LIGHTWEIGHT CONCRETE—MATERIAL PROPERTIES FOR STRUCTURAL DESIGN

Henry G. Russell, Henry G. Russell, Inc., Glenview, IL

ABSTRACT

This paper contains a compilation and synthesis of research relating to lightweight concrete and its use in highway bridges as designed using the AASHTO LRFD Bridge Design Specifications. Specific topics include creep, shrinkage, modulus of elasticity, modulus of rupture, flexural and axial force design, shear design, loss of prestress, and development length. For these topics, the existing LRFD provisions are generally adequate for the design of lightweight concrete members with concrete compressive strength up to 10.0 ksi. Refinement of some provisions would improve their consistency and accuracy.

Keywords: Axial Force Design, Creep, Development Length, Flexural Design, Lightweight Concrete, Loss of Prestress, Material Properties, Modulus of Elasticity, Modulus of Rupture, Shear Design, Shrinkage, Splitting Tensile Strength, and Structural Design.

NOTE:

This paper was first presented at the ESCSI Special Workshop on Lightweight Aggregate Concrete Bridges that was held May 7, 2008, in St. Louis, MO. The workshop was held in conjunction with the 2008 Concrete Bridge Conference. This paper appeared as Paper 141 in the proceedings for the conference.

INTRODUCTION

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 use with higher concrete compressive strengths. This includes the use of lightweight concrete. As part of this effort, a compilation and synthesis of research relating to lightweight concrete and its use in highway bridges as related to the AASHTO LRFD Bridge Design Specifications was developed (Russell, 2007). This paper contains a comparison of the AASHTO LRFD design provisions with available research data for those provisions with the most available data. For details of all the provisions affected by lightweight concrete and those provisions for which research data with lightweight concrete are lacking, the reader is referred to the original report (Russell, 2007). Those provisions are not addressed in this paper.

For purposes of this paper, lightweight concrete is assumed to have a density between about 0.100 kcf and that of normal weight concrete. The paper does not differentiate between all-lightweight and sand-lightweight concrete.

The literature search concentrated on publications concerning North American materials and design practices. The research results are presented as a comparison with the AASHTO LRFD Bridge Design Specifications. Consequently, the units of the LRFD Specifications are generally used throughout the paper.

ARTICLE 5.4 MATERIAL PROPERTIES

5.4.2.3.2 CREEP

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. 1. 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. 1 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. 1. It is generally assumed that total creep strain is proportional to stress level up to a stress level of at least 40 percent of the concrete compressive strength at the age of loading.

In addition to the data shown in Fig. 1, 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.

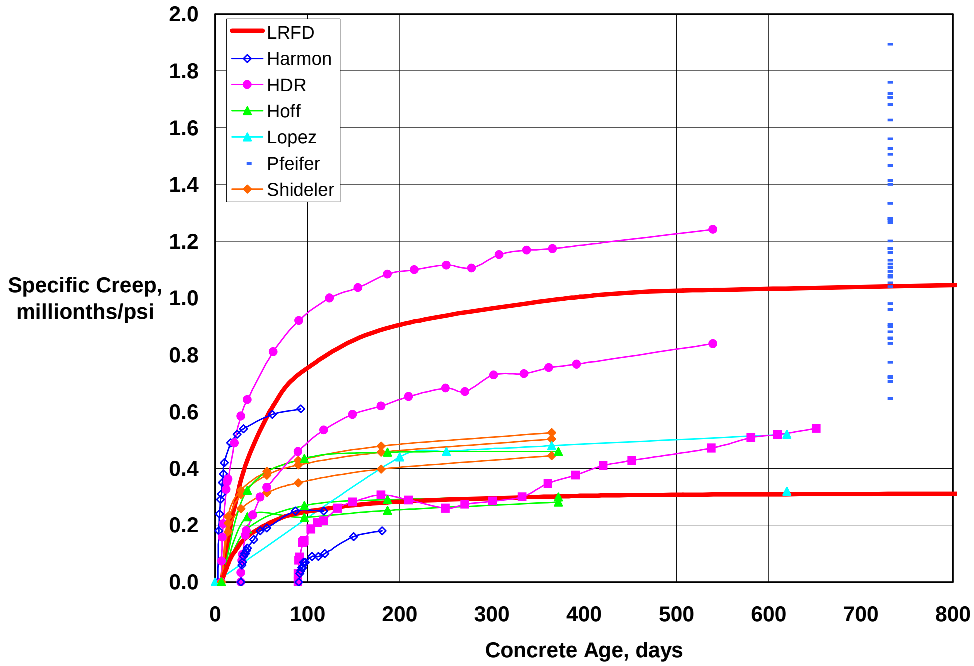

Fig. 1 Creep Data

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. 1. 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 Article 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).

