Synthesis of Research and Provisions Regarding the Use of Lightweight Concrete in Highway Bridges

August 2007

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U.S. Department of Transportation
Federal Highway Administration

FOREWORD

Improvements in the structural behaviors of lightweight concretes, combined with an increasing demand to use these types of concretes in highway bridge structures, has illuminated the need to address perceived shortcomings in AASHTO specifications pertaining to these types of concrete. The Federal Highway Administration is embarking on a significant research effort aimed at better characterizing the structural behaviors of lightweight concretes, particularly those whose equilibrium densities fall between that of traditional lightweight concrete and that of normal weight concrete. This synthesis is intended to provide a reference point for this and future research efforts focusing on the use of lightweight concrete in bridge structures. Through this document, the reader will become aware of the history and shortcomings of the relevant AASHTO specifications as they pertain to lightweight concrete.

This report corresponds to the TechBrief titled, “Current Provisions and Needed Research for Lightweight Concrete in Highway Bridges” (FHWA-HRT-07-051). This report only is being distributed through the National Technical Information Service for informational purposes. The content in this report is being distributed “as is” and may contain editorial or grammatical errors.

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16. Abstract
The AASHTO LRFD Bridge Design Specifications and the AASHTO LRFD Bridge Construction Specifications were reviewed to identify provisions that affect the use of lightweight concrete. A synthesis of research relevant to the provisions was then compiled. Proposed research problem statements were developed to generate data to update the specifications in areas where gaps exist for lightweight concrete.

This report corresponds to the TechBrief titled, "Current Provisions and Needed Research for Lightweight Concrete in Highway Bridges" (FHWA-HRT-07-051).

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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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INTRODUCTION

BACKGROUND

Significant research efforts are currently being performed under the National Cooperative Highway Research Program (NCHRP) and others to update and modify the AASHTO LRFD Bridge Design Specifications for the use of normal weight concrete with compressive strengths up to 15 or 18 ksi. These efforts do not address the research needs of the lightweight concrete classifications currently permitted in the specifications. Also, the current provisions that apply to the use of lightweight concrete need to be verified or may need to be modified for their applicability at higher strength levels.

There is, therefore, a need to review the AASHTO specifications to identify the relevant articles that address or should address the use of lightweight concrete in highway bridges and to synthesize existing research that is relevant to those articles. This information can then be used to develop working agenda items for consideration by AASHTO Committee T-10—Concrete Structures and to define further research needs.

SCOPE

The scope of this report consists of the following:

1. Compilation of the relevant provisions of the AASHTO LRFD Bridge Design Specifications and the AASHTO LRFD Bridge Construction Specifications that address or should address the use of lightweight concrete in highway bridges. The review includes versions of the Design Specifications and the Construction Specifications through the 2006 Interim Revisions.

2. Compilation and synthesis of research relating to lightweight concrete and its use in highway bridges as related to the AASHTO LRFD Specifications.

3. Development of working agenda items for consideration by AASHTO Committee T-10, where adequate knowledge exists to substantiate updating the current specifications.

4. Development of research needs statements detailing the scope and extent of work required to generate data sufficient for updating the current specifications in areas where gaps exist in the collective body of knowledge.

For purposes of this report, lightweight concrete is assumed to have a density between about 100 pcf and that of normal weight concrete. The report does not differentiate between all-lightweight and sand-lightweight concrete.

In general, the articles in the LRFD Design and Construction Specifications can be classified into the following three categories:

1. Not affected by lightweight concrete.

2. Clearly identified as being affected by lightweight concrete through the use of the word "lightweight" or reference to other articles that include lightweight concrete. The word "lightweight" appears 45 times in Section 5: Concrete Structures of the Design Specifications.

3. Lightweight concrete is not specifically mentioned but the potential exists for lightweight concrete to have an effect on the article.

REPORT FORMAT

The body of this report is divided into three parts. Part 1 addresses the AASHTO LRFD Bridge Design Specifications. Part 2 addresses the AASHTO LRFD Bridge Construction Specifications. Part 3 contains conclusions, recommendations, and references. Five research problem statements are included in an Appendix at the end of the report.

Part 1 lists all of the articles in Sections 3 and 5 of the LRFD Bridge Design Specifications that are affected or impacted by the use of lightweight concrete. Each major article begins a new section. The sub articles affected by lightweight concrete are then shown in a rectangular box followed by a discussion of the relevant background and research for the previous article or articles. For some articles, the full text has not been reproduced in the interest of brevity.

Part 2 lists the few articles in Section 8 of the LRFD Bridge Construction Specifications that address lightweight concrete.

The literature search concentrated on publications concerning North American materials and design practices. Whenever possible, the research results are presented as a comparison with the LRFD Specifications. Figures are numbered using the article number followed by a letter. Conclusions and research recommendations are provided in Part 3.

PART 1–AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS

ARTICLE 3.5 PERMANENT LOADS

ARTICLE 3.5 PERMANENT LOADS
3.5.1 Dead Loads: *DC, DW,* and *EV*
In the absence of more precise information, the unit weights, specified in Table 1, may be used for dead loads.

Table 3.5.1-1 Unit Weights.

Material		Unit Weight (kcf)
Concrete	Lightweight	0.110
	Sand-Lightweight	0.120

C3.5.1
The unit weight of concrete is primarily affected by the unit weight of the aggregate, which varies be geographic location and increases with concrete compressive strength.

RELEVANT BACKGROUND AND RESEARCH

With high-strength sand-lightweight concretes, the unit weight can be higher than 0.120 kcf. This is somewhat addressed in the commentary, which indicates that unit weight increases with concrete compressive strength.

ARTICLE 5.1 SCOPE

5.1 SCOPE

The provisions in this section apply to the design of bridge and retaining wall components constructed of normal weight or lightweight concrete and reinforced with steel bars, welded wire reinforcement, and/or prestressing strands, bars, or wires. The provisions are based on concrete strengths varying from 2.4 ksi to 10.0 ksi, except where higher strengths are allowed.

RELEVANT BACKGROUND AND RESEARCH

When the first edition of the LRFD Specifications was being prepared, there was a lack of research data concerning the design of structural members with specified concrete compressive strengths greater than 10.0 ksi. Since then, considerable research has been performed so that strengths greater than 10.0 ksi for normal weight concrete are now allowed by exception for selected articles. It is anticipated that the number of articles with exceptions will increase as recent research sponsored by NCHRP and the Federal Highway Administration (FHWA) is completed. These exceptions, however, will still only be applicable for normal weight concretes. Although compressive strengths greater than 10.0 ksi can be achieved with lightweight concretes, it is unlikely that specified compressive strengths and compressive strengths used for structural design will exceed 10.0 ksi based on current materials. The LRFD Specifications are set up, however, so that exceptions for lightweight concrete can be made, if appropriate.

ARTICLE 5.2 DEFINITIONS

5.2 DEFINITIONS

Lightweight Concrete—Concrete containing lightweight aggregate and having an air-dry unit weight not exceeding 0.120 kcf, as determined by ASTM C 567. Lightweight concrete without natural sand is termed "all-lightweight concrete" and lightweight concrete in which all of the fine aggregate consists of normal weight sand is termed "sand-lightweight concrete."

Normal Weight Concrete—Concrete having a weight between 0.135 and 0.155 kcf.

RELEVANT BACKGROUND AND RESEARCH

Lightweight concrete is defined as having a unit weight not exceeding 0.120 kcf. Normal weight concrete begins at a unit weight of 0.135 kcf. Consequently, there is a gap between 0.120 and 0.135 kcf for which the LRFD Specifications can be deemed not applicable. In the terminology of the American Concrete Institute, concretes in this gap range are called specified density concretes. Because some high-strength lightweight concretes are likely to have a unit weight greater than 0.120 kcf, the LRFD Specifications need to address this gap. Ideally, there should be a gradual transition in properties and design criteria from lightweight concrete to normal weight concrete rather than the steps that now exist based on the types of aggregate.

ARTICLE 5.3 NOTATION

5.3 NOTATION

f_{ct} = average splitting tensile strength of lightweight aggregate concrete (ksi) (5.8.2.2)

RELEVANT RESEARCH

This is a definition so research is not needed.

ARTICLE 5.4 MATERIAL PROPERTIES

5.4 MATERIAL PROPERTIES
5.4.2 Normal and Structural Lightweight Concrete
5.4.2.1 Compressive Strength
Design concrete strengths above 10.0 ksi shall be used only when allowed by specific articles or when physical tests are made to establish the relationships between the concrete strength and other properties.

For lightweight structural concrete, air dry unit weight, strength and any other properties required for the application shall be specified in the contract documents.

C5.4.2.1
Lightweight concrete is generally used only under conditions where weight is critical.

Table C5.4.2.1-1 Concrete Mix Characteristics By Class.

Class of Concrete	Minimum Cement Content	Maximum W/C Ratio	Air Content Range	Coarse Aggregate Per AASHTO M 43 (ASTM D 448)	28-Day Compressive Strength
	pcy	lbs Per lbs	%	Square Size of Openings (in.)	ksi
Lightweight	564	As specified in the contract documents			

RELEVANT BACKGROUND AND RESEARCH

Many publications have reported measured lightweight concrete compressive strengths greater than 10.0 ksi. However, it is unlikely that design concrete strengths above 10 ksi will be specified because of the overdesign requirements of the LRFD Construction Specifications Article 8.4.1.2, which references AASHTO M 241 (AASHTO, 1997). The corresponding table to C5.4.2.1-1 in the LRFD Construction Specifications is Table 8.2.2-1, which does not list a lightweight class of concrete.

5.4.2.2 Coefficient of Thermal Expansion
In the absence of more precise data, the thermal coefficient of expansion may be taken as:

For lightweight concrete: 5.0×10^{-6}/°F

C5.4.2.2
Only limited determinations of these coefficients have been made for lightweight concretes. They are in the range of 4.0 to 6.0 x 10^{-6}/°F and depend on the amount of natural sand used.

RELEVANT BACKGROUND AND RESEARCH

The coefficient of thermal expansion for lightweight concrete was reported by Hoff (1992) for four different concretes tested at relative humidities of 0, 50, and 100 percent, and Vincent et al. (2004) for one concrete. Measured values ranged from 3.2 to 7.1 millionths/°F with an average value of 4.7 millionths/°F. Price and Cordon (1949) reported values of 4 to 5 millionths/°F depending on the amount of natural sand in the concrete. The lightweight concrete used on the Benicia-Martinez Bridge had a measured coefficient of 4.3 millionths/°F (Caltrans 2007). Kahn and Lopez (2005) reported values of 5.1 and 5.6 millionths/°F.

5.4.2.3 Shrinkage and Creep

5.4.2.3.1 General

These provisions shall be applicable for specified concrete strengths up to 15.0 ksi. In the absence of more accurate data, the shrinkage coefficients may be assumed to be 0.0002 after 28 days and 0.0005 after one year of drying.

5.4.2.3.2 Creep

The creep coefficient may be taken as:

$$\psi(t,t_i) = 1.9 k_{vs} k_{hc} k_f k_{td} t_i^{-0.118} \qquad (5.4.2.3.2\text{-}1)$$

in which:

$$k_{vs} = 1.45 - 0.13(V/S) \geq 0.0 \qquad (5.4.2.3.2\text{-}2)$$

$$k_{hc} = 1.56 - 0.008H \qquad (5.4.2.3.2\text{-}3)$$

$$k_f = \frac{5}{1 + f'_{cl}} \qquad (5.4.2.3.2\text{-}4)$$

$$k_{td} = \left(\frac{t}{61 - 4f'_{ci} + t} \right) \qquad (5.4.2.3.2\text{-}5)$$

where:

H = relative humidity (%). In the absence of better information, H may be taken from Figure

		5.4.2.3.3-1.
k_{vs}	=	factor for the effect of the volume-to-surface ratio of the component
k_f	=	factor for the effect of concrete strength
k_{hc}	=	humidity factor for creep
k_{td}	=	time development factor
t	=	maturity of concrete (day), defined as age of concrete between time of loading for creep calculations, or end of curing for shrinkage calculations, and time being considered for analysis of creep or shrinkage effects
t_i	=	age of concrete when load is initially applied (day)
V/S	=	volume-to-surface ratio (in.)
f'_{ci}	=	specified compressive strength of concrete at time of prestressing for pretensioned members and at time of initial loading for nonprestressed members. If concrete age at time of initial loading is unknown at design time, f'_{ci} may be taken as 0.80 f'_c (ksi).

RELEVANT BACKGROUND AND RESEARCH

A comparison of specific creep versus time for data by Harmon (2005), HDR (1998), Hoff (1992), Lopez et al. (2004), Pfeifer (1968), and Shideler (1957) is shown in Fig. 5.4.2.3.2-A. The data are plotted as specific creep versus concrete age. Specific creep, defined as creep strain divided by applied stress, is used because it does not depend on the initial elastic strain or modulus of elasticity of the concrete. The data in Fig. 5.4.2.3.2-A are for a variety of concrete unit weights, compressive strengths, stress levels, aggregate sources, and loading ages. All of the creep data except those by Lopez et al. are based on 6x12-in. cylinders stored at approximately 73°F and 50 percent relative humidity during the tests. Lopez et al. used 4x15-in. cylinders. The effect of stress level can be taken into account by using creep strain per unit stress or specific creep as plotted in Fig. 5.4.3.2.2-A. It is generally assumed that total creep strain is proportional to stress level up to a stress level of about 40 percent of the compressive strength at the age of loading.

Vincent et al. (2004) performed creep tests on four batches of concrete. However, it is difficult to determine the ages of loading and the applied stress level from their report. It also appears that the load was increased during the test.

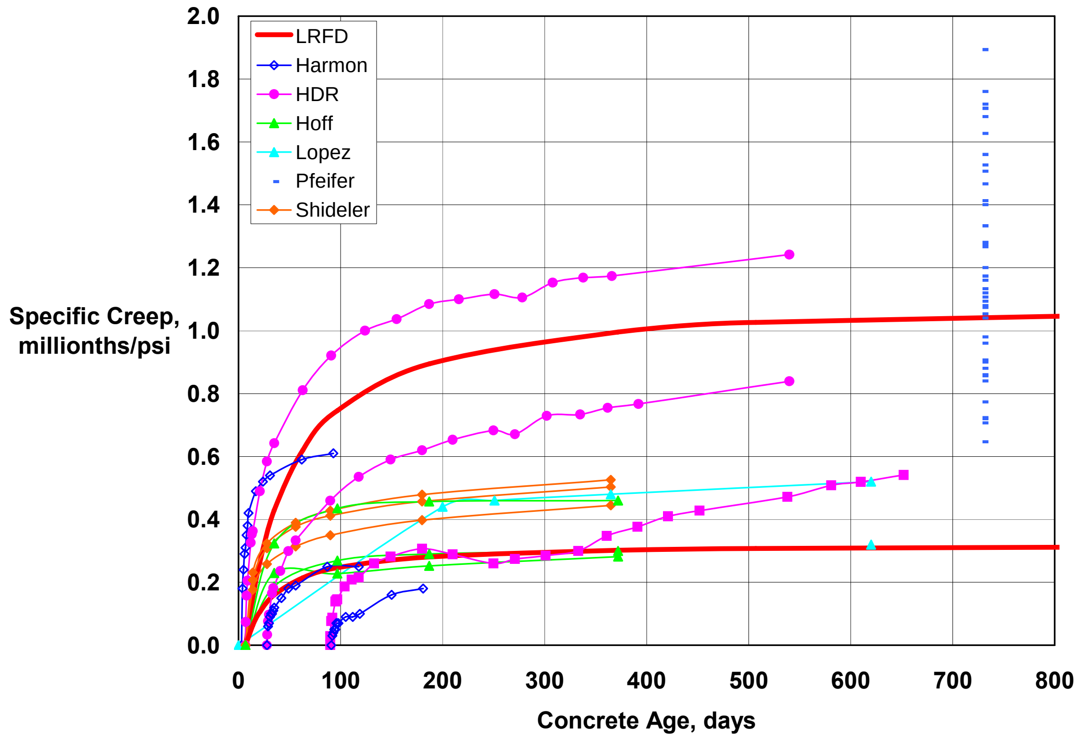

Figure 5.4.2.3.2-A. Creep data

For comparison with the measured values, creep calculated using Eq. 5.4.2.3.2-1 of the LRFD Specifications is also shown in Fig. 5.4.2.3.2-A. The upper line is for a concrete compressive strength of 4.0 ksi and a modulus of elasticity of 2000 ksi at a loading age of 7 days. The lower line is based on concrete compressive strength of 8.0 ksi and a modulus of elasticity of 3.75 ksi at a loading age of 7 days. These lines correspond to unit weights of about 110 and 130 pcf according to the revised equation for modulus of elasticity discussed in Article 5.4.2.4. Both lines are based on 6x12-in cylinders. The two lines correspond to a wide range of creep properties but encompass most of the data except those of Pfeifer (1968). In many cases, the compressive strength of his concrete at the loading age of 7 days was less than 2.0 ksi and the ratio of stress to strength exceeded 0.40.

The commentary to 5.4.2.3.1 states that without specific physical tests or prior experience with the materials, the use of the empirical methods referenced in the specifications cannot be expected to yield results with errors less than ±50 percent. A more detailed analysis is needed to determine if the separate variables in Eq. 5.4.2.3.2-1 represent the true behavior of lightweight concrete since the equation was based on normal weight concrete (Tadros et al., 2003).

During construction of the Benicia-Martinez Bridge in California, the contractor was required to measure the creep of concrete used in the first segment of each cantilever. The test procedure, based on ASTM C 512, required that 6x12-in. cylinders be loaded at an age of 28 days and that creep be measured after 28, 56, and 90 days of loading. In addition, Caltrans measured the creep of concrete used in selected segments. The complete data representing samples of production concrete are shown in Fig. 5.4.2.3.2-B (Caltrans, 2007). Concrete age is used because some of the specimens were not loaded at a concrete age of 28 days as intended and creep is influenced by age of loading. For comparison purposes, the creep calculated using Eq. 5.4.2.3.2-1 is shown as the red line. This calculated line is based on a concrete compressive strength of 10.0 ksi at 28 days and a unit weight of 0.125 pcf. These two numbers were used to calculate the modulus of elasticity for conversion of the creep coefficient from Eq. 5.4.2.3.2-1 to specific creep for the figure according to the following equation:

Specific creep = creep coefficient/modulus of elasticity

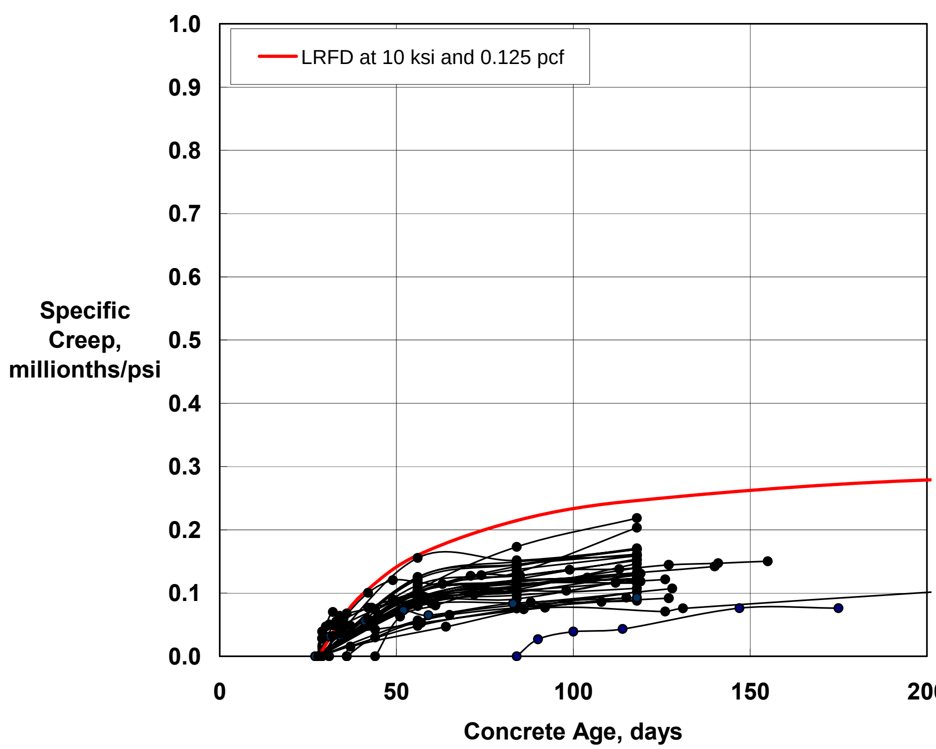

Figure 5.4.2.3.2-B. Creep data from Benicia-Martinez Bridge

5.4.2.3.3 Shrinkage

For concretes devoid of shrinkage-prone aggregates, the strain due to shrinkage, ε_{sh}, at time, *t*, may be taken as:

$$\varepsilon_{sh} = -k_{vs} k_{hs} k_f k_{td} \, 0.48\text{x}10^{-3} \qquad (5.4.2.3.3\text{-}1)$$

in which:

$$k_{hs} = (2.00 - 0.014 \, H) \qquad (5.4.2.3.3\text{-}2)$$

where:

k_{hs} = humidity factor for shrinkage

RELEVANT BACKGROUND AND RESEARCH

A comparison of shrinkage versus drying time for data by Hanson (1968), Hoff (1992), Holm (1980), Leeming (1990), Lopez et al. (2004), Malhotra (1990), Ozyildirim and Gomez (2005), Pfeifer (1968), Rogers (1957), Shideler (1957), and Vincent et al. (2004) is shown in Fig. 5.4.2.3.3-A. These data are for a variety of concrete unit weights, compressive strengths, aggregate sources, curing conditions, and specimen sizes. These variations may contribute to the scatter in the data.

