with the proposed aggregate.

For HPC, testing should be conducted or field experience should be used to determine if a given source of aggregate can be safely used with specific cementitious materials.

ACTION: Revisions referencing AASHTO M 6 and M 80 are proposed.

5.12.3 Concrete Cover

Cover for unprotected prestressing and reinforcing steel shall not be less than that specified in table 1 and modified for the W/C ratio, unless otherwise specified either herein or in Article 5.12.4.

Concrete cover and placing tolerances shall be shown in the contract documents.

Cover for pretensioned prestressing strand, anchorage hardware, and mechanical connections for reinforcing bars or post-tensioned prestressing strands shall be the same as for reinforcing steel.

Cover for metal ducts for post-tensioned tendons shall not be less than:

- *That specified for main reinforcing steel,*
- *One-half the diameter of the duct, or*
- *That specified in table 1.*

For decks exposed to tire studs or chain wear, additional cover shall be used to compensate for the expected loss in depth due to abrasion, as specified in Article 2.5.2.4.

Modification factors for the W/C ratio shall be the following:

- *For W/C $\leq$ 0.40 ....0.8*
- *For W/C $\geq$ 0.50...1.2*

Minimum cover to main bars, including bars protected by epoxy coating, shall be 1.0 inch.

Cover to ties and stirrups may be 0.5 inch less than the values specified in table 1 for main bars, but shall not be less than 1.0 inch.

This article provides minimum cover requirements for a range of exposure conditions. HPC is less permeable than conventional concrete, and a longer service life is expected.

The second bullet after the fourth paragraph that says "One-half the diameter of the duct" should be revised to allow the use of wider ducts in 152-mm- (6-inch-) thick webs.

In this article, modification factors for w/c ratios of 0.40 and 0.50 are given. A gradual transition would be more logical. Furthermore, there are ways to reduce the permeability without lowering the w/c ratio.

ACTION: None.

5.13 SPECIFIC MEMBERS

5.13.2 Diaphragms, Deep Beams, Brackets, Corbels, and Beam Ledges

5.13.2.4 BRACKETS AND CORBELS

5.13.2.4.2 Alternative to Strut-and-Tie Model

- *For normal-weight concrete, nominal shear resistance, V_n, shall be taken as the lesser of:*

$$V_n = 0.2\ f_c'\ b_w d_e \qquad (5.13.2.4.2\text{-}1) \qquad \text{[Equation 97]}$$

and

$$V_n = 0.8\ b_w d_e \qquad (5.13.2.4.2\text{-}2) \qquad \text{[Equation 98]}$$

- *For all-lightweight or sand-lightweight concretes, nominal shear resistance, V_n, in kips, shall be taken as the lesser of:*

$$V_n = (0.2 - 0.07\ a_v/d)\ f_c'\ b_w d_e \qquad (5.13.2.4.2\text{-}3) \qquad \text{[Equation 99]}$$

or

$$V_n = (0.8 - 0.28 a_v/d_e)\ b_w d \qquad (5.13.2.4.2\text{-}4) \qquad \text{[Equation 100]}$$

Equations 5.13.2.4.2-1 and 5.13.2.4.2-2 impose a limit of 28 MPa (4.0 ksi) on the compressive strength of concrete that can be used in design. The limits of equation 5.13.2.4.2-2 and 0.2 f_c' need to be evaluated for HSC.

ACTION: A research problem statement is proposed.

5.13.2.5 BEAM LEDGES

5.13.2.5.4 Design for Punching Shear

Nominal punching shear resistance, V_n, in kips, shall be taken as:

- *At interior pads:*

$$V_n = 0.125\sqrt{f_c'}\left(W + 2L + 2d_e\right)d_e \qquad \textit{(5.13.2.5.4-1)} \qquad \text{[Equation 101]}$$

- *At exterior pads:*

$$V_n = 0.125\sqrt{f_c'}\left(W + L + d_e\right)d_e \qquad \textit{(5.13.2.5.4-2)} \qquad \text{[Equation 102]}$$

The equations in this article should be verified for use with HSC.

ACTION: A research problem statement is proposed.

5.13.2.5.5 Design of Hangar Reinforcement

Using the notation in Figure 2, the nominal shear resistance of the ledges of inverted T-beams shall be the lesser of that specified by Equation 2 and Equation 3.

- *At interior pads:*

$$V_n = \left(0.063\sqrt{f_c'}\,b_f d_f\right) + \frac{A_{hr} f_y}{s}\left(W + 2d_f\right) \qquad \textit{(5.13.2.5.5-3)} \qquad \text{[Equation 103]}$$

Equation 5.13.2.5.5-3 should be verified for use with HSC.

ACTION: A research problem statement is proposed.

5.13.3 Footings

5.13.3.6 SHEAR IN SLABS AND FOOTINGS

5.13.3.6.3 Two-Way Action

For two-way action for sections without transverse reinforcement, the nominal shear resistance, V_n, in kips, of the concrete shall be taken as:

$$V_n = \left(0.063 + \frac{0.126}{\beta_c}\right)\sqrt{f_c'}\,b_o d_v \le 0.126\sqrt{f_c'}\,b_o d_v$$

(5.13.3.6.3-1) [Equation 104]

For two-way action for sections with transverse reinforcement, the nominal shear resistance, V_n, in kips, shall be taken as:

$$V_n = V_c + V_s \le 0.192\sqrt{f_c'}\,b_o d_v$$

(5.13.3.6.3-2) [Equation 105]

for which

$$V_c = 0.1264\sqrt{f_c'}\,b_o d_v$$

(5.13.3.6.3-3) [Equation 106]

and

$$V_s = \frac{A_v f_y d_v}{s}$$

(5.13.3.6.3-4) [Equation 107]

The equations in this article should be verified for use with HSC.

ACTION: A research problem statement is proposed.

5.13.4 Concrete Piles

5.13.4.4 PRECAST PRESTRESSED PILES

5.13.4.4.1 Pile Dimensions

The wall thickness of cylinder piles shall not be less than 5.0 inches.

A wall thickness of less than 127 mm (5.0 inches) should be allowed when HPC is used.

ACTION: A revision to eliminate the minimum wall thickness is proposed.

5.14 PROVISIONS FOR STRUCTURE TYPES

5.14.1 Beams and Girders

5.14.1.2 PRECAST BEAMS

5.14.1.2.5 Concrete Strength

For slow-curing concretes, the 90-day compressive strength may be used for all stress combinations that occur after 90 days.

For normal-weight concrete, the 90-day strength of slow-curing concretes may be estimated at 115 percent of their 28-day strength.

This article is very relevant to HSC. It should be revised to clarify the meaning of slow-curing concrete and allow the use of 56-day strengths. The factor of 115 percent should be evaluated based on data from the FHWA HPC showcase bridges.

ACTION: A research problem statement is proposed.

5.14.2 Segmental Construction

5.14.2.3 DESIGN

15.4.2.3.3 Construction Load Combinations at the Service Limit State

Tensile stresses in concrete due to construction loads shall not exceed the values specified in table 1; except for structures with Type A joints and less than 60 percent of their tendon capacity provided by internal tendons, the tensile stresses shall not exceed $0.095\sqrt{f_c'}$ *. For structures with Type B joints, no tensile stresses shall be permitted.*

HSC has a relatively higher tensile strength than conventional strength concrete. The tensile stress limit in this article and in table 5.14.2.3.3-1 should be evaluated for HSC.

ACTION: A research problem statement is proposed.

5.14.2.4 TYPES OF SEGMENTAL BRIDGES

5.14.2.4.7 Precast Segmental Beam Bridges

5.14.2.4.7b Segment Reinforcement

Segments of segmental beam bridges shall preferably be pretensioned for dead load and all construction loadings to limit the tensile stress in the concrete to $0.0948\sqrt{f_c'}$ *.*

The tensile stress limit should be evaluated for use with HSC.

ACTION: A research problem statement is proposed.

5.14.5 Additional Provisions for Culverts

5.14.5.3 DESIGN FOR SHEAR IN SLABS OF BOX CULVERTS

$$V_c = \left(0.0676\sqrt{f_c'} + 4.6\,\frac{A_s}{bd_e}\frac{V_u d_e}{M_u} \right) bd_e \qquad \textit{(5.14.5.3-1)} \qquad \text{[Equation 108]}$$

but V_c shall not exceed $0.126\sqrt{f_c'}\,bd_e$.

Although HSC may not be used in slabs for box culverts, the limits for V_c in this article should be evaluated for use with HSC.

ACTION: A research problem statement is proposed.

Section 9: DECKS AND DECK SYSTEMS

This section contains provisions for the analysis and design of bridge decks and deck systems of concrete, metal, and wood, or combinations thereof, subject to gravity loads. No provisions affected by HPC were identified in the section.

ACTION: None.

AASHTO LRFD BRIDGE CONSTRUCTION SPECIFICATIONS

Section 8 of the *AASHTO LRFD Bridge Construction Specifications* deals with concrete structures. The technical provisions are essentially the same as those in the AASHTO *Standard Specifications for Highway Bridges*, Division II, Section 8, Concrete Structures, except for the use of metric units in the LRFD version. Consequently, a separate review of the LRFD specifications was not undertaken. Proposed revisions to the AASHTO LRFD Specifications, similar to those proposed for division II, section 8 of the AASHTO Standard Specifications, are included in appendix D. The proposed revisions are based on *AASHTO LRFD Bridge Construction Specifications*, First Edition, 1998, and the 1999, 2000, and 2001 interim revisions.[11-14]

CHAPTER 4. RECOMMENDATIONS FOR NEEDED RESEARCH

The objective of task E was to develop specific recommendations for needed research where existing research was not sufficient to allow development of proposed revisions to the AASHTO specifications. During the review of the AASHTO specifications, many areas of needed research were identified. For some areas, the amount of needed research was minimal, while major research projects would be needed for others. Many of the areas requiring a small amount of research of a similar nature were combined into a single, more substantial research project. At the same time, it was recognized that the following NCHRP projects were underway or in the process of development and will address some of the research needs identified in this project:

- Project 18-07, Prestress Losses in Pretensioned High-Strength Concrete Bridge Girders.
- Project 12-56, Application of the LRFD Bridge Design Specifications to High-Strength Structural Concrete: Shear Provisions.
- Project 12-60, Transfer, Development, and Splice Length for Strand/Reinforcement in High-Strength Concrete.
- Project 12-64, Application of the LRFD Bridge Design Specifications to High-Strength Structural Concrete: Flexure and Compression Provisions.

Consequently, none of the research problem statements developed on this project overlap the four NCHRP projects listed above.

The end result of the above-mentioned process was a series of 10 proposed research projects. Six of the projects have a strong emphasis on concrete materials technology, while four relate primarily to structural design.

MATERIALS RESEARCH

The following six research projects have a strong emphasis on concrete materials technology:

1. Performance Requirements for HPC.
2. Use of Wash Water in HPC.
3. Air-Void Requirements and Freeze-Thaw Testing Requirements for Durability of HPC.
4. Penetrability Criteria for HPC.
5. Curing of HPC.
6. Procedures for Measuring Compressive and Flexural Strengths of High-Strength Concrete.

STRUCTURAL RESEARCH

The following four projects relate to structural design requirements:

1. Application of Bridge Design Specifications to High-Strength Concrete Structural Members: Material Properties.
2. Application of Bridge Design Specifications to High-Strength Concrete Structural Members: Shear Provisions Except Prestressed Concrete Beams.
3. Verification of Stress Limits and Resistance Factors for HPC.
4. Confinement of High-Strength Concrete Columns for Seismic and Non-Seismic Regions.

For each of these projects, a research problem statement using the NCHRP format is included in appendix F.

CHAPTER 5. EVALUATION OF "HIGH-PERFORMANCE CONCRETE DEFINED FOR HIGHWAY STRUCTURES"

SUMMARY

A review of the FHWA definition of HPC was made to identify whether the performance characteristics, test methods, and range of grades were appropriate and to propose any modifications based on experience with the definition since it was published in 1996. Based on the review, the eight existing characteristics of freeze-thaw durability, scaling resistance, abrasion resistance, chloride penetration, compressive strength, modulus of elasticity, shrinkage, and creep are appropriate, with the addition of alkali-silica reactivity, sulfate resistance, and flowability. However, abrasion resistance and creep should only be specified for special situations. Three grades should be assigned to each characteristic and the values in each grade should be revised to reflect recent data and experience and to raise the performance level of each characteristic. Several modifications to the test methods are suggested.

INTRODUCTION

In 1993, FHWA initiated a national program to implement the use of HPC in bridges. As part of the program, FHWA produced a definition of HPC that identified a set of concrete performance characteristics sufficient to estimate long-term concrete durability and strength for highway structures. Standard laboratory tests, specimen preparation procedures, and grades of performance were suggested for each characteristic. Estimates of relationships between each performance grade and severity of field conditions were provided to assist designers in selecting the grade of HPC for a particular project.

The definition was published in 1996.[31] Subsequently, numerous bridges have been built with HPC. The purpose of this article is to report on a review of the FHWA definition in light of experiences and data collected since 1996 and to address the following questions:

- Are the performance characteristics appropriate?
- Are the test methods appropriate?
- Are modifications to the test methods needed?
- Is the range of grades appropriate?
- Do the values assigned to each grade need modification?

FHWA HPC DEFINITION

Performance Characteristics

Eight performance characteristics were used in the definition of HPC to encompass both durability and structural design. The four performance characteristics related to durability are: freeze-thaw resistance, scaling resistance, abrasion resistance, and chloride ion penetration. The four structural design characteristics are: compressive strength, modulus of elasticity, shrinkage, and creep.

Test Methods

For each characteristic, there is a standard test method published by ASTM or AASHTO. The eight performance characteristics and the corresponding test methods are listed in table 12, which is reproduced from table 1 of reference 31.

Since standard test methods sometimes offer different options, the specimens and procedures listed in table 13 (table 2 of reference 31) were adopted. Unless listed otherwise, the following were also stipulated in the original paper:

- Cylinder size: 100 by 200 mm (4 by 8 inches) or 150 by 300 mm (6 by 12 inches).
- Specimen curing: For non-steam-cured products, moist cure specimens for 56 days or until the listed test age, whichever is longer, or match cure and use a maturity meter. For steam-cured products, cure specimens with the member or match cure until test age.

Footnote 2 of table 12 states that all tests are to be performed on concrete samples moist or submersion cured for 56 days, with a reference to table 13 for exceptions. The only exceptions in table 13 are for creep and shrinkage.

Performance Grades

For each FHWA HPC performance characteristic, a range of two to four performance grades was established as listed in table 12. A higher number for the grade indicates a higher level of performance. An empty box in table 12 indicates that the grade is not applicable. With this approach, it is not necessary to specify every characteristic or to specify the same grade for different characteristics. The characteristics and grades should be selected to match the intended application and its environment. For example, a bridge deck supported on girders needs a specified compressive strength, but is unlikely to require specified values for modulus of elasticity and creep. It is not necessary to require all performance characteristics for a given application.

Exposure Conditions

Recommendations for the application of performance grades for the durability characteristics were provided separately as given in table 14 (table 3 of reference 31). Because there is a lack of information correlating field condition severity and laboratory test performance, the relationships shown in table 14 serve only as suggestions and local experience should receive careful consideration in selecting the grades. For scaling resistance, recommendations in table 14 are only provided for grade 1, whereas three grades are listed in table 12.

Table 12. Grades of performance characteristics for high-performance structural concrete.[1]

Performance Characteristic[2]	Standard Test Method	FHWA HPC Performance Grade[3]			
		1	2	3	4
Freeze-Thaw Durability[4] (x = relative dynamic modulus of elasticity after 300 cycles)	AASHTO T 161 ASTM C 666 Proc. A	$60\% \leq x < 80\%$	$80\% \leq x$		
Scaling Resistance[5] (x = visual rating of the surface after 50 cycles)	ASTM C 672	x = 4,5	x = 2,3	x = 0,1	
Abrasion Resistance[6] (x = avg. depth of wear in mm)	ASTM C 944	$2.0 > x \geq 1.0$	$1.0 > x \geq 0.5$	$0.5 > x$	
Chloride Penetration[7] (x = coulombs)	AASHTO T 277 ASTM C 1202	$3000 \geq x > 2000$	$2000 \geq x > 800$	$800 \geq x$	
Strength (x = compressive strength)	AASHTO T 2* ASTM C 39	$41 \leq x < 55$ MPa ($6 \leq x < 8$ ksi)	$55 \leq x < 69$ MPa ($8 \leq x < 10$ ksi)	$69 \leq x < 97$ MPa ($10 \leq x < 14$ ksi)	$x \geq 97$ MPa ($x \geq 14$ ksi)
Elasticity[10] (x = modulus of elasticity)	ASTM C 469	$28 \leq x < 40$ GPa ($4 \leq x < 6 \times 10^6$ psi)	$40 \leq x < 50$ GPa ($6 \leq x < 7.5 \times 10^6$ psi)	$x \geq 50$ GPa ($x \geq 7.5 \times 10^6$ psi)	
Shrinkage[8] (x = microstrain)	ASTM C 157	$800 > x \geq 600$	$600 > x \geq 400$	$400 > x$	
Creep[9] (x = microstrain/ pressure unit)	ASTM C 512	$75 \geq x >$ 60/MPa ($0.52 \geq x >$ 0.41/psi)	$60 \geq x >$ 45/MPa ($0.41 \geq x >$ 0.31/psi)	$45 \geq x >$ 30/MPa ($0.31 \geq x >$ 0.21/psi)	30 MPa $\geq x$ (0.21 psi $\geq x$)

[1] This table does not represent a comprehensive list of all characteristics that good concrete should exhibit. It does list characteristics that can quantifiably be divided into different performance groups. Other characteristics should be checked. For example, HPC aggregates should be tested for detrimental alkali-silica reactivity according to ASTM C 227, cured at 38 °C, and tested at 23 °C, and should yield less than 0.05 percent mean expansion at 3 months and less than 0.10 percent expansion at 6 months (based on Strategic Highway Research Program (SHRP) C-342, p. 83). Consideration should also be paid to (but not necessarily limited to) acidic environments and sulfate attack.