5.4.2.3.3 SHRINKAGE

A comparison of shrinkage versus drying time for data by Hanson (1968), Hoff (1992), Holm (1980), Leming (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. 2.

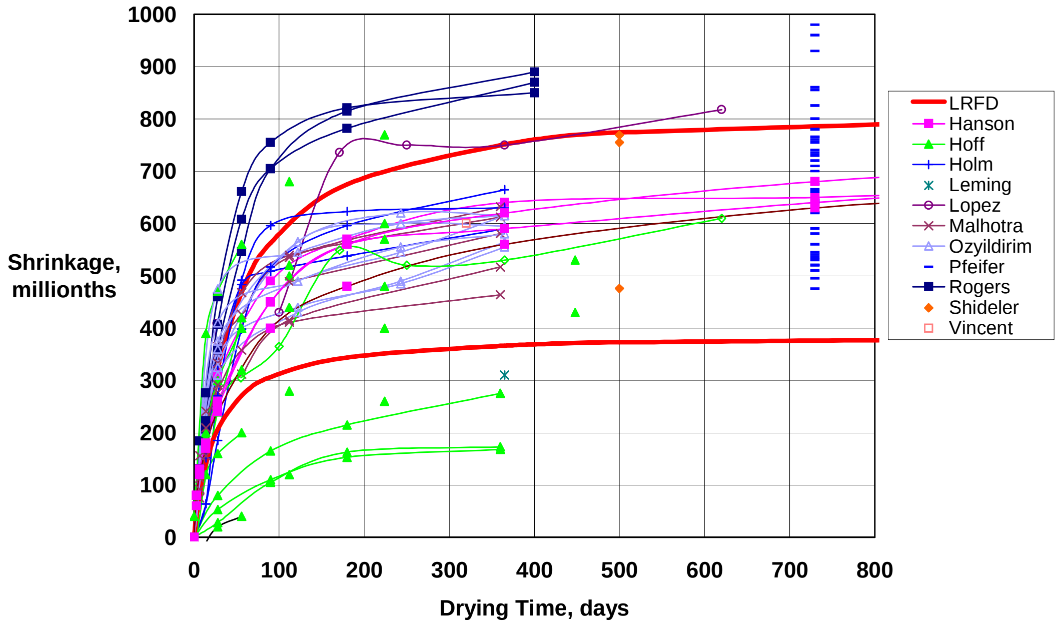

Fig. 2 Shrinkage Data

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.

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.

5.4.2.4 MODULUS OF ELASTICITY

The existing equation (5.4.2.4-1) for calculating modulus of elasticity is

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

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)

f'_c = specified compressive strength of concrete (ksi)

The NCHRP Project No. 12-64 titled "Application of the LRFD Bridge Design Specifications to High-Strength Structural Concrete: Flexure and Compression Provisions" (Rizkalla, 2007) compiled 4388 data points for concrete unit weights ranging from 0.090 to 0.176 kcf, 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:

$$E_c = 310{,}000 K_1 w_c^{2.5} f_c^{'0.33} \tag{2}$$

Comparisons of the measured data with values predicted using Eq. 5.4.2.4-1 and the proposed equation are shown in Figs. 3 and 4, respectively. The proposed equation improves the overall agreement between measured and predicted values and results in lower predicted values for lightweight concrete compared to values calculated using the existing equation. It should be noted that the spread of data is about ± 25 percent of the predicted values.