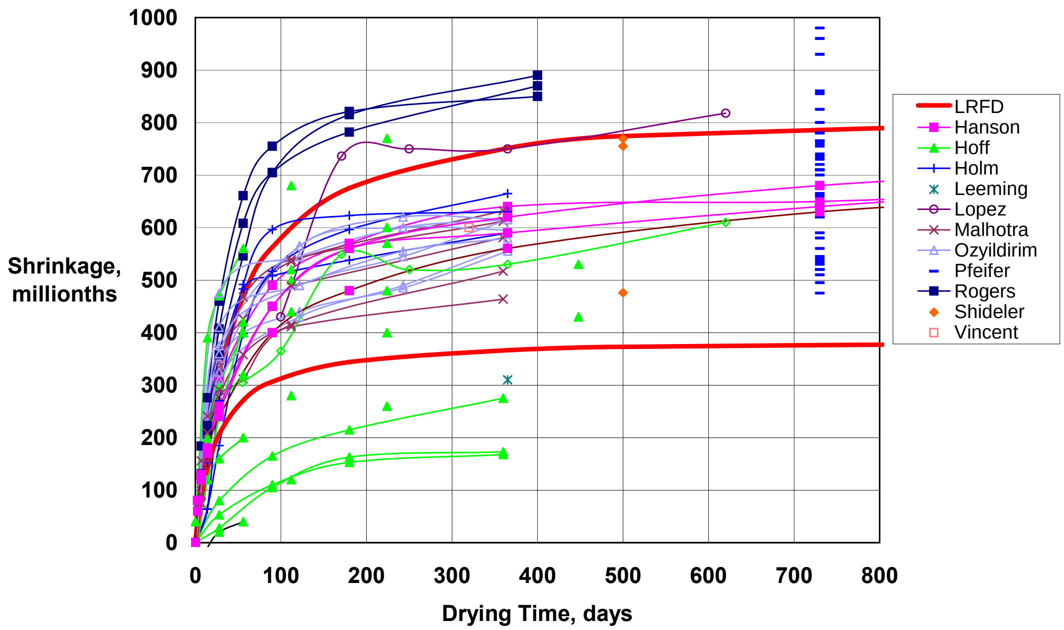

Figure 5.4.2.3.3-A. Shrinkage data

For comparison with the measured values, the shrinkage calculated using Eq. 5.4.2.3.3-1 of the LRFD Specifications is also shown. The upper line is based on a 3x3-in. prism using a concrete compressive strength of 4.0 ksi at the start of shrinkage measurements. The lower line is based on a 6x12-in. cylinder using a concrete compressive strength of 9.0 ksi.

The equations for shrinkage and creep were developed based on normal weight concretes. (Tadros et al. 2003) Nevertheless, the upper and lower limits do encompass most of the range for lightweight concrete.

During construction of the Benicia-Martinez Bridge in California, the contractor was required to measure the shrinkage of the concrete used in the first segment of each cantilever. The test procedure required that the 100x100 mm (4x4-in.) prisms be moist cured for 7 days followed by air drying at 23°C (73°F) and 50 percent relative humidity. In addition, Caltrans measured the shrinkage of concrete used in selected segments. The complete data representing 34 samples of production concrete are shown in Fig. 5.4.2.3.3-B.

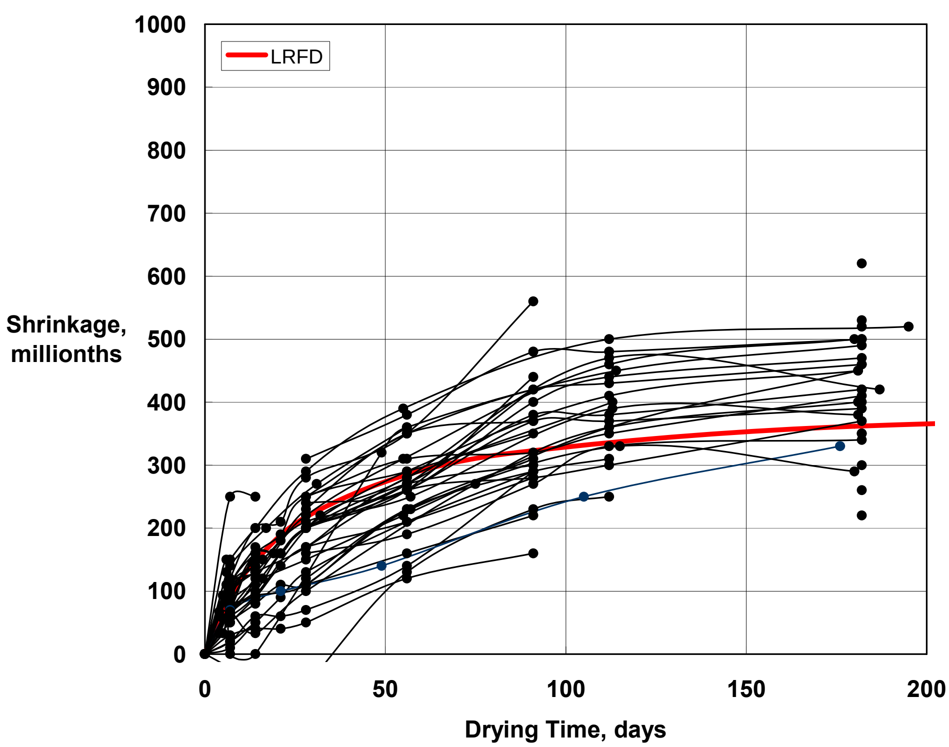

Figure 5.4.2.3.3-B. Shrinkage data from Benicia-Martinez Bridge

For comparison purposes, the shrinkage calculated using Eq. 5.4.2.3.3-1 is shown as the red line. The calculated line is based on a concrete compressive strength of 9.0 ksi at 7 days. The concrete was specified to have a maximum shrinkage of 500 millionths after 180 days of drying in accordance with ASTM C 157. The concrete mix proportions selected by the contractor included a shrinkage-reducing admixture.

5.4.2.4 Modulus of Elasticity
In the absence of measured data, the modulus of elasticity, E_c, for concretes with unit weights between 0.090 and 0.155 kcf and specified compressive strengths up to 15.0 ksi may be taken as:

$$E_c = 33{,}000 K_1 w_c^{1.5} \sqrt{f_c'} \qquad (5.4.2.4\text{-}1)$$

where:

K_1 = correction factor for source of aggregate to be taken as 1.0 unless determined by physical test, and as approved by the authority of jurisdiction
w_c = unit weight of concrete (kcf); refer to Table 3.5.1-1 or Article C5.4.2.4
f'_c = specified compressive strength of concrete (ksi)

RELEVANT BACKGROUND AND RESEARCH

The NCHRP Project No. 12-64 titled "Application of the LRFD Bridge Design Specifications to High-Strength Structural Concrete: Flexure and Compression Provisions" compiled 4388 data points for concrete unit weights ranging from 0.090 to 0.176 pcf, concrete compressive strengths from 0.4 to 24.0 ksi, and concrete modulus of elasticity from 710 to 10,780 ksi. Based on these data, the following equation was recommended to replace Eq. 5.4.2.4-1:

$$E_c = 310{,}000 K_1 w_c^{2.5} f_c'^{0.33}$$

Comparisons of the measured data with values predicted using Eq. 5.4.2.4-1 and the proposed equation are shown in Figs 5.4.2.4-A and 5.4.2.4-B, respectively. A comparison between Eq. 5.4.2.4-1 and the proposed equation for two different concrete unit weights is shown in Fig. 5.4.2.4-C. The effect of the proposed equation is to reduce the predicted values of modulus of elasticity more for the lightweight concretes than for the normal weight concretes.

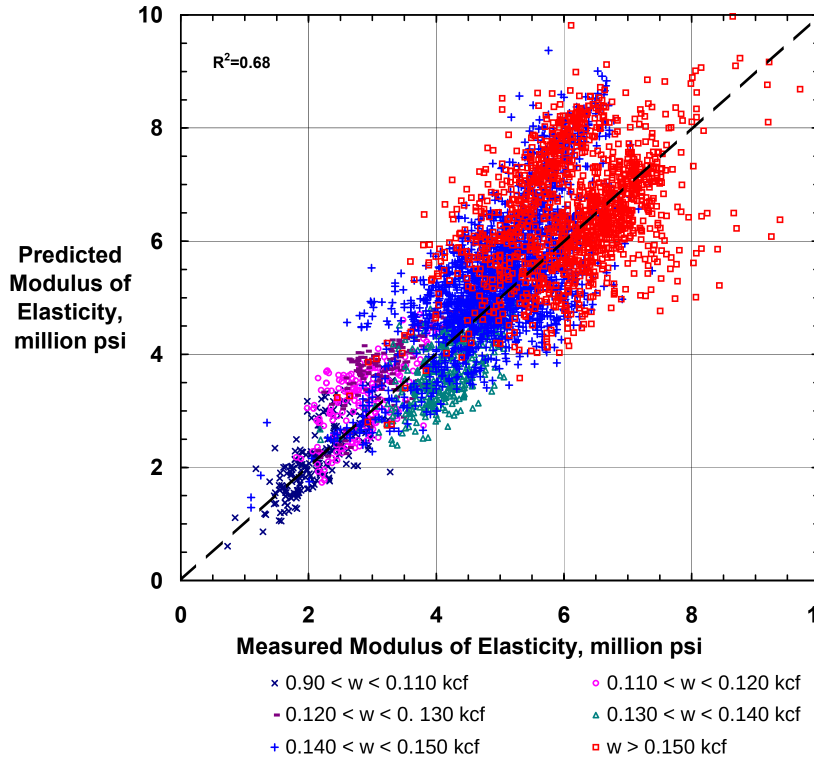

Figure 5.4.2.4-A. Comparison of predicted and measured modulus of elasticity for Eq. 5.4.2.4-1

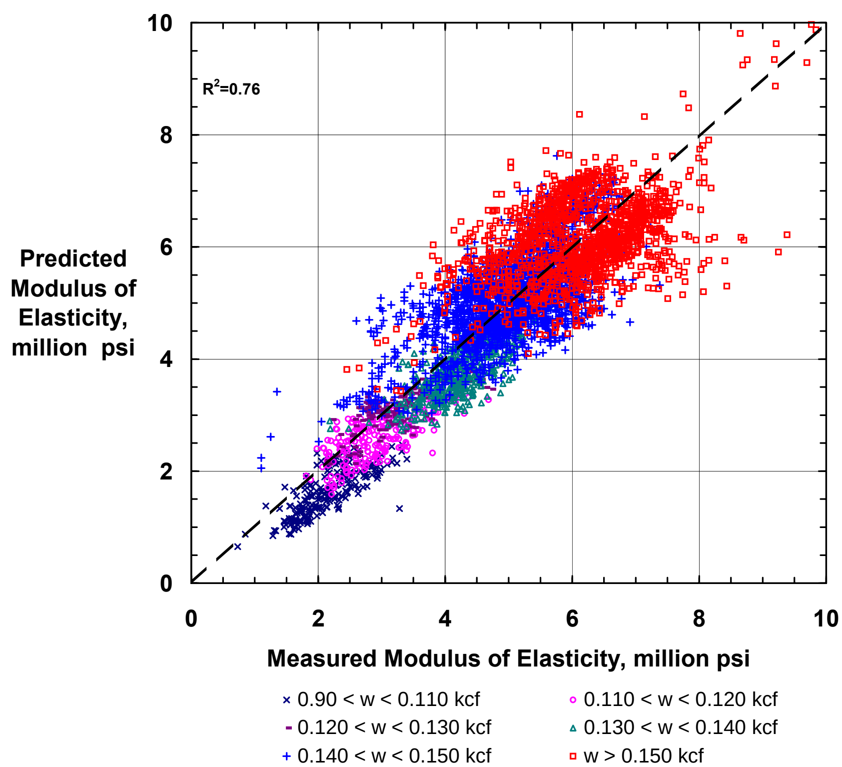

Figure 5.4.2.4-B. Comparison of predicted and measured modulus of elasticity for proposed equation

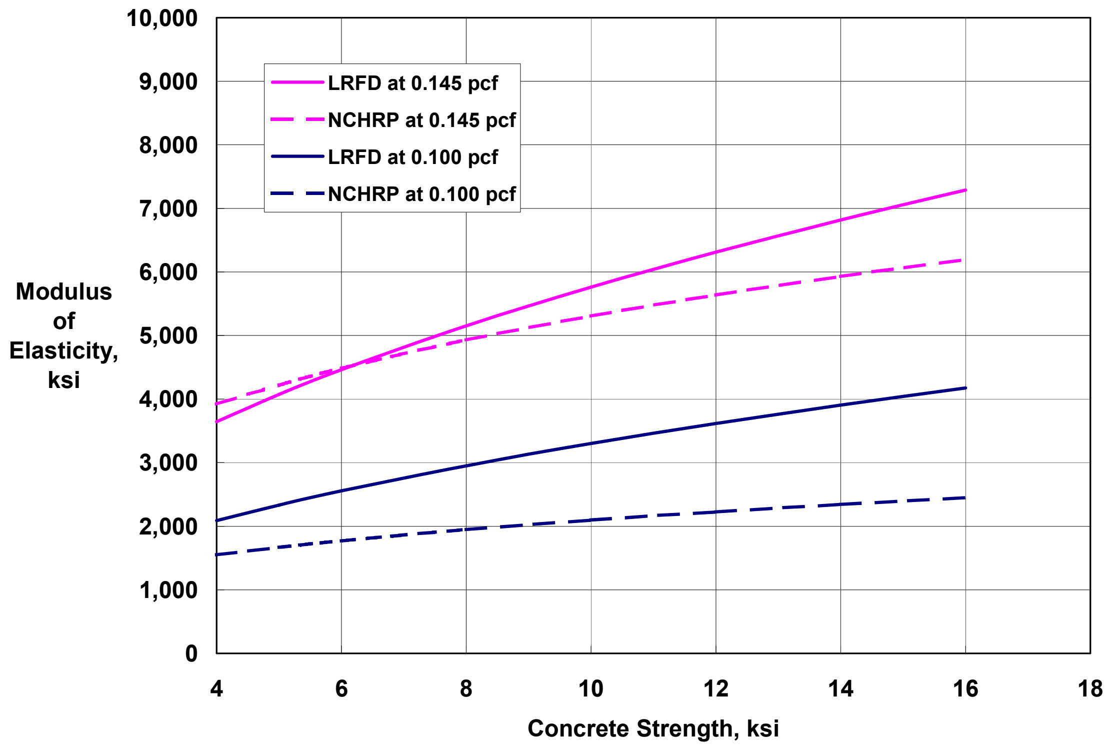

Figure 5.4.2.4-C Comparison of existing and proposed equations

5.4.2.5 Poisson's Ratio
Unless determined by physical tests, Poisson's ratio may be assumed as 0.2.

RELEVANT BACKGROUND AND RESEARCH

Values of Poisson's ratio have been reported by Hanson (1958, 1964a), Harmon (2005), Hoff (1992), Pfeifer and Hanson (1967), and Ramirez et al. (2000). A graph of their measured values versus concrete compressive strength is shown in Fig. 5.4.2.5-A. The average value of all the data is 0.19 compared to the value of 0.20 used in Article 5.4.2.5. Slate et al. (1986) reported that Poisson's ratio for lightweight concrete is close to 0.20 regardless of concrete strength, curing conditions, and test age.

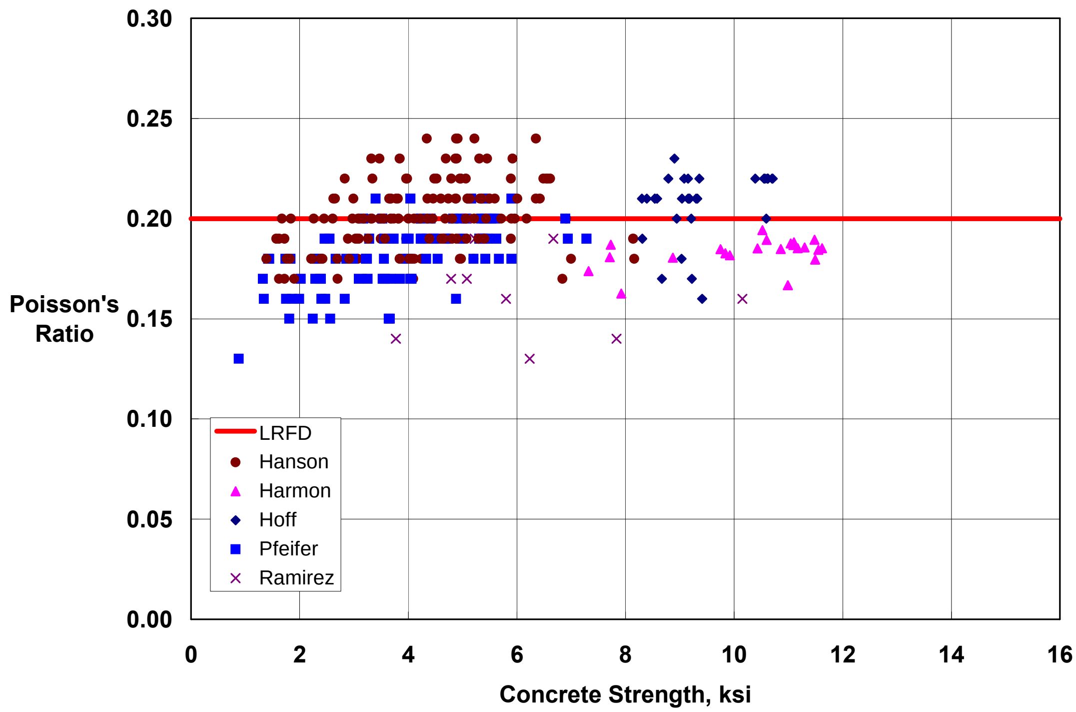

Figure 5.4.2.5-A. Comparison of Poisson's ratio versus concrete compressive strength

5.4.2.6 Modulus of Rupture
Unless determined by physical tests, the modulus of rupture, f_r in ksi, for specified concrete strengths up to 15.0 ksi, may be taken as:

- For lightweight concrete:
  - For sand-lightweight concrete ....$0.20\sqrt{f_c^{'}}$
  - For all-lightweight concrete ....$0.17\sqrt{f_c^{'}}$

RELEVANT BACKGROUND AND RESEARCH

A comparison of modulus of rupture versus concrete compressive strength is shown in Fig. 5.4.2.6-A for data by Harmon (2005), Heffington (2000), Hoff (1992), Malhotra (1990), Meyer (2002), Ozyildirim and Gomez (2005), Ramirez et al. (2000), Shideler (1957), and Tasillo et al. (2004). These data are for a variety of concrete unit weights, aggregate sources, curing conditions, and specimen sizes. Slate et al. (1986) recommended a modulus of rupture of $0.21\sqrt{f_c'}$ for compressive strengths from 3.0 to 9.0 ksi for moist cured lightweight concretes. No satisfactory correlation was found for dry-cured lightweight concrete.

The measured modulus of rupture is sensitive to the curing conditions because specimens that are allowed to dry develop tensile stresses near the surfaces. This in turn results in a reduced measured value of the modulus of rupture. Specimens that are moist cured until test age have a higher measured modulus of rupture compared to specimens that are allowed to dry out. This difference is often larger than would be expected from the change in compressive strength.

For comparison purposes, the two red lines in Fig. 5.4.2.6-A show the modulus of rupture calculated using the provisions of Article 5.4.2.6 for sand-lightweight and all-lightweight concrete.

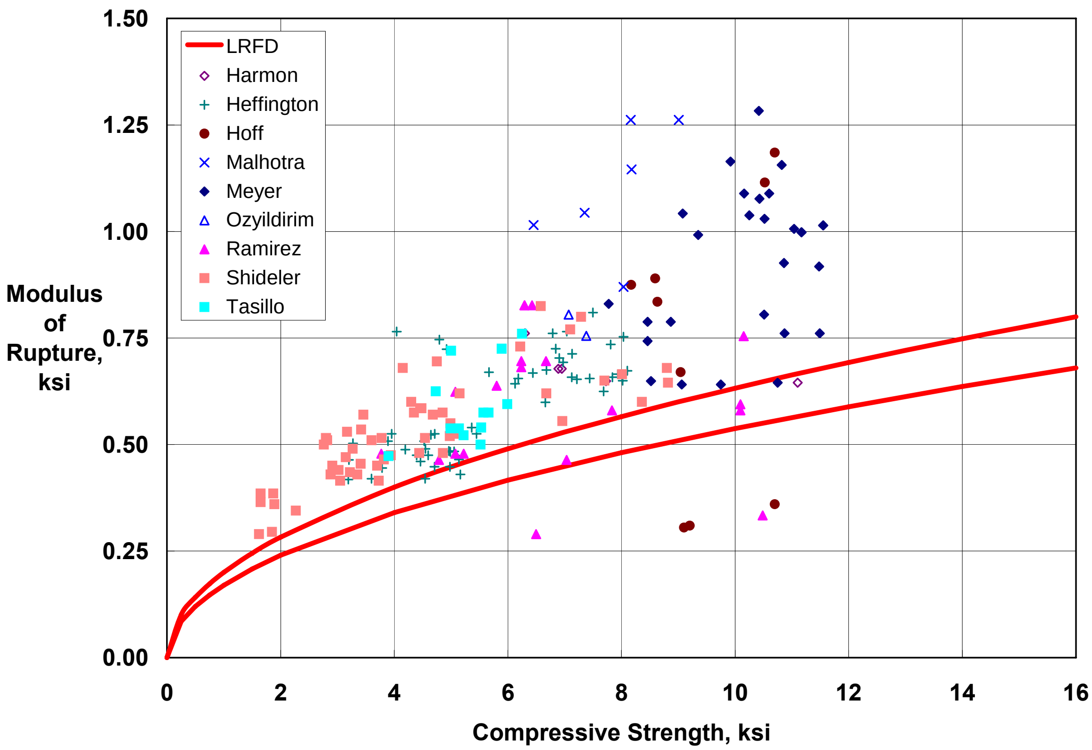

Figure 5.4.2.6-A. Modulus of rupture versus concrete compressive strength

5.4.2.7 Tensile Strength
C5.4.2.7
For most regular concretes, the direct tensile strength may be estimated as $f_r = 0.23\sqrt{f_c^{'}}$.

RELEVANT BACKGROUND AND RESEARCH

A comparison of splitting tensile strength versus concrete compressive strength is shown in Fig. 5.4.2.7-A for data by Hanson (1961, 1965, 1968), Heffington (2000), Hoff (1992), Ivey and Buth (1966), Khaloo and Nakseok (1999), Malhotra (1990), Mattock et al. (1976a), Ozyildirim and Gomez (2005), Pfeifer (1967), Ramirez et al. (2000, 2004), and Vincent et al. (2004). These data are for a variety of concrete unit weights, aggregate sources, and curing conditions. Slate et al. (1986) recommended a splitting strength of $0.16\sqrt{f_c^{'}}$ for lightweight concrete with compressive strengths from 3.0 to 9.0 ksi.

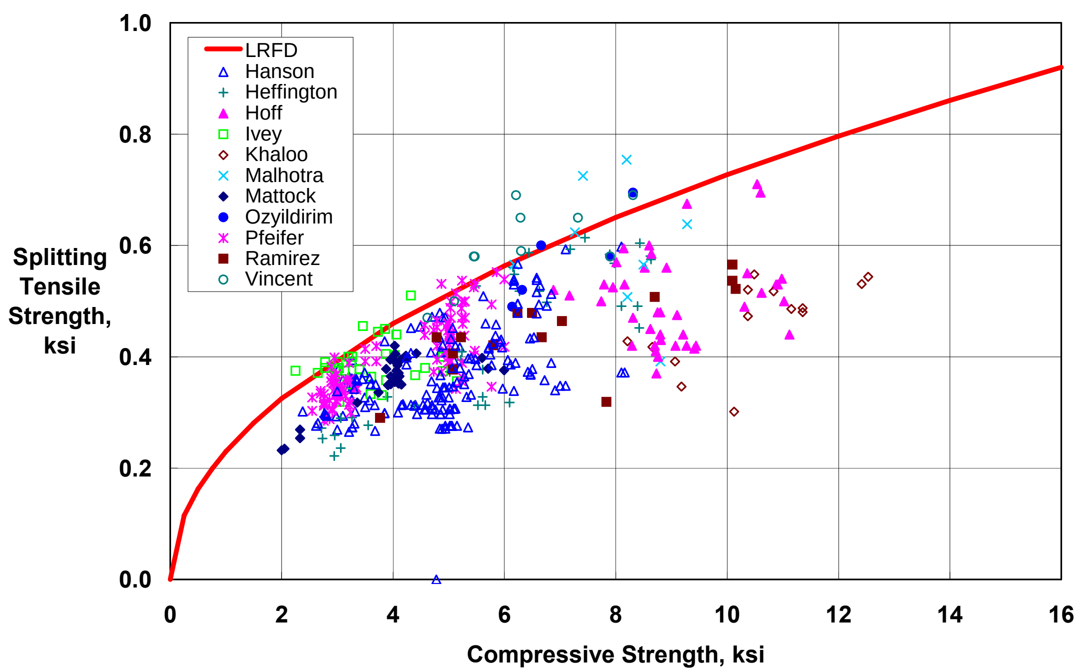

Figure 5.4.2.7-A. Splitting tensile strength versus concrete compressive strength

The effects of drying on splitting tensile strengths are well known. The drying of the outer regions of the test specimen establishes differential shrinkage within the concrete which, in turn, leads to the development of tensile strains in the outer surfaces of the specimen. These strains reduce the measured value of splitting tensile strength. (Hoff, 1992)

For comparison purposes, the relationship from Commentary C5.4.2.7 is plotted in Fig. 5.4.2.7-A. This comparison assumes that the direct tensile strength referred to in the commentary is the same as the strength measured in the splitting tensile strength test. For most of the data, the relationship overestimates the measured values. The equation was probably developed for normal weight concretes and there are no reductions in the coefficient for lightweight concrete similar to those used for modulus of rupture. A proposed revision restricting this article to normal weight concrete has been submitted to AASHTO Technical Committee T-10, Concrete Design.

ARTICLE 5.5 LIMIT STATES

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.
Fatigue of the reinforcement need not be checked for fully prestressed components designed to have extreme fiber tensile stress due to Service III Limit State within the tensile stress limit specified in Table 5.9.4.2.2-1.

C5.5.3.1
Stresses measured in concrete deck slabs of bridges in service are far below infinite fatigue life, most probably due to internal arching action; see Article C9.7.2.
For fully prestressed components, the net concrete stress is usually significantly less than the concrete tensile stress limit specified in Table 5.9.4.2.2-1. Therefore, the calculated flexural stresses are significantly reduced. For this situation, the calculated steel stress range, which is equal to the modular ratio times the concrete stress range, is almost always less than the steel fatigue stress range limit specified in Article 5.5.3.3.

RELEVANT BACKGROUND AND RESEARCH

No research specifically addressing the fatigue limit state for lightweight concrete was identified.

Ramakrishnan et al. (1992) conducted tests to determine flexural fatigue strength of lightweight concretes made using expanded shale aggregates. The endurance limit, defined as the ratio of modulus of rupture after 2 million cycles of loading to the modulus of rupture before loading, ranged from 0.55 to 0.72. The researchers concluded that the performance of the lightweight concretes was similar to that of normal weight concretes.

Hoff (1994) reviewed the research on the compressive and flexural fatigue behavior of high-strength lightweight concrete. He concluded that the fatigue behavior of high-strength lightweight concrete was comparable or somewhat better than high-strength normal density concrete tested under the same conditions.

5.5.4 Strength Limit State
5.5.4.2 Resistance Factors
5.5.4.2.1 Conventional Construction
Resistance factor ϕ shall be taken as:

- For shear and torsion:

  lightweight concrete 0.70

- For compression in anchorage zones:

  lightweight concrete............................ 0.65

C5.5.4.2.1
The ϕ-factor of 0.65 for lightweight concrete reflects its often lower tensile strength and is based on the multipliers used in ACI 318-89, Section 11.2.1.2.

5.5.4.2.2 Segmental Construction
Resistance factors for the strength limit state shall be taken as provided in Table 1 for the conditions indicated and in Article 5.5.4.2.1 for conditions not covered in Table 1.

Table 5.5.4.2.2-1 Resistance Factor for Joints in Segmental Construction.

	ϕ_f Flexure	ϕ_v Shear
Sand-Lightweight Concrete		
Fully Bonded Tendons	0.90	0.70
Unbonded or Partially Bonded Tendons	0.85	0.65

RELEVANT BACKGROUND AND RESEARCH

No research addressing the resistance factors for the strength limit state with lightweight concrete was identified.

ARTICLE 5.6 DESIGN CONSIDERATIONS

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:

$$f_{cu} = \frac{f_c'}{0.8 + 170\varepsilon_1} \le 0.85 f_c' \qquad (5.6.3.3.3\text{-}1)$$

in which:

$$\varepsilon_1 = \varepsilon_s + (\varepsilon_s + 0.002)\cot^2 \alpha_s \qquad (5.6.3.3.3\text{-}2)$$

α_s = the smallest angle between the compressive strut and adjoining tension ties (°)
ε_s = the tensile strain in the concrete in the direction of the tension tie (in./in.)
f'_c = specified compressive strength (ksi)

5.6.3.3.4 Reinforced Strut
If the compressive strut contains reinforcement that is parallel to the strut and detailed to develop its yield stress in compression, the nominal resistance of the strut shall be taken as:

$$P_n = f_{cu}A_{cs} + f_y A_{ss} \qquad (5.6.3.3.4\text{-}1)$$

where:

A_{ss} = area of reinforcement in the strut (in.^{2})

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: $0.85\ \phi f'_c$
- For node regions anchoring a one-direction tension tie: $0.75\ \phi f'_c$
- For node regions anchoring tension ties in more than one direction: $0.65\ \phi f'_c$

where:

ϕ = the resistance factor for bearing on concrete as specified in Article 5.5.4.2.

RELEVANT BACKGROUND AND RESEARCH

Based on the analysis of six test girders failing in shear, Meyer (2002) concluded that a variable angle truss model provided an overly conservative prediction of shear capacity.

ARTICLE 5.7 DESIGN FOR FLEXURAL AND AXIAL FORCE EFFECTS

5.7 DESIGN FOR FLEXURAL AND AXIAL FORCE EFFECTS
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 unconfined, the maximum usable strain at the extreme concrete compression fiber is not greater than 0.003.

- If the concrete is confined, a maximum usable strain exceeding 0.003 in the confined core may be utilized if verified. Calculation of the factored resistance shall consider that the concrete cover may be lost at strains compatible with those in the confined concrete core.

- Balanced strain conditions exist at a cross-section when tension reinforcement reaches the strain corresponding to its specified yield strength f_y just as the concrete in compression reaches its assumed ultimate strain of 0.003.

- Sections are compression-controlled when the net tensile strain in the extreme tension steel is equal to or less than the compression-controlled strain limit at the time the concrete in compression reaches its assumed strain limit of 0.003. The compression-controlled strain limit is the net tensile strain in the reinforcement at balanced strain conditions. For Grade 60 reinforcement, and for all prestressed reinforcement, the compression-controlled strain limit may be set equal to 0.002.

- Sections are tension-controlled when the net tensile strain in the extreme tension steel is equal to or greater than 0.005 just as the concrete in compression reaches its assumed strain limit of 0.003. Sections with net tensile strain in the extreme tension steel between the compression-controlled strain limit and 0.005 constitute a transition region between compression-controlled and tension-controlled sections.

C5.7.2.1
The nominal flexural strength of a member is reached when the strain in the extreme compression fiber reaches the assumed strain limit of 0.003. The net tensile strain ε_t is the tensile strain in the extreme tension steel at nominal strength, exclusive of strains due to prestress, creep, shrinkage, and temperature. The net tensile strain in the extreme tension steel is determined from a linear strain distribution at nominal strength, as shown in Figure C5.7.2.1-1, using similar triangles.

Figure C5.7.2.1-1 Strain Distribution and Net Tensile Strain.

When the net tensile strain in the extreme tension steel is sufficiently large (equal to or greater than 0.005), the section is defined as tension-controlled where ample warning of failure with excessive deflection and cracking may be expected. When the net tensile strain in the extreme tension steel is small (less than or equal to the compression-controlled strain limit), a brittle failure condition may be expected, with little warning of impending failure. Flexural members are usually tension-controlled, while compression members are usually compression-controlled. Some sections, such as those with small axial load and large bending moment, will have net tensile strain in the extreme tension steel between the above limits. These sections are in a transition region between compression- and tension-controlled sections. Article 5.5.4.2.1 specifies the appropriate resistance factors for tension-controlled and compression-controlled sections, and for intermediate cases in the transition region.

Before the development of these provisions, the limiting tensile strain for flexural members was not stated, but was implicit in the maximum reinforcement limit that was given as $c/d_e \lesssim 0.42$, which corresponded to a net tensile strain at the centroid of the tension reinforcement of 0.00414. The net tensile strain limit of 0.005 for tension-controlled sections was chosen to be a single value that applies to all types of steel (prestressed and nonprestressed) permitted by this Specification.

Unless unusual amounts of ductility are required, the 0.005 limit will provide ductile behavior for most designs. One condition where greater ductile behavior is required is in design for redistribution of moments in continuous members and frames. Article 5.7.3.5 permits redistribution of negative moments. Since moment redistribution is dependent on adequate ductility in hinge regions, moment redistribution is limited to sections that have a net tensile strain of at least 0.0075.

For beams with compression reinforcement, or T-beams, the effects of compression reinforcement and flanges are automatically accounted for in the computation of net tensile strain ε_t.

RELEVANT BACKGROUND AND RESEARCH

The concept of tension-controlled and compression-controlled sections is similar to that in the ACI Building Code, which was based on work by Mast (1992).

A comparison of maximum usable concrete compressive strain versus concrete compressive strength is shown in Fig. 5.7.2.1-A for data by Ahmad and Barker (1991), Ahmad and Batts (1991), Hoff (1992), Kaar et al. (1978), and Thatcher et al. (2002). For most of the data, the current assumed maximum usable strain of 0.003 for unconfined concrete is a conservative value for lightweight concrete.

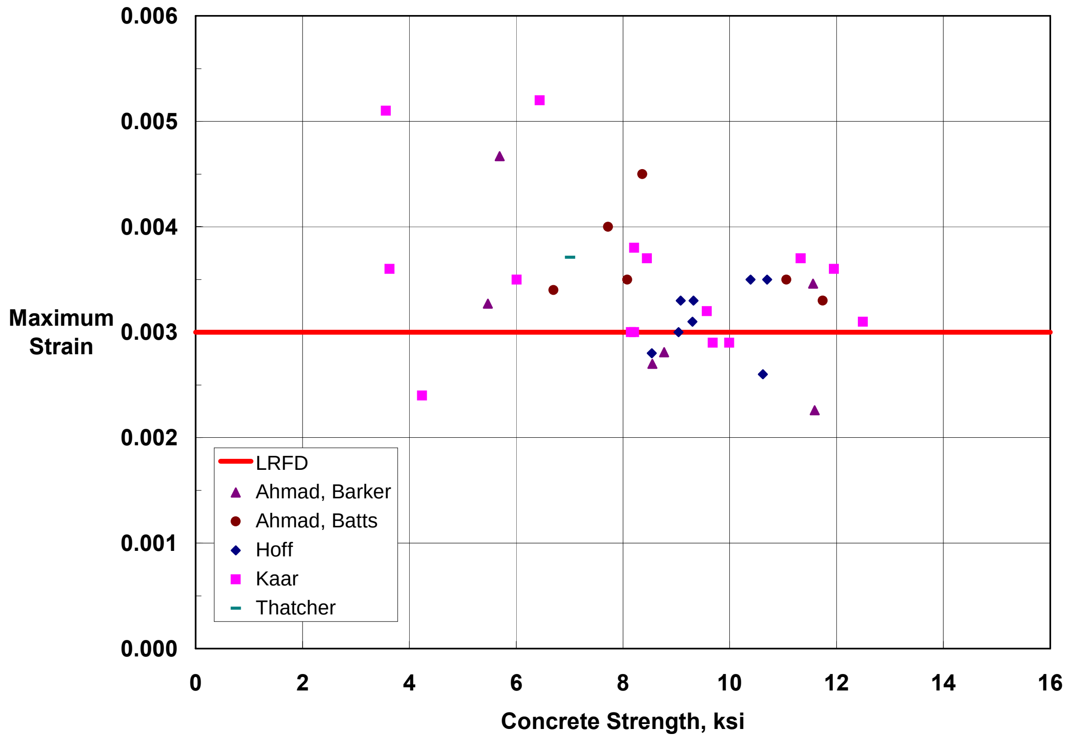

Figure 5.7.2.1-A. Maximum strain versus concrete compressive strength

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 to be less than 0.65.

C5.7.2.2

For sections that consist of a beam with a composite slab of different concrete strength, and the compression block includes both types of concrete, it is conservative to assume the composite beam to be of uniform strength at the lower of the concrete strengths in the flange and web. If a more refined estimate of flexural capacity is warranted, a more rigorous analysis method should be used.

RELEVANT BACKGROUND AND RESEARCH

The equivalent rectangular stress block assumes a uniform compressive stress equal to $0.85f'_c$. The 0.85 multiplier is sometimes called the α_1 factor. Values of α_1 versus concrete compressive strengths are shown in Fig. 5.7.2.2-A for data by Hoff (1992) and Kaar et al. (1978). In addition, there were 15 tests by Hanson reported by Hognestad et al. (1956) but specific values were not reported.

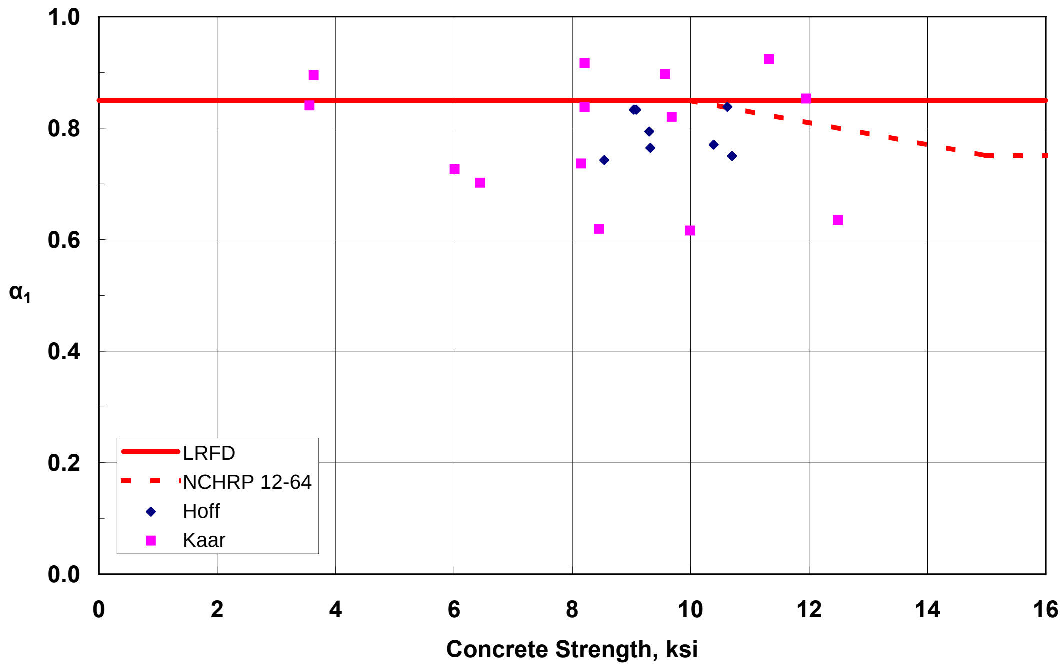

Fig. 5.7.2.2-A. Variation of α_1 factor with concrete compressive strength

For comparison purposes, the value of 0.85 used in the LRFD Specifications is also shown in Fig. 5.7.2.2-A as the solid red line. Based on test data for normal weight concrete, NCHRP Project 12-64 has proposed that for concrete compressive strengths greater than 10.0 ksi, the value of α_1 shall be reduced by 0.02 for each 1.0 ksi in excess of 10.0 ksi but shall not be less than 0.75. This proposed relationship is also shown in Fig. 5.7.2.2-A as the broken red line. Based on the limited data, it would seem that the existing and proposed relationships overestimate the value of α_1 for lightweight concrete. Kaar et al. (1978) suggested that α_1 should be taken as 0.65 for all strengths of lightweight concrete.

Values of β_1 for different concrete compressive strengths are shown in Fig. 5.7.2.2-B for data by Hoff (1992) and Kaar et al. (1978). For comparison purposes, the value of β_1 used in the LRFD Specifications is also shown. For all values except one, the measured values exceed the LRFD values.

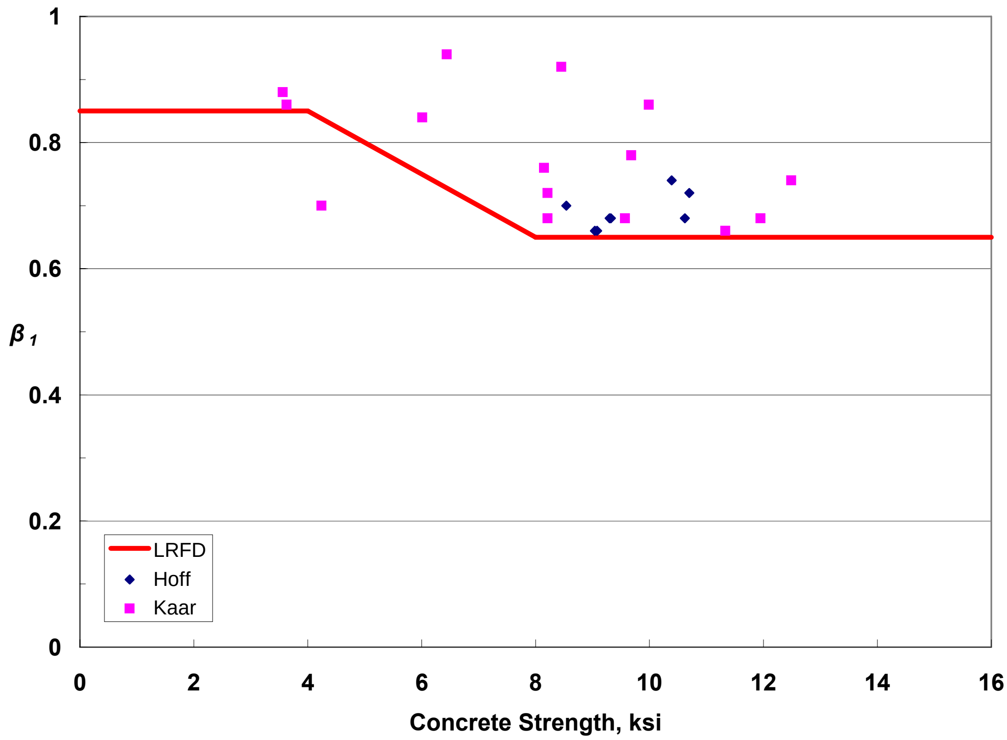

Figure 5.7.2.2-B. Variations of β_1 factor with concrete compressive strength

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:

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right) \qquad (5.7.3.1.1\text{-}1)$$

in which:

$$k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right) \quad (5.7.3.1.1\text{-}2)$$

for T-section behavior:

$$c = \frac{A_{ps} f_{pu} + A_s f_y - A_s^{'} f_y^{'} - 0.85 f_c^{'} (b - b_w) h_f}{0.85 f_c^{'} \beta_1 b_w + k A_{ps} \dfrac{f_{pu}}{d_p}} \quad (5.7.3.1.1\text{-}3)$$

for rectangular section behavior:

$$c = \frac{A_{ps} f_{pu} + A_s f_y - A_s^{'} f_y^{'}}{0.85 f_c^{'} \beta_1 b + k A_{ps} \dfrac{f_{pu}}{d_p}} \quad (5.7.3.1.1\text{-}4)$$

where:

A_{ps} = area of prestressing steel (in.^{2})
f_{pu} = specified tensile strength of prestressing steel (ksi)
f_{py} = yield strength of prestressing steel (ksi)
A_s = area of mild steel tension reinforcement (in.^{2})
A'_s = area of compression reinforcement (in.^{2})
f_y = yield strength of tension reinforcement (ksi)
f'_y = yield strength of compression reinforcement (ksi)
b = width of compression flange (in.)
b_w = width of web (in.)
h_f = depth of compression flange (in.)
d_p = distance from extreme compression fiber to the centroid of the prestressing tendons (in.)
c = distance between the neutral axis and the compressive face (in.)
β_1 = stress block factor specified in Article 5.7.2.2

The stress level in the compressive reinforcement shall be investigated, and if the compressive reinforcement has not yielded, the actual stress shall be used in Eq. 3 instead of f'_y.

5.7.3.1.2 Components with Unbonded Tendons
For rectangular or 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, the average stress in unbonded prestressing steel may be taken as:

$$f_{ps} = f_{pe} + 900\left(\frac{d_p - c}{\lambda_e}\right) \le f_{py} \quad (5.7.3.1.2\text{-}1)$$

in which:

$$\lambda_e = \left(\frac{2\lambda_i}{2 + N_s} \right) \quad (5.7.3.1.2\text{-}2)$$

for T-section behavior:

$$c = \frac{A_{ps} f_{ps} + A_s f_y - A_s^{'} f_y^{'} - 0.85 f_c^{'} (b - b_w) h_f}{0.85 f_c^{'} \beta_1 b_w} \quad (5.7.3.1.2\text{-}3)$$

for rectangular section behavior:

$$c = \frac{A_{ps} f_{ps} + A_s f_y - A_s^{'} f_y^{'}}{0.85 f_c^{'} \beta_1 b} \quad (5.7.3.1.2\text{-}4)$$

where:

c = distance from extreme compression fiber to the neutral axis assuming the tendon prestressing steel has yielded, given by Eqs. 3 and 4 for T-section behavior and rectangular section behavior, respectively (in.)
ℓ_e = effective tendon length (in.)
ℓ_i = length of tendon between anchorages (in.)
N_s = number of support hinges crossed by the tendon between anchorages or discretely bonded points
f_{py} = yield strength of prestressing steel (ksi)
f_{pe} = effective stress in prestressing steel at section under consideration after all losses (ksi)

The stress level in the compressive reinforcement shall be investigated, and if the compressive reinforcement has not yielded, the actual stress shall be used in Eq. 3 instead of f'_y.

RELEVANT BACKGROUND AND RESEARCH

No research specific to the calculation of the stress in prestressing steel at nominal flexural resistance for lightweight concrete members was identified.

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 where the compression flange depth is less than $a=\beta_1 c$, as determined in accordance with Eqs. 5.7.3.1.1-3, 5.7.3.1.1-4, 5.7.3.1.2-3, or 5.7.3.1.2-4, the nominal flexural resistance may be taken as:

$$M_n = A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right) + A_s f_y\left(d_s - \frac{a}{2}\right) - A_s^{'} f_y^{'}\left(d_s^{'} - \frac{a}{2}\right) + 0.85 f_c^{'}(b - b_w)h_f\left(\frac{a}{2} - \frac{h_f}{2}\right) \quad (5.7.3.2.2\text{-}1)$$

where:

A_{ps} = area of prestressing steel (in.^{2})
f_{ps} = average stress in prestressing steel at nominal bending resistance specified in Eq. 5.7.3.1.1-1 (ksi)
d_p = distance from extreme compression fiber to the centroid of prestressing tendons (in.)
A_s = area of nonprestressed tension reinforcement (in.^{2})
f_y = specified yield strength of reinforcing bars (ksi)
d_s = distance from extreme compression fiber to the centroid of nonprestressed tensile reinforcement (in.)
A'_s = area of compression reinforcement (in.^{2})
f'_y = specified yield strength of compression reinforcement (ksi)
d'_s = distance from extreme compression fiber to the centroid of compression reinforcement (in.)
f'_c = specified compressive strength of concrete at 28 days, unless another age is specified (ksi)
b = width of the compression face of the member (in.)
b_w = web width or diameter of a circular section (in.)
β_1 = stress block factor specified in Article 5.7.2.2
h_f = compression flange depth of an I or T member (in.)
a = $c\beta_1$; depth of the equivalent stress block (in.)

RELEVANT BACKGROUND AND RESEARCH

Comparison of measured and calculated flexural strengths have been reported by Ahmad and Barker (1991), Ahmad and Batts (1991), Meyer (2002), Peterman et al. (1999), and Thatcher et al. (2002). In some cases, the flexural strengths were measured as part of a program to determine development length. A graph of measured strength divided by calculated strength versus concrete compressive strength is shown in Fig. 5.7.3.2-A. In all cases, the calculated flexural strengths were determined using the procedures of the Standard Specifications (AASHTO, 1996) or the ACI Building Code (ACI Committee 318, 1983a). However, since the procedures of the LRFD Specifications result in similar strengths to those calculated using the Standard Specifications and the ACI Building Code, the ratios of measured to calculated strengths should not be too different using the LRFD Specifications. Further analysis of the test results using the LRFD Specifications may be warranted.

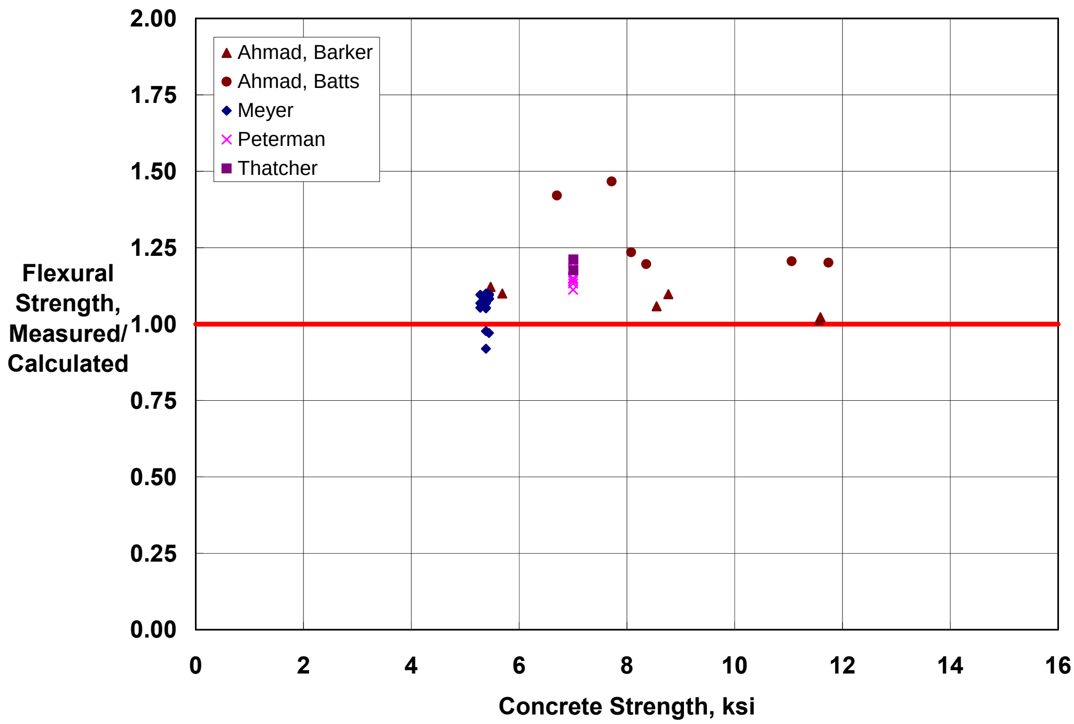

Figure 5.7.3.2-A. Comparison of the ratio of measured to calculated flexural strength with concrete compressive strengths

5.7.3.5 Moment Redistribution
In lieu of more refined analysis, where bonded reinforcement that satisfies the provisions of Article 5.11 is provided at the internal supports of continuous reinforced concrete beams, negative moments determined by elastic theory at strength limit states may be increased or decreased by not more than 1000 ε_t percent, with a maximum of 20 percent. Redistribution of negative moments shall be made only when ε_t is equal to or greater than 0.0075 at the section at which moment is reduced.

RELEVANT BACKGROUND AND RESEARCH

No research specific to the calculation of moment redistribution for lightweight concrete members was identified.

5.7.3.6 Deformations

5.7.3.6.2 Deflection and Camber

In the absence of a more comprehensive analysis, instantaneous deflections may be computed using the modulus of elasticity for concrete as specified in Article 5.4.2.4 and taking the moment of inertia as either the gross moment of inertia, I_g, or an effective moment of inertia, I_e, given by Eq. 1:

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \le I_g \qquad (5.7.3.6.2\text{-}1)$$

in which:

$$M_{cr} = f_r \frac{I_g}{y_t} \qquad (5.7.3.6.2\text{-}2)$$

where:

M_{cr} = cracking moment (kip-in.)
f_r = modulus of rupture of concrete as specified in Article 5.4.2.6 (ksi)
y_t = distance from the neutral axis to the extreme tension fiber (in.)
M_a = maximum moment in a component at the stage for which deformation is computed (kip-in.)

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: $3.0–1.2(A'_s/A_s) \geq 1.6$

where:

A'_s = area of compression reinforcement (in.^{2})
A_s = area of nonprestressed tension reinforcement (in.^{2})

RELEVANT BACKGROUND AND RESEARCH

No research specifically dealing with the calculation of deflection and camber for lightweight concrete was identified.

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:

$$\frac{A_s}{A_g} + \frac{A_{ps} f_{pu}}{A_g f_y} \leq 0.08 \qquad (5.7.4.2\text{-}1)$$

and

$$\frac{A_{ps} f_{pe}}{A_g f_c^{'}} \leq 0.30 \qquad (5.7.4.2\text{-}2)$$

The minimum area of prestressed and nonprestressed longitudinal reinforcement for noncomposite compression components shall be such that:

$$\frac{A_s f_y}{A_g f_c^{'}} + \frac{A_{ps} f_{pu}}{A_g f_c^{'}} \geq 0.135 \qquad (5.7.4.2\text{-}3)$$

where:

A_s = area of nonprestressed tension steel (in.^{2})
A_g = gross area of section (in.^{2})
A_{ps} = area of prestressing steel (in.^{2})
f_{pu} = specified tensile strength of prestressing steel (ksi)
f_y = specified yield strength of reinforcing bars (ksi)
f'_c = specified compressive strength of concrete (ksi)
f_{pe} = effective prestress (ksi)

RELEVANT BACKGROUND AND RESEARCH

The requirement for minimum reinforcement in columns was selected to ensure that column reinforcement would not yield in compression as a result of elastic shortening, creep, and shrinkage. In NCHRP Project 12-64, it was identified that Eq. 5.7.4.2-3 can result in high ratios for the minimum area of reinforcement with high-strength concrete. Consequently, the project has proposed that the value of A_s/A_g should not be greater than 0.0225.

No research specifically addressing the maximum and minimum area of prestressed and nonprestressed longitudinal reinforcement in lightweight concrete members was identified. Pfeifer (1969) conducted creep tests on 6-in. diameter lightweight concrete columns with reinforcement ratios ranging from 0.0117 to 0.0838. The highest stress in the reinforcement was 44.0 ksi after 2-1/2 years of sustained loading.

5.7.4.4 Factored Axial Resistance

The factored axial resistance of concrete compressive components, symmetrical about both principal axes, shall be taken as:

$$P_r = \phi P_n \qquad (5.7.4.4\text{-}1)$$

in which:

- For members with spiral reinforcement:

$$P_n = 0.85\begin{bmatrix} 0.85 f_c'\left(A_g - A_{st} - A_{ps}\right) \\ + f_y A_{st} - A_{ps}\left(f_{pe} - E_p \varepsilon_{cu}\right) \end{bmatrix} \qquad (5.7.4.4\text{-}2)$$

- For members with tie reinforcement:

$$P_n = 0.80\begin{bmatrix} 0.85 f_c'\left(A_g - A_{st} - A_{ps}\right) \\ + f_y A_{st} - A_{ps}\left(f_{pe} - E_p \varepsilon_{cu}\right) \end{bmatrix} \qquad (5.7.4.4\text{-}3)$$

where:

P_r = factored axial resistance, with or without flexure (kip)
P_n = nominal axial resistance, with or without flexure (kip)
f'_c = specified strength of concrete at 28 days, unless another age is specified (ksi)
A_g = gross area of section (in.^{2})
A_{st} = total area of longitudinal reinforcement (in.^{2})
f_y = specified yield strength of reinforcement (ksi)
ϕ = resistance factor specified in Article 5.5.4.2
A_{ps} = area of prestressing steel (in.^{2})
E_p = modulus of elasticity of prestressing tendons (ksi)
f_{pe} = effective stress in prestressing steel after losses (ksi)
ε_{cu} = failure strain of concrete in compression (in./in.)

RELEVANT BACKGROUND AND RESEARCH

Pfeifer (1969) tested twenty 6-in. diameter lightweight concrete columns with concrete compressive strengths ranging from 4.42 to 7.60 ksi, steel yield strengths ranging from 50.0 to 92.5 ksi, and percentage of reinforcement ranging from 0 to 8.38 percent. Measured strengths were compared with the equivalent of Eq. 5.7.4.4-2. The measured strengths were slightly less than predicted in most cases and significantly less than predicted when the steel yield strength was 92.5 ksi.

Research about the α_1 factor in Article 5.7.2.2 is relevant because the same 0.85 factor appears in Eq. 5.7.4.4-2 and 5.7.4.4-3.

5.7.4.5 Biaxial Flexure

In lieu of an analysis based on equilibrium and strain compatibility for biaxial flexure, noncircular members subjected to biaxial flexure and compression may be proportioned using the following approximate expressions:

- If the factored axial load is not less than 0.10 $\phi f'_c A_g$:

$$\frac{1}{P_{rxy}} = \frac{1}{P_{rs}} + \frac{1}{P_{ry}} - \frac{1}{\phi P_o} \qquad (5.7.4.5\text{-}1)$$

in which:

$$P_o = 0.85 f'_c \left(A_g - A_{st} - A_{ps}\right) + f_y A_{st} - A_{ps}\left(f_{pc} - E_p \varepsilon_{cu}\right) \qquad (5.7.4.5\text{-}2)$$

where:

ϕ = resistance factor for members in axial compression
P_{rxy} = factored axial resistance in biaxial flexure (kip)
P_{rx} = factored axial resistance determined on the basis that only eccentricity e_y is present (kip)
P_{ry} = factored axial resistance determined on the basis that only eccentricity e_x is present (kip)
P_u = factored applied axial force (kip)
M_{ux} = factored applied moment about the X-axis (kip-in.)
M_{uy} = factored applied moment about the Y-axis (kip-in.)
e_x = eccentricity of the applied factored axial force in the X direction, i.e., = M_{uy}/P_u (in.)
e_y = eccentricity of the applied factored axial force in the Y direction, i.e., = M_{ux}/P_u (in.)
P_o = nominal axial resistance of a section at 0.0 eccentricity

RELEVANT BACKGROUND AND RESEARCH

No research specifically addressing the biaxial flexural strength of lightweight concrete was identified. However, research about the α_1 factor in Article 5.7.2.2 is relevant because the same 0.85 factor appears in Eq. 5.7.4.5-2.

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 satisfy:

$$\rho_s \geq 0.45\left(\frac{A_g}{A_c}-1\right)\frac{f_c'}{f_{yh}} \qquad (5.7.4.6\text{-}1)$$

where:

A_g = gross area of concrete section (in.^{2})

A_c = area of core measured to the outside diameter of the spiral (in.^{2})

f'_c = specified strength of concrete at 28 days, unless another age is specified (ksi)

f_{yh} = specified yield strength of spiral reinforcement (ksi)

RELEVANT BACKGROUND AND RESEARCH

Martinez et al. (1984) tested 47 short columns with lightweight concrete having compressive strengths between 3.66 and 9.88 ksi. All specimens contained spiral reinforcement and were loaded concentrically. They note that the ACI equation for spiral reinforcement, which is the same as Eq. 5.7.4.6-1 is based on the premise that the increment in column capacity provided by the spiral should at least equal the capacity lost when the cover spalls. A similar derivation based on their test results for lightweight concrete would result in the following equation for lightweight concrete columns:

$$\rho_s = 1.13\left(\frac{A_g}{A_c}-1\right)\frac{f_c'}{f_{yh}}$$

A graph of axial strain at maximum stress versus concrete compressive strength is shown in Fig. 5.7.4.6-A for tests by Martinez et al. Most specimens had measured strains in excess of 0.003 assumed for unconfined concrete.

Ahmad and Shah (1982) developed stress-strain curves for two confined lightweight concretes and concluded that the use of lightweight aggregate decreased the effectiveness of spiral reinforcement. Measured strains at the peak of the stress-strain curves ranged from 0.0027 to 0.0035 and are included in Fig. 5.7.4.6-A.

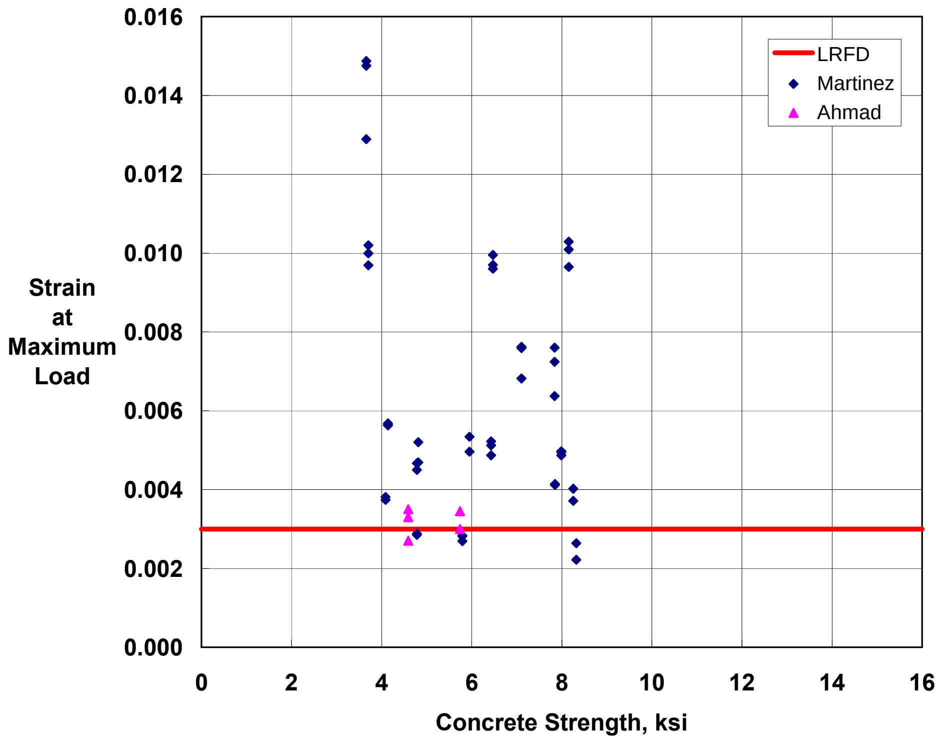

Figure 5.7.4.6-A. Variation of strain at maximum load with concrete compressive strength

5.7.4.7 Hollow Rectangular Compression Members
5.7.4.7.2 Limitations on the Use of the Rectangular Stress Block Method
5.7.4.7.2a General
Where the wall slenderness ratio is less than 15, the rectangular stress block method may be used based on a compressive strain of 0.003.

5.7.4.7.2b Refined Method for Adjusting Maximum Usable Strain Limit
Where the wall slenderness ratio is 15 or greater, the maximum usable strain at the extreme concrete compression fiber is equal to the lesser of the computed local buckling strain of the widest flange of the cross-section, or 0.003.

5.7.4.7.2c Approximate Method for Adjusting Factored Resistance
The factored resistance of a hollow column, determined using a maximum usable strain of 0.003, and the resistance factors specified in Article 5.5.4.2 shall be further reduced by a factor ϕ_w taken as:

- If $\lambda_w \leq 15$ then $\phi_w = 1.0$ (5.7.4.7.2c-1)

- If $15 < \lambda_w \leq 25$, then $\phi_w = 1 - 0.025(\lambda_w - 15)$ (5.7.4.7.2c-2)
- If $25 < \lambda_w \leq 35$, then $\phi_w = 0.75$ (5.7.4.7.2c-3)

RELEVANT BACKGROUND AND RESEARCH

No research addressing the use of the rectangular stress block in hollow rectangular compression members made of lightweight concrete was identified.

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:

$$P_r = \phi P_n \qquad (5.7.5\text{-}1)$$

in which:

$$P_n = 0.85 f_c' A_1 m \qquad (5.7.5\text{-}2)$$

where:

P_n = nominal bearing resistance (kip)
A_1 = area under bearing device (in.^{2})
m = modification factor
A_2 = a notional area defined herein (in.^{2})

RELEVANT BACKGROUND AND RESEARCH

No research addressing bearing resistance of lightweight concrete was identified. However, research about the α_1 factor in Article 5.7.2.2 is relevant because the same 0.85 factor appears in Eq. 5.7.5-2.

ARTICLE 5.8 SHEAR AND TORSION

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: $4.7 f_{ct} \leq \sqrt{f'_c}$

- Where f_{ct} is not 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.

C5.8.2.2
The tensile strength and shear capacity of lightweight concrete is typically somewhat less than that of normal weight concrete having the same compressive strength.

RELEVANT BACKGROUND AND RESEARCH

The opening sentence in Article 5.8.2.2 states that the modifications shall apply in determining resistance to torsion and shear. Yet the two bulleted items limit the modifications to Articles 5.8.2 and 5.8.3. Other articles dealing with shear that involve $\sqrt{f'_c}$ are 5.8.5, 5.8.6, 5.13.2.5, 5.10.4.3.1, 5.10.11.4, 5.13.3.6, and 5.14.5.3 but these are excluded from the modifications.

The use of f_{ct} in place of $\sqrt{f'_c}$ is based on the research by Hanson (1961), who developed a correlation between the shear cracking strength of beams and the splitting tensile strength of cylinders.

The origin of 0.75 and 0.85 factors listed with the second bullet appear to have been developed by ACI Committee 213 Lightweight Concrete and recommended to ACI Committee 318 (Ivey and Buth, 1967). A waiver to allow higher shear stresses when justified by splitting tensile tests was also included. Ivey and Buth (1967) compared the results of tests at the Texas

Transportation Institute, Portland Cement Association, and the University of Texas with the ACI shear strength equations modified by the 0.75 and 0.85 factors and found reasonable conservatism. Although the shear design approach has changed from that used when the factors were developed, the modification factors remain the same.

Data on the measured splitting tensile strength from 14 reports are shown in Fig. 5.8.2.2-A. (Hanson, 1961, 1965, 1968; Heffington, 2000; Hoff, 1992; Ivey and Buth, 1966; Khaloo and Nakseok, 1999; Malhotra, 1990; Mattock et al., 1976a; Ozyildirim and Gomez, 2005; Pfeifer, 1967; Ramirez et al., 2000, 2004; and Vincent et al., 2004) This figure is the same as Fig. 5.4.2.7-A with the addition of the lines representing the above multipliers for lightweight concrete. It is assumed that the line labeled "LRFD at 0.23" applies to normal weight concrete. The LRFD lines for lightweight concrete tend to overestimate the measured strengths.

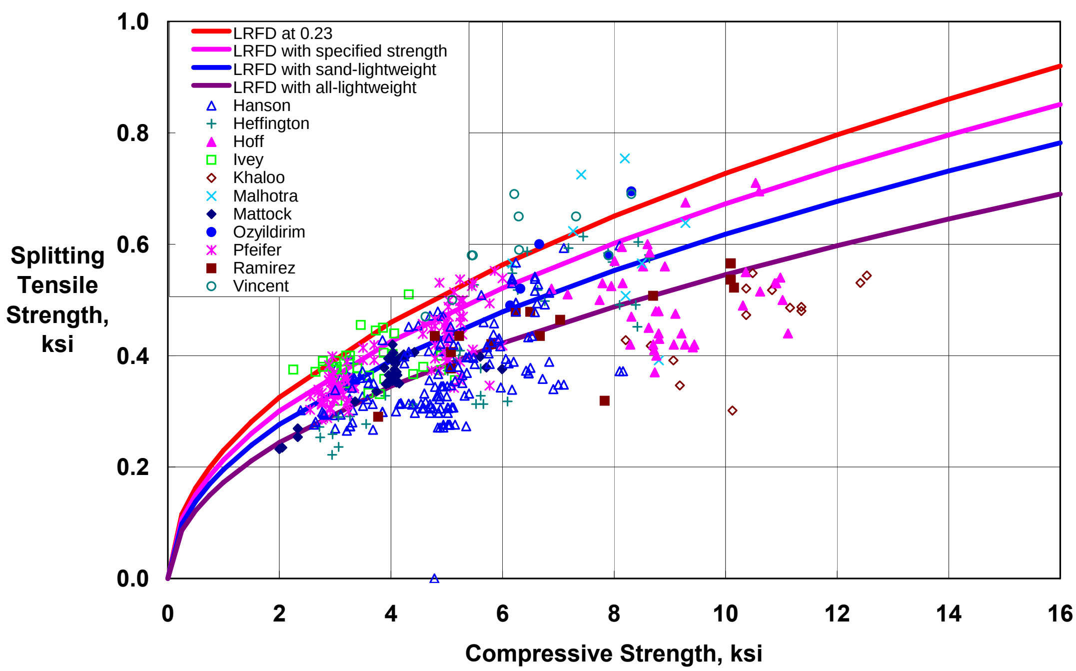

Figure 5.8.2.2-A. Splitting tensile strength versus concrete compressive strength

Moore (1982) compared the shear strength and response of short columns made with lightweight and normal weight concrete subject to cyclic loading. He concluded that for columns with no axial load, the 15 percent reduction for shear specified in Chapter 11 of ACI 318-77 (ACI Committee 318, 1977) for lightweight concrete was adequate but for columns with axial compression, the 15 percent reduction was not adequate. The 15 percent reduction refers to the use of $0.85\sqrt{f_c^{'}}$ for sand-lightweight concrete in Article 5.8.2.2.

5.8.2.5 Minimum Transverse Reinforcement

Except for segmental post-tensioned concrete box girder bridges, where transverse reinforcement is required, as specified in Article 5.8.2.4, the area of steel shall satisfy:

$$A_v \geq 0.0316\sqrt{f_c^{'}}\frac{b_v s}{f_y} \qquad (5.8.2.5\text{-}1)$$

where:

A_v = area of a transverse reinforcement within distance s (in.^{2})
b_v = width of web adjusted for the presence of ducts as specified in Article 5.8.2.9 (in.)
s = spacing of transverse reinforcement (in.)
f_y = yield strength of transverse reinforcement (ksi)

5.8.2.7 Maximum Spacing of Transverse Reinforcement

The spacing of the transverse reinforcement shall not exceed the maximum permitted spacing, s_{max}, determined as:

- If $v_u < 0.125 f'_c$, then:

  $$s_{max} = 0.8d_v \leq 24.0 \text{ in.} \qquad (5.8.2.7\text{-}1)$$

- If $v_u \geq 0.125 f'_c$, then:

  $$s_{max} = 0.4\, d_v \leq 12.0 \text{ in.} \qquad (5.8.2.7\text{-}2)$$

RELEVANT BACKGROUND AND RESEARCH

A minimum area of transverse reinforcement is specified to prevent shear failures when inclined cracking occurs. The modification factors of Article 5.8.2.2 result in a requirement for more minimum transverse reinforcement with lightweight concrete.

A maximum spacing of transverse reinforcement is specified to ensure that diagonal cracks will be intersected by the transverse reinforcement. No modification is required for lightweight concrete.

No research specifically addressing the maximum and minimum transverse reinforcement or maximum spacing of transverse reinforcement for lightweight concrete members was identified.

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:

$$V_n = V_c + V_s + V_p \qquad (5.8.3.3\text{-}1)$$

$$V_n = 0.25 f'_c\, b_v\, d_v + V_p \qquad (5.8.3.3\text{-}2)$$

in which:

$$V_c = 0.0316\beta\sqrt{f'_c}\, b_v d_v \qquad (5.8.3.3\text{-}3)$$

$$V_s = \frac{A_v f_y d_v (\cot\theta + \cot\alpha)\sin\alpha}{s} \qquad (5.8.3.3\text{-}4)$$

where:

b_v = effective web width taken as the minimum web width within the depth d_v as determined in Article 5.8.2.9 (in.)
d_v = effective shear depth as determined in Article 5.8.2.9 (in.)
s = spacing of stirrups (in.)
β = factor indicating ability of diagonally cracked concrete to transmit tension as specified in Article 5.8.3.4
θ = angle of inclination of diagonal compressive stresses as determined in Article 5.8.3.4 (°)
α = angle of inclination of transverse reinforcement to longitudinal axis (°)
A_v = area of shear reinforcement within a distance s (in.^{2})
V_p = component in the direction of the applied shear of the effective prestressing force; positive if resisting the applied shear (kip)

5.8.3.4 Determination of β and θ

5.8.3.4.2 General Procedure

C5.8.3.4.2

The values of β and θ listed in Table 1 and Table 2 are based on calculating the stresses that can be transmitted across diagonally cracked concrete. As the cracks become wider, the stress that can be transmitted decreases. For members containing at least the minimum amount of transverse reinforcement, it is assumed that the diagonal cracks will be spaced about 12.0 in. apart. For members without transverse reinforcement, the spacing of diagonal cracks inclined at θ° to the longitudinal reinforcement is assumed to be $s_x/\sin\theta$, as shown in Figure 3. Hence, deeper members having larger values of s_x are calculated to have more widely spaced cracks and hence, cannot transmit such high shear stresses. The ability of the crack surfaces to transmit shear stresses is influenced by the aggregate size of the concrete. Members made from concretes that have a smaller maximum aggregate size will have a larger value of s_{xe} and hence, if there is no transverse reinforcement, will have a smaller shear strength.

RELEVANT BACKGROUND AND RESEARCH

The shear design provisions of the LRFD Specifications have been evolving since their introduction in the first edition. The fourth edition (AASHTO, 2007) now includes a new simplified procedure for prestressed and nonprestressed sections in addition to the general procedure (Article 5.8.3.4.2) and the simplified procedure for nonprestressed sections (Article 5.8.3.4.1). As a result of NCHRP Project 12-56, some additional changes may also occur.

Salandra and Ahmad (1989) tested eight reinforced lightweight concrete beams with shear reinforcement but only two failed due to diagonal tension cracking with the rest failing in flexure due to crushing of concrete in the constant moment region. Ramirez et al. (2004) tested five reinforced and four prestressed lightweight concrete beams. Measured strengths were compared with strengths calculated using the general and simplified methods of the LRFD Specifications (AASHTO, 1998) through the 2001 Interim Revisions. Meyer (2002) tested six prestressed lightweight concrete beams that failed primarily due to shear. Measured strengths were compared with strengths calculated using the 1998 LRFD Specifications (AASHTO, 1998). Meyer (2002) concluded that the 1998 Specifications provided a conservative prediction of shear strength.

A comparison of the ratio of measured to calculated strengths versus concrete compressive strength for the tests by Meyer (2002) and Ramirez et al. (2004) is shown in Fig. 5.8.3.3-A. All measured strengths were greater than the calculated strengths. Although measured shear capacities exceeded calculated values, Ramirez et al (2000) cautioned that the degree of conservatism was less with high-strength lightweight concrete. They recommended more research in the area of high-strength prestressed lightweight concrete beams especially with regard to the requirements for minimum transverse reinforcement.

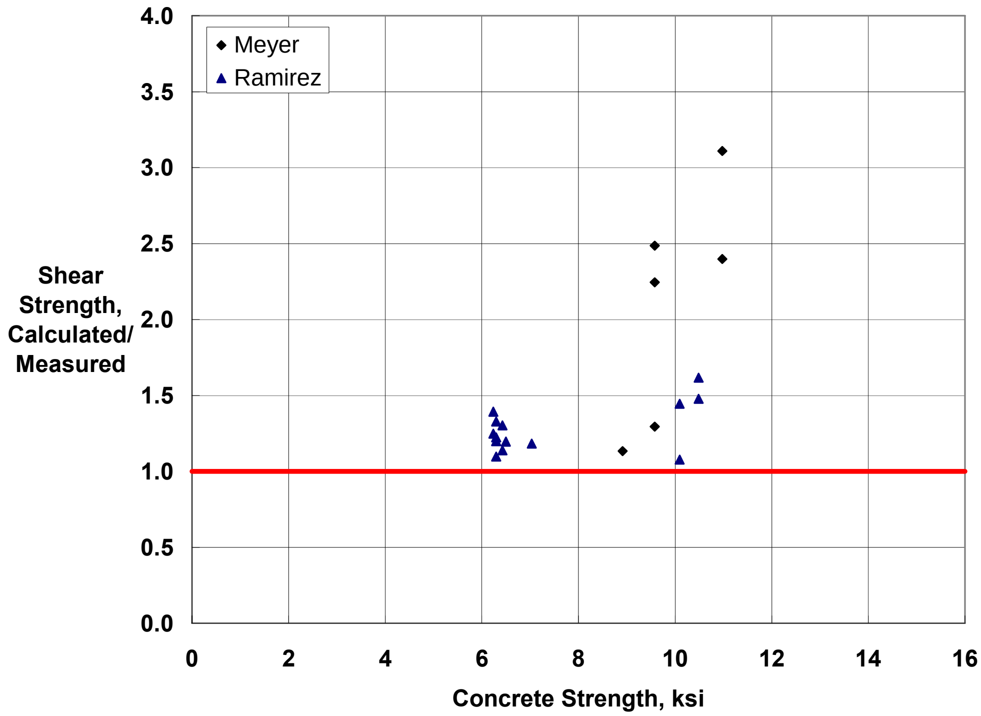

Figure 5.8.3.3-A. Comparison of the ratio of measured to calculated shear strengths with concrete compressive strengths

The following articles are taken from the 4th Edition of the LRFD Specifications (AASHTO, 2007) because extensive changes were made and these affect the use of lightweight concrete.

5.8.4 Interface Shear Transfer—Shear Friction
5.8.4.1 General

The nominal shear resistance of the interface plane shall be taken as:

$$V_{ni} = cA_{cv} = \mu\left(A_{vf} f_y + P_c\right) \qquad (5.8.4.1\text{-}5)$$

The nominal shear resistance, V_{ni}, used in the design shall not be greater than the lesser of:

$$V_{ni} \leq K_1 f'_c A_{cv}, \text{ or} \qquad (5.8.4.1\text{-}4)$$

$$V_{ni} \leq K_2 A_{cv} \qquad (5.8.4.1\text{-}5)$$

in which:

A_{cv} = $b_{vi}\, L_{vi}$ (5.8.4.1-6)

where:

A_{cv} = area of concrete considered to be engaged in interface shear transfer (in.^{2})
A_{vf} = area of interface shear reinforcement crossing the shear plane within the area A_{cv} (in.^{2})
b_{vi} = interface width considered to be engaged in shear transfer (in.)
L_{vi} = interface length considered to be engaged in shear transfer (in.)
c = cohesion factor specified in Article 5.8.4.3 (ksi)
μ = friction factor specified in Article 5.8.4.3 (dim)
f_y = yield stress of reinforcement but design value not to exceed 60 (ksi)
P_c = permanent net compressive force normal to the shear plane; if force is tensile, P_c = 0.0 (kip)
f'_c = specified 28-day compressive strength of the weaker concrete on either side of the interface (ksi)
K_1 = fraction of concrete strength available to resist interface shear, as specified in Article 5.8.4.3
K_2 = limiting interface shear resistance in Article 5.8.4.3 (ksi)

C5.8.4.1
The interface shear strength Eqs. 3, 4 and 5 are based on experimental data for normal weight, nonmonolithic concrete strengths ranging from 2.5 ksi to 16.5 ksi; normal weight, monolithic concrete strengths from 3.5 ksi to 18.0 ksi; sand-lightweight concrete strengths from 2.0 ksi to 6.0 ksi; and all-lightweight concrete strengths from 4.0 ksi to 5.2 ksi.

5.8.4.3 Cohesion and Friction Factors
The following values shall be taken for cohesion, c, and friction factor, μ:

- For a cast-in-place concrete slab on clean concrete girder surfaces, free of laitance with surface roughened to an amplitude of 0.25 in.

  c = 0.28 ksi
  μ = 1.0
  K_1 = 0.3
  K_2 = 1.8 ksi for normal-weight concrete
  = 1.3 ksi for lightweight concrete

- For normal-weight concrete placed monolithically:

  c = 0.40 ksi
  μ = 1.4
  K_1 = 0.25
  K_2 = 1.5 ksi

- For lightweight concrete placed monolithically, or nonmonolithically, against a clean concrete surface, free of laitance with surface intentionally roughened to an amplitude of

0.25 in.:

c = 0.24 ksi
μ = 1.0
K_1 = 0.25
K_2 = 1.0 ksi

- For normal-weight concrete placed against a clean concrete surface, free of laitance, with surface intentionally roughened to an amplitude of 0.25 in.:

c = 0.24 ksi
μ = 1.0
K_1 = 0.25
K_2 = 1.5 ksi

- For concrete placed against a clean concrete surface, free of laitance, but not intentionally roughened:

c = 0.075 ksi
μ = 0.6
K_1 = 0.2
K_2 = 0.8 ksi

- 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 ksi
μ = 0.7
K_1 = 0.2
K_2 = 0.8 ksi

- For brackets, corbels, and ledges, the cohesion factor, c, shall be taken as 0.0.

C5.8.4.3
Available experimental data demonstrates that only one modification factor is necessary, when coupled with the resistance factors of Article 5.5.4.2, to accommodate both all-lightweight and sand-lightweight concrete. Note this deviates from earlier specifications that distinguished between all-lightweight and sand-lightweight concrete.

Due to the absence of existing data, the prescribed cohesion and friction factors for nonmonolithic lightweight concrete are accepted as conservative for application to monolithic lightweight concrete.

RELEVANT BACKGROUND AND RESEARCH

Equations 5.8.4.1-4 and 5.8.4.1-5 taken in combination mean that the highest value of concrete compressive strength that can be used in design is given by

$$f'_c = K_2/K_1$$

For cast-in-place lightweight concrete, f'_c = 4.33 ksi

For all other situations, f'_c = 4.00 ksi.

Given the lack of data for higher strength lightweight concretes as indicated in C5.8.4.1, this limit is probably appropriate and conservative. The same two equations also restrict the maximum compressive strength that can be used for design with normal weight concrete.

Hoff (1992) reported the results of 18 tests of precracked specimens with three different lightweight concretes having compressive strengths between 8.3 and 11.0 ksi. Test results were compared with values predicted using ACI 318-89 (ACI Committee 318, 1989). Hoff concluded that the shear friction capacity for two of the concretes can be predicted by the code provisions but designs should be approached with caution. For the third concrete, the measured strengths exceeded the calculated values. The ACI provisions for lightweight concrete were based on research by Mattock et al. (1976a) and are different from the new provisions of the LRFD Specifications.

5.8.5 Principal Stresses in Webs of Segmental Concrete Bridges
The provisions specified herein shall apply to all types of segmental bridges with internal and/or external tendons.

The principal tensile stress resulting from the long-term residual axial stress and maximum shear and/or maximum shear combined with shear from torsion stress at the neutral axis of the critical web shall not exceed the tensile stress limit of Table 5.9.4.2.2-1 at the Service III limit state of Article 3.4.1 at all stages during the life of the structure, excluding those during construction. When investigating principal stresses during construction, the tensile stress limits of Table 5.14.2.3.3-1 shall apply.

RELEVANT BACKGROUND AND RESEARCH

The first paragraph of this article states that the provisions shall apply to all types of segmental bridges. This can be interpreted to mean that they apply to lightweight concrete. The principal tensile stresses at Service III are limited to $0.110\sqrt{f_c^{'}}$ ksi per Table 5.9.4.2.2.1. This stress is the same for both normal weight and lightweight concrete.

The principal tensile stresses during construction are limited to either $0.110\sqrt{f_c^{'}}$ or $0.126\sqrt{f_c^{'}}$ per Table 5.14.2.3.3-1. These limits do not differentiate between normal and lightweight concretes. However, at the beginning of Article 5.14.2.1, it is stated that the provisions shall only apply to segmental construction using normal weight concrete. The modification factors for lightweight concrete given in Article 5.8.2.2 only apply to Articles 5.8.2 and 5.8.3 and not Article 5.8.5.

No research addressing principal tensile stresses in segmental lightweight concrete bridges was identified.

5.8.6 Shear and Torsion for Segmental Box Girder Bridges
5.8.6.3 Regions Requiring Consideration of Torsional Effects
For normal weight concrete, torsional effects shall be investigated where:

5.8.6.5 Nominal Shear Resistance
In lieu of the provisions of Article 5.8.3, the provisions herein shall be used to determine the nominal shear resistance of post-tensioned concrete box girders in regions where it is reasonable to assume that plane sections remain plane after loading.

The nominal shear resistance, V_n, shall be determined as the lesser of:

$$V_n = V_c + V_s \qquad (5.8.6.5\text{-}1)$$

$$V_n = 0.379\sqrt{f_c^{'}}b_v d_v \qquad (5.8.6.5\text{-}2)$$

and, where the effects of torsion are required to be considered by Article 5.8.6.2, the cross-sectional dimensions shall be such that:

$$\left(\frac{V_u}{b_v d_v}\right)+\left(\frac{T_u}{2A_o b_e}\right)\leq 0.474\sqrt{f_c^{'}} \qquad (5.8.6.5\text{-}3)$$

in which:

$$V_c = 0.0316\ K\sqrt{f_c^{'}}\, b_v\, d_v \qquad (5.8.6.5\text{-}4)$$

$$V_s = \frac{A_v f_y d_v}{s} \qquad (5.8.6.5\text{-}5)$$

where:

b_v = effective web width taken as the minimum web width within the depth d_v as determined in Article 5.8.6.1 (in.)
d_v = $0.8h$ or the distance from the extreme compression fiber to the centroid of the prestressing reinforcement, whichever is greater (in.)
s = spacing of stirrups (in.)
K = stress variable computed in accordance with Article 5.8.6.3.
A_v = area of shear reinforcement within a distance s (in.^{2})
V_u = factored design shear including any normal component from the primary prestressing force (kip)
T_u = applied factored torsional moment (kip-in.)
A_o = area enclosed by shear flow path, including area of holes, if any (in.^{2})
b_e = the effective thickness of the shear flow path of the elements making up the space truss model resisting torsion calculated in accordance with Article 5.8.6.3 (in.)
ϕ = resistance factor for shear specified in Article 5.5.4.2

5.8.6.6 Reinforcement Details

The spacing of the transverse reinforcement shall not exceed the maximum permitted spacing, s_{max}, determined as:

- If $v_u < 0.19\sqrt{f_c^{'}}$, then:

  $s_{max} = 0.8d \leq 36.0$ in. (5.8.6.6-1)

- If $v_u \geq 0.19\sqrt{f_c^{'}}$, then:

  $s_{max} = 0.4d \leq 18.0$ in. (5.8.6.6-2)

where:

v_u = the shear stress calculated in accordance with Eq. 5.8.6.5-3 (ksi)
d_v = effective shear depth as defined in Article 5.8.6.5 (in.)

RELEVANT BACKGROUND AND RESEARCH

Article 5.8.6 contains provisions for shear and torsion in segmental box girder bridges. Article 5.8.6.3 defines when torsional effects shall be investigated for normal weight concrete. No similar article is provided for lightweight concrete.

Article 5.8.6.5 contains provisions to be used to calculate the shear resistance of post-tensioned concrete box girders instead of using the sectional design model of Article 5.8.3. Although not explicitly stated, it needs to be assumed that the modifications for lightweight concrete of Article 5.8.2.2 would apply to Article 5.8.6.5 if used to determine the shear resistance with lightweight concrete. However, Article 5.14.2.1 and Commentary C5.14.2.1 discourage the use of lightweight concrete in segmental construction.

Article 5.8.6.6 is confusing because Eqs. 5.8.6.6-1 and 5.8.6.6-2 use d whereas the definition is d_v.

No specific research addressing shear and torsion in lightweight concrete segmental box girders was identified.

ARTICLE 5.9 PRESTRESSING AND PARTIAL PRESTRESSING

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).

5.9.4.1.2 Tension Stresses
The limits in Table 1 shall apply for tensile stresses.

5.9.4.2 For Stresses at Service Limit State After Losses—Fully Prestressed Components
5.9.4.2.1 Compression Stresses
Compression shall be investigated using the Service Limit State Load Combination I specified in Table 3.4.1-1. The limits in Table 1 shall apply.

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.

RELEVANT BACKGROUND AND RESEARCH

The tensile stress limits in Tables 5.9.4.1.2-1 and 5.9.4.2.2-1 are given as functions of the concrete compressive strength and do not differentiate between normal weight and lightweight concrete. Yet, Article 5.4.2.6 provides different values for modulus of rupture of sand-lightweight and all-lightweight concretes, and Article 5.5.2 states that the cracking stress shall be taken as the modulus of rupture specified in Article 5.4.2.6. The data provided in the discussion of Article 5.4.2.7 indicates that the splitting tensile strength is also less for lightweight concrete than normal weight concrete.

No research specifically addressing the stress limits for lightweight concrete was identified.

5.9.5 Loss of Prestress
5.9.5.1 Total Loss of Prestress
Values of prestress losses specified herein shall be applicable for specified concrete strengths up to 15.0 ksi.

C5.9.5.1
For segmental construction, lightweight concrete construction, multi-stage prestressing, and bridges where more exact evaluation of prestress losses is desired, calculations for loss of prestress should be made in accordance with a time-step method supported by proven research data. See references cited in Article C5.4.2.3.2.

5.9.5.3 Approximate Estimate of Time-Dependent Losses
For members other than those made with composite slabs, stressed after attaining a compression strength of 3.5 ksi, an approximate 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 members made from structural lightweight concrete, the values specified in Table 1 shall be increased by 5.0 ksi.

5.9.5.4 Refined Estimates of Time-Dependent Losses
5.9.5.4.1 General
For concrete containing lightweight aggregates, very hard aggregates, or unusual chemical admixtures, the estimated material properties used in this article and Article 5.4.2.3 may be inaccurate. Actual test results should be used for their estimation.

5.9.5.5 Losses For Deflection Calculations
For camber and deflection calculations of prestressed nonsegmental members made of normal weight concrete with a strength in excess of 3.5 ksi at the time of prestress, f_{cgp} and Δf_{cdp} may be computed as the stress at the center of gravity of prestressing steel averaged along the length of the member.

RELEVANT BACKGROUND AND RESEARCH

The provisions of Article 5.9.5 for prestress losses were revised to a great extent based on NCHRP Project 18-07 (Tadros et al., 2003), which only investigated normal weight concrete. The commentary C5.9.5.1 clarifies that for lightweight concrete construction, an alternative method should be used. However, Article 5.9.5.3 allows the use of the losses in Table 5.9.5.3-1 for lightweight concrete members other than those with composite slabs. A proposed revision to provide a more consistent approach has been submitted to AASHTO Technical Committee T-10, Concrete Design.

Hanson (1964b) measured the effect of type of curing on prestress losses of concretes made using two different lightweight aggregates. Prestressed concrete members were simulated using short post-tensioned members of two different sizes. Companion creep and shrinkage tests were made on 6x12-in. cylinders.

Cousins (2005) reported on the measurement of prestress losses in three lightweight concrete girders of the Chickahominy River Bridge, Virginia. The bridge is a three-span structure made continuous for live load with two end spans of 81 ft 10 in. and a center span of 82 ft 10 in. Each span consists of five AASHTO Type IV girders at 10 ft centers with an 8.5-in. thick lightweight concrete deck.

Measured values of prestress losses were compared with those predicted using the procedures of the LRFD Specifications, NCHRP Report No. 469 (Tadros et al., 2003), and several other methods. Cousins (2005) concluded that the refined and approximate methods of the NCHRP report were suitable for a conservative estimate of total losses.

Meyer (2002) monitored the prestress losses at three locations in each of six girders on beams being used to determine transfer and development lengths. Measured values were compared with calculated losses using the AASHTO Standard Specifications (AASHTO, 1996). All measured values were less than the calculated values as shown in Figure 5.9.5-A.

Kahn et al. (2005) reported prestress losses in four girders using two different concrete strengths. They compared the measured values with calculated values using the refined method and the lump sum methods of the LRFD Specifications (AASHTO 1998). Their results are included in Fig. 5.9.5-A. In general, the refined method overestimated the losses, whereas the lump sum losses underestimated the losses for one of the mixes.

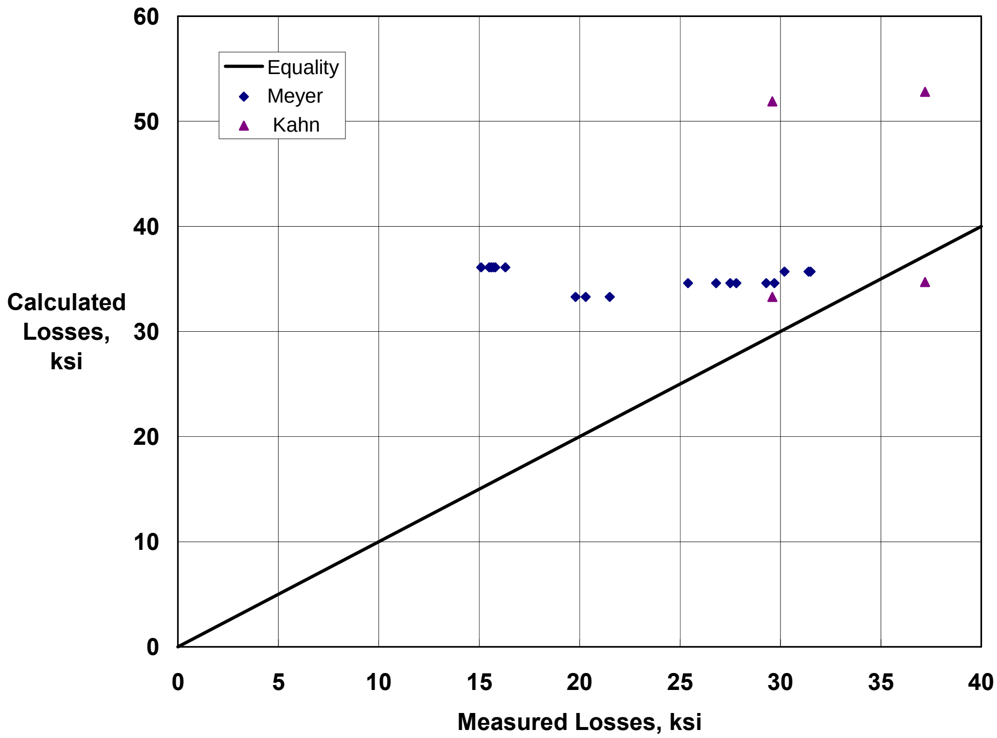

Figure 5.9.5-A. Comparison of calculated and measured prestress losses

ARTICLE 5.10 DETAILS OF REINFORCEMENT

5.10 DETAILS OF REINFORCEMENT
5.10.2 Hooks and Bends
5.10.2.1 Standard Hooks
For the purpose of these Specifications, the term "standard hook" shall mean one of the following:

- For longitudinal reinforcement:

(a) 180°-bend, plus a 4.0 d_b extension, but not less than 2.5 in. at the free end of the bar, or
(b) 90°-bend, plus a 12.0 d_b extension at the free end of the bar.

- For transverse reinforcement:

(a) No. 5 bar and smaller—90°-bend, plus a 6.0 d_b extension at the free end of the bar,
(b) No. 6, No. 7 and No. 8 bars—90°-bend, plus a 12.0 d_b extension at the free end of the bar; and
(c) No. 8 bar and smaller—135°-bend, plus a 6.0 d_b extension at the free end of the bar.

where:

d_b = nominal diameter of reinforcing bar (in.)

RELEVANT BACKGROUND AND RESEARCH

No research addressing the definition of the "standard hook" in lightweight concrete was identified.

5.10.4 Tendon Confinement
5.10.4.3 Effects of Curved Tendons
5.10.4.3.1 In-Plane Force Effects
The shear resistance of the concrete cover against pull-out by deviation forces, V_r, shall be taken as:

$$V_r = \phi V_n \qquad (5.10.4.3.1\text{-}2)$$

in which:

$$V_n = 0.125 d_c \sqrt{f'_{ci}} \qquad (5.10.4.3.1\text{-}3)$$

where:

V_n = nominal shear resistance of two shear planes per unit length (kips/in.)
ϕ = resistance factor for shear specified in Article 5.5.4.2
d_c = minimum concrete cover over the tendon duct, plus one-half of the duct diameter (in.)
f'_{ci} = specified compressive strength of concrete at time of initial loading or prestressing (ksi)

RELEVANT BACKGROUND AND RESEARCH

Article 5.5.4.2 specifies a resistance factor of 0.70 for shear and torsion with lightweight concrete compared to 0.90 for normal weight concrete. This accounts for the lower tensile strength of lightweight concrete and results in a lower value for V_r in Eq. 5.10.4.3.1-2.

No research specifically dealing with in-plane force effects with lightweight concrete was identified.

5.10.6 Transverse Reinforcement for Compression Members
5.10.6.2 Spirals
Anchorage of spiral reinforcement shall be provided by 1.5 extra turns of spiral bar or wire at each end of the spiral unit. For Seismic Zones 3 and 4, the extension of transverse reinforcement into connecting members shall meet the requirements of Article 5.10.11.4.3.

Splices in spiral reinforcement may be one of the following:

- Lap splices of 48.0 uncoated bar diameters, 72.0 coated bar diameters, or 48.0 wire diameters;

RELEVANT BACKGROUND AND RESEARCH

No research addressing anchorage and lap splices of spirals in lightweight concrete was identified.

5.10.8 Shrinkage and Temperature Reinforcement
For bars or welded wire fabric, the area of reinforcement per foot, on each face and in each direction**,** shall satisfy:

$$A_s \geq \frac{1.30bh}{2(b+h)f_y} \qquad (5.10.8\text{-}1)$$

$$0.11 \leq A_s \leq 0.60 \qquad (5.10.8\text{-}2)$$

where:

A_s = area of reinforcement in each direction and each face (in.^{2}/ft.)
b = least width of component section (in.)
h = least thickness of component section (in.)
f_y = specified yield strength of reinforcing bars ≤75 ksi
f_y = specified yield strength of reinforcing bars (ksi)

RELEVANT BACKGROUND AND RESEARCH

Equations 5.10.8-1 and 5.10.8-2 were introduced in the 2006 Interim Revisions of the LRFD Specifications to provide a more uniform approach for components of any size. The relevant research for Article 5.4.2.3.3 indicates that shrinkage of lightweight concrete is similar to that for normal weight concrete. Based on this, it seems that Eq. 5.10.8-1 and 5.10.8-2 would be equally applicable to normal weight and lightweight concrete.

No research specifically addressing the shrinkage and temperature reinforcement in lightweight concrete was identified.

5.10.9 Post Tensioned Anchorage Zones

Provisions intentionally not included.

RELEVANT BACKGROUND AND RESEARCH

Article 5.10.9 contains extensive provisions based on NCHRP Report 356 (Breen et al., 1994). The provisions were originally developed for the Standard Specifications and modified for the LRFD Specifications. Thirty-one test specimens were used to evaluate the behavior, test criteria, and design procedures for the local zone. From Report 356, it is not possible to tell if lightweight concrete was included in the test program. The report did, however, include recommendations that the resistance factors for anchorage zones be 0.85 for normal weight concrete and 0.70 for lightweight concrete. Article 5.5.4.2 specifies a resistance factor for compression in anchorage zones of 0.80 for normal weight concrete and 0.65 for lightweight concrete.

No other research addressing post-tensioned anchorage zones with lightweight concrete was identified.

5.10.10 Pretensioned Anchorage Zones

5.10.10.1 Factored Bursting Resistance

The bursting resistance of pretensioned anchorage zones provided by vertical reinforcement in the ends of pretensioned beams at the service limit state shall be taken as:

$$P_r = f_s A_s \qquad (5.10.10.1\text{-}1)$$

where:

f_s = stress in steel not exceeding 20 ksi
A_s = total area of vertical reinforcement located within the distance $h/4$ from the end of the beam (in.^{2})
h = overall depth of precast member (in.)

The resistance shall not be less than 4 percent of the prestressing force at transfer.

5.10.10.2 Confinement Reinforcement

For the distance of 1.5*d* from the end of the beams other than box beams, reinforcement shall be placed to confine the prestressing steel in the bottom flange. The reinforcement shall not be less than No. 3 deformed bars, with spacing not exceeding 6.0 in. and shaped to enclose the strands. For box beams, transverse reinforcement shall be provided and anchored by extending the leg of stirrup into the web of the girder.

RELEVANT BACKGROUND AND RESEARCH

No research on the requirements for reinforcement in the pretensioned anchorage zones of lightweight concrete beams was identified.

5.10.11 Provisions for Seismic Design
5.10.11.4 Seismic Zones 3 and 4
5.10.11.4.1 Column Requirements
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 satisfy either that required in Article 5.7.4.6 or:

$$\rho_s \geq 0.12 \frac{f_c'}{f_y} \qquad (5.10.11.4.1d\text{-}1)$$

where:

f'_c = specified compressive strength of concrete at 28 days, unless another age is specified (ksi)
f_y = yield strength of reinforcing bars (ksi)

For a rectangular column, the total gross sectional area, A_{sh}, of rectangular hoop reinforcement shall satisfy either:

$$A_{sh} \geq 0.30 s h_c \frac{f_c'}{f_y} \left[\frac{A_g}{A_c} - 1 \right] \qquad (5.10.11.4.1d\text{-}2)$$

or

$$A_{sh} \geq 0.12 s h_c \frac{f_c'}{f_y} \qquad (5.10.11.4.1d\text{-}3)$$

where:

s = vertical spacing of hoops, not exceeding 4.0 in. (in.)
A_c = area of column core (in.^{2})
A_g = gross area of column (in.^{2})
A_{sh} = total cross-sectional area of tie reinforcement, including supplementary cross-ties having a vertical spacing of s and crossing a section having a core dimension of h_c (in.^{2})
f_y = yield strength of tie or spiral reinforcement (ksi)
h_c = core dimension of tied column in the direction under consideration (in.)

5.10.11.4.2 Requirements for Wall-Type Piers

The factored shear resistance, V_r, in the pier shall be take as the lesser of:

$$V_r = 0.253\sqrt{f_c^{'}}bd \text{, and} \qquad (5.10.11.4.2\text{-}1)$$

$$V_r = \phi V_n \qquad (5.10.11.4.2\text{-}2)$$

in which:

$$V_n = \left[0.063\sqrt{f_c^{'}} + \rho_h f_y\right]bd \qquad (5.10.11.4.2 - 3)$$

5.10.11.4.3 Column Connections

The nominal shear resistance, V_n, provided by the concrete in the joint of a frame or bent in the direction under consideration, shall satisfy:

For lightweight aggregate concrete:

$$V_n \leq 0.285\, bd\sqrt{f_c^{'}} \qquad (5.10.11.4.3\text{-}2)$$

C5.10.11.4.3

The factored shear resistance for joints made with lightweight aggregate concrete has been based on the observation that shear transfer in such concrete has been measured to be approximately 75 percent of that in normal weight aggregate concrete.

5.10.11.4.4 Construction Joints in Piers and Columns

Where shear is resisted at a construction joint solely by dowel action and friction on a roughened concrete surface, the nominal shear resistance across the joint, V_n, shall be taken as:

$$V_n = (A_{vf} f_y + 0.75\, P_u) \qquad (5.10.11.4.4\text{-}1)$$

where:

A_{vf} = the total area of reinforcement, including flexural reinforcement (in.^{2})

P_u = the minimum factored axial load as specified in Article 3.10.9.4 for columns and piers (kip)

C5.10.11.4.4

Eq. 1 is based on Eq. 11-26 of ACI 378-89 but is restated to reflect dowel action and frictional resistance.

Author's note: ACI 378-89 should be ACI 318-89.

RELEVANT BACKGROUND AND RESEARCH

According to the commentary at the beginning of Article 5.10.11, the specifications are based on the work by the Applied Technology Council in 1979-1980 and insights from the Loma Prieta earthquake of 1989. The commentary also indicates that the California Department of Transportation (Caltrans) has a number of research projects underway. This same statement appeared in the first edition of the Design Specifications published in 1994 although some changes to the Specifications have been made since 1994.

An additional literature search is needed to identify any reports from the Caltrans research and to determine if lightweight concrete was included.

Ghosh et al. (1992) conducted reversed cyclic loading tests of two sets of beams with compressive strengths of 5 ksi at 28 days and 9 ksi at 56 days. Stable hysteretic behavior was obtained up to the limiting stroke of the testing machines.

ARTICLE 5.11 DEVELOPMENT AND SPLICES OF REINFORCEMENT

5.11 DEVELOPMENT AND SPLICES OF REINFORCEMENT
5.11.2 Development of Reinforcement
5.11.2.1 Deformed Bars and Deformed Wire in Tension
5.11.2.1.2 Modification Factors That Increase ℓ_d
The basic development length, ℓ_{db}, shall be multiplied by the following factor or factors, as applicable:

- For lightweight aggregate concrete where f_{ct} (ksi) is specified…… $\dfrac{0.22\sqrt{f_c'}}{f_{ct}} \geq 1.0$
- 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.

RELEVANT BACKGROUND AND RESEARCH

In the 1989 edition of the ACI Building Code Requirements for Reinforced Concrete (ACI Committee 318, 1989), the factor for lightweight aggregate concretes was made equal to 1.3 for all types of aggregates when f_{ct} is not specified. According to the ACI 318-89 Commentary, research on hooked bar anchorages did not support the variations specified in previous codes for all-lightweight and sand-lightweight concrete. Similar changes were not made in the AASHTO LRFD Specifications. The ACI 318-89 commentary does not identify the specific research on which the changes were based or the research for the original factors that are used in Article 5.11.2.1.

Mitchell and Marzouk (2007) tested 72 pull-out and push-in specimens to evaluate the bond behavior under monotonic and cyclic loading using No. 8 and No. 11 deformed reinforcement embedded in lightweight concrete with a compressive strength of 11.6 ksi. They concluded that high-strength, lightweight concrete behaves in a manner similar to high-strength, normal weight concrete and that the 30 percent increase in development length required by ACI 318-05 (ACI Committee 318, 2005) is not justified for high-strength lightweight concrete. The 30 percent increase in ACI 318-05 corresponds to the 1.3 factor in Article 5.11.2.1.2.

According to ACI Committee 408 (2003) and ACI Committee 213 (2003), design provisions generally require longer development lengths for lightweight concrete although test results from previous research are contradictory. The report states that early research by Lyse (1934), Peterson (1948), and Shideler (1957), and more recent research by Martin (1982) and Clarke and Birjandi (1993) concluded that the bond behavior of reinforcing steel in lightweight concrete was comparable to that in normal weight concrete. In contrast, Baldwin (1965), Robins and Standish (1982), and Mor (1992) reported bond strengths in lightweight concrete that were less than bond strengths in normal weight concrete. Overall, the data indicate that the use of lightweight concrete can result in bond strengths that range from nearly equal to 65 percent of the values obtained with normal weight concrete (ACI Committee 408, 2003). Further discussion of the 1.3 factor is provided with Article 5.11.2.4.

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 8.0 in.

The basic development length, ℓ_{db}, for deformed bars in compression shall satisfy:

$$\lambda_{db} \geq \frac{0.63 d_b f_y}{\sqrt{f_c^{'}}} \text{, or} \qquad (5.11.2.2.1\text{-}1)$$

$$\lambda_{db} \geq 0.3 d_b f_y \qquad (5.11.2.2.1\text{-}2)$$

where:

f_y = specified yield strength of reinforcing bars (ksi)
f'_c = specified compressive strength of concrete at 28 days, unless another age is specified (ksi)
d_b = diameter of bar (in.)

RELEVANT BACKGROUND AND RESEARCH

Although modification factors for lightweight concrete are included for deformed bars in tension, there is no equivalent for deformed bars in compression.

No research addressing the development length of deformed bars in compression in lightweight concrete was identified.

5.11.2.4 Standard Hooks in Tension

5.11.2.4.2 Modification Factors

Basic hook development length, ℓ_{hb}, shall be multiplied by the following factor of factors, as applicable, where:

- Lightweight aggregate concrete is used……..……………………………………1.3

RELEVANT BACKGROUND AND RESEARCH

According to the Commentary in the ACI Building Code (ACI Committee 318, 1983b), the modification factors applied to the basic hook development length are based on recommendations by ACI Committee 408 (1979) and Jirsa et al. (1979). ACI Committee 408 (1979) recommended that the modification factor be 1.25 for all concretes containing lightweight aggregates. According to Jirsa et al. (1979), the increase of 25 percent was recommended as a simplification over the procedure in ACI 318-77 (ACI Committee 318, 1977), in which the increase varied from 18 to 33 percent depending on the amount of lightweight aggregate. Presumably, the modification factor of 1.3 used in 5.11.2.4 was also used in 5.11.2.1. In the report, ACI Committee 408 (1979) noted that research is needed in the area of lightweight aggregate to improve the understanding of the behavior of anchored bars under such conditions.

5.11.2.5 Welded Wire Fabric

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 in. 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:

$$\lambda_d = 8.50 \frac{A_w f_y}{s_w \sqrt{f_c^{'}}} \qquad (5.11.2.5.2\text{-}1)$$

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.

RELEVANT BACKGROUND AND RESEARCH

No research on the development length of welded wire fabric in lightweight concrete was identified.

5.11.4 Development of Prestressing Strand

5.11.4.1 General

For the purpose 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 section required to develop f_{ps} for a development length, ℓ_d, in in., where ℓ_d shall satisfy:

$$\lambda_d \geq \kappa\left(f_{ps} - \frac{2}{3} f_{pe}\right) d_b \qquad (5.11.4.2\text{-}1)$$

where:

d_b = nominal strand diameter (in.)
f_{ps} = average stress in prestressing steel at the time for which the nominal resistance of the member is required (ksi)
f_{pe} = effective stress in the prestressing steel after losses (ksi)
κ = 1.0 for pretensioned panels, piling, and other pretensioned members with a depth of less than or equal to 24.0 in.
κ = 1.6 for pretensioned members with a depth greater than 24.0 in.

RELEVANT BACKGROUND AND RESEARCH

Measurements of strand transfer length in lightweight concrete have been reported by Kozlos (2000), Thatcher et al. (2002), and Ozyildirim and Gomez (2005) for 0.5-in. diameter strand; Peterman et al. (1999, 2000a) for 0.5-in. special strand; and Meyer (2002) for 0.6-in. diameter strand. A graph of the ratio of measured to calculated transfer lengths versus concrete compressive strength is shown in Fig. 5.11.4-A. All of the data for Meyer and Ozyildirim had measured lengths less than the calculated length of 60 strand diameters. The data of Thatcher et al. has measured lengths less than and greater than the calculated length. Peterman et al. did not report actual values but concluded that the measured lengths were less than 50 strand diameters—the value used in the Standard Specifications.

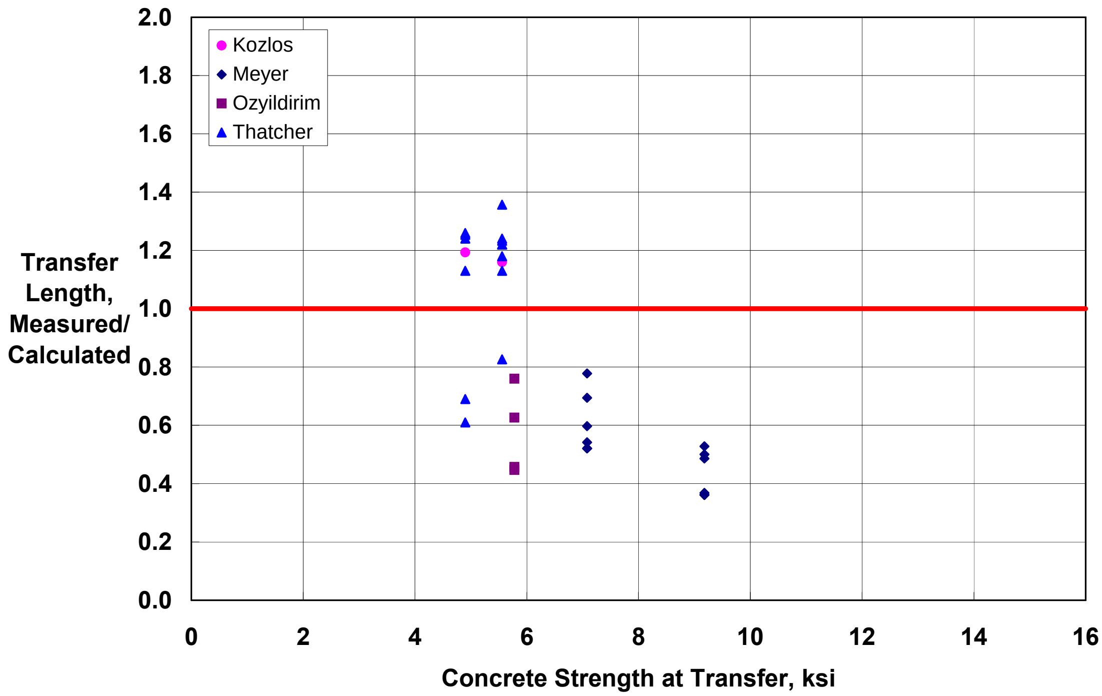

Figure 5.11.4-A. Comparison of the ratio of measured to calculated transfer length with concrete compressive strengths

Thatcher et al. (2002) and Sylva III et al. (2002) reported on transfer lengths of 3/8-in. diameter strand in two 4-in. thick lightweight concrete panels. They concluded that the measured length was less than calculated using the 60 strand diameters of the LRFD Specifications.

Equation 5.11.4.2-1 without the κ factor was based largely on data from tests conducted by Hanson and Kaar (1959), which did not include lightweight concrete. The history of the development length equation was described by Tabatabai and Dickson (1993).

Peterman et al. (1999, 2000) conducted 12 development length tests on rectangular single-strand beams made with lightweight concrete and strand from two different manufacturers. The results indicated that the development length calculated using Eq. 5.11.4.2-1 provided sufficient embedment to develop the full capacity of a single strand. When the same combinations of strands and concrete were tested in multi-strand T-beams, the results were mixed. For one strand, flexural failures occurred indicating that the development length per Eq. 5.11.4.2-1 was adequate. For the other strand, bond, flexure, and a combination of bond and web shear failures occurred in different beams.

Based on his tests, Meyer (2002) concluded that there was no need to differentiate between normal weight concrete and lightweight concrete made with a slate aggregate for concrete compressive strengths greater than 8.0 ksi.

Thatcher et al (2002) performed 10 tests of lightweight concrete beams with 1/2-in. diameter strands and embedment lengths of 80, 70, and 60 in. They concluded that the embedment length was less than 60 in. as all specimens failed in flexure. The calculated embedment per Eq. 5.11.4.2-1 with $\kappa = 1.0$ was 86 in.

Ozyildirim and Gomez (2005) determined that the measured development length of 1/2-in. diameter strand was less than calculated using Eq. 5.11.4.2-1 with $\kappa = 1.0$.

ARTICLE 5.12 DURABILITY

5.12 Durability

Provisions intentionally not included.

The provisions of Article 5.12 are intended to prevent corrosion of reinforcing steel and do not differentiate between normal weight and lightweight concrete.

ARTICLE 5.13 SPECIFIC MEMBERS

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
The section at the face of the support for brackets and corbels may be designed in accordance with either the strut-and-tie method specified in Article 5.6.3 or the provisions of Article 5.13.2.4.1, with the following exceptions:

For all lightweight or sand-lightweight concretes, nominal shear resistance, V_n, shall satisfy:

$$V_n = (0.2 - 0.07 a_v / d) f_c^{'} b_w d_e \text{ (kips) and} \qquad (5.13.2.4.2\text{-}3)$$

$$V_n = (0.8 - 0.28 a_v / d_e) b_w d \text{ (kips)} \qquad (5.13.2.4.2\text{-}4)$$

RELEVANT BACKGROUND AND RESEARCH

According to the Commentary of ACI 318-05 (ACI Committee 318, 2005), the maximum shear strength of lightweight concrete corbels or brackets is a function of both f'_c and a_v/d based on the tests by Mattock et al. (1976b). Mattock et al. tested six all-lightweight concrete corbels with concrete compressive strengths ranging from 3.645 to 4.035 ksi and proposed Eqs. 5.13.2.4.2-3 and 5.13.2.4.2-4. No data are available for corbels or brackets made of sand-lightweight concrete. As a result, the same limitations were placed on both all lightweight and sand-lightweight brackets and corbels.

No other research addressing the shear resistance of corbels and brackets with lightweight concrete was identified.

5.13.2.5 Beam Ledges
5.13.2.5.4 Design for Punching Shear
Nominal punching shear resistance, V_n, kip, shall be taken as:

- At interior pads, or exterior pads where the end distance *c* is greater than *S/2*:

$$V_n = 0.125\sqrt{f_c^{'}}(W + 2L + d_e)d_e \qquad (5.13.2.5.4\text{-}1)$$

- At exterior pads where the end distance c is less than $S/2$ and $c-0.5W$ is less than d_e:

$$V_n = 0.125\sqrt{f_c'}\left(W + L + d_e\right)d_e \qquad (5.13.2.5.4\text{-}2)$$

- At exterior pads where the end distance c is less than $S/2$, but $c-0.5W$ is greater than d_e:

$$V_n = 0.125\sqrt{f_c'}\left(0.5W + L + d_e + C\right)d_e \quad (5.13.2.5.4\text{-}3)$$

where:

f'_c = specified strength of concrete at 28 days (ksi)
W = width of bearing plate or pad as shown in Figure 1 (in.)
L = length of bearing pad as shown in Figure 1 (in.)
d_e = effective depth from extreme compression fiber

RELEVANT BACKGROUND AND RESEARCH

The punching shear resistance in this article is the same for normal weight and lightweight concrete of the same compressive strength. This would seem inconsistent with the modifications for lightweight concrete required in Article 5.8.2.2 although the lower resistance factor of 0.70 for lightweight concrete in Article 5.5.4.2.1 would apply.

No research addressing the punching shear resistance of lightweight concrete ledges was identified.

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 kip, of the concrete shall be taken as:

$$V_n = \left(0.063 + \frac{0.126}{\beta_c}\right)\sqrt{f_c'}b_o d_v \le 0.126\sqrt{f_c'}b_o d_v \qquad (5.13.3.6.3\text{-}1)$$

where:

β_c = ratio of long side to short side of the rectangle through which the concentrated load or reaction force is transmitted
b_o = perimeter of the critical section (in.)
d_v = effective shear depth (in.)

Where $V_u > \phi V_n$, shear reinforcement shall be added in compliance with Article 5.8.3.3, with angle θ taken as 45°.

For two-way action for sections with transverse reinforcement, the nominal shear resistance, in kip, shall be taken as:

$$V_n = V_c + V_s \leq 0.195\sqrt{f'_c}\, b_o d_v \qquad (5.13.3.6.3\text{-}2)$$

in which:

$$V_c = 0.0632\sqrt{f'_c}\, b_o d_v \text{, and} \qquad (5.13.3.6.3\text{-}3)$$

$$V_s = \frac{A_v f_y d_v}{s} \qquad (5.13.3.6.3\text{-}4)$$

RELEVANT BACKGROUND AND RESEARCH

The nominal shear resistance by Eq. 5.13.3.6.3-1 and 5-13.3.6.3-2 is the same for normal weight and lightweight concrete of the same compressive strength. This would seem inconsistent with the modifications for lightweight concrete required in Article 5.8.2.2 although the lower resistance factor of 0.70 for lightweight concrete in Article 5.5.4.2.1 would apply.

Hognestad et al. (1964) tested six lightweight concrete slabs in shear and found that their shear strength was characterized by the splitting tensile strength of the concrete rather than by the compressive strength. The equivalent equation to 5.13.3.6.3-1 in the ACI Building Code does require modification for lightweight concrete.

5.14 PROVISIONS FOR STRUCTURE TYPES
5.14.2 Segmental Construction
5.14.2.1 General
The provisions herein shall apply only to segmental construction using normal weight concrete.

C5.14.2.1
Lightweight concrete has been infrequently used for segmental bridge construction. Provision for the use of lightweight aggregates represents a significant complication of both design and construction specifications. Given this complication and questions concerning economic benefit, use of lightweight aggregates for segmental bridges is not explicitly covered.

RELEVANT BACKGROUND AND RESEARCH

No research on the use of lightweight concrete for segmental construction was identified. Data during the construction of the Benicia-Martinez Bridge in California was included with the relevant research for Article 5.4. In addition, several sections of the bridge have been instrumented to measure long-term strains, deflections, length changes, and rotations. This information is not yet available.

5.14.5 Additional Provisions for Culverts
5.14.5.3 Design for Shear in Slabs of Box Culverts
The provisions of Article 5.8 apply unless modified herein. For slabs of box culverts under 2.0 ft. or more fill, shear strength V_c may be computed by:

$$V_c = \left(0.0676\sqrt{f_c^{'}} + 4.6\frac{A_s}{bd_e}\frac{V_u d_e}{M_u}\right) bd_e \qquad (5.14.5.3\text{-}1)$$

but V_c shall not exceed $0.126\sqrt{f_c^{'}}\ bd_e$

where:

A_s = area of reinforcing steel in the design width (in.^{2})
d_e = effective depth from extreme compression fiber to the centroid of the tensile force in the tensile reinforcement (in.)
V_u = shear from factored loads (kip)
M_u = moment from factored loads (kip-in.)
b = design width (in.)

For single-cell box culverts only, V_c for slabs monolithic with walls need not be taken to be less than $0.0948\sqrt{f'_c}\, bd_e$, and V_c for slabs simply supported need not be taken to be less than $0.0791\sqrt{f'_c}\, bd$. The quantity $V_u d_e/M_u$ shall not be taken to be greater than 1.0 where M_u is the factored moment occurring simultaneously with V_u at the section considered.

RELEVANT BACKGROUND AND RESEARCH

Because the provisions of Article 5.8 apply unless modified, the modifications for lightweight concrete in 5.8.2.2 are applicable. However, no research about the use of Eq. 5.14.5.3-1 with lightweight concrete modifiers was identified. It is unlikely that lightweight concrete would be used for cast-in-place culverts although it could be used for precast culverts to reduce shipping weight.

SECTION 9: DECK AND DECK SYSTEMS

There are no articles in Section 9 that specifically address lightweight concrete. No provisions were identified that should address lightweight concrete.

PART 2–AASHTO LRFD BRIDGE CONSTRUCTION SPECIFICATIONS

ARTICLE 8.2 CLASSES OF CONCRETE

8.2 CLASSES OF CONCRETE
8.2.3 Lightweight (Low-Density) Concrete
Lightweight (low-density) concrete shall conform to the requirements specified in the contract documents. When the contract documents require the use of natural sand for a portion or all of the fine aggregate, the natural sand shall conform to AASHTO M 6.

RELEVANT RESEARCH

Research is not relevant to this article.

ARTICLE 8.3 MATERIALS

8.3 MATERIALS
8.3.6 Lightweight (Low-Density) Aggregate
Lightweight (low-density) aggregate for concrete shall conform to the requirements of AASHTO M 195 (ASTM C 330).

RELEVANT RESEARCH

Research is not relevant to this article.

ARTICLE 8.4 PROPORTIONING OF CONCRETE

8.4 PROPORTIONING OF CONCRETE
8.4.1 Mix Design
8.4.1.1 Responsibility and Criteria
For lightweight (low-density) 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.

C8.4.1.1
Lightweight (low-density) mix design refers to the ACI Publication 211.2, 1998.

8.4.2 Water Content
The amount of water used shall no exceed the limits listed in Table 8.2.2-1 and shall be further reduced as necessary to produce concrete of the consistencies listed in Table 8.4.2-1 at the time of placement.

RELEVANT BACKGROUND AND RESEARCH

Table 8.4.2-1 is only applicable to normal weight concrete.

A similar table for lightweight concrete may be desirable.

APPENDIX A8 PROPOSED STANDARD SPECIFICATION FOR COMBINED AGGREGATES FOR HYDRAULIC CEMENT CONCRETE

1. SCOPE
1.1 This specification covers the requirements for combined aggregates for hydraulic cement concrete having a nominal maximum aggregate size of 50 mm (2.0 in.) or less. Fine and coarse aggregate shall be blended to achieve the desired properties. Two approaches are given. One is based on performance and the other on method type.

RELEVANT BACKGROUND AND RESEARCH

This proposed standard specification was developed for the AASHTO Standard Specifications for Transportation Materials and Methods of Sampling, Part 1: Specifications. It was included in the LRFD Bridge Construction Specifications on an interim basis. The appendix is applicable to both normal weight and lightweight concrete and is primarily based on practical experience with concrete mix proportioning.

PART 3–CONCLUSIONS AND RECOMMENDATIONS

This synthesis indicates that the majority of provisions of the LRFD Design Specifications are based on normal weight concrete with some form of modification factor applied for all-lightweight and sand-lightweight concretes. This modification is handled in several different ways:

1. Different strength reduction factors (Article 5.5.4.2.1)

2. Modification factors for shear (Article 5.8.2.2)

3. Multipliers for development length (Article 5.11.2.1.2)

4. Separate provisions for lightweight concrete (Articles 5.8.4.3 and 5.13.2.4.2)

In some articles, more than one modification applies. The modifications seem to address the fact that the tensile strength of lightweight concrete is less than that of normal weight concrete with the same compressive strength. The modifications depend on the amount and type of fine aggregate. Whether this approach is consistent or the best with today's materials and the ability to produce higher strength concretes needs to be assessed. In conjunction with this, an overall approach needs to be developed for concrete with densities between 0.120 and 0.135 kcf.

The following recommendations for each article of the LRFD Design Specification are based on the research and background discussions of each article in Part 1.

Article 5.1 Scope

No research is needed in connection with Article 5.1.

Article 5.2 Definitions

Research and specification revisions are needed to address the gap between lightweight concrete with an air dry unit weight of 0.120 kcf and normal weight concrete with a unit weight of 0.135 kcf.

Article 5.3 Notation

No research is needed in connection with Article 5.3.

Article 5.4 Material Properties

Research is needed to identify the changes needed, if any, to make the creep and shrinkage equations applicable to lightweight concrete (Article 5.4.2.3).

Research is needed to develop a relationship between concrete tensile strength and compressive strength for lightweight concrete (Article 5.4.2.7).

Article 5.5 Limit States

Research is needed to address the lower resistance factors for lightweight concrete in combination with other modification factors that are used to accommodate the reduction in tensile strength (Article 5.5.4).

Article 5.6 Design Considerations

Research is needed to evaluate the proportioning of compressive struts and node regions with lightweight concrete (Articles 5.6.3.3 and 5.6.3.5).

Article 5.7 Design for Flexural and Axial Force Effects

Research is needed to assess the α_1 factor for the rectangular stress block for use in flexure and axial compression (Articles 5.7.2.2 and 5.7.4.4) and bearing (Article 5.7.5) for lightweight concrete.

Research is needed to assess the minimum amount of reinforcement in columns (Article 5.7.4.2) and the effectiveness of spirals in columns with lightweight concrete (Article 5.7.4.6).

Article 5.8 Shear and Torsion

Research is needed on the modification factors for lightweight concrete (Article 5.8.2.2), minimum transverse reinforcement (Article 5.8.2.5), maximum spacing of transverse reinforcement (Article 5.8.2.7), applicability of the latest versions of the various sectional design procedures (Article 5.8.3), and shear friction (Article 5.8.4) for lightweight concrete.

Article 5.9 Prestressing and Partial Prestressing

Research is needed on the tensile stress limits (Article 5.9.4) and loss of prestress (Article 5.9.5).

Article 5.10 Details of Reinforcement

Research is needed on tendon confinement (Article 5.10.4.3), spiral anchorages (Article 5.10.6.2), and anchorage zones (Articles 5.10.9 and 5.10.10).

Article 5.11 Development and Splices of Reinforcement

Research in needed on the modification factors for deformed bars in lightweight concrete (Article 5.11.2), transfer length (Article 5.11.4.1), and development length (Article 5.11.4.2).

Article 5.12 Durability

No structural research is needed in connection with Article 5.12. The need for durability research is not addressed in this synthesis.

Article 5.13 Specific Members

Research is needed on the shear strength of brackets and corbels (Article 5.13.2.4), beam ledges (Article 5.13.2.5), and slabs and footings (Article 5.13.3.6) for lightweight concrete.

Article 5.14 Provisions for Structure Types

Research is needed on the use of lightweight concrete in segmental construction (Article 5.14.2) and box culverts (Article 5.14.5). Clarification is also needed with regard to lightweight concrete in other articles that address segmental construction (Articles 5.5.4.2.2, 5.8.5, and 5.8.6).

All research programs should consider that lightweight aggregates can be produced with different materials and this may affect the structural behavior. Consequently, the research programs should include several different lightweight aggregates.

Research problem statements based on the needed research are included in an appendix at the end of this report.

Additional research in connection with the LRFD Construction Specifications does not seem necessary.

No new working agenda items for consideration by AASHTO Committee T-10 were identified in the preparation of this synthesis.

APPENDIX: RESEARCH PROBLEM STATEMENTS

The following five research problem statements have been developed and are included in this appendix.

1. Lightweight Structural Concrete for Bridges—Material Properties

2. Lightweight Structural Concrete for Bridges—Flexural and Compression Provisions

3. Lightweight Structural Concrete for Bridges—Shear and Torsion Provisions

4. Lightweight Structural Concrete for Bridges—Reinforcement Details

5. Lightweight Structural Concrete for Bridges—Segmental Construction

I. PROBLEM STATEMENT No. 1

II. PROBLEM STATEMENT TITLE

Lightweight Structural Concrete for Bridges—Material Properties

III. RESEARCH OBJECTIVE

The research objective is to validate the applicability or develop proposed revisions, where necessary, of the following articles of the AASHTO LRFD Bridge Design Specifications for use with lightweight concrete having design concrete compressive strengths up to 10.0 ksi or greater:

5.4.2.3 Shrinkage and Creep

C5.4.2.7 Tensile Strength

5.9.4 Stress Limits for Concrete

5.9.5 Loss of Prestress

5.10.10 Pretensioned Anchorage Zones

IV. SCOPE

The scope of this project should include the following tasks:

Task 1

Compile existing data for shrinkage, creep, and prestress losses of lightweight concrete and compare with the existing LRFD specification developed for normal weight concrete.

Task 2

Develop and conduct a test program to obtain shrinkage, creep, and prestress loss data for a broader range of lightweight aggregate concretes.

At least three different lightweight aggregates and three different concrete compressive strengths should be included although a full parametric study is probably not necessary. The highest compressive strength level must be greater than 10.0 ksi at 28 days.

Shrinkage and creep tests should be conducted in accordance with ASTM C 157 and C 512, respectively. The use of 4-in. diameter cylinders for the creep tests is permitted provided a minimum gage length for strain measurements of 8 in. is used. Two ages of loading for the creep tests should be included—18 hours and 28 days. The specimens to be loaded at 18 hours should be heat or steam cured. A minimum of 10 creep tests will be necessary. Duration of testing should be at least 12 months.

Prestress losses should be measured on at least five full-size precast, prestressed concrete beams stored in an outdoor environment. The stress levels in the beams should represent the stress conditions of beams in an actual bridge. The concretes used in the beams should be the same as those used in the shrinkage and creep tests. The ends of the beams shall be reinforced in accordance with Article 5.10.10 to verify the adequacy of this article. Duration of testing should be at least 12 months.

Task 3

Based on the results from Task 1 and 2, develop proposed revisions, if necessary, to make Articles 5.4.2.3 and 5.9.5 applicable to lightweight concrete with compressive strengths up to 10.0 ksi or greater.

Task 4

Based on existing data, develop a proposed revision to Commentary C5.4.2.7 to address lightweight concrete.

Task 5

Analyze existing data to determine if the tensile stress limits of Article 5.9.4 are applicable for lightweight concrete beams. If additional data are required, load the prestressed concrete beams at the completion of Task 2 to determine their tensile stress at cracking. Data will also be available from the prestressed concrete beams in Task 5 of Statement 2 and Task 6 of Statement 4.

Task 6

Prepare a technical report suitable for publication to document the research findings, conclusions, and recommendations. Any proposed revisions to the AASHTO LRFD Design Specifications shall be in the format required by AASHTO Technical Committee T-10, Concrete Design.

I. PROBLEM STATEMENT No. 2

II. PROBLEM STATEMENT TITLE

Lightweight Structural Concrete for Bridges—Flexural and Compression Provisions

III. RESEARCH OBJECTIVE

The research objective is to validate the applicability or develop proposed revisions, where necessary, of the following articles of the AASHTO LRFD Bridge Design Specifications for use with lightweight concrete having design concrete compressive strengths up to 10.0 ksi or greater:

5.5.4.2 Resistance Factors (Compression in Anchorage Zones)

5.7.2.2 Rectangular Stress Distribution

5.7.4.2 Limits for Reinforcement

5.7.4.4 Factored Axial Resistance

5.7.4.6 Spirals and Ties

5.7.5 Bearing

5.10.10 Pretensioned Anchorage Zones

IV. SCOPE

The scope of this project should include the following tasks:

Task 1

Perform tests to obtain additional data for the α_1 and β_1 factors used in the equivalent rectangular stress block design. At least three different lightweight aggregates with three different concrete compressive strength levels at test age should be included. The highest strength levels should be the maximum that can be achieved with each aggregate. A full parametric study is desirable.

The recommended test specimen is that used on NCHRP No. 12-64 and consists of a C-shaped specimen with a 9-in. square test section. The upper and lower arms of the C consist of reusable steel members that are bolted to the concrete test specimen. The testing concept is similar to the original concept used by Hognestad et al. (1955) except for the steel arms.

Task 2

Conduct tests of reinforced concrete beams in pure flexure and under a combination of axial load and flexure. As a minimum, tests should be made using each of the aggregates tested in Task 1 at the highest concrete compressive strength level.

Task 3

Conduct tests of reinforced concrete columns with a small eccentricity. As a minimum, tests should be made using each of the aggregates tested in Task 1 at the highest concrete compressive strength level.

Task 4

Compare the measured strengths of the tested beams and columns from Tasks 2 and 3 with interaction diagrams constructed using the current LRFD specifications and with any proposed revisions for α_1 and β_1.

Task 5

Conduct tests of prestressed concrete beams in flexure. As a minimum, tests should be made using each of the aggregates tested in Task 1 at the highest concrete compressive strength level. These tests could be made on the same beams used for measurement of prestress losses in Statement No. 1. For these tests, the compression block needs to be made of lightweight concrete. The ends of the beams shall be reinforced per Article 5.10.10 to verify the adequacy of this article. Compare the measured beam strengths with values calculated using the current LRFD specifications and with any proposed revisions for α_1 and β_1

Task 6

Conduct analyses to determine if the minimum reinforcement required by Article 5.7.4.2 is appropriate for lightweight concrete. A procedure for the analysis was developed in NCHRP Project No. 12-64. A similar analysis should be made for lightweight concrete based on the shrinkage and creep data obtained from the research described in Statement No. 1.

Task 7

Evaluate the data by Martinez et al. (1984) and Ahmad and Shah (1982) to determine if additional tests are needed and if proposed revisions to Article 5.7.4.6 are needed. If necessary, perform additional tests of circular and rectangular columns.

Task 8

Conduct tests to verify that the 0.85 factor in Eq. 5.7.5.2 and the strength reduction factor of 0.65 of Article 5.5.4.2.1 for anchorage zones are applicable or whether they should be modified.

Task 9

Prepare a technical report suitable for publication to document the research findings, conclusions, and recommendations. Any proposed revisions to the AASHTO LRFD Design Specifications shall be in the format required by AASHTO Technical Committee T-10, Concrete Design.

I. PROBLEM STATEMENT No. 3

II. PROBLEM STATEMENT TITLE

Lightweight Structural Concrete for Bridges—Shear and Torsion Provisions

III. RESEARCH OBJECTIVE

The research objective is to validate the applicability or develop proposed revisions, where necessary, of the following articles of the AASHTO LRFD Bridge Design Specifications for use with lightweight concrete having design concrete compressive strengths up to 10.0 ksi or greater:

5.5.4 Strength Limit State (Shear and Torsion)

5.6.3 Strut-and-Tie Model

5.8.2.2 Modifications for Lightweight Concrete

5.8.2.5 Minimum Transverse Reinforcement

5.8.2.7 Maximum Spacing of Transverse Reinforcement

5.8.3 Sectional Design Model

5.8.4 Interface Shear Transfer—Shear Friction

5.13.2.4 Brackets and Corbels

5.13.2.5 Beam Ledges

5.13.3.6 Shear in Slabs and Footings

IV. SCOPE

The scope of this project should include the following tasks:

Task 1

Conduct tests to investigate the applicability of Articles 5.8.2.2, 5.8.2.5, 5.8.2.7, and 5.8.3 to full-size reinforced and prestressed lightweight concrete beams. The tests should include three different lightweight aggregates at the highest concrete compressive strength possible. Test results should be compared with the existing provisions and, if necessary, modified provisions should be developed.

Task 2

Conduct limited tests to investigate the applicability of Articles 5.8.4, 5.13.2.4, 5.13.2.5, and 5.13.3.6 to high-strength lightweight concrete. At least three different lightweight aggregates at the highest concrete compressive strength should be included.

Task 3

Based on available data, determine if the resistance factor of 0.70 for shear and torsion of lightweight concrete of Article 5.5.4.2.1 is appropriate or whether an alternative value should be proposed.

Task 4

Based on available data, determine if the limiting compressive stresses of Articles 5.6.3.3.3 and 5.6.3.5 are applicable for use with lightweight concrete or whether alternative values should be proposed.

Task 5

Prepare a technical report suitable for publication to document the research findings, conclusions, and recommendations. Any proposed revisions to the AASHTO LRFD Design Specifications shall be in the format required by AASHTO Technical Committee T-10, Concrete Design.

I. PROBLEM STATEMENT No. 4

II. PROBLEM STATEMENT TITLE

Lightweight Structural Concrete for Bridges—Reinforcement Details

III. RESEARCH OBJECTIVE

The research objective is to validate the applicability or develop proposed revisions, where necessary, of the following articles of the AASHTO LRFD Bridge Design Specifications for use with lightweight concrete having design concrete compressive strengths up to 10.0 ksi or greater:

5.10.4.3 Effects of Curved Tendons

5.10.6.2 Spirals

5.10.9 Post-Tensioned Anchorage Zones

5.11.2 Development of Reinforcement

5.11.4 Development of Prestressing Strand

IV. SCOPE

The scope of this project should include the following tasks:

Task 1

Conduct analyses to determine if the provisions of Article 5.10.4.3.1 in combination with the strength reduction factor of Article 5.5.4.2 are adequate for use with lightweight concrete. If necessary, conduct a limited number of tests to verify the analyses.

Task 2

Conduct tests of short axially loaded circular columns with lap splices of the spirals at mid height to verify that the provisions of Article 5.10.6.2 are applicable for lightweight concrete. Because the provisions are independent of concrete strength, the tests should be conducted using a concrete compressive strength of about 4 ksi with three different aggregates. If the provisions are not applicable, proposed revisions should be developed and verified by testing.

Task 3

Conduct limited tests similar to those described in NCHRP Report 356 (Breen et al., 1994) to verify the applicability of Article 5.10.9 for lightweight concrete.

Task 4

Analyze existing data relative to the modification factors of Article 5.11.2.1.2 as the existing test data appear to be inconsistent. If necessary, conduct flexural tests of beams with lap splices in a constant moment region to obtain additional data.

Task 5

Perform tests of hooked bars of different diameters anchored in lightweight concrete blocks of three different concrete compressive strengths to verify the 1.3 modification factor of Article 5.11.2.4.2. At least three different lightweight aggregates should be used although a full parametric study is probably not needed.

Task 6

Conduct a systematic laboratory test program of simple rectangular prestressed concrete beams to measure transfer length and to determine development length. Controlled test variables should be strand diameter, concrete strength, and aggregate type.

Task 7

Prepare a technical report suitable for publication to document the research findings, conclusions, and recommendations. Any proposed revisions to the AASHTO LRFD Design Specifications shall be in the format required by AASHTO Technical Committee T-10, Concrete Design.

I. PROBLEM STATEMENT No. 5

II. PROBLEM STATEMENT TITLE

Lightweight Structural Concrete for Bridges—Segmental Construction

III. RESEARCH OBJECTIVES

The first research objective is to determine if additional research to extend the LRFD Bridge Design Specifications for segmental construction to lightweight concrete is warranted. If warranted, the second research objective is to validate the applicability or develop proposed revisions, where necessary, to the following articles of the AASHTO LRFD Bridge Design Specifications for use with lightweight concrete having design concrete compressive strengths up to 10.0 ksi or greater:

5.5.4.2.2 Segmental Construction (Resistance Factors)

5.8.5 Principal Stresses in Webs of Segmental Concrete Bridges

5.8.6 Shear and Torsion for Segmental Box Girder Bridges

5.14.2 Segmental Construction

IV. SCOPE

The scope of this project should be performed in two phases. Phase I consists of identification of research needed for the use of lightweight concrete in segmental construction in addition to that described in Research Problem Statements 1 through 4. An estimate of the cost and time to perform the research should be made. In consultation with FHWA, State DOTs, and industry (ASBI, ESCSI, PCI), a determination should be made if the cost of the research is justified based on the current and anticipated use of lightweight concrete in segmental construction.

If justified in Phase I, the research would be conducted in Phase II. The end product will be a technical report suitable for publication to document the research findings, conclusions, and recommendations. Any proposed revisions to the AASHTO LRFD Design Specifications shall be in the format required by AASHTO Technical Committee T-10, Concrete Design.

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