[2] All tests to be performed on concrete samples moist or submersion cured for 56 days. See table 13 for additional information and 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 scaling resistance, and grade 2 in all 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.

* AASHTO T 2 should be AASHTO T 22.

Table 13. Details of test methods for determining HPC performance grades.

Performance Characteristic	Standard Test Method	Notes[1]
Freeze/Thaw Durability	AASHTO T 161 ASTM C 666 Proc. A	1. Test specimen 76.2 by 76.2 by 279.4 mm (3 by 3 by 11 inches) as cast or cut from 152.4- by 304.8-mm (6- by 12-inch) cylinder. 2. Acoustically measure dynamic modulus until 300 cycles.
Scaling Resistance	ASTM C 672	1. Test specimen to have a surface area of 46,451 mm^2 (72 $inch^2$). 2. Perform visual inspection after 50 cycles.
Abrasion	ASTM C 944	1. Concrete shall be tested at three different locations. 2. At each location, 98 Newtons (N), 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 C 799*, Procedure B.
Chloride Penetration	AASHTO T 277 ASTM C 1202	1. Test per standard test method.
Strength	AASHTO T 22 ASTM C 39	1. Molds shall be rigid metal or one-time-use rigid plastic. 2. Cylinders shall be 100 mm diameter by 200 mm long (3.9 by 7.8 inches) or 150 mm diameter by 300 mm long (5.9 by 11.2 inches). 3. Ends shall be capped with high-strength capping compound, ground parallel, or placed onto neoprene pads per AASHTO Specifications for Concretes. 4. Use of neoprene pads on early-age testing of concrete exceeding 70 MPa at 56 days should use neoprene pads on the 56-day tests. 5. The 56-day strength is recommended.
Elasticity	ASTM C 469	1. Test per standard test method.
Shrinkage	ASTM C 157	1. Use 76.2- by 76.2- by 285-mm (3- by 3- by 11.25-inch) specimens. 2. Shrinkage measurements are to start 28 days after moist curing and be taken for a drying period of 180 days.
Creep	ASTM C 512	1. Use 152- by 305-mm (6- by 12-inch) specimens. 2. Cure specimens at 73 ^{o}F and 50 percent relative humidity after 7 days until loading at 28 days. 3. Creep measurements to be taken for a creep loading period of 180 days.

[1] See footnote to table 12 for the curing periods to be used before testing.

* ASTM C 799 should be ASTM C 779.

Table 14. Recommendations for the application of HPC grades.

	Recommended HPC Grade for Given Exposure Condition				
Exposure condition	**N/A[2]**	**Grade 1**	**Grade 2**	**Grade 3**	**Grade 4**
Freeze/Thaw Durability Exposure (x= F/T cycles per year)[1]	x < 3	3 ≤ x < 50	50 ≤ x		
Scaling Resistance Applied Salt[3] (x = tons/lane-mile year)	x < 5.0	5.0 < x			
Abrasion Resistance (x = average daily traffic, studded tires allowed)	no studs/ chains	x ≤ 50,000	50,000 < x < 100,000	100,000 ≤ x	
Chloride Penetration Applied Salt[3] (x = tons/lane-mile year)	x < 1	1.0 ≤ x < 3.0	3.0 ≤ x < 6.0	6.0 ≤ x	

[1] F/T = freeze/thaw. A freeze/thaw cycle is defined as an event where saturated concrete is subjected to an ambient temperature, which drops below -2.2 ^{o}C (28 ^{o}F), followed by a rise in temperature above freezing.
[2] N/A = not applicable. NA indicates a situation in which specifications for an HPC performance grade are unnecessary.
[3] As defined in SHRP S-360.

PERFORMANCE GRADES IN FHWA SHOWCASE BRIDGES

Information on concrete mixtures, concrete properties, research projects, girder fabrication, bridge construction, live-load tests, and specifications from 19 HPC bridge projects was compiled in task A. The information was placed on a CD-ROM for easy retrieval and viewing.(32)

To provide a basis for evaluating the FHWA definition of HPC, data on the eight performance characteristics were extracted from the compilation and are summarized in tables 15 through 18. Tables 15 and 16 list specified and measured durability characteristics for precast, prestressed concrete girders and cast-in-place concrete decks, respectively. Tables 17 and 18 list specified and measured strength characteristics for precast, prestressed concrete girders and cast-in-place concrete decks, respectively. The data are based on a combination of measurements made as part of the quality control tests and measurements made as part of the research that was conducted on the bridges. It should be noted that not all of the tests were made exactly in accordance with the procedures defined in the FHWA HPC definition. Where the procedures were reported in sufficient detail to identify exceptions, the exceptions are noted as footnotes to the tables. A dash indicates that the characteristic was not specified or reported.

The data in tables 15 and 16 indicate that the primary characteristic specified for durability was chloride penetration, with values ranging from 1000 to 3000 coulombs (C). Approximately three-quarters of the specified values are grade 2 in the performance definition. The majority of the measured values are less than 2000 C, with half of the values less than 2000 C corresponding to grade 3 of the definition.

Table 15. Specified and measured durability characteristics for precast, prestressed concrete girders.

State	Bridge	Freeze-Thaw, %		Scaling		Abrasion, inches		Chloride Penetration,[1] C	
		Spec.	Meas.	Spec.	Meas.	Spec.	Meas.	Spec.	Meas.
AL	AL 199	—	98[2]	—	—	—	—	—	2,720
CO	Yale Ave.	—	—	—	—	—	—	—	—
GA	S.R. 920	—	—	—	—	—	—	3,000	3,963
LA	Charenton	—	—	—	—	—	—	2,000	1,355
NE	120th St.	—	—	—	—	—	0.024[3] 0.040[4]	—	292 377
NH	Route 104	—	107[5]	—	—	—	—	1,000	1,590
NH	Route 3A	—	—	—	—	—	—	1,500	1,100
NM	Rio Puerco	—	—	—	—	—	—	—	—
NC	U.S. 401	—	—	—	—	—	—	2,000	3,704-4,557
OH	U.S. 22	—	86	—	—	—	—	1,000	213[6]
SD	I-29 NB	—	—	—	—	—	—	—	65
SD	I-29 SB	—	—	—	—	—	—	—	88
TN	Porter	—	—	—	—	—	—	2,500[7]	390
TN	Hickman	—	—	—	—	—	—	2,500[7]	496
TX	Louetta NB	—	—	—	—	—	—	—	≤ 1,000
TX	Louetta SB	—	—	—	—	—	—	—	≤ 1,000
TX	San Angelo EB	—	—	—	—	—	—	—	≤ 1,000
TX	San Angelo WB	—	—	—	—	—	—	—	≤ 1,000
VA	Route 40	—	—	—	—	—	—	1,500[7]	228[7]
VA	VA Ave.	—	26[8] 91[8]	—	0-1[8]	—	—	1,500[7]	125[7]
WA	S.R. 18	—	100	—	—	—	—	1,000	496

[1] Tested at 56 days unless noted otherwise.
[2] Tested at 14 days after storage in limewater.
[3] After 30 min.
[4] After 60 min.
[5] Tested at 306 days.
[6] At 320 days.
[7] Tested at 28 days after 21 days at 100 °F.
[8] At 2 months using AASHTO T 161, procedure A, with test water containing a 2% NaCl solution.

Specimens were steam cured and air dried.

1 inch = 25.4 mm

Table 16. Specified and measured durability characteristics for cast-in-place concrete decks.

State	Bridge	Freeze-Thaw, %		Scaling		Abrasion, inches		Chloride Penetration,[1] C	
		Spec.	Meas.	Spec.	Meas.	Spec.	Meas.	Spec.	Meas.
AL	AL 199	—	92[2]	—	—	—	—	—	2,873
CO	Yale Ave.	—	—	—	—	—	0.025[3]	—	5,597
GA	S.R. 920	—	—	—	—	—	—	2,000	198
LA	Charenton	—	—	—	—	—	—	2,000	1,390
NE	120th St.	—	—	—	—	—	0.033[4] 0.059[5]	1,800	507 671
NH	Route 104	—	97[6]	—	0-1	—	—	1,000	753
NH	Route 3A	—	—	—	—	—	—	1,000	1,060
NM	Rio Puerco	—	—	—	—	—	—	—	—
NC	U.S. 401	—	—	—	—	—	—	—	—
OH[7]	U.S. 22	80	80	—	—	—	< 0.02	1,000	—
SD	I-29 NB	—	—	—	—	—	—	—	461
SD	I-29 SB	—	—	—	—	—	—	—	1,058
TN	Porter	—	—	—	—	—	—	1,500[8]	3,280
TN	Hickman	—	—	—	—	—	—	1,500[8]	317[8]
TX	Louetta NB	—	—	—	—	—	—	—	1,730
TX	Louetta SB	—	—	—	—	—	—	—	900
TX	San Angelo EB	—	97.9	—	2-3	—	0.04	—	703
TX	San Angelo WB	—	97.3	—	0	—	0.07	—	2,573
VA	Route 40	—	—	—	—	—	—	2,500[8]	778[8]
VA	VA Ave.	—	108[9]	—	0-1[9]	—	—	2,500[8]	1,457[8]
WA	S.R. 18	—	—	—	—	—	—	—	2,164-3,434[10]

[1] Tested at 56 days unless noted otherwise.
[2] At 14 days.
[3] ASTM C 779, procedure A, after 30 min.
[4] After 30 min.
[5] After 60 min.
[6] At 140 days.
[7] Values for abutments.
[8] Tested at 28 days after 21 days at 100 ºF.
[9] Tested at 6 months using AASHTO T 161, procedure A, with the test water containing a 2 percent NaCl solution. Specimens were moist cured for 2 months and then air dried.
[10] Between 3.5 and 6.5 months.

1 inch = 25.4 mm

Table 17. Specified and measured strength characteristics for precast, prestressed concrete girders.

State	Bridge	Compressive Strength,[1] psi		Modulus of Elasticity, ksi		Shrinkage, millionths		Creep, millionths/psi	
		Spec.	Meas.	Spec.	Meas.	Spec.	Meas.	Spec.	Meas.
AL	AL 199	10,000[1]	8,440-11,060	—	5,200-7,100[1]	—	370[2]	—	0.18[2]
CO	Yale Ave.	10,000	7,800-14,000	—	5,000	—	551[3]	—	0.493[3]
GA	S.R. 920	10,000	13,300	—	4,970	—	—	—	—
LA	Charenton	10,000	10,502-12,023	—	5,976	—	489[4]	—	0.315
NE	120th St.	12,000	13,944	—	6,280	—	410[5]	—	—
NH	Route 104	8000[1]	7,755[1]	—	5,350	—	—	—	—
NH	Route 3A	8000[1]	11,200[1]	—	—	—	—	—	—
NM	Rio Puerco	10,000	10,151	—	—	—	—	—	—
NC	U.S. 401	10,000[1]	11,800[1] -15,000	—	3,816[1]-5,240	—	470 510	—	0.214[6] 0.254[6]
OH	U.S. 22	10,000	9,570-12,920	—	4,647	—	900[7] 1,050	—	0.268[8]
SD	I-29 NB	9900[1]	15,900[1]	—	7,180	—	150[9]	—	—
SD	I-29 SB	9900[1]	13,250[1]	—	7,030[10]	—	140[9]	—	—
TN	Porter	10,000[1]	9,651[1]	—	6,660	—	—	—	
TN	Hickman	10,000[1]	10,529[1]	—	6,239	—	—	—	
TX	Louetta NB	13,100	14,440[1]	—	6,030-6,840	—	392[11]	—	0.222[11]
TX	Louetta SB	13,100	14,550[1]	—		—		—	
TX	San Angelo EB	14,000	15,240[1]	—	5,560-7,370	—	382[11]	—	0.205[11]
TX	San Angelo WB	8900[1]	10,130[1]	—	—	—	298[11]	—	0.420[11]
VA	Route 40	8000[1]	9,060[1] 11,490[1]	—	5,960-6,660	—	290-500[12]	—	—
VA	VA Ave.	10,000[1]	11,200[1]	—	5,900-6,300[12]	—	330-450[12]	—	—
WA	S.R. 18	10,000	12,220	—	—	—	—	—	—

[1] At 28 days. All other values at 56 days.
[2] 4- by 8-inch cylinders at 180 days after starting at a concrete age of 24 h.
[3] At 196 days. Measurements started at a concrete age of 1 day.
[4] 6- by 12-inch cylinders. Measurements started at a concrete age of 2 days.
[5] 2- by 2- by 10-inch prism at 64 weeks.
[6] 4- by 8-inch cylinder at 120 days.
[7] At 150 days.
[8] At 180 days after loading at 7 days.
[9] 6- by 6- by 12-inch prism in outdoor environment.
[10] At 90 days.
[11] Tests started at 2 days on 4- by 20-inch cylinders.
[12] At 1 year.

1000 psi = 6.895 MPa, 1000 ksi = 6.895 gigapascals (GPa), 1 millionth/psi = 145.04 millionths/MPa

Table 18. Specified and measured strength characteristics for cast-in-place concrete decks.

State	Bridge	Compressive Strengths,[1] psi		Modulus of Elasticity, ksi		Shrinkage, Millionths		Creep, millionths/psi	
		Spec.	Meas.	Spec.	Meas.	Spec.	Meas.	Spec.	Meas.
AL	AL 199	6,000	7,370	—	4,950–6,500	—	—	—	—
CO	Yale Ave.	5,076	5,310	—	—	—	—	—	—
GA	S.R. 920	7,250[1]	7,740	—	3,610	—	—	—	—
LA	Charenton	4,200	5,493	—	4,161	—	—	—	—
NE	120th St.	8,000[1]	10,433[1]	—	5,440	—	390[2]	—	
NH	Route 104	6,000	9,020	—	4,250	—	—	—	—
NH	Route 3A	6,000	9,004	—	—	—	—	—	—
NM	Rio Puerco	6,000	6,160	—	—	—	—	—	—
NC	U.S. 401	6,000	7,150 5,700	—	—	—	—	—	—
OH[3]	U.S. 22	8,000[1]	8,565[1]	—	4,447[1]	—	410[4]	—	—
SD	I-29 NB	4,500	7,070	—	5,220	—	200[5]	—	—
SD	I-29 SB	4,500	6,170	—	5,620	—	0[5]	—	—
TN	Porter	5,000	8,265	—	4,501	—	—	—	—
TN	Hickman	5,000	6,460	—	—	—	—	—	—
TX	Louetta NB	4,000	5,700	—	4,520	—	296[6]	—	0.317[6]
TX	Louetta SB	8,000	9,100	—	5,170	—	344[6]	—	0.320[6]
TX	San Angelo EB	6,000	7,345	—	4,920–6,060[1]	—	265[7]	—	0.240[7]
TX	San Angelo WB	4,000	6,120	—	4,310–5,170[1]	—	462[7]	—	0.722[7]
VA	Route 40	4,000	6,600	—	5,590–6,320[8]	—	450–590[9]	—	—
VA	VA Ave.	5,000	5,400	—	5,240[8]	—	—	—	—
WA	S.R. 18	4,000	5,490	—	—	—	—	—	—

[1] At 56 days. All other values at 28 days.
[2] 2- by 2- by 10-inch prism at 64 weeks.
[3] Values for abutments.
[4] At 124 days.
[5] 6- by 6- by 12-inch prism in outdoor environment.
[6] 4- by 20-inch cylinders loaded at 2 days.
[7] 4- by 20-inch cylinders.
[8] At 1 year.
[9] At 64 weeks.

1000 psi = 6.895 MPa, 1000 ksi = 6.895 GPa, 1 millionth/psi = 145.04 millionths/MPa.

Freeze-thaw resistance was specified for only one bridge and scaling resistance and abrasion resistance were not specified for any bridges. However, these characteristics were measured for several bridges.

The observation that only chloride penetration was frequently specified for durability may reflect reluctance on the part of States to specify other characteristics. This reluctance may be a result of a combination of the following factors:

- Lack of familiarity with the test methods.
- Lack of in-house capability to perform the tests.
- Impact on the cost of concrete when additional characteristics are specified.
- Increased time needed to perform the tests.

For the strength characteristics, compressive strength was the only characteristic specified for girders and decks of all bridges. For the majority of the bridges, the specified strength for the girder concrete was 69 MPa (10,000 psi), corresponding to the lower limit of grade 3. The majority of the measured strengths were in the range of 69 to 97 MPa (10,000 to 14,000 psi), corresponding to grade 3. For the decks, the specified strengths ranged from 28 to 41 MPa (4000 to 6000 psi), except for Georgia, Nebraska, and one bridge in Texas. The range of 28 to 41 MPa (4000 to 6000 psi) is outside the range of the strength performance characteristics for HPC. This is to be expected since there is no reason to specify an HSC for the deck in most slab and girder bridges. For decks, the emphasis should be on durability. An HSC does not ensure a durable concrete.

The observation that compressive strength was the only strength characteristic specified for the HPC bridges probably indicates that it was not necessary to specify modulus of elasticity, shrinkage, or creep for the types of bridges that were built. These characteristics are more significant in long-span structures where control of deflections and prestress losses are more important. Most of the bridges described in this report consisted of cast-in-place concrete deck slabs on precast, prestressed concrete girders. A difficulty in specifying shrinkage and creep criteria is that the tests must be run for 180 days. For most bridges, this means a delay between the time of awarding a contract and casting the HPC members. For shrinkage of deck concrete, selection of a specific value to reduce cracking is somewhat arbitrary because there is no direct correlation between the shrinkage of a laboratory specimen and the likelihood of cracking in the deck.

For most of the bridges listed in table 17, the modulus of elasticity was measured. Values ranged from about 28 to 48 GPa (4000 to 7000 kips/inch^2), corresponding to grades 1 and 2.

Although shrinkage and creep were measured on several projects, it is difficult to draw any conclusions since the specimen sizes, curing conditions, and ages at which values are reported varied considerably and often deviated from the procedures described in table 13.

REVIEW OF EXISTING CHARACTERISTICS, GRADES, AND TEST METHODS

General

Various definitions of HPC have been developed over the years.(33) Most of these definitions define HPC in a qualitative manner. The FHWA definition is the first to quantify the definition for a variety of characteristics and provides criteria for HPC. The word *definition* implies a few words or a phrase, whereas the FHWA definition provides a classification system for HPC and the word *classification* is suggested for future use.

The existing definition uses a maximum of four grades to classify HPC for each performance characteristic. However, all four grades are not used for each characteristic. This appears to cause confusion. For one characteristic, grade 2 is the highest and, for another, grade 4 is the highest. To simplify the system, it is recommended that each characteristic have three grades.

With the change to provide three grades for each characteristic, revisions to table 14 for freeze-thaw durability and scaling resistance are needed. These should be done by the original authors since they know the basis for the original recommendations.

Another item that has caused confusion is that each grade has an upper and lower limit. This is necessary when determining the grade for a particular test result. However, when specifying a grade, an upper or lower limit may not be appropriate. For example, concrete that is specified to have a scaling resistance of grade 2 is still acceptable if the actual scaling resistance is grade 3. Consequently, the grades should always be considered minimum performance values, even though upper and lower limits are specified. In other words, any actual grade that is higher than that specified is acceptable.

Freeze-Thaw Durability

Resistance to cycles of freezing and thawing is important for structures that can become critically saturated and are exposed to a severe freeze-thaw environment. Thus, it is only important in regions experiencing cycles of freezing and thawing. Without appropriate measures, cracking, scaling, and disintegration of the concrete can occur.

For proper resistance to freezing and thawing, concretes must have sound aggregates, a proper air-void system, and have matured prior to freezing. This generally means the development of a compressive strength of about 28 MPa (4000 psi).(34) The freezing and thawing damage can occur in both the cement paste and the aggregate.(35)

Some rocks have pore sizes that are not large enough to expel water, so hydraulic pressures in the aggregates can occur and lead to a distress known as D-cracking. Optimizing the air-void system in the concrete cannot mitigate this distress.(36) Proper selection of the coarse aggregate or the reduction of the coarse aggregate size may be needed.(37) When the resistance of the paste is of concern, as in the majority of cases, the air-void system can be evaluated instead of testing for freezing and thawing. ASTM C 457 describes the determination of the air-void parameters and can be used to predict the performance of concrete exposed to a severe environment.

The most commonly used test for resistance to freezing and thawing is AASHTO T 161 (ASTM C 666). There are two procedures: Procedure A involves rapid freezing and thawing in water; procedure B requires rapid freezing in air and thawing in water. Procedure A is a severe test and concretes performing well in this test have done well in field applications. However, concretes failing the test may also have satisfactory field performance; however, such performance must be proven in the field. In procedure A, the specimen is subjected to cycles of freezing and thawing much faster than expected in the field. Specimens are also continuously kept moist and are moist cured for two weeks and tested without allowing any drying. Critics raise concerns because of the harsh environment of the test procedure. HPC designed for longevity is less likely to become saturated because of its low permeability. In fact, the ACI 318 Building Code permits the reduction of the required air contents by 1 percent for concretes with specified strengths exceeding 34 MPa (5000 psi).(18) The ACI commentary states that HSC has low water-cementitious materials ratios and porosity and, therefore, improved frost resistance. However, it should be emphasized that strength alone does not ensure freeze-thaw durability and a proper air-void system is required. Also, in procedure A, the specimens are moist cured for 2 weeks without a drying period. In the field, concretes are generally dry before exposure to cycles of freezing and thawing. Therefore, in procedure A, a more realistic approach to curing all specimens (not just HPC specimens) would be to extend the required 2 weeks of curing to a longer time period of 1 or 2 months, and then to let the concrete dry for at least a week before starting the test.

Grade 1 of the current definition has a lower limit for the relative dynamic modulus of 60 percent after 300 cycles. This limit corresponds with the end point given in the test procedure, which was developed for conventional concretes. For HPC, a higher standard needs to be set. Consequently, a new set of values, as shown in table 19, is proposed since HPC structures are expected to have longer service lives.

Scaling Resistance

Scaling is a freezing- and thawing-related deterioration starting at the surface. It progresses into the concrete with additional freezing and thawing. Hydraulic and osmotic pressures cause great stresses that lead to cracking of the concrete. Osmotic pressure is enhanced by the presence of deicing salts. The deterioration is greater for intermediate concentrations of chemicals (3 to 4 percent salt solution by weight) than for lower or higher concentrations.(38) The resistance to scaling is improved by providing an adequate air-void system, proper finishing and curing, a drying period before the salt application, and reduced permeability and water-cementitious materials ratios. Air-void analysis (ASTM C 457) can be used to indicate the level of resistance to scaling of concretes properly proportioned, placed, finished, and cured.

The standard test for scaling resistance is ASTM C 672. In ASTM C 672, specimens are subjected to a 4 percent solution of calcium chloride and are rated from 0 to 5 (0 represents sound concrete with no scaling and 5 represents severe scaling with coarse aggregate visible over the entire surface). However, it is also possible to modify the ASTM C 666 test with the addition of sodium chloride to water surrounding the specimens to determine the effects of salt scaling. Virginia DOT uses 2 percent sodium chloride in the test water of ASTM C 666, procedure A.(39)

Table 19. Proposed grades of performance characteristics for high-performance structural concrete.[1]

Performance Characteristic[2]	Standard Test Method	FHWA HPC Performance Characteristic Grade[3]		
		1	2	3
Freeze-thaw durability[4] (F/T = relative dynamic modulus of elasticity after 300 cycles)	AASHTO T 161 ASTM C 666 Proc. A	70% ≤ F/T < 80%	80% ≤ F/T < 90%	90% ≤ F/T
Scaling resistance[5] (SR = visual rating of the surface after 50 cycles)	ASTM C 672	3.0 ≥ SR > 2.0	2.0 ≥ SR > 1.0	1.0 ≥ SR ≥ 0.0
Abrasion resistance[6] (AR = avg. depth of wear in mm)	ASTM C 944	2.0 > AR ≥ 1.0	1.0 > AR ≥ 0.5	0.5 > AR
Chloride penetration[7] (CP = coulombs)	AASHTO T 277 ASTM C 1202	2500 ≥ CP > 1500	1500 ≥ C P> 500	500 ≥ CP
Alkali-silica reactivity (ASR = expansion at 56 days) (%)	ASTM C 441	0.20 ≥ ASR > 0.15	0.15 ≥ ASR > 0.10	0.10 ≥ ASR
Sulfate Resistance (SR = expansion) (%)	ASTM C 1012	SR ≤ 0.10 at 6 months	SR ≤ 0.10 at 12 months	SR ≤ 0.10 at 18 months
Flowability (SL = slump, SF = slump flow)	AASHTO T 119 ASTM C 143, and proposed slump flow test	SL > 190 mm (SL > 7.5 inches), and SF < 500 mm (SF < 20 inches)	500 ≤ SF ≤ 600 mm (20 ≤ SF ≤ 24 inches)	600 mm < SF (24 inches < SF)
Strength (f'_c = compressive strength)	AASHTO T 22 ASTM C 39	$55 \leq f'_c < 69$ MPa ($8 \leq f'_c < 10$ ksi)	$69 \leq f'_c < 97$ MPa ($10 \leq f'_c < 14$ ksi)	97 MPa $\leq f'_c$ (14 ksi $\leq f'_c$)
Elasticity[8] (E_c = modulus of elasticity)	ASTM C 469	$34 \leq E_c < 41$ GPa ($5 \leq E_c < 6 \times 10^6$ psi)	$41 \leq E_c < 48$ GPa ($6 \leq E_c < 7 \times 10^6$ psi)	48 GPa $\leq E_c$ (7×10^6 psi $\leq E_c$)
Shrinkage[9] (S = microstrain)	AASHTO T 160 ASTM C 157	800 > S ≥ 600	600 > S ≥ 400	400 > S
Creep[10] (C = microstrain/pressure unit)	ASTM C 512	75 ≥ C > 55/MPa (0.52 ≥ C > 0.38/psi)	55 ≥ C > 30/MPa (0.38 ≥ C > 0.21/psi)	30/MPa ≥ C (0.21/psi ≥ C)

[1] This table does not represent a comprehensive list of all characteristics that good concrete should exhibit. It does list characteristics that can quantifiably be divided into different performance groups. Other characteristics should be checked. Only one characteristic is sufficient for HPC.
[2] For non-heat-cured products, all tests are to be performed on concrete samples moist, submersion, or match cured for 56 days or until test age. For heat-cured products, all tests are to be performed on concrete samples cured with the member or match cured until test age. See table 13 for additional information and exceptions.
[3] A given HPC mix design is specified by a grade for each desired performance characteristic. A higher grade indicates a higher level of performance. Performance characteristics and grades should be selected for the particular project. For example, a concrete may perform at grade 3 in strength and elasticity, grade 2 in shrinkage and scaling resistance, and grade 2 in all 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, RD104.
[8] Based on SHRP C/FR-91-103, p. 3.17.
[9] Based on SHRP C/FR-91-103, p. 3.25.
[10] Based on SHRP C/FR-91-103, p. 3.30.

The lower limit for grade 1 in the current definition is a rating of 5.0. Concrete that has deteriorated to the extent that coarse aggregate is visible over the entire surface should not be considered as HPC. Consequently, a revision to the grades, as shown in table 19, is proposed. A range of ratings is proposed rather than single values since the rating should be the average of at least two specimens.

Sometimes, the use of pozzolans or slag in air-entrained HPC exposed to a severe environment has been attributed to causing increased scaling. Such scaling has been limited to only a very thin surface layer. When pozzolans or slag are substituted for a portion of the portland cement, concrete strength and maturity develop at a slower rate, especially in cold weather. Concretes with lower strengths and less maturity are expected to scale more. Consequently, it is important to ensure that the concretes with pozzolans or slag have the same strength as the control concretes when this test is used for comparison purposes.(40)

Abrasion Resistance

Abrasion resistance is defined by ACI 116R as the ability of a surface to resist wear from rubbing and friction.(41) The abrasion resistance is improved by increasing the compressive strength, using hard and dense aggregates, and proper finishing and curing methods.(42-44) This property is of great importance in transportation facilities. The traveled surfaces must have adequate skid resistance for proper vehicular control. Skid resistance is affected by both the microtexture provided by the aggregate particles and the macrotexture mainly provided by the grooves formed on freshly mixed concrete or the grooves cut in the hardened concrete (ACI 325.6R).(45) Deep texture also enables the drainage of water, preventing hydroplaning as the tire loses contact with the pavement surface.

In some locations on the roadway, such as the acceleration and deceleration lanes, tollbooth areas, where chains or studded tires are used, and for elements in water exposed to abrasive material, abrasion resistance is required. Studded tires can cause considerable wear and an NCHRP report addresses pavement wear in the presence of studded tires.(46) Abrasive materials such as sand are used widely with deicing salts and have caused little damage to quality concrete surfaces. Thus, in bridge structures, the need for an abrasion test is limited. ASTM C 944 is the standard test procedure used for HPC. This test method is similar to procedure B of ASTM C 779. A rotating cutter abrades the surface of the concrete under load. It has been successfully used in the quality control of highway and bridge concrete subjected to traffic. The equipment is not common and many laboratories do not test for abrasion. Also, the test results have a large variability of 12.6 percent for the single-operator coefficient of variation.

It is recommended that this test be specified only for locations where abrasion resistance is a critical issue. No change to FHWA's abrasion resistance values is recommended. However, clarification is needed in the FHWA definition concerning the force to be applied to the test specimen.(31) In table 13, a force of 98 N is listed. However, in the text of the original article, a force of 196 N is listed. Both force levels are permitted by the test procedure.

Chloride Penetration

The durability of concrete exposed outdoors depends largely on its ability to resist the penetration of water and aggressive solutions. There are four major types of environmental distress in reinforced concrete: (1) corrosion of the reinforcement, (2) alkali-aggregate reactivity, (3) freezing and thawing deterioration, and (4) attack by sulfates.(47) Corrosion of the reinforcing steel is the most extensive of these. In each case, water or solutions penetrating into the concrete initiates or accelerates the distress, making costly repairs necessary. Air-entrained concretes that have low penetrability are necessary to resist infiltration of aggressive liquids and provide the necessary resistance to freezing and thawing when exposed to the environment.

The recommended chloride penetrability test for HPC is AASHTO T 277 (ASTM C 1202). In this test, the charge passed in coulombs through a saturated specimen 50-mm (2-inches) thick and 100 mm (4 inches) in diameter, and subjected to 60 volts (V) direct current (dc) in a 6-h period is determined.(48) Low values indicate high resistance to penetration by solutions. This test gives a good indication of permeability with proper testing, the absence of interferences, and proper interpretation.

All concretes exposed outdoors or cured in a moist room exhibit a reduction in coulomb values with time, and different concretes have different rates of reduction.(23) However, specimens air-dried in the laboratory do not exhibit the expected reduction.(49) Thus, the method and duration of curing of the concretes subjected to the AASHTO T 277 test are important. Virginia DOT moist cures specimens 1 week at 23 ^{o}C (73 ^{o}F) and 3 weeks in water at 38 ^{o}C (100 ^{o}F) to obtain coulomb values similar to those obtained after 6 months of curing at 23 ^{o}C (73 ^{o}F). The test age of 56 days in the original HPC definition paper is too early for concretes with pozzolans and slag. The classification should allow testing at later ages when the construction schedule permits.

Interferences, such as the presence of calcium nitrite, can produce misleading results from the test method. One option is to test concrete with and without the interfering ingredient and make corrections to the required values. Another option is the new test developed for FHWA. In this new test, the actual penetration of chlorides is measured.(50) The chlorides are driven into the concrete using a direct voltage similar to AASHTO T 277. Then the sample is split open and treated with a silver nitrate solution. Reaction of the silver nitrate with chlorides forms white silver chloride, which can be visually detected. If there is doubt about the effect of ingredients on the results of the electrical tests, a correlation with the ponding test (AASHTO T 259) is recommended.

The limits for grade 1 in the current definition are 3000 and 2000 C. Data from the FHWA showcase bridges show that most States specified 2500 C or less for both girders and deck. The measured results showed many values less than 1000 C. Consequently, it is recommended that the upper limit be lowered to 2500 C and the ranges for each grade be adjusted accordingly. A new set of values, shown in table 19, is suggested. These are more conservative than the current values since HPC is expected to have a longer service life. Coulomb values should be as low as possible as long as it is practical, economical, and does not have adverse effects on other properties. A possible revision to AASHTO T 277 would allow testing at a concrete age of 28 days, provided that accelerated curing of concrete is used prior to the test. This accelerated test will make it easier to meet the requirements in less time since it produces similar values to tests at later ages.

Compressive Strength

Concrete compressive strength is the only performance characteristic that is always specified for both conventional concrete and HPC. With time, the upper strength level of HSC has increased. However, in the United States, the lower limit has remained at 41 MPa (6000 psi) for many years.[15] This is reflected in the lower limit for grade 1 compressive strength in the current definition.

The primary application of HSC in bridge structures is in precast, prestressed concrete girders. With today's technology, most precast concrete producers can achieve compressive strengths of 41 MPa (6000 psi) relatively easily. Therefore, it is recommended that the lower limit for the strength characteristic be raised to 55 MPa (8000 psi) and that the following performance grades be used:

Grade 1: $55 \leq f_c' < 69$ MPa ($8000 \leq f_c' < 10{,}000$ psi)

Grade 2: $69 \leq f_c' < 97$ MPa ($10{,}000 \leq f_c' < 14{,}000$ psi)

Grade 3: $f_c' \geq 97$ MPa ($f_c' \geq 14{,}000$ psi)

Modulus of Elasticity

The modulus of elasticity of concrete is the ratio of stress to strain in the elastic range of a stress-strain curve. According to ASTM C 469, it is calculated as the slope of a straight line between two points on the stress-strain curve. The upper point corresponds to a stress equal to 40 percent of the measured compressive strength. The lower point corresponds to a strain of 50 millionths. It has long been accepted that the modulus of elasticity is approximately proportional to the square root of the concrete compressive strength, i.e., $E_c = 57{,}000\sqrt{f_c'}$.

The current lower limit for the modulus of elasticity in grade 1 is 28 GPa (4000 ksi). Using the current conventional equation relating modulus of elasticity to compressive strength, this corresponds to a compressive strength of about 34 MPa (5000 psi). Based on the data shown in table 17, and the proposed changes in the grades for strength, it is recommended that the lower limit for the modulus of elasticity be increased to 34 GPa (5000 ksi) and the following three grades be used:

Grade 1: $34 \leq E_c < 41$ GPa ($5000 \leq E_c < 6000$ ksi)
Grade 2: $41 \leq E_c < 48$ GPa ($6000 \leq E_c < 7000$ ksi)
Grade 3: $E_c \geq 48$ GPa ($E_c \geq 7000$ ksi)

It is also possible to have situations such as bridge decks or seismic zones where a maximum value for modulus may be appropriate.

Shrinkage

Drying shrinkage is a shortening that results from loss of moisture from the concrete. The magnitude and rate of shrinkage depend on many factors, including concrete constituent materials, size of the member, amount of nonprestressed reinforcement, and ambient environment. Consequently, in a test procedure for shrinkage, it is important to specify the conditions for the test. The test procedure, AASHTO T 160 (ASTM C 157), involves measuring the length change of a concrete prism made of concrete similar to that to be used in the actual structure. The length change is generally measured using a comparator. For this test, it is important that the concrete be stored at a constant temperature and humidity.

The curing procedure for the shrinkage specimens states that measurements are to start at 28 days after moist curing. To be consistent with the curing procedures of ASTM C 157, measurements should begin at a concrete age of 28 days. Generally, this includes 27 days of moist curing.

The stated curing procedure in the current definition provides a means to compare the shrinkage of concretes subjected to the same curing procedures. It does not provide data that represent shrinkage in a heat-cured product since heat curing affects the amount of shrinkage. If the intent of the test is to provide a measure of the shrinkage in the product, the specimen curing should be as close as possible to that of the product. This is true for all concretes and needs to be clarified in the procedure.

It should be noted that shrinkage measured in the ASTM C 157 test is that of a small specimen stored at 50 percent relative humidity. Shrinkage in a bridge member is less because the member is thicker and the outdoor relative humidity is generally higher than 50 percent.

Creep

Creep is the change in length of a concrete member when subjected to a sustained load. The amount and rate of creep depend on concrete constituent materials, age and strength of concrete at time of load application, length of time under load, size of the member, amount of nonprestressed reinforcement, and ambient environment. In the FHWA definition, creep is defined as specific creep, which is the change in length divided by the applied sustained stress. It does not include the initial length change that occurs when the load is first applied.

Creep tests in accordance with ASTM C 512 are made using special rigs so that the stress on the specimen remains reasonably constant over the duration of the test. These special rigs are expensive and only a few testing laboratories in the United States are equipped to perform the test. This may explain some of the reluctance to specify the creep characteristic. It is recommended that the creep characteristic only be specified for special structures where creep deformations are significant in the structural design, such as long-span segmental box girder bridges and highly prestressed girders.

The current definition for creep includes four grades. For consistency with other characteristics, it is proposed that the number of grades be reduced to three as follows:

Grade 1: $75 \geq C > 55$/MPa ($0.52 \geq C > 0.38$/psi)
Grade 2: $55 \geq C > 30$/MPa ($0.38 \geq C > 0.21$/psi)
Grade 3: $C \leq 30$/MPa ($C \leq 0.21$/psi)

The curing procedure for heat-cured products needs clarification. Table 13 states that the specimens are to be cured at 23 ^{o}C (73 ^{o}F) and 50 percent relative humidity after 7 days until loading at 28 days. The general statement about curing for steam-cured products states that specimens are to be cured as close as possible to the curing of the product until test age. If the intent of the test is to provide a measure of the creep in the product, the specimen curing should be as close as possible to that of the product. This is true for all concretes and needs to be clarified in the procedure.

ADDITIONAL CHARACTERISTICS, GRADES, AND TEST METHODS

FHWA's HPC criteria has four durability parameters: (1) resistance to freezing and thawing, (2) scaling resistance, (3) abrasion resistance, and (4) chloride penetration.[31] These are valid properties; however, some additional properties such as resistance to alkali-silica reaction, resistance to sulfate attack, and flowability are desirable and are explained below.

The current HPC definition covers only the hardened concrete properties; however, the fresh concrete properties are also important in the development of HPC. It is not possible to obtain HPC with specified durability and strength unless workable concretes are used. Concrete that is not workable cannot be properly consolidated; therefore, permeability and strength are adversely affected. The consolidation effort needed is related to the consistency or the workability of the concrete. There is also a self-consolidating concrete (SCC) that does not need any vibration. The flow characteristics of SCC are measured by a proposed flow test.

Curing is difficult to measure. Many specifications are prescriptive and specify the method of curing. Sometimes the properties are tested to ensure that they can be achieved by the curing method used. For example, if strength is specified, the maturity method can be used. In this method, the time-temperature history is used as a measure of the concrete maturity. It is assumed that there is sufficient water for hydration. In the laboratory, maturity is correlated with strength (or another property) on the same concrete mixture to be used in the field. Then the maturity calculated from the actual structure is used to determine the strength (or another property) from the laboratory-generated relationship. The maturity method is given in AASHTO TP52, ASTM C 918, and ASTM C 1074. Further work is needed before this method can be recommended to measure characteristics.

A performance characteristic related to cracking is highly desirable since the performance of bridge decks is generally better when they do not exhibit cracking. Cracking, however, is not an inherent property of concrete, depend as it does on other characteristics such as shrinkage, creep, heat of hydration, environmental temperature changes, and degree of external restraint. Consequently, it is not possible to recommend cracking as a performance characteristic until additional research is performed and a generally accepted test method is available.

Alkali-Silica Reactivity

A chemical reaction between aggregates containing reactive silica, and the alkalies in cement can produce an alkali-silica gel. The gel swells when water is absorbed, causing concrete distress. The reactivity of aggregates varies depending on the presence of noncrystalline or poorly or imperfectly crystalline silica.[42,51] Amorphous forms, such as opal and volcanic ashes, are highly reactive.

When the alkalies are present in a sufficient amount, the resulting high hydroxide ion concentrations can cause the formation of an expansive gel. Traditionally, a total alkali content below 0.60 percent is considered sufficiently low to avoid expansion; however, this limit may not provide the needed protection in all cases.[51] However, alkali-silica reactivity (ASR) has not been evident when a limit of 0.40 percent was used.[52] In many areas, it is not practical or economical to restrict the use of reactive siliceous aggregates to prevent ASR. The use of pozzolans or slag is effective in preventing ASR damage since they: (1) tie up hydroxide ions, preventing the formation of expansive gel; (2) reduce the concentration of alkalies to a safe level by replacing portions of portland cement; or (3) lower the permeability of concretes, thus preventing the penetration of alkalies from outside sources. Lithium salts have also been found to mitigate ASR and are also being tried to reduce ASR in existing structures.

ASTM C 441 covers the determination of the effectiveness of pozzolans or slag in preventing excessive expansion of concrete as a result of ASR. It is proposed that alkali-silica reactivity using ASTM C 441 be added to the definition. Three grades are proposed and listed in table 19. Grades are selected based on the reactivity of the aggregates. ASTM C 1260 is the standard test method for the reactivity of aggregates. If the aggregates are reactive, a pozzolan or slag is added and the effectiveness of this mixture in preventing excessive expansion is determined by ASTM C 441. It is recommended that the three grades be applied as follows:

- Grade 1 is for moderately reactive aggregates, expansion equals or exceeds 0.1 percent at 14 days when tested in accordance with ASTM C 1260.

- Grade 2 is for highly reactive aggregates, expansion equals or exceeds 0.2 percent at 14 days when tested in accordance with ASTM C 1260.

- Grade 3 is for aggregates that exhibit very high reactivity, expansion equals or exceeds 0.4 percent at 14 days when tested in accordance with ASTM C 1260.[53-54]

The above recommendations are shown in table 20 and should be added to table 14 (table 3 of reference 31).

Sulfate Resistance

Sulfates in solution react with the aluminate hydrates of the cement. Since the final reaction product occupies a larger volume than the original constituents, this can result in cracking, scaling, and disintegration of concrete.[55-56] To protect against sulfate attack, the use of cement with low tricalcium aluminate, certain pozzolanic materials that tie up lime and lower the tricalcium aluminate content when used as a replacement, or low-permeability concrete is recommended.(42) Sulfate damage in transportation structures is very limited because moderately sulfate-resisting cements (type II cements) are normally used. These cements have a maximum tricalcium aluminate content of 8 percent.

ASTM C 1012 is the test method for determining the length change of mortar bars immersed in a sulfate solution. It is commonly used to determine the sulfate resistance of concretes. Three grades as listed in table 19 are proposed for sulfate resistance. It is recommended that the three grades be applied as follows:

- Grade 1 corresponds to mild exposure (sulfates less than or equal to 150 parts per million (ppm)).
- Grade 2 applies to moderate exposure (sulfates between 150 and 1500 ppm).
- Grade 3 applies to severe exposure (sulfates greater than 1500 ppm).(44)

The above recommendations are shown in table 20.

Table 20. Recommendations for the application of HPC grades for new durability characteristics.

Exposure Condition	Recommended HPC Grade for Given Exposure Condition			
	N/A	Grade 1	Grade 2	Grade 3
Alkali-Silica Reactivity (x = expansion in percent[1])	$x < 0.1$	$0.1 \leq x < 0.2$	$0.2 \leq x < 0.4$	$0.4 \leq x$
Sulfate Resistance (x = sulfates in ppm)	$x = 0$	$0 < x \leq 150$	$150 < x \leq 1500$	$1500 < x$

[1] At 14 days when tested in accordance with ASTM C 1260.

Flowability

It is important that the concrete mixture has the consistency that enables easy mixing, placing, consolidating, and finishing without segregation. Consistency is the ability of concrete to flow. Workability is the property that determines the ease with which concrete can be mixed, placed, consolidated, and finished to a homogeneous condition.(40) Factors affecting workability are water content; maximum size, grading, shape, and texture of the aggregates; and the water-cementitious materials ratios.(40) Slump test AASHTO T 119 measures the consistency of freshly mixed concrete and is also used to indicate workability. Low slump values require special equipment and are generally difficult to work with. Specifications generally require a lower limit to enable proper placement and consolidation and an upper limit to prevent segregation. With the use of high- or mid-range water-reducing admixtures, concretes with higher slump values can be prepared without segregation. Flowing concrete is characterized by slump greater than 190 mm (7.5 inches), while remaining cohesive (ASTM C 1017). A new family of high-range water-reducing admixtures (HRWRA) is available to produce flowing concrete that does not require any consolidation effort. They have high flowability. In these concretes, the slump flow, which is the diameter of the spread rather than the slump value, is determined.

It is recommended that workability, as shown in table 19, be added to the performance characteristics using the following grades:

- Grade 1 is for concretes that do not segregate and have a slump greater than 190 mm (7.5 inches) measured using AASHTO T 119 (ASTM C 143), and slump flow less than 500 mm (20 inches) measured by the proposed slump flow test.
- Grade 2 is for flowing concretes with a slump flow of 500 to 600 mm (20 to 24 inches) measured using a proposed slump flow test.
- Grade 3 is for flowing concretes with a slump flow greater than 600 mm (24 inches) measured using a proposed slump flow test.

CONCLUSIONS

The purpose of this review is to address the following questions:

1. Are the performance characteristics appropriate?
2. Are the test methods appropriate?
3. Are modifications to the test methods needed?
4. Is the range of grades appropriate?
5. Do the values assigned to each grade need modification?

The eight existing characteristic are appropriate with the addition of performance characteristics for alkali-silica reactivity, sulfate resistance, and flowability. However, any characteristic should only be specified when it is needed for the intended application. Abrasion resistance and creep should only be specified for special situations.

The answers to questions 2 through 5 are summarized in tables 14 and 15 for durability and strength characteristics, respectively.

Table 21. Summary of recommendations for durability characteristics.

<table>
<tr><th>Question</th><th>Freeze-Thaw Durability</th><th>Scaling Resistance</th><th>Abrasion Resistance</th><th>Chloride Penetration</th></tr>
<tr><td>2. Test method appropriate?</td><td>Yes, with modifications</td><td>Yes</td><td>Yes, however, it has great variability and equipment is not readily available.</td><td>Yes, except for curing</td></tr>
<tr><td>3. Any modification?</td><td>a. Longer curing.
b. Drying after moist curing.</td><td>ASTM C 666 can be used with modification.</td><td>Clarify force level</td><td>Use accelerated curing or allow testing later than 56 days.</td></tr>
<tr><td>4. Range of grades appropriate?</td><td colspan="4" rowspan="2">Three grades are now proposed for each characteristic. Since, there is lack of information correlating field condition severity and laboratory performance, the values in the grades serve only as suggestions. As more data are collected, the appropriateness of grades and values in each grade can be evaluated.</td></tr>
<tr><td>5. Values in grades appropriate?</td></tr>
<tr><td>Comments</td><td>Air-void analysis is an alternative method (ASTM C 457).</td><td>Air-void analysis is an alternative method (ASTM C 457).</td><td>Use only in special cases for comparing different mixtures.</td><td>Consider the rapid migration test when interferences are present.</td></tr>
</table>

Table 22. Summary of recommendations for strength characteristics.

<table>
<tr><th>Question</th><th>Compressive Strength</th><th>Modulus of Elasticity</th><th>Shrinkage</th><th>Creep</th></tr>
<tr><td>2. Test method appropriate?</td><td>Yes</td><td>Yes</td><td>Yes</td><td>Yes</td></tr>
<tr><td>3. Any modification?</td><td>No</td><td>No</td><td>Revise curing of specimens before start of the test measurements.</td><td>Revise curing of specimens before start of the test measurements.</td></tr>
<tr><td>4. Range of grades appropriate?</td><td colspan="4">Three grades are now proposed for each characteristic.</td></tr>
<tr><td>5. Values in grades appropriate?</td><td colspan="4">Strength grades are selected by the engineer for the intended application. It is not appropriate to assign values in the definition.</td></tr>
<tr><td>Comments</td><td>None</td><td>None</td><td>None</td><td>Use only in special structures.</td></tr>
</table>

RECOMMENDATIONS

Based on the above discussions, the following recommendations are made:

- Refer to the HPC definition as a “classification” and not as a “definition.”
- Include ASR and sulfate resistance in the durability characteristics.
- Add workability as another characteristic that affects both durability and strength.
- For all tests, define the curing procedure to be used for test specimens when accelerated curing is used for the concrete member.
- For all characteristics, provide three grades.
- Revise the values in each grade to reflect recent data and experience, and to raise the performance levels.
- For resistance to freezing and thawing and scaling, consider air-void analysis as an alternative.
- Extend the curing time and dry the specimens before testing for resistance to freezing and thawing.
- Only specify the abrasion resistance test for concretes in special areas, such as tollbooths, acceleration and deceleration lanes, and when the use of studded tires or heavy chains is anticipated.
- For chloride penetration, offer an accelerated test as an alternative and consider the rapid migration test, especially when interferences are present.
- Only specify creep for special structures.

AASHTO AND ASTM SPECIFICATIONS

The following AASHTO and ASTM specifications are mentioned in this article:

AASHTO Specifications

AASHTO T 22 (ASTM C 39) Compressive Strength of Cylindrical Concrete Specimens

AASHTO T 119 (ASTM C 143) Slump of Hydraulic Cement Concrete

AASHTO T 160 (ASTM C 157) Length Change of Hardened Hydraulic Cement Mortar and Concrete

AASHTO T 161 (ASTM C 666) Resistance of Concrete to Rapid Freezing and Thawing

AASHTO T 259 Resistance of Concrete to Chloride Ion Penetration

AASHTO T 277 (ASTM C 1202) Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration

AASHTO T XX1 Proposed Slump Flow Test

AASHTO TP52 Estimating the Strength of Concrete in Transportation Construction by Maturity Tests

ASTM Specifications

ASTM C 227 Potential Alkali Reactivity of Cement-Aggregate Combinations (Mortar-Bar Method)

ASTM C 441 Effectiveness of Mineral Admixtures or Ground Blast-Furnace Slag in Preventing Excessive Expansion of Concrete Due to the Alkali-Silica Reaction

ASTM C 457 Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete

ASTM C 469 Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression

ASTM C 512 Creep of Concrete in Compression

ASTM C 672 Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals

ASTM C 779 Abrasion Resistance of Horizontal Concrete Surfaces

ASTM C 918 Early-Age Compressive Strength and Projecting Later Age Strength

ASTM C 944 Abrasion Resistance of Concrete or Mortar Surfaces by the Rotating-Cutter Method

ASTM C 1012 Length Change of Hydraulic-Cement Mortar Bars Exposed to a Sulfate Solution

ASTM C 1017 Chemical Admixtures for Use in Producing Flowing Concrete

ASTM C 1074 Estimating Concrete Strength by the Maturity Method

ASTM C 1260 Potential Alkali Reactivity of Aggregates (Mortar-Bar Method)

CHAPTER 6. CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

Based on the work described in this report, the following general conclusions are made:

- A significant amount of information is available as a result of the joint State-FHWA HPC bridge projects. This information includes construction practices, specifications, and research.
- A large number of provisions of the AASHTO Standard Specifications need to be revised for use with HPC.
- Based on the available information, proposed revisions to the AASHTO Standard Specifications can be developed.
- Additional research is still needed to address all provisions of the specifications that are impacted by the use of HPC.
- The FHWA definition of HPC needs to be revised to reflect new characteristics and to update the grades.

RECOMMENDATIONS

Based on the work described in this report, the following specific recommendations are made:

- The compilation of information and specifications from HPC bridges should be made available from FHWA as a CD-ROM and for downloading from the FHWA HPC Web site and the National Concrete Bridge Council Web site.
- The proposed revisions to 15 AASHTO materials specifications plus 1 new specification should be submitted to the AASHTO Highway Subcommittee on Materials for consideration and adoption.
- The proposed revisions to 14 AASHTO test methods plus 1 new test method should be submitted to the AASHTO Highway Subcommittee on Materials for consideration and adoption.
- The proposed revisions to 30 articles of the AASHTO *Standard Specifications for Highway Bridges* should be made available to the highway departments that continue to use these specifications.
- The proposed revisions to 17 articles of the *AASHTO LRFD Bridge Design Specifications* should be submitted to the AASHTO Highway Subcommittee on Bridges and Structures for consideration and adoption.

- The proposed revisions to 16 articles of the *AASHTO LRFD Bridge Construction Specifications* should be submitted to the AASHTO Highway Subcommittee on Bridges and Structures for consideration and adoption.

- The 10 research problem statements should be submitted to the appropriate Transportation Research Board committees and to the AASHTO Standing Committee on Research for their endorsement.

- The 10 research problem statements should be submitted to other organizations that sponsor concrete research, such as the American Concrete Institute, Portland Cement Association, and Precast/Prestressed Concrete Institute.

- The FHWA HPC definition should be referred to as a *classification* and not a *definition*.

- The proposed revisions to the characteristics and grades should be incorporated into the FHWA HPC classification.

- The review of the HPC definition should be submitted to a national technical journal, such as *Concrete International*, for publication.

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