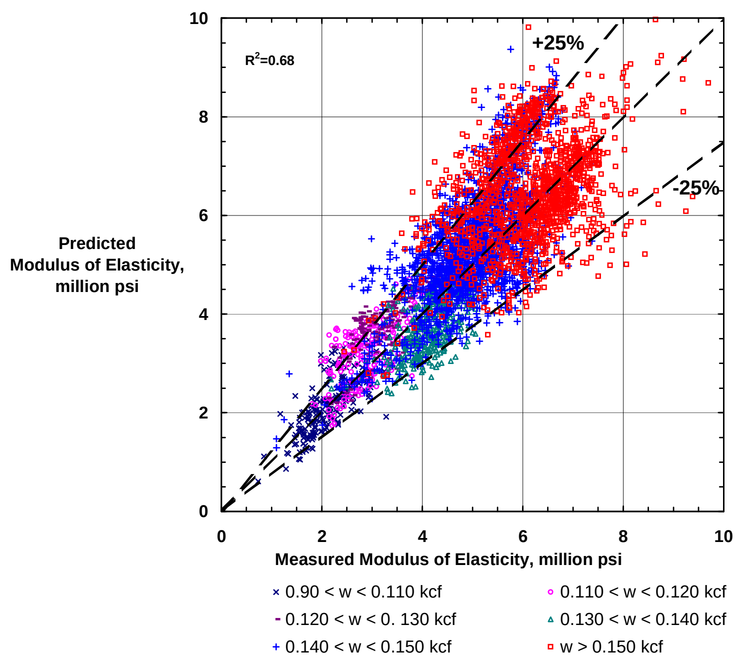

Fig. 3 Comparison of Predicted and Measured Modulus of Elasticity for Eq. 5.4.2.4-1

5.4.2.6 MODULUS OF RUPTURE

A comparison of modulus of rupture versus concrete compressive strength is shown in Fig. 5 for data by Harmon (2005), Heffington (2000), Hoff (1992), Malhotra (1990), Meyer (2002),

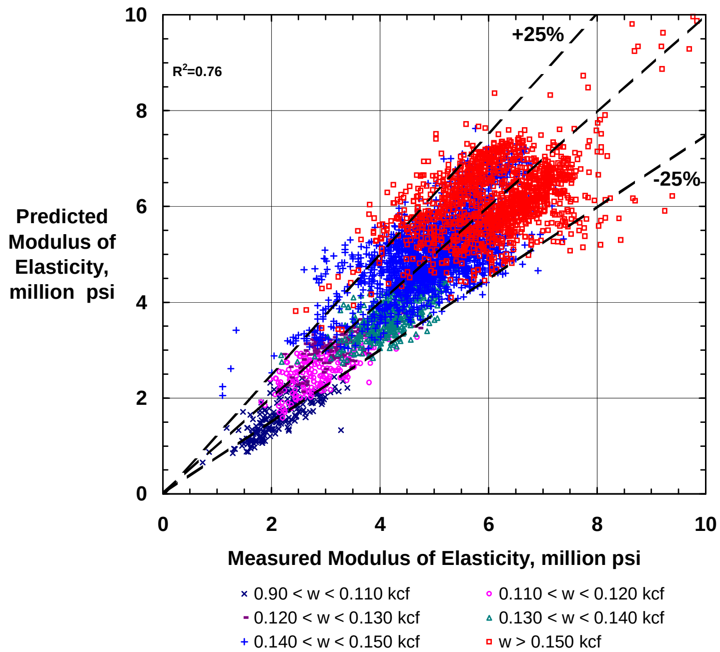

Fig. 4 Comparison of Predicted and Measured Modulus of Elasticity for Proposed Equation

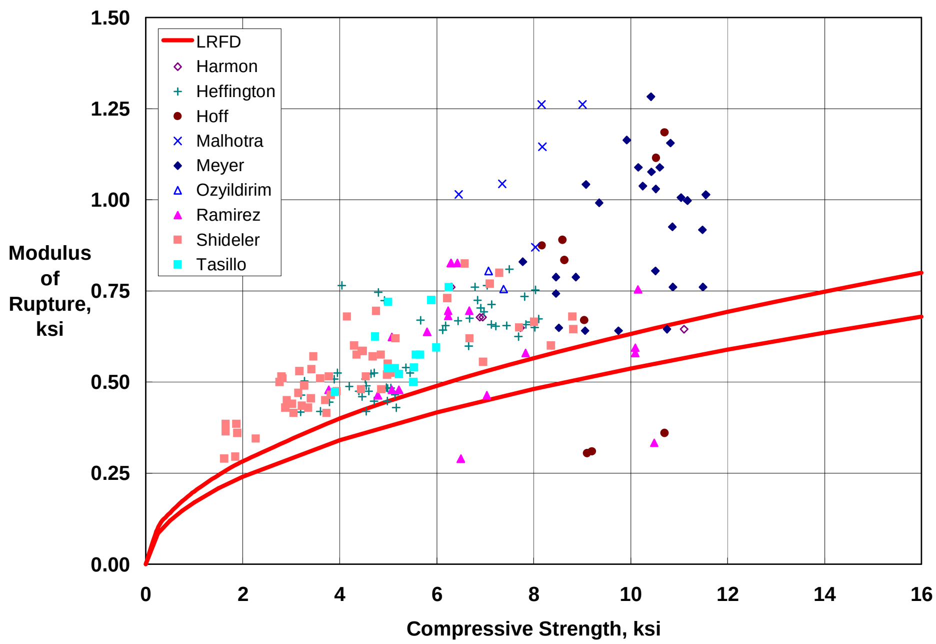

Fig. 5 Modulus of Rupture Versus Concrete Compressive Strength

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 show the modulus of rupture calculated using the provisions of Article 5.4.2.6 for sand-lightweight and all-lightweight concrete. The variation of splitting tensile strength with concrete compressive strength is presented later in ARTICLE 5.8 SHEAR AND TORSION.

ARTICLE 5.7 DESIGN FOR FLEXURAL AND AXIAL FORCE EFFECTS

A comparison of maximum usable concrete compressive strain versus concrete compressive strength is shown in Fig. 6 for lightweight concrete 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.

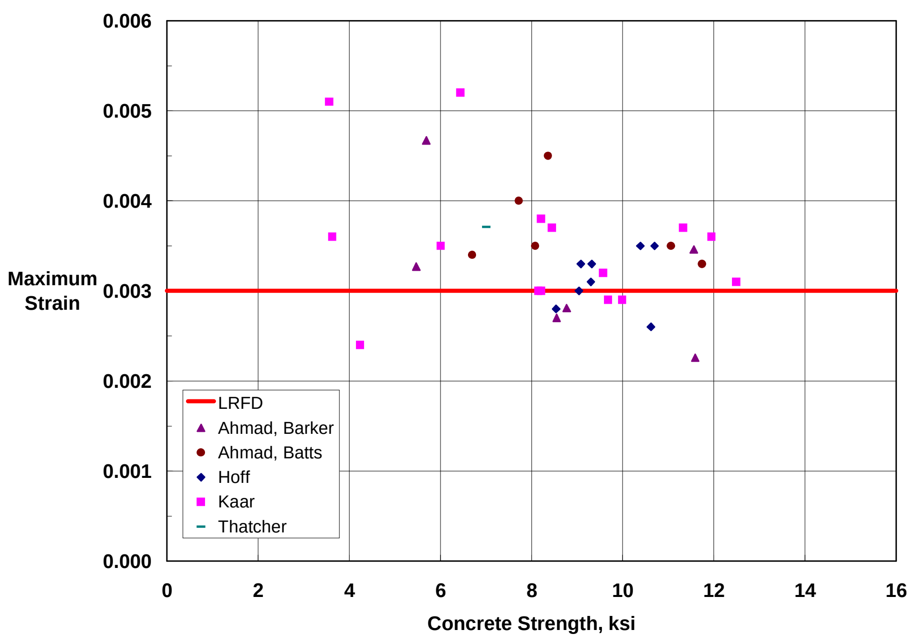

Fig. 6 Maximum Strain Versus Concrete Compressive Strength

5.7.2.2 RECTANGULAR STRESS DISTRIBUTION

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. 7 for lightweight concrete 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.

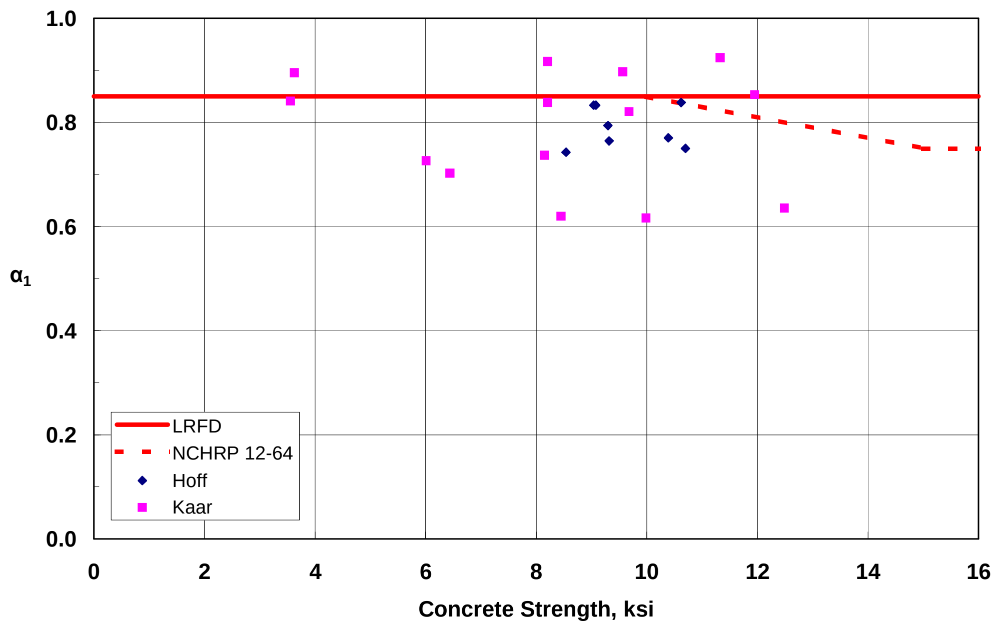

Fig. 7 Variation of α_1 Factor with Concrete Compressive Strength

For comparison purposes, the value of 0.85 used in the LRFD Specifications is also shown in Fig. 7 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. 7 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. The value of α_1 has little effect on the calculated flexural strength of under-reinforced sections but does affect the calculated axial load capacity where concrete compression controls.

The equivalent rectangular stress block assumes that the uniform compressive stress acts over a depth of $\beta_1 c$, where c is the depth of the neutral axis from the extreme compression fiber. Values of β_1 for different concrete compressive strengths are shown in Fig. 8 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.

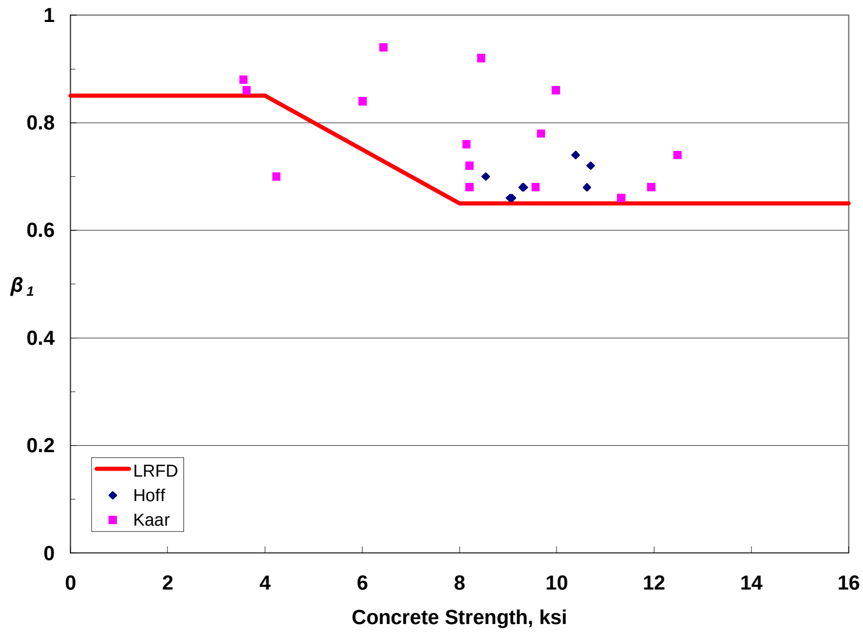

Fig. 8 Variations of β_1 Factor with Concrete Compressive Strength

5.7.3.2 FLEXURAL RESISTANCE

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. 9. 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, 1983). However, since the procedures of the LRFD Specifications result in similar flexural 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.

5.7.4.4 FACTORED AXIAL RESISTANCE

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.

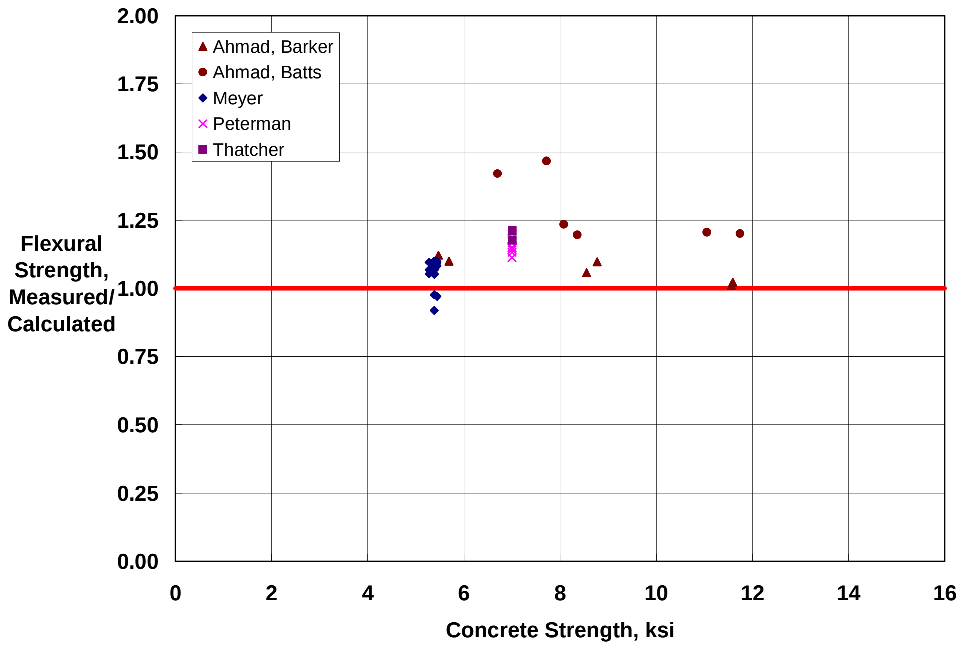

Fig. 9 Comparison of the Ratio of Measured to Calculated Flexural Strength with Concrete Compressive Strength

ARTICLE 5.8 SHEAR AND TORSION

5.8.2.2 MODIFICATIONS FOR LIGHTWEIGHT CONCRETE

Article 5.8.2.2 states that modifications shall apply in determining resistance to torsion and shear of lightweight concrete.

Where the average splitting tensile strength of lightweight concrete, f_{ct}, is specified, the term $\sqrt{f_c'}$ may be replaced by 4.7 $f_{ct} \leq \sqrt{f_c'}$. 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.

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 $\sqrt{f_c'}$. The origin of 0.75 and 0.85 factors 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. 10 (Hanson, 1961, 1965, 1968; Heffington, 2000; Hoff, 1992; Ivey and Buth, 1966; Khaloo and Nakseok, 1999; Malhotra, 1990; Mattock et al., 1976; Ozyildirim and Gomez, 2005; Pfeifer, 1967; Ramirez et al., 2000, 2004; and Vincent et al., 2004). The LRFD lines represent the above modifications for lightweight concrete. The line labeled "LRFD at 0.23" applies to normal weight concrete. The LRFD lines for lightweight concrete tend to overestimate the measured strengths.

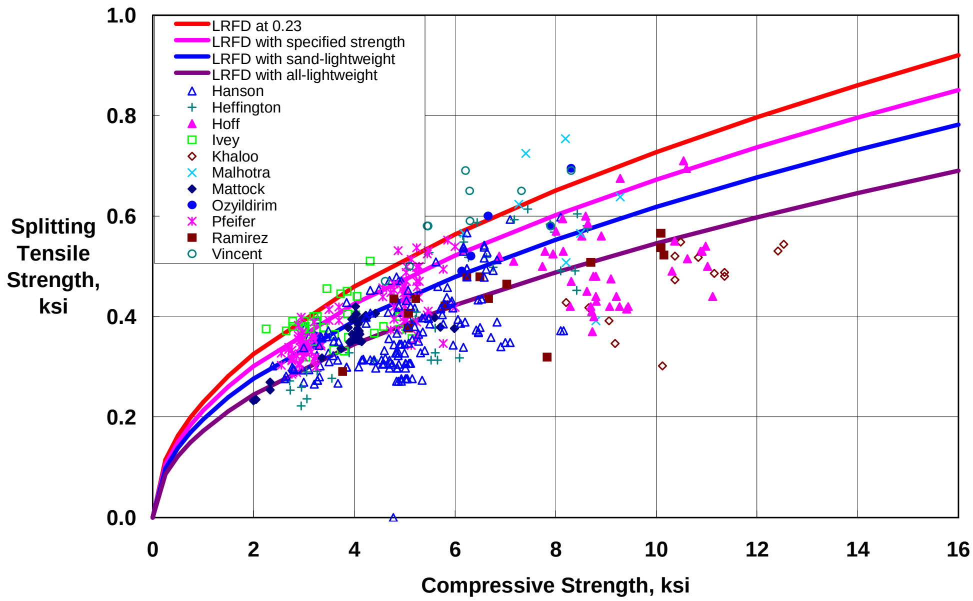

Fig. 10 Splitting Tensile Strength Versus Concrete Compressive Strength

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.

5.8.3.3 NOMINAL SHEAR RESISTANCE

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 titled "Applications of LRFD Bridge Design Specifications to High-Strength Structural Concrete: Shear Provisions" (Hawkins et al., 2007), some additional changes may also occur in the future.

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. 11. 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.

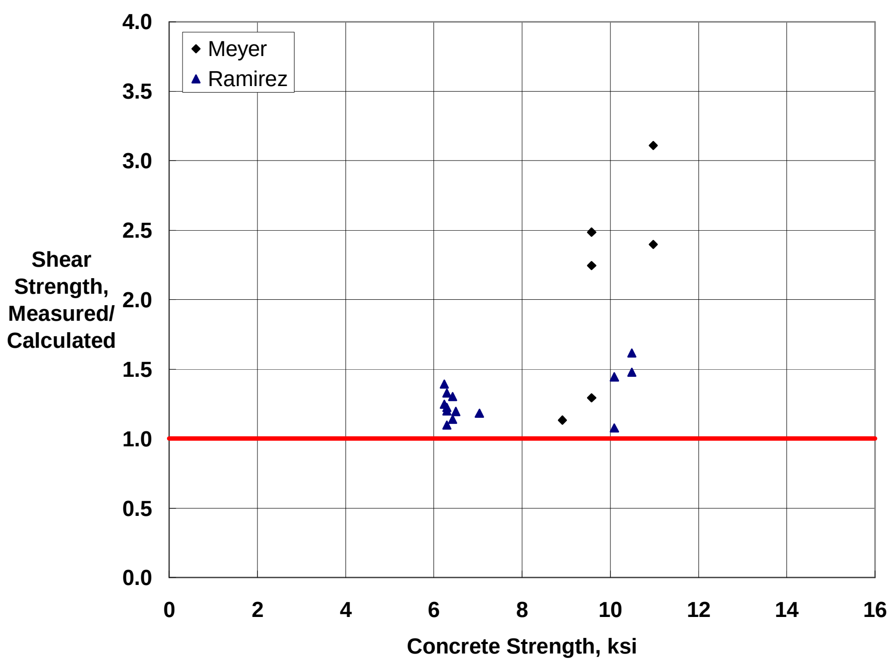

Fig. 11 Comparison of the Ratio of Measured to Calculated Shear Strengths with Concrete Compressive Strength

ARTICLE 5.9 PRESTRESSING AND PARTIAