Synthesis and Evaluation of Lightweight Concrete Research Relevant to the AASHTO LRFD Bridge Design Specifications: Identification of Articles for Further Evaluation and Potential Revision

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U.S. Department of Transportation
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FOREWORD

Broad-based advancements in the field of concrete materials have led to significant enhancements in the performance of lightweight concrete. Although the value of using lightweight concrete within the constructed infrastructure is clear, decades-old performance perceptions continue to raise barriers that hinder wider use of the concrete. Additionally, the lack of modern updates to structural design provisions for lightweight concrete has perpetuated additional barriers to the use of lightweight concrete. In 2007, the Federal Highway Administration (FHWA) embarked on a research program aimed at investigating the structural performance of modern lightweight concretes. This effort both engaged the academic, public sector, and private sector communities to compile the body of knowledge on lightweight concrete while also conducting nearly 100 full-scale structural tests on multiple lightweight concretes.

The American Association of State Highway and Transportation Officials (AASHTO) Subcommittee on Bridges and Structures (SCOBS) Technical Committee 10 (T-10) has expressed interest in updating the AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications to more accurately and consistently reflect the performance of lightweight concrete. FHWA researchers were engaged to compile the overall body of knowledge on this topic then to report back to T-10 with an overall synopsis and proposals for addressing perceived shortcomings in the current design specifications. This report represents the document developed for and delivered to T-10 in March 2012 as part of their ongoing efforts to address the lightweight concrete provisions in the bridge design specifications.

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<tr><td colspan="4">16. Abstract
Much of the fundamental basis for the current lightweight concrete provisions in the AASHTO LRFD Bridge Design Specifications is based on research of lightweight concrete (LWC) from the 1960s. The LWC that was part of this research used traditional mixes of coarse aggregate, fine aggregate, portland cement, and water. Broad-based advancement in concrete technology over the past 50 years has given rise to significant advancements in concrete mechanical and durability performance. The purpose of this document is to give members of AASHTO Subcommittee on Bridges and Structures T-10 the opportunity to begin considering whether revisions to provisions related to LWC within Chapter 5 of the AASHTO LRFD Specifications are warranted. Five specific topics are described in this document where recent LWC research may be sufficient to warrant consideration of a revision to the AASHTO LRFD Specifications. For each topic, the current articles in AASHTO LRFD are described, some of the recent research efforts are mentioned, and then potential revisions are outlined. This document served as supplemental information in support of the discussion that occurred at the March 2012 T-10 meeting in Chicago, Illinois.</td></tr>
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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)

TABLE OF CONTENTS

I. INTRODUCTION

Much of the fundamental basis for the current lightweight concrete provisions in the AASHTO LRFD Bridge Design Specifications is based on research of lightweight concrete (LWC) from the 1960s (ACI Committee 213 1967, Hanson 1961, Ivey and Buth 1967, Pauw 1960). The LWC that was part of this research used traditional mixes of coarse aggregate, fine aggregate, portland cement, and water. Broad-based advancement in concrete technology over the past 50 years has given rise to significant advancements in concrete mechanical and durability performance. Research during the past 30 years including the recent NCHRP studies on different aspects of high-strength concrete has resulted in revisions to the AASHTO LRFD Specifications to capitalize on the benefits of high-strength normal weight concrete (NWC). However, as described by Russell (2007), many of the design equations in the AASHTO LRFD Specifications are based on data that do not include tests of LWC specimens, particularly with regard to structural members with compressive strengths in excess of 6 ksi.

I.A. PURPOSE OF THIS DOCUMENT

The purpose of this document is to give members of AASHTO SCOBS T-10 the opportunity to begin considering whether revisions to provisions related to LWC within Chapter 5 of the AASHTO LRFD Specifications are warranted. The authors would like to solicit feedback as to the importance of focusing on each of a variety of potential revisions. Five specific topics are described in this document where recent LWC research may be sufficient to warrant consideration of a revision to the AASHTO LRFD Specifications. For each topic, the current articles in AASHTO LRFD are described, some of the recent research efforts are mentioned, and then potential revisions are outlined. This document can serve as supplemental information in support of the discussion scheduled to occur at the 2012 PCI Committee Days T-10 meeting in Chicago. Feedback from T-10 will then be used to guide the direction of the synthesis of past work and the development of specific revision proposals.

The document is divided into three sections. The first section is an introduction, which also includes a summary of the mechanical properties of LWC and a description of the gap of equilibrium densities that currently exists in AASHTO LRFD. The second section describes topics where the structural performance of LWC has been explicitly included in AASHTO LRFD, and also covers recent LWC research whose results may merit consideration of a revision to AASHTO LRFD. The third section summarizes topics that, although relevant to LWC, fall outside the bounds of recent research and thus will not merit significant discussion. The units in all design expressions are assumed to be kips and inches unless stated otherwise. Additional information on the types of concrete mixes that will be used to evaluate the AASHTO LRFD design expressions is in Appendix A. Further supplemental information relevant to the following discussion can be found in Appendix B through Appendix G.

I.B. MECHANICAL PROPERTIES OF LIGHTWEIGHT CONCRETE

The aggregate in LWC can either be manufactured or natural, with a cellular pore system providing for a lower density particle. The density of lightweight aggregate is approximately half of that of normal weight rock. The reduced dead weight of the LWC has many benefits in building and bridge construction such as smaller, lighter members, longer spans, and reduced substructures and foundations requirements (ACI Committee 213 2003).

As compared to NWC, LWC tends to exhibit two specific mechanical property reductions. The modulus of elasticity and the tensile strength of LWC tend to be reduced as compared to a similar compressive strength NWC. These differences are generally attributed to the characteristics of the lightweight aggregate. The reduced modulus of elasticity results in larger deflections, larger prestress losses, and longer transfer lengths. The tensile strength of the lightweight aggregate is typically less than that of normal weight aggregate. The performance of concrete structures is affected by the tensile strength of concrete in several significant ways. The reduced tensile strength of LWC can affect the shear strength, cracking strength at the release of prestress, and bond strength of prestressed and non-prestressed reinforcement (ACI Committee 213 2003).

I.C. THE EQUILIBRIUM DENSITY GAP IN AASHTO LRFD

The definition for LWC in AASHTO LRFD covers concrete having lightweight aggregate and an air-dry unit weight less than or equal to 0.120 kcf. Normal weight concrete is defined as having a unit weight from 0.135 to 0.155 kcf. Concretes in the gap of densities between 0.120 and 0.135 kcf are commonly referred to as "specified density concrete" and are not directly addressed by the AASHTO LRFD Specifications. Specified density concrete (SDC) typically contains a mixture of normal weight and lightweight coarse aggregate.

Modifications to AASHTO LRFD are needed to remove the SDC-related ambiguity, to give the designer the freedom of specifying a slightly lower density than NWC, and to allow for appropriate design with SDC. The inclusion of SDC into AASHTO LRFD could take many forms, but would likely require modifications to both terminology and design expressions.

I.D. FACTOR FOR LWC TENSILE STRENGTH

The tendency for LWC to have a reduced tensile strength is not treated consistently in the AASHTO LRFD Specifications. There are many articles where the $\sqrt{f_c'}$ term is used to represent concrete tensile strength. The provisions for shear and tension development length of mild reinforcement currently include a modification for LWC. However, the tensile stress limits in prestressed concrete do not include a modification for LWC. A potential option to provide a more uniform treatment of LWC tensile strength would be to add the definition of a modification factor for LWC, such as λ, to Section 5.4 which could then be referenced in other articles. Then the factor could be added to design expressions where the $\sqrt{f_c'}$ term is used to represent concrete tensile strength.

II. ARTICLES WHERE REVISION MAY BE WARRANTED

Research has indicated that the structural performance of LWC may not be appropriately reflected by some of the current provisions in the AASHTO LRFD Specifications. Areas of particular interest include shear behavior, short-term mechanical properties such as elastic modulus and tensile strength, and the bond of mild reinforcement and prestressing strand. Research has also shown that the provisions in AASHTO LRFD regarding the performance of NWC and LWC in the areas of predicting the development length of mild reinforcement and the transfer and development length of prestressing strands may warrant revision.

II.A. SHEAR CAPACITY

This section describes how the current AASHTO LRFD provisions account for the use of LWC, lists several recent studies involving the shear strength of LWC, and then outlines two potential options for revisions to the AASHTO LRFD Specifications. The revisions have the goal of including SDC in AASHTO LRFD and using a ϕ factor for LWC in shear based on tests of contemporary mix designs. Additional background information and a description of the data analysis methods that will be used in the synthesis are included in Appendix B.

DISCUSSION OF CURRENT AASHTO LRFD PROVISIONS

The shear provisions of AASHTO LRFD account for LWC by using a modification factor in specific articles where the $\sqrt{f_c'}$ term appears. Article 5.8.2.2 states that, when the splitting tensile strength (f_{ct}) is specified, the $\sqrt{f_c'}$ in Articles 5.8.2 and 5.8.3 should be replaced by $4.7f_{ct}$ as long as it is not greater than $\sqrt{f_c'}$. This is equivalent to the modification factor for LWC (λ) in the ACI 318-08 Building Code (ACI Committee 318 2008), where λ is equal to $f_{ct}/(6.7\sqrt{f_c'})$, but not greater than unity. Where f_{ct} is not specified by the designer, AASHTO LRFD requires that $\sqrt{f_c'}$ is reduced by using a multiplier of 0.75 for all-lightweight concrete and by 0.85 for sand-lightweight concrete.

The modification factor for LWC affects the requirements for the minimum transverse reinforcement (Article 5.8.2.5) as well as the component of the shear resistance that relies on the tensile strength of concrete (V_c term) when using the Simplified Procedure for Prestressed and Non-prestressed Sections (Article 5.8.3.3 or Article 5.8.3.4.3). The Simplified Procedure involves taking the lesser of the nominal shear resistance provided by concrete when inclined cracking results from combined shear and moment (V_{ci} term) or the nominal shear resistance provided by concrete when inclined cracking results from excessive principal tensions in the web (V_{cw} term) given in Article 5.8.3.4.3. The modification factor for LWC also affects the inclination angle of diagonal compression (θ) used to calculate the shear resistance using the Simplified Procedure.

The design of concrete members with lightweight aggregate is also affected by the resistance factor (ϕ). The ϕ factor is used to reduce the calculated shear resistance to account for variations in material properties, uncertainty in the calculation method, and differences in the performance of shear tests on normal weight and lightweight members. AASHTO LRFD currently prescribes a ϕ of 0.90 and 0.80 (Article 5.5.4.2) for the calculated shear resistance of normal weight and lightweight members, respectively.

TESTS ON HIGH-PERFORMANCE LWC IN THE LITERATURE

Recent tests to evaluate the shear capacity of LWC have been undertaken by many researchers. Shear tests on high-strength LWC girders were performed by Virginia Tech (VT) as part of a National Cooperative Highway Research Program (NCHRP) project. Eighteen tests on AASHTO Type II girders made with high-strength LWC were conducted at Georgia Tech (Meyer et al. 2002, Meyer and Kahn 2004). Twelve shear tests on rectangular reinforced LWC beams and four tests on AASHTO Type I prestressed LWC girders were performed at Purdue (Ramirez et al. 2000 and 2004). FHWA conducted thirty shear tests on AASHTO Type II and AASHTO/PCI BT-54 prestressed girders made with high-strength SDC (Greene and Graybeal 2011). Other recent studies that include shear tests on high-strength lightweight reinforced concrete were completed by Ahmad (Salandra et al. 1989, and Ahmad et al. 1994), Hamadi and Regan (1980), and Walraven and Al-Zubi (1995).

PHILOSOPHY OF POTENTIAL REVISIONS TO AASHTO LRFD

Two revisions may be warranted. The first revision would aim to eliminate the gap in equilibrium densities within AASHTO LRFD. The second revision pertains to the ϕ factor for LWC in shear. The two options shown below provide potential avenues through which these concepts could be addressed. Option #1 would include a change in the definition of LWC and a potential revision of the ϕ factor for LWC in shear. Option #2 would add SDC to AASHTO LRFD by including SDC with LWC and would utilize a LWC modification factor that is dependent upon unit weight. Option #2 may also include a revision of the ϕ factor for LWC in shear if merited.

<u>Proposed Change Philosophy: Option #1</u>

Article 5.2: Definition of "Lightweight Concrete" – Revise to included unit weights up to 0.135 kcf. This would include SDC as sand-lightweight concrete.

Article 5.5.4.2: Resistance Factors – Evaluate 0.80 as the ϕ factor for LWC in shear and potentially revise.

<u>Proposed Change Philosophy: Option #2</u>

Article 5.2: Definition of "Lightweight Concrete" – Revise to included unit weights up to 0.135 kcf and remove the definitions of "all-lightweight" and "sand-lightweight" concrete.

Article 5.8.2.2: Modification for Lightweight Concrete – When f_{ct} is not specified, revise modification factor to make it dependent upon unit weight. For example, the modification factor could range from $0.75\sqrt{f_c'}$ at a density of 0.110 kcf up to $1.0\sqrt{f_c'}$ at density of 0.135 kcf.

Article 5.5.4.2: Resistance Factors – Evaluate 0.80 as the ϕ factor for LWC in shear and revise the resistance factor for lightweight concrete in shear to be a function of unit weight. For example, the modification factor could range from 0.80 at a density of 0.110 kcf up to 0.90 at density of 0.135 kcf.

II.B. MECHANICAL PROPERTIES OF LWC

This section describes the design expressions in AASHTO LRFD that pertain to modulus of elasticity, modulus of rupture, and direct tensile strength. Other mechanical properties such as the coefficient of thermal expansion and Poisson's ratio are described in this section, but there is no intent to recommend revisions pertaining to these properties due to the large scatter in the data available in the literature. Time-dependent material properties such creep and shrinkage are described in a later section of this document. Additional background information and a description of the data analysis methods that will be used in the synthesis are included in Appendix C.

This section then lists several recent studies on material properties of LWC and outlines three proposed options for potential revision to the AASHTO LRFD Specifications. The revisions have the goal of including SDC in AASHTO LRFD and improving the design expression for elastic modulus.

DISCUSSION OF CURRENT AASHTO LRFD PROVISIONS

The expression for modulus of elasticity in AASHTO LRFD (Article 5.2.2.4) is given by Eq. (1). The expression is stated as being applicable to concrete with a unit weight between 0.090 and 0.155 kcf and compressive strengths up to 15.0 ksi. The correction factor, K_1, for the aggregate source must be approved by the local jurisdiction.

$$E_c = 33{,}000 K_1 w_c^{1.5} \sqrt{f_c'} \qquad (1)$$

where the units for w_c are kcf and E_c and f_c' are ksi

The expressions for modulus of rupture (f_r) in AASHTO LRFD (Article 5.4.2.6) are dependent upon the type of concrete and how the calculated f_r will be used. In each expression, f_r is given as a factor multiplied by $\sqrt{f_c'}$.

For NWC, a factor of 0.24 is used for the cracking control (Article 5.7.3.4) and for calculating deflection through the effective moment of inertia (Art. 5.7.3.6.2), a factor of 0.36 is used for the calculation of the minimum area of flexural reinforcement in prestressed and non-prestressed members (Article 5.7.3.3.2), and a factor of 0.20 is used to calculate the cracking moment for V_{ci} ("Simplified Procedure" in Article 5.8.3.4.3). Factors of 0.20 and 0.17 are used for sand-lightweight and all-lightweight, respectively.

Direct tensile stress is covered in Article 5.4.2.7 which states that the direct tensile stress may be determined using ASTM C900 or using the splitting tensile strength method of AASHTO T198 (ASTM C496). The use of the direct tensile strength in AASHTO LRFD is less clear than for the modulus of rupture. The commentary for Article 5.4.2.7 gives an expression of $0.23\sqrt{f_c'}$ as an estimate of the direct tensile strength. In the commentary of Article 5.4.2.6, it states that the calculated modulus of rupture values may be unconservative when tensile cracking is caused by effects other than flexure. The direct tensile strength is then allowed for use with restrained shrinkage, anchor zone splitting, and other non-flexural related tensile stresses.

Limited data is available in the literature on tests that evaluate Poisson's ratio and the coefficient of thermal expansion for LWC. Poison's Ratio is given as 0.2 (Article 5.4.2.5) for concrete regardless of the mix design. Article 5.4.2.2 in AASHTO LRFD specifies that the coefficient of thermal expansion (CTE) may be taken as 5.0×10^{-6} in./in./°F in the absence of more precise data. The commentary states that only limited data is available for LWC and the range of CTE is from 4.0 to 6.0×10^{-6} in./in./°F.

RELEVANT TESTS ON LWC IN THE LITERATURE

Considerable research has been performed on LWC at Georgia Tech (GT), Purdue, the University of Texas at Austin (UTA), and Virginia Tech (VT). The research performed at these universities has included numerous evaluations of the short-term mechanical properties and time-dependent properties of LWC. Results from the tests performed at GT, Purdue, UTA, and VT provide a considerable amount of the data on high-performance LWC. Extensive tests on traditional LWC mixes will also be included in the database for comparison (Hanson 1961, Ivey and Buth 1966, Pfeifer 1967, Richart and Jensen 1930, Shideler 1957). Any differences in performance should be evident after analysis of the complete database.

PHILOSOPHY OF POTENTIAL REVISIONS TO AASHTO LRFD

The design expression for E_c in AASHTO LRFD is currently dependent on the design unit weight. As such, modification may not be necessary it if is determined to be appropriate for the applicable range of unit weights. However, the current expression does tend to overestimate E_c for both high-strength NWC and high-strength LWC, so an alternate expression may be warranted. Additionally, the provisions relating to f_r may require modification to appropriately address the applicable range of unit weights.

Three potential options are proposed to revise AASHTO LRFD to address these issues. Option #1 is a change to the definition of LWC. Option #2 would add SDC to AASHTO LRFD by including SDC with LWC and would utilize a LWC modification factor for f_r. Option #3 is similar to Option #2 with the addition of a revised expression for E_c.

<u>Proposed Change Philosophy: Option #1</u>

Article 5.2: Definition of "Lightweight Concrete" – Revise to included unit weights up to 0.135 kcf. This would include SDC as sand-lightweight concrete.

<u>Proposed Change Philosophy: Option #2</u>

Article 5.2: Definition of "Lightweight Concrete" – Revise to included unit weights up to 0.135 kcf and remove the definitions of "all-lightweight" and "sand-lightweight" concrete.

Article 5.4.2.6: Modulus of Rupture – Revise the three expressions for NWC to include a modification factor for lightweight concrete similar to the one used for shear. Remove the two expressions for lightweight concrete.

Proposed Change Philosophy: Option #3

Article 5.2: Definition of “Lightweight Concrete” – Revise to included unit weights up to 0.135 kcf and remove the definitions of “all-lightweight” and “sand-lightweight” concrete.

Article 5.4.2.4: Modulus of Elasticity – Revise expressions to be similar to the one from NCHRP Project 12-64.

Article 5.4.2.6: Modulus of Rupture – Revise the three expressions for NWC to include a modification factor for lightweight concrete similar to the one used for shear. Remove the two expressions for lightweight concrete.

II.C. DEVELOPMENT OF MILD STEEL REINFORCEMENT

This section describes the design expressions in AASHTO LRFD that pertain to the development of mild steel reinforcement in LWC. This section then lists a few of the limited number of studies on development length of mild steel reinforcement in LWC and outlines three potential options for revisions to the AASHTO LRFD Specifications. The proposed revisions have the goal of including SDC in AASHTO LRFD. Additional background information and a description of the data analysis methods that will be used in the synthesis are included in Appendix D.

Although modifying the basic tension development length equation for NWC members is beyond the scope of this document, some of the concerns with the current expressions are discussed in Appendix D. The third proposed revision has the additional goal of improving the design expression for the basic tension development length of mild reinforcement in both NWC and LWC.

DISCUSSION OF CURRENT AASHTO LRFD PROVISIONS

The tension development length of mild steel reinforcing bars is in Article 5.11.2.1. The development length, ℓ_d, is the product of the basic development length, ℓ_{db}, multipliers that increase the development length (Article 5.11.2.1.2), and multipliers that decrease the development length (Article 5.11.2.1.3). The equation for basic development length in AASHTO LRFD for No. 11 bars and smaller is given by Eq. (2).

$$\text{For No. 11 bars and smaller: } \ell_{db} = \frac{1.25A_b f_y}{\sqrt{f_c'}} \geq 0.4 d_b f_y \qquad (2)$$

The modification factors that increase the development length include a factor for use of top bars, for use of LWC, and for use of epoxy-coated bars. When LWC is used, the factor is 1.3 for all-lightweight concrete and 1.2 for sand-lightweight concrete if the splitting tensile strength is not specified, and $0.22\sqrt{f_c'}/f_{ct}$ (but not less than unity) when the splitting tensile strength is specified.

TESTS ON LWC IN THE LITERATURE

The body of experimental tests completed to assess the bond strength of LWC is limited. Many of the existing test results come from simple pull-out tests. The large amount of compression induced in a pull-out test introduces a stress condition in the specimen that does not represent the stress condition of a bar under tension in a typical structure. Test specimens geometries that give more realistic stress conditions are the beam end-block and the splice beam. Clarke and Birjandi (1993) evaluated the bond strength of four different lightweight aggregates in tests on beam end-block specimens. Petersen (1948) evaluated two different lightweight aggregates in a beam test. As part of the testing on high-performance specified-density concrete at FHWA, forty splice beams were tested to evaluate concrete mixes with three different lightweight aggregates and four rebar sizes (Greene and Graybeal 2010b).

PHILOSOPHY OF POTENTIAL REVISIONS TO AASHTO LRFD

Two specific revisions may be warranted. The first revision would aim to eliminate the gap in equilibrium densities within AASHTO LRFD. The second is a revised expression for the basic development length of both NWC and LWC to reflect the results of research from the last thirty years that has resulted in an improved understanding of bond strength since the time the current expression in AASHTO LRFD was originally proposed.

Three options are proposed to revise AASHTO LRFD to address these issues. Option #1 is essentially a change in the definition of LWC. Option #2 outlines a proposal that would add SDC to AASHTO LRFD by including SDC with LWC and would utilize the modification factor for LWC already in AASHTO LRFD. Option #3 outlines a change to the basic development length expression in AASHTO LRFD.

<u>Proposed Change Philosophy: Option #1</u>

Article 5.2: Definition of "Lightweight Concrete" – Revise to included unit weights up to 0.135 kcf. This would include SDC as sand-lightweight concrete.

<u>Proposed Change Philosophy: Option #2</u>

Article 5.2: Definition of "Lightweight Concrete" – Revise to included unit weights up to 0.135 kcf and remove the definitions of "all-lightweight" and "sand-lightweight" concrete.

Article 5.11.2.1.1: Modification Factors which increase ℓ_d – When f_{ct} is not specified, revise modification factor to make it dependent upon unit weight.

<u>Proposed Change Philosophy: Option #3</u>

Article 5.2: Definition of "Lightweight Concrete" – Revise to included unit weights up to 0.135 kcf and remove the definitions of "all-lightweight" and "sand-lightweight" concrete.

Article 5.11.2.1.1: Tension Development Length – Revise expressions to be similar to those from NCHRP Project 12-60 or those from ACI Committee 408.

Article 5.11.2.1.2: Modification Factors which increase ℓ_d – When f_{ct} is not specified, revise modification factor to make it dependent upon unit weight.

II.D. TRANSFER AND DEVELOPMENT LENGTH OF PRESTRESSING STRAND

Several researchers have found that there is large variation in the comparison of the transfer and development length predicted by the AASHTO LRFD expressions and measurements of transfer and development length. An important reason for this variability is that the AASHTO LRFD expressions for transfer length and development length of pretensioned strands do not account for the influence of the strength and stiffness of the surrounding concrete. Additional background information and a description of the data analysis methods that will be used in the synthesis are included in Appendix E.

This section describes the design expressions in AASHTO LRFD that pertain to the transfer length and development length of pretensioned strands and lists several recent studies in this area. There are currently no provisions which specify that the transfer or development length of prestressing strand be modified when embedded in LWC. Then three potential options for revisions to the AASHTO LRFD Specifications are outlined. The first two options use a LWC modification factor to improve the prediction transfer and development length for LWC given by the current expressions in AASHTO LRFD. The third option outlines the use of different design expressions for transfer and development length.

DISCUSSION OF CURRENT AASHTO LRFD PROVISIONS

Equation (3) is provided by AASHTO LRFD (Article 5.11.4.1) for the transfer length of prestressing strands. This equation relates a calculated transfer length that is 20% longer than the expression from the Standard Specifications for Highway Bridges, 16th Edition (AASHTO 1996). AASHTO LRFD requires prestressing strand to be bonded beyond the critical section a distance not less than the development length, which is given by Eq. (4). The expression depends upon the effective stress in the prestressing strands after losses, f_{pe}, and the stress in the prestressing steel at the nominal resistance of the member, f_{ps}, in addition to the nominal strand diameter. The factor κ was added to the AASHTO LRFD Specifications as a result of an FHWA 1988 Memorandum (Lane 1998), to account for the reduced bond characteristics of some strand and has a value of 1.6 for members with a depth greater than 24 inches (AASHTO 2010).

$$\ell_t = 60 d_b \tag{3}$$

$$\ell_d = \kappa\left(f_{ps} - \frac{2}{3} f_{pe}\right) d_b \tag{4}$$

TESTS ON HIGH-PERFORMANCE LWC IN THE LITERATURE

Recent studies on the transfer and development length of prestressing strand in full-scale AASHTO girders made with LWC have been performed by researchers at Georgia Tech (GT), Purdue, the University of Texas at Austin (UTA), and Virginia Tech (VT). Studies on smaller scale LWC beams were conducted at Kansas State University (KSU) and the University of Maryland (UM). Evaluation of the transfer and development length of SDC was performed by FHWA at the Turner Fairbank Highway Research Center (TFHRC). Results from these tests provide a considerable amount of data on the transfer and development length of LWC. It is proposed that the tested development length of strand in LWC members will be compared to the development length observed in NWC members.

PHILOSOPHY OF POTENTIAL REVISIONS TO AASHTO LRFD

AASHTO LRFD Specifications do not currently differentiate between LWC and NWC in the articles that pertain to transfer and development length of prestressing strand. Some research studies, however, have found an increase in the transfer and development length of strand in LWC. Three methods are proposed to revise AASHTO LRFD to address these issues. Option #1 would apply a modification factor to transfer and development length based on unit weight, similar to the modification factor for shear. Option #2 outlines a proposal to apply a modification factor to transfer and development length based on concrete stiffness. Option #3 outlines a proposal to change the expression for transfer and development length of prestressing strands in AASHTO LRFD.

Proposed Change Philosophy: Option #1

Article 5.2: Definition of "Lightweight Concrete" – Revise to included unit weights up to 0.135 kcf and remove the definitions of "all-lightweight" and "sand-lightweight" concrete.

Article 5.11.4.1: Development of Prestressing Strand, General – Add a modification factor that would be dependent upon unit weight which would increase the transfer length when f_{ct} is not specified.

Article 5.11.4.2: Bonded Strand – Add a modification factor to the expression for ℓ_d that would be dependent upon unit weight which would increase the development length when f_{ct} is not specified.

Proposed Change Philosophy: Option #2

Article 5.2: Definition of "Lightweight Concrete" – Revise to included unit weights up to 0.135 kcf and remove the definitions of "all-lightweight" and "sand-lightweight" concrete.

Article 5.11.4.1: Development of Prestressing Strand, General – Add a modification factor that would be dependent upon the LWC elastic modulus.

Article 5.11.4.2: Bonded Strand – Add a modification factor to the expression for ℓ_d that would be dependent upon the LWC elastic modulus.

Proposed Change Philosophy: Option #3

Article 5.2: Definition of "Lightweight Concrete" – Revise to included unit weights up to 0.135 kcf and remove the definitions of "all-lightweight" and "sand-lightweight" concrete.

Article 5.11.4.1: Development of Prestressing Strand, General – Revise the expression for transfer length to be similar to the one from NCHRP Project 12-60.

Article 5.11.4.2: Bonded Strand – Revise the expression for development length to be similar to the one from NCHRP Project 12-60.

II.E. TIME-DEPENDENT MATERIAL PROPERTIES AND THEIR EFFECT ON PRESTRESS LOSSES AND DEFLECTION

Considerable research on LWC during the past 20 years has resulted in an improved understanding of the long-term structural performance of contemporary LWC. Specifically, this research has evaluated the creep and shrinkage behavior of LWC and its impact on the long-term deflection of prestressed structures. AASHTO LRFD does not explicitly account for any potential difference in performance between LWC and NWC on this topic. However, Article 5.9.5.3 on Approximate Estimate of Time-Dependent Losses is stated to pertain only to NWC.

Based on a preliminary analysis of the individual research studies, it seems unlikely that a significant revision to the AASHTO LRFD Specifications to account for the performance of LWC is necessary. However, due to the importance of the topic, and the number of recent research studies in the topic area, an analysis of data obtained from multiple studies is justified. A revision to expand the applicability of Article 5.9.5.3 might be proposed.

Many of the research studies that evaluated short-term LWC mechanical properties also assessed the time-dependent proprieties of LWC such as creep and shrinkage. Some studies have evaluated the effects of creep and shrinkage on prestress losses. The long-term performance of LWC structures has been evaluated by monitoring deflections and concrete strain of structures in the field.

This section reviews some of the recent research that has been performed on LWC to evaluate creep and shrinkage and their effect on prestress losses. The current expressions in AASHTO LRFD are reviewed. However, due to the complexity of the concrete behavior involved in the loss of prestressing and the interaction of the factors involved, there is no intent to recommend specification revisions. A short description of tests on LWC in the literature is included in Appendix F.

DISCUSSION OF CURRENT AASHTO LRFD PROVISIONS

The stress in the prestressing strand is immediately reduced at transfer due to elastic losses in the concrete. Additional time-dependant losses such as creep and shrinkage further reduce the long-term effective prestressing force. Prestress losses reduce the beneficial effect of prestressing at service load which can cause additional cracking and deformation in bridge girders.

The expressions in AASHTO LRFD (Article 5.4.2.3) for the creep coefficient (ψ), given by Eq. (5), and concrete shrinkage strain (ε_{sh}), given by Eq. (6), do not include any modifications for the use of lightweight aggregate. AASHTO LRFD states that concrete creep and shrinkage shall be considered in the calculation of deflection and chamber; however it does not prescribe a specific method for calculating camber or long-term deflection.

$$\psi(t,t_i) = 1.9 k_s k_{hc} k_f k_{td} t_i^{-0.1} \tag{5}$$

$$\varepsilon_{sh} = k_s k_{hs} k_f k_{td} 0.48 \times 10^{-3} \tag{6}$$

PHILOSOPHY OF POTENTIAL REVISIONS TO AASHTO LRFD

Article 5.9.5.3 on Approximate Estimate of Time-Dependent Losses is stated to pertain only to NWC. A potential revision to expand the applicability of Article 5.9.5.3 is outlined by Option 1. Option 2 outlines a proposal to modify the creep coefficient and the strain due to shrinkage to account for LWC.

Proposed Change Philosophy: Option #1

Article 5.9.5.3: Approximate Estimate of Time-Dependent Losses – Revise to included lightweight concrete.

Proposed Change Philosophy: Option #2

Article 5.4.2.3: Shrinkage and Creep – Revise creep coefficient and strain due to shrinkage to account for lightweight concrete.

Article 5.9.5.3: Approximate Estimate of Time-Dependent Losses – Revise to included lightweight concrete.

III. ARTICLES UNLIKELY TO BE EVALUATED

There are several topics pertaining to the structural performance of LWC which will not be evaluated for potential revisions to AASHTO LRFD. Some of these topics are the flexural resistance, and compressive resistance, and shear friction resistance of LWC members. A brief summary of these topics is included in Appendix G. Other topics not discussed are: 1) the resistance factor for compression and bursting resistance in LWC anchorage zones, 2) limiting compressive stresses for strut-and-tie models, 3) LWC columns including biaxial flexure, requirements for spirals and ties, hollow rectangular cross sections, 3) bearing stresses, 4) principle stresses in webs, and the effects of shear and torsion in segmental concrete bridges, 5) development length of mild reinforcement in compression or ending in hooks and under tension, 6) provisions for seismic design, 7) specific members like diaphragms, deep beams, brackets, corbels, and beam ledgers, and 8) shear in footings and box culverts. Only a limited number of studies are available in the literature, and typically no more than one study using LWC could be identified. Additional information can be found in Russell (2007). Because of limited data, no revisions to AASHTO LRFD will be proposed for these topics.

IV. ACKNOWLEDGEMENTS

This document was developed to assist AASHTO SCOBS T-10 as they consider revisions to Chapter 5 of the AASHTO LRFD Bridge Design Specification. It does not constitute a policy statement or a recommendation from FHWA. Additionally, the publication of this article does not necessarily indicate approval or endorsement of the findings, opinions, conclusions, or recommendations either inferred or specifically expressed herein by FHWA or the United States Government. This document was created by PSI on behalf of FHWA as part of contract DTFH61-10-D-00017.

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V. NOTATION

A_b	=	area of an individual bar
d_b	=	nominal diameter of reinforcing bar or prestressing strand
E_c	=	modulus of elasticity of concrete
f_c'	=	concrete compressive strength
f_{ct}	=	concrete splitting tensile strength
f_{pe}	=	effective stress in the prestressing steel after losses
f_{ps}	=	average stress in prestressing steel at the time the nominal resistance of the member is required
f_r	=	modulus of rupture of concrete
f_y	=	yield strength of reinforcing bars
K_1	=	correction factor for source of aggregate
k_{hc}	=	humidity factor for creep
k_{hs}	=	humidity factor for shrinkage
k_f	=	factor of the effect of concrete strength
k_s	=	factor for the effect of the volume-to-surface ratio
k_{td}	=	time development factor
ℓ_d	=	development length
ℓ_{db}	=	basic development length for straight reinforcement
ℓ_t	=	transfer length of prestressing strand
M_{cre}	=	moment causing flexural cracking due to externally applied loads
M_{max}	=	maximum factored moment due to externally applied loads
t_i	=	age of concrete when load is initially applied
V_c, V_p	=	components of V_n provided by concrete and prestressing force
V_{ci}	=	nominal shear resistance provided by concrete when inclined cracking results from combined shear and moment
V_{cw}	=	nominal shear resistance provided by concrete when inclined cracking results from excessive principle tensions in the web
V_n	=	nominal shear resistance
w_c	=	unit weight of concrete
ε_{sh}	=	concrete shrinkage at a given time
θ	=	inclination angle of diagonal compressive stresses
κ	=	multiplier for strand development length
λ	=	modification factor lightweight concrete in ACI-318 Building Code
ϕ	=	resistance factor
$\psi(t, t_i)$	=	creep coefficient; the ratio of the creep strain that exists t days after casting to the elastic strain caused when load is applied t_i days after casting

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APPENDIX A

MIX DESIGNS AND LIGHTWEIGHT AGGREGATE TO BE INCLUDED IN THIS SYNTHESIS EFFORT

There is a wide variety of materials and mix designs that are considered LWC. The type of lightweight aggregate can be natural like pumice and scoria, or manufactured. Many different processes have been used to bloat natural materials. These processes include the rotary kiln process that produces much of the expanded clay, shale, and slate used to produce structural lightweight concrete. Other processes like sintering have been used to expand fly ash, shale, slag, and slate. There are also differences in the raw material used and aggregates from different regions around the globe that have been used in the production of LWC. Another significant variation on LWC comes from the addition of mineral and chemical admixtures to produce HPC. The material properties of HPC using lightweight aggregate may be considerably different than "traditional" mixes using only lightweight aggregate, cement, and water.

The mechanical properties of "traditional" mix designs may differ from those of "contemporary" mix designs with the same compressive strength. In this document, a traditional LWC mix refers to concrete that consists of only portland cement, water, lightweight coarse aggregate, and either lightweight fine aggregate or sand. The term contemporary LWC mix refers to concrete that also includes chemical admixtures or supplementary cementitious materials such as fly ash, silica fume, or natural pozzolans. The development of contemporary mixes has led to LWC with high compressive strength and greater durability.

This compilation effort intends to evaluate a large database of tests using lightweight aggregate from different sources and concrete with different mix designs, including both traditional LWC mixes and contemporary LWC mixes. It is common for a research study to focus on a limited number of different lightweight aggregates, and the mechanical properties of one type of lightweight aggregate (i.e. clay, shale, or slate) can vary from different sources. The mechanical properties of contemporary mixes may be improved over those made from traditional mixes because contemporary mixes can have a denser matrix surrounding the aggregate. Also, trends in data that are used to develop design expressions for traditional mixes may not be compatible with contemporary mixes and lead to large underestimation or overestimation of mechanical properties. A comprehensive effort is needed to evaluate a broad range of lightweight aggregates and mix designs, and compare the performance of LWC to NWC.

Analysis of mechanical properties will be performed on all tests of structural lightweight concrete in the database. Structural lightweight concrete has a compressive strength of at least 2.5 ksi and is made with lightweight aggregate that meets the requirements of ASTM C330. Preference will not be given to tests performed on LWC using lightweight aggregate from a particular region, produced by a particular manufacturing process, or based on the use of natural aggregate. The mechanical properties resulting from traditional mix designs, however, will be compared to HPC mixes incorporating mineral and chemical admixtures.

APPENDIX B

ANALYSIS OF SHEAR TEST DATA AND REFINEMENT OF THE Φ FACTOR FOR LWC IN SHEAR

Analysis of Shear Test Data

Several issues can affect the shear capacity measured from a test. The ratio of the shear span to effective member depth (a/d ratio) has a significant effect on the measured result. The methods for calculating the V_n (Article 5.8.3.3) assume that, after diagonal cracking caused by the applied shear forces, the shear is carried by a "truss" consisting of concrete struts and steel reinforcement ties. When the shear span is small ($a/d < 2$), the compressive stresses in the concrete strut can flow directly into the support, resulting in larger applied shear forces at failure. This behavior is not accounted for in the truss-model concept used by AASHTO, so comparing V_n to a shear capacity tested with a small a/d ratio creates an apparent conservatism in the resistance calculation. Previous research has shown that for reinforced concrete members, tests on beams with an a/d ratio greater than around 3.0 are not affected by the proximity of the support. Another issue that can affect the result of a shear test is yielding of the longitudinal reinforcement due to flexural stresses. When the longitudinal reinforcement yields, the shear cracks become wider and are less capable of transmitting shear stresses between the concrete struts due to aggregate-interlock. The overall effect is a reduction in the tested shear capacity.

Refinement of the ϕ factor for LWC in shear

An effort to refine the estimate of the ϕ factor for LWC in shear was described in a paper by Paczkowski and Nowak (2010). Nowak used statistical data from compression test performed on LWC and NWC cylinders (Nowak and Rakoczy 2010) and data from thirteen pairs of shear tests from the literature (Hamadi and Regan 1980, Ramirez et al. 2000, Walraven and Al-Zubi 1995). These shear tests were selected because similar specimens were fabricated out of LWC and NWC, allowing the shear resistance of a LWC beam to be directly compared the shear resistance of a NWC beam. Paczkowski and Novak compared the tested shear capacities to the shear resistance predicted using the General Procedure of AASHTO (Article 5.8.3.4.2) and included the reduction factors of $0.75\sqrt{f_c'}$ for all-lightweight concrete and $0.85\sqrt{f_c'}$ for sand-lightweight concrete. The authors performed a reliability analysis and concluded that a ϕ factor of 0.80 was adequate for shear in lightweight reinforced concrete members.

It is proposed that, as part of this compilation and specification revision effort, a reliability study will be performed on both reinforced and prestressed LWC members. The database will include tests on members with LWC and SDC. A limited number of tests on members with NWC will be included. The NWC members selected for inclusion will have a similar range of compressive strengths and member depths as the members with lightweight aggregate.

The difference in ϕ factor between NWC and LWC imposes a significant reduction factor for calculating the design capacity of LWC sections under shear stresses. Several research projects have investigated the use of LWC under shear and have found that the nominal capacity calculated according to AASHTO is conservative without any modification (Dymond et al. 2010, Meyer and Kahn 2004, Ramirez et al. 2000). The ϕ factor for LWC needs to be evaluated using as large of a database of shear tests as possible, especially a database that includes tests on high-strength LWC.

Notation

a	=	shear span
d	=	effective shear depth
f_c'	=	concrete compressive strength
V_n	=	nominal shear resistance
ϕ	=	resistance factor

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APPENDIX C

IMPORTANCE OF THE PREDICTED MODULUS OF ELASTICITY AND MODULUS OF RUPTURE AND ANALYSIS OF TEST DATA

Importance of the Predicted Modulus of Elasticity and Modulus of Rupture

The accuracy of the predicted modulus of elasticity (E_c) is very important for many types of concrete structures. Modulus of elasticity is used directly to calculate deflections (Articles 5.7.3.6.2 and 4.5.2.2) and in the estimation of prestress losses. The calculations for prestress losses use E_c in the expression for elastic losses (Article 5.9.2.3), and if the refined estimate of losses is used (Art. 5.9.5.4), E_c also affects shrinkage, creep, and possibly relaxation. For steel structures, E_c is used to calculate fiber stresses in composite sections (Article 6.10.1.1.1b).

The accuracy of the expression for E_c, through the calculation of prestress losses (and as a result the effective prestress (f_{pe})), affects many significant aspects in the design of prestressed members. Several important aspects include the calculation of concrete fiber stresses, the nominal shear resistance (through β and V_p, Article 5.8.3.3), the average stress in unbonded strands (through f_{pe}, Article 5.7.3.1.2) used to calculate the nominal moment capacity, and the development length of prestressing strand (Article 5.11.4.2).

The expressions of f_r for use with LWC are independent of how the calculated cracking moment is used. This creates varying levels of conservatism in the calculations of cracking control, effective moment of inertia, and cracking moment for V_{ci} when used in members made from LWC. However, when used in the calculation of the minimum area of flexural reinforcement, the result could be that approximately half the minimum flexural reinforcement is required for LWC members.

Analysis of Test Data

Many expressions for E_c have been proposed to account for the reduced stiffness of LWC. The expression for E_c given by Eq. (1) was proposed by Pauw (1960) and became part of design practice with its introduction into the ACI 318-63 code (ACI Committee 318 1963). Since then it has been evaluated in many studies and found to consistently overestimate the prediction of E_c for NWC and LWC members with a compressive strength above 6 ksi (ACI Committee 363 2010, Slate et al. 1986, Stiffey 2005, Thatcher et al 2002). Other design expressions for E_c have been suggested by ACI Committee 213 (2003), Meyer et al. (2002), Rizkalla et al. (2007), Slate et al. (1986), and Stiffey (2005). The Slate et al. expression is given by Eq. (7) and the Rizkalla et al. expression from NCHRP Project 12-64 is given by Eq. (8). Additional expressions for E_c that are found in the literature can also be evaluated, however most use $(f_c')^n$ or $(w_c)^n$, where n is not 0.5 or 1.5, respectively, as typically used in expressions in AASHTO LRFD.

$$E_c = \left(40{,}000\sqrt{f_c'} + 1{,}000{,}000\right)\left(\frac{w_c}{145}\right)^{1.5} \tag{7}$$

where units are in inches and pounds

$$E_c = 310000K_1(w_c)^{2.5}(f_c')^{0.33} \tag{8}$$

where the units for w_c are kcf and E_c and f_c' are ksi

Expressions for f_r other than the ones in AASHTO LRFD have also been proposed in literature (ACI Committee 363 2010, Slate et al. 1983). For NWC, these expressions are typically in the form of a factor multiplied by $\sqrt{f_c'}$. Expressions of f_r for LWC are typically multiplied by another reduction factor for the use of LWC. The expressions in AASHTO LRFD and other expressions in the literature will be evaluated as part of this study.

Notation

E_c	=	modulus of elasticity of concrete
f_{pe}	=	effective stress in the prestressing steel after losses
f_r	=	modulus of rupture of concrete
V_c, V_p	=	components of V_n provided by concrete and prestressing force
V_{ci}	=	nominal shear resistance provided by concrete when inclined cracking results from combined shear and moment
w_c	=	unit weight of concrete
β	=	factor relating effect of longitudinal strain on the shear capacity of concrete, as indicated by the ability of diagonally cracked concrete to transmit tension

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APPENDIX D

DEVELOPMENT OF MILD REINFORCEMENT

This appendix describes the importance of the test specimen for measuring bond and then the design philosophy underlying the current design expressions for basic development length of mild steel reinforcement in NWC. The analysis of test data is also described.

Test Specimens for Bond Measurements

The Russell synthesis report recognized the paucity of mild steel bond test data for LWC (Russell 2007). Also, most of the bond tests for LWC referenced in ACI 213-03 (ACI Committee 213 2003) utilized a pullout test which, although easy to fabricate and simple to perform, is known to produce an unrealistic stress field within the specimen (ACI Committee 408 2003). More realistic measures of bond strength can be made in beam-end specimens and splice beam specimens. Current AASHTO and ACI 318 design provisions for the development of mild steel bars in NWC are mostly based on tests of splice beam specimens. It is not appropriate to compare design expressions for tension development length to the results of pull-out tests. At best, these tests can be used to compare the performance of LWC to NWC between specimens in the same study. However, because of the stress condition in the specimen, these types of comparisons may not correlate to development of a bar in an actual member.

Concerns with the Design Expressions for Development Length in the AASHTO LRFD Specifications

Although specifying the basic tension development length equation for NWC members is beyond the scope of this paper, some of the deficiencies of the current expressions are be discussed. The main deficiencies are that the design expressions for development length in AASHTO 1) do not account for the presence of transverse reinforcement, 2) relate increasingly unconservative predictions of bond strength as the concrete compressive strength increases, and 3) provide a large variability in the ratio of tested to predicted bond strength.

The transverse reinforcement confines the bar being developed and limits the progression of splitting cracks (ACI Committee 408 2003). The increase in total bond strength due to confining reinforcement has been recognized since the mid-1970s (Orangun et al. 1975 and 1977), and was introduced into ACI 318-83 (ACI Committee 318 1983). The confining effect of transverse reinforcement also provides considerable ductility into a spliced connection. The use of a design expression that incorporates the effect of transverse reinforcement could promote the use of transverse reinforcement and improve member ductility and safety.

Another deficiency is that the design expressions for basic development length use the term $\sqrt{f_c'}$ to account for the contribution of concrete. As discussed in ACI 408, although the term $\sqrt{f_c'}$ may be useful to describe the tensile strength of concrete, it overestimates the additional bond strength of high strength concrete. This is due to the bond strength being more controlled by fracture energy of the aggregate rather than tensile strength. The term $f_c'^{0.25}$ is used in ACI 408 to account for the contribution of concrete and gives improved predictions of bond strength at all ranges of concrete strength. Comparisons in ACI 408 show that regardless of the design expressions used, the expressions that used $\sqrt{f_c'}$ to account for concrete strength give unconservative predictions of bond strength for high strength concrete.

The design expressions for basic development length in AASHTO were originally introduced into the ACI 318-71 code. The design expression given by Eq. (2) can be derived by using the average bond stress value of $9.5\sqrt{f_c'}/d_b$ used in the ACI 318-63 code multiplied by a 1.2 reduction factor or closely spaced bars (ACI Committee 408 2003). A paper discussing the design expression showed that the ratio of the tested bond strength to the predicted average bond stress had large variability and many predicted bond stresses that were unconservative (Jirsa et al. 1979). Because the expressions for development length are empirical, they reflect a fit of the data available at the time the expressions were proposed. In intervening 50 years since the expressions were first used as code provisions, the number of tests on development length has increased significantly and the basic understanding of the most important factors affecting the prediction of bond strength, such as the important roles of splitting cracks and confining reinforcement, has improved. As such, it may be appropriate to reevaluate the design expressions in AASHTO for both NWC and LWC.

Analysis of Test Data

Due to the limited number of tests on beam end-block or splice beam specimens, an evaluation of LWC specimens alone may not be appropriate. Instead, the ratio of tested-to-calculated bond strength for specimens made with LWC will be compared to specimens made from NWC. Then differences between the predictions for LWC and NWC can be made.

ACI Committee 408 has a published database (408 Database) of over 600 tests on development length and splice length specimens (ACI Committee 408 2003). There is a large range of variables included in the 408 Database, such as splice length, bar size, concrete compressive strength, steel yield strength, concrete side and bottom cover, and bar spacing. Comparing the test-to-predicted ratios, or bond strength ratio, for the limited number LWC specimens to the full 408 Database of NWC specimens could skew the results of the analysis. This is because the design expression may over-predict or under-predict the bond strength of variables only tested in the NWC specimens, which could ultimately increase or decrease the average bond strength ratio for NWC and not result in a proper comparison to LWC.

Instead of using the full 408 Database of tests on NWC, it is proposed that a subset of the database be selected with similar ranges of important variables as those from beam end and splice beam tests on LWC from the literature. These important variables include concrete strength, bar diameter, bar yield strength, splice length-to-bar diameter ratio, and effective cover-to-bar diameter ratio. Then the bond strength ratio of LWC can be compared to NWC to better evaluate the effect of the reduced tensile strength of lightweight aggregate on bond strength of mild reinforcement.

The calculated bond strength in AASHTO is given in terms of the length of bar required to develop the yield stress of the bar. The length is then increased by an additional 25% as a factor of safety. In order to evaluate the bond stress directly, the expression for development length is rearranged to give an expression for bar stress at bond failure (f_s) as a function of tested splice length (ℓ_s). This is done by replacing ℓ_{db} with ℓ_s, and the bar yield stress (f_y) is replaced with f_s. After rearranging, the expression for calculated bond strength is given by Eq. (9) and was based on Eq. (2), (Article 5.11.2.1.1) for bars smaller than No. 11. Similar expressions for calculated bond strength can be derived the other design expressions for development length as well.

$$\text{For No.11 bars and smaller: } f_s = \frac{\ell_s \sqrt{f_c'}}{1.25 A_b} \tag{9}$$

The calculated bond strength will be based on several design expressions. The current design expressions in AASHTO LRFD will be evaluated. The expression given by ACI Committee 408 and the expression proposed by NCHRP Project 12-60, which is based on the expression in the ACI 318 code, will also be evaluated. The design expressions will be evaluated 1) using a modification factor of 0.75 for all-lightweight and 0.85 for sand-lightweight, 2) using a modification factor based on a measured splitting tensile strength, and 3) without using any modification for LWC.

Additionally, any proposed modification pertaining to the unit weight gap between sand-lightweight concrete and NWC will need to be investigated in relation to the mild steel development length formulation.

Notation

d_b = nominal diameter of reinforcing bar or prestressing strand
f_c' = concrete compressive strength

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APPENDIX E

TRANSFER AND DEVELOPMENT LENGTH OF PRESTRESSING STRAND

This appendix gives background information on transfer and development length of prestressing strand and describes some concerns with the current design expressions in the AASHTO LRFD Specifications. Then recent tests on LWC in the literature are summarized and the analysis of the test data is described.

Background Information

Transfer Length

The transfer length of prestressing strands is defined as the embedment length required to transfer the effective prestressing force in the strands to the surrounding concrete. An accurate estimation of the transfer length is important for several reasons: calculation of the concrete stresses at transfer and under service loads, design of anchorage zone reinforcement for strut-and-tie models, and design of shear reinforcement which requires knowledge of the level of precompression in the concrete (Barnes et al. 1999).

The two most significant mechanisms that contribute to prestress transfer bond are friction and mechanical resistance (Barnes et al. 2003). Radial compressive stress, commonly attributed to the Hoyer Effect, is required to develop frictional bond stresses. In the short region of the transfer length where the concrete remains elastic, the radial compressive stress depends directly on the elastic modulus of the concrete. In the inelastic region, the radial compressive stress depends on both the elastic modulus and the tensile capacity of concrete.

Both the elastic modulus and tensile capacity of LWC are less than NWC of the same compressive strength. Previous tests on LWC members have had varied results as to the whether AASHTO relates a conservative prediction of the transfer length (Cousins and Nassar 2003, Meyer and Kahn 2004, Meyer et al. 2002).

There are many variables that affect transfer length. Transfer length has been shown by previous research to be proportional to strand diameter (Barnes et al. 2003, Buckner 1994, Hanson and Kaar 1959, Mitchell et al. 1993, Zia and Mostafa 1977). Transfer length is also strongly influenced by the stress level in the strand. Other variables that can affect the transfer length include surface condition of the steel (clean, oiled, rusted), time-dependent effects (concrete creep and shrinkage, strand relaxation), method of release (flame cut, gradual release), and concrete properties (compressive strength, tensile strength, and modulus of elasticity) (Barnes et al. 1999, Base 1957, Russell and Burnes 1993, Zia and Mostafa 1977). In many previous investigations the transfer length was measured at release of the prestress. Previous research has shown differing results as to whether the transfer length changes significantly after release (Barnes et al. 2003, Base 1957).

Development Length

The development length of prestressing strands is defined as the embedment length required to attain the nominal design strength of the strand. Inadequate development length could result in strand slip before the strand reaches its nominal strength, potentially leading to non-attainment of the calculated nominal resistance for flexure or shear. Previous tests on the development length of LWC specimens have had varied results as to whether AASHTO relates a conservative prediction of the development length for prestressed members made from LWC (Cousins and Nassar 2003, Meyer et al. 2002, Peterman et al. 1999, Thatcher et al. 2002).

The development length is typically determined by testing both ends of a prestressed member to flexural failure. This is an indirect method employed in numerous studies (Barnes et al. 1999, Cousins and Nassar 2003, Meyer et al. 2002, Peterman et al. 1999, Russell and Burns 1993, Thatcher et al. 2002). When the first test at one end of the specimen results in a ductile flexural failure, then the tested embedment length is assumed to be greater than the development length. However, if strand slip occurs first, then the tested embedment length is assumed to be less than the development length. In this manner, two tests on one specimen can “bracket” the development length; however, the development length cannot be specifically determined.

Concerns with the Design Expressions for Transfer Length and Development Length of Prestressing Strand in the AASHTO LRFD Specifications

Several researchers have found that there is large variation in the comparison of the transfer and development length predicted by the AASHTO expressions and measurements of transfer and development length. An important reason for this variability is that the AASHTO expressions for transfer length and development length of pretensioned strands do not account for the influence of the strength and stiffness of the surrounding concrete.

Results from several studies have shown that transfer length is affected by concrete strength. Cousins et al. (1993) found that transfer lengths measured in high-strength NWC beams was 25% less than in normal-strength NWC beams. A primary focus of NCHRP Project 12-60 was the transfer and development of pretensioned strand in high-strength NWC (Ramirez and Russell 2008). In the studies by both Zia and Mostafa (1977) and Mitchell et al. (1993), a correlation was found between the concrete compressive strength and the transfer and development of prestressing strands.

Other researchers have found a relationship between modulus of elasticity and transfer length. In a study evaluating the measured transfer length of prestressing strands in NWC reported in the literature, Buckner (1994) concluded that the difference in peak measured concrete strain between similar specimens was due to an “apparent elastic modulus” for concrete and he proposed an expression for transfer length that is a function of E_c. Thatcher et al. (2002) followed Buckner’s work and proposed a similar expression for the transfer length based on their study of transfer and development length of prestressing strands in LWC.

Tests on High-Performance LWC in the Literature

Georgia Tech (GT)

Six AASHTO Type II girders were tested at GT to evaluate the transfer and development length of prestressing strands in LWC (Meyer et al. 2002, Meyer and Kahn 2004). The 28-day compressive strengths ranged from 8.8 to 11.0 ksi. The bond of prestressing strands in NWC has also been evaluated at GT (Kahn et al. 2005).

Virginia Tech (VT)
The transfer and development length of prestressing strands in LWC was evaluated on three AASHTO Type II girders and two AASHTO Type IV girders (Cousins and Nassar 2003). The tested average 28-day compressive strength of the girders was 6.4 ksi. Transfer and development lengths of high-strength LWC girders were also a part of NCHRP 18-15. These results can be included once they are available to the public.

Purdue
Both single-strand rectangular beams and multiple-strand T-beams were tested at Purdue to evaluate the transfer and development length of prestressing strands in SDC (Peterman et al. 1999). The nine single strand beams and three multi-strand T-beams had 28-day compressive strengths that ranged from 8.0 to 12.1 ksi and an average unit weight of 0.138 kcf.

University of Texas at Austin (UTA)
At UTA, the transfer and development length of prestressing strands in LWC was evaluated on seven AASHTO Type I girders (Kolozs 2000, Thatcher et al. 2002). The 28-day compressive strengths averaged nearly 8.0 ksi. A large number of tests on NWC have also been performed at UTA (Barnes et al 1999, Russell and Burnes 1993).

Federal Highway Administration at TFHRC (FHWA)
Prestress transfer length was evaluated by FHWA using twelve AASHTO Type II girders and six AASHTO/PCI BT-54 girders made from SDC (Greene and Graybeal 2008). The twelve AASHTO Type II girders were then tested to failure to evaluate the development length of prestressing strand (Greene and Graybeal 2010a). The three mix designs had 28-day compressive strengths that ranged from 8.6 to 9.7 ksi and unit weights ranging from 0.125 to 0.133 kcf.

University of Maryland (UM)
Twelve square and rectangular beams were tested at UM to evaluate the transfer and development length of prestressing strands in LWC (Zena 1996). The compressive strength of the LWC ranged from 5.3 to 7.2 ksi.

Kansas State University (KSU)
The development length of prestressing strands in eight single-strand rectangular beams and four multiple strand T-beams were evaluated at KSU (Grother and Peterman 2009). The beams had 28-day compressive strengths that ranged from 4.7 to 5.1 ksi. The transfer length was measured on eight separate inverted T-beams with a range of 28-day compressive strengths of 3.3 to 5.2 ksi.

Analysis of Test Data

This section includes a discussion of the method typically used for experimentally determining the transfer length of prestressing strands. Next, the method used to evaluate the results of development length tests is described. Lastly, several design expressions for ℓ_t and ℓ_d of prestressing stress found in the literature is described.

Transfer Length

Two different methods have been commonly used to measure the transfer length of prestressing strand. These methods are the measurement of concrete surface strains (CSS) at the level of the prestressing steel and the strand draw-in method. Most studies in the literature have reported transfer length based on the CSS method and only a few have reported transfer length based on the strand draw-in method. In order to have an adequate amount of data from different studies in the analysis, the CSS method will be used in this analysis.

The CSS method typically uses a demountable mechanical strain gage (DMSG) to read the distance between two target points (DMSG points) in the concrete. The average strain at the surface of the concrete girders is calculated by taking the difference between readings made before and after the release of the prestressing. The concrete surface strain is then calculated by taking the difference between the initial and final measurements, and dividing by the gage length adjusted for the initial measurement. The strain data is also typically “smoothed” by averaging the strain for three consecutive points and applying their average to the middle point. The CSS gives a reasonable estimate of the strain in the prestressing strand due to strain compatibility. For members with fully bonded strand, the strain measurements typically start near zero at the end of the girder and increase approximately linearly until they reach a constant value. A plot of the CSS with respect to the distance from the girder end is the strain profile. An ideal strain profile shows a plateau beginning at the theoretical transfer length. In order for the strain profile to have sufficient resolution, the DMSG points are typically spaced approximately 2 inches apart and the DMSG instrument must have sufficient accuracy to take readings to the nearest 0.0001 inch.

There have been several different methods used in previous studies to analyze the strain profile to determine the transfer length. The most common method is the 95% Average Maximum Strain Method (95% AMS). This method was developed Russell and Burns (1993) and has also been used by researchers in several recent investigations (Cousins and Nassar 2003, Meyer et al. 2002, Thatcher et al. 2002) to evaluate the CSS data for transfer length. The 95% AMS method involves calculating the average of all the strain data points on the strain plateau (the AMS), constructing a line on the strain profile at the strain equal to 95% of the AMS, then determining the transfer length at the intersection of the 95% AMS line and the smoothed strain profile.

The most common variations of the 95% AMS method found in literature determine the transfer length based on the intersection of a line fitted to the increasing part of the strain profile, and the 95% AMS line. The method of fitting the line typically varies from study to study so this variation of the 95% AMS Method tends to add some subjectivity to the calculation of transfer length.

In the present compilation and analysis effort, the transfer length will be based on the intersection of the strain profile with the 95% AMS line. The test data used for comparisons in this study that do not use this method will be reevaluated so that all of the measured transfer lengths used in this study will be determined using a consistent method. Strain data measured at DMSG points spaced greater than 4 inches apart or resulting in measured tensile strains will be evaluated, but will not be used in the final comparison of design expressions.

The transfer length has been shown not to vary significantly with time for NWC (Base 1957). A limited number of LWC tests will be evaluated at different times to demonstrate whether or not this concept is also appropriate for LWC. If transfer length is found to stay relatively constant over time, then the measurements taken immediately following transfer of prestressed will be used in the final comparison of design expressions.

Development length

Different methods have been used in the literature to evaluate the tested embedment length (Cousins and Nassar 2003, Meyer et al. 2002, Peterman et al. 1999, Russell and Burns 1993). A common method is to first subtract the measured transfer length from the tested embedment length, resulting in the tested flexural bond length. Then the flexural bond length can be compared to the design expression for development length minus the calculated transfer length. This method will be used to evaluate the development length results in the present compilation and analysis effort.

Calculation of the expression in AASHTO for development length first requires calculations of f_{ps} and f_{pe}. Different methods for calculating f_{ps} and f_{pe} have been used in the literature (Cousins and Nassar 2003, Meyer et al. 2002, Peterman et al. 1999, Russell and Burns 1993). The calculation of f_{pe} is especially varied because it depends upon the estimation of time-dependent prestress losses. This study will used the prestress loss calculations described in AASHTO LRFD combined with the method for making conservative assumptions in the calculations of f_{ps} and f_{pe} proposed by Peterman et al. (1999). This method uses a conservative calculated value of f_{pe} to determine the likely smallest possible calculated value of development length, and a conservative comparison with the tested development length.

In addition to the expressions for transfer and development length in AASHTO, other expressions have also been proposed. These include the expressions from NCHRP Project 12-60, the expression in ACI 318-11, and the expressions by Mitchell et al., Zia and Mostafa, Buckner, and Thatcher et al. A short description of each expression follows. The database assembled in this study will also be used to evaluate these additional design expressions for transfer and development length of prestressing strands in LWC.

NCHRP Project 12-60 (Report 603)

NCHRP Project 12-60 evaluated the provisions in AASHTO relating to transfer and development length of prestressing strands in high-strength NWC (Ramirez and Russell 2008). The study included development length tests of rectangular beams and I-beams. Based on the results of their tests, the researchers proposed Eq. (10) for transfer length and Eq. (11) for development length.

$$\ell_t = \frac{120 d_b}{\sqrt{f_{ci}'}} \geq 40 d_b \tag{10}$$

$$\ell_d = \left[\frac{120}{\sqrt{f_{ci}'}} + \frac{225}{\sqrt{f_c'}}\right] d_b \geq 100 d_b \tag{11}$$

ACI 318-08

The expression for transfer length in the ACI 318-08 Building Code (ACI Committee 318 2008) is given by Eq. (12). This expression was derived by Mattock (1962) who assumed a uniform bond stress of 0.40 ksi based on the research of Hanson and Kaar (1959). Eq. (12) was developed for Grade 250 prestressing strands (250 ksi ultimate strength). Assuming a 150 ksi effective stress (f_{pe}), then Eq. (12) simplifies to the expression in Eq. (13).

$$\ell_t = \frac{f_{pe} d_b}{3} \tag{12}$$

$$\ell_t = 50 d_b \tag{13}$$

Since the development of Eq. (13), construction practice has changed and Grade 270 strands (270 ksi ultimate strength) are now widely used. If a 180 ksi effective stress is assumed for the Grade 270 strands, then this represents a 20% increase in the effective stress over the stress assumed for the Grade 250 strand. Equation (3), which is the expression for ℓ_t in AASHTO, assumes the same uniform bond stress of 0.40 ksi and incorporates the 20% increase in effective stress over Eq. (13).

The expression for development length in ACI 318 is given by Eq. (14) and is similar to the expression in the AASHTO Specifications (after rearranging and changing units from ksi to psi) without the κ factor.

$$\ell_d = \frac{f_{pe}}{3} d_b + \left(f_{ps} - f_{pe}\right) d_b \tag{14}$$

Mitchell et al.

Equation (15) for transfer length and Eq. (16) for development length was the result of research by Mitchell et al. (1993) on 22 precast, pretensioned NWC beams to investigate the effect of the compressive strength and strand diameter on transfer and development length. The beams had a small cross section with a single strand and the prestress was released gradually. The compressive strength at release varied from 3.0 to 7.3 ksi, and the nominal strand diameters varied from 3/8 to 0.62 inches.

$$\ell_t = 0.33 f_{pt} d_b \sqrt{\frac{3}{f_{ci}'}} \tag{15}$$

$$\ell_d = 0.33 f_{pt} d_b \sqrt{\frac{3}{f_{ci}'}} + \left(f_{ps} - f_{pe}\right) d_b \sqrt{\frac{4.5}{f_c'}} \tag{16}$$

Zia and Mostafa

The empirical expressions for transfer length and development length proposed by Zia and Mostafa (1977) are given by Eq. (17) and Eq. (18), respectively, and are based on data available in the literature. The data was from NWC specimens with nominal strand diameters that ranged from 1/4 to 3/4 inches. The investigators stated that their expression was applicable to concrete strengths ranging from 2.0 to 8.0 ksi.

$$\ell_t = 1.5\frac{f_{pt}d_b}{f_{ci}'} - 4.6 \tag{17}$$

$$\ell_d = \left[1.5\frac{f_{pt}d_b}{f_{ci}'} - 4.6\right] + 1.25\left(f_{ps} - f_{pe}\right)d_b \tag{18}$$

Buckner

Bucker performed a review of the literature related to transfer and development length and then analyzed the data from several studies that were published in the early 1990s (Buckner 1994). As part of his analysis, he developed Eq. (19) for ℓ_t and Eq. (20) for ℓ_d based on the data from normal weight specimens that had only one 1/2 inch nominal diameter fully bonded strand. Buckner's study indicated an influence of the modulus of elasticity of concrete at time of transfer of prestress (E_{ci}) on transfer length.

$$\ell_t = \frac{1250 f_{pt}d_b}{E_{ci}} \tag{19}$$

$$\ell_d = \frac{f_{pt}d_b}{3} + \lambda_{Buckner}\left(f_{ps} - f_{ps}\right)d_b \tag{20}$$

Thatcher et al.

The study by Thatcher et al. (2002) also indicated an influence of the modulus of elasticity on transfer length. They studied the transfer length of AASHTO Type II girders made with LWC. Their expression for transfer length is given by Eq. (21) and is 72% of the value calculated by Eq. (19).

$$\ell_t = \frac{900 f_{pt}d_b}{E_{ci}} \tag{21}$$

Notation

d_b = nominal diameter of reinforcing bar or prestressing strand
E_c = modulus of elasticity of concrete
E_{ci} = modulus of elasticity of concrete at prestress transfer
f_c' = concrete compressive strength
f_{ci}' = concrete compressive strength at prestress transfer
f_{pe} = effective stress in the prestressing steel after losses
f_{ps} = average stress in prestressing steel at the time the nominal resistance of the member is required
f_{pt} = stress in prestressing steel immediately after transfer
ℓ_d = development length
ℓ_t = transfer length of prestressing strand
$\lambda_{Buckner}$ = multiplying factor applied to the flexural bond length; taken as 0.6+40ε_{ps} where

ε_{ps} is the average strain in the prestressing corresponding to f_{ps}

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APPENDIX F

TIME-DEPENDENT MATERIAL PROPERTIES OF LWC AND THEIR EFFECTS ON PRESTRESS LOSSES AND DEFLECTION

This appendix summarizes some recent tests on LWC in the literature.

Tests on High-Performance LWC in the Literature

There have been a limited number of recent studies on prestress losses in high-strength LWC. One study has shown that the reduced elastic modulus of LWC can cause elastic losses that are considerably larger than for NWC (Sylva 2004). Another study showed that the creep and shrinkage of LWC is similar to the normal weight high-performance concrete (Lopez et al. 2004). The same study showed that LWC girders had measured prestress losses that were less than the losses predicted by AASHTO.

Results from other studies (Hanson 1968, Hoff 1992, Leming 1988, Malhotra 1990, Ozyildirim and Gomez 2005) will also be compared to the expressions in AASHTO for creep, shrinkage, and prestress losses. However, because the losses due to creep and shrinkage are not independent of other factors involved like relaxation, there is no intent to recommend revisions to the AASHTO LRFD Specifications.

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APPENDIX G

DISCUSSION OF SELECTED ARTICLES UNLIKELY TO BE EVALUATED

This section briefly discusses a few select topics pertaining to the structural performance of LWC which will not be evaluated for potential revisions to AASHTO. These topics are the flexural resistance, and compressive resistance, and shear friction resistance of LWC members. Only a limited number of studies are available in the literature, and typically no more than one study using LWC could be identified. The following summarizes some of the findings presented in the synthesis of LWC provisions in AASHTO that was completed by Russell (2007). Because of limited data, no revisions to AASHTO will be proposed.

Design for Flexure and Axial Force Effects

The behavior of LWC can affect the two important aspects of the design assumptions for flexure and compression members. These aspects are the maximum usable compressive strain, and the assumed rectangular stress block for concrete compressive stresses. The maximum usable concrete compressive strain for unconfined concrete is specified as 0.003 (Article 5.7.2.1). Research on LWC has shown that this value is conservative for most of the data (Hoff 1992, Kaar et al. 1977, Thatcher et al. 2002). An assumed uniform compressive stress of $0.85\sqrt{f_c'}$ is permitted by AASHTO (Article 5.7.2.2) when calculating the flexural or axial capacity of a member. A limited number of eccentric bracket tests on LWC have shown that the factor of 0.85 is appropriate for LWC. The depth of the cross-section over which the assumed rectangular stress block acts is related to the factor β_1. Data from a limited number of studies has shown that the expression for β_1 in AASHTO LRFD (Article 5.7.2.2) may not be appropriate for LWC (Hoff 1992, Kaar et al. 1977), and a lower value of 0.65 was suggested by Kaar et al. (1977).

Flexural Members

More recent research on the flexural capacity of LWC members has shown that the method of calculating the nominal moment capacity in AASHTO (Article 5.7.3) is conservative for reinforced and prestressed LWC members (Ahmad and Barker 1991, Ahmad and Batts 1991, Meyer et al. 2002, Peterman et al. 1999, Thatcher et al. 2002).

Compression Members

Tests on LWC have been performed in several studies (Basset and Uzumeri 1986, Bresler 1971, Marzouk et al. 2000, Pfeifer 1969). Axial resistance is covered by Article 5.7.4.4. Similar to the 0.85 factor for flexure, the assumed uniform compressive stress for compression members also uses the 0.85 factor for the rectangular stress block.

Interface Shear Friction

Article 5.8.4 specifies the interface shear transfer (shear friction). Shear friction factors specifically related to LWC include cast-in-place concrete slabs on a concrete girder, LWC placed monolithically, or concrete placed non-monolithically against an intentionally roughened surface. A limited number of tests can be found in the literature on test of shear friction with LWC (Hoff 1992, Mattock et al. 1976). Additional data may be available once the results of the recently completed NCHRP Project 18-15 are published.

Notation

f_c' = concrete compressive strength

β_1 = ratio of the depth of the equivalent uniformly stressed compression zone assumed in the strength limit state to the depth of the actual compression zone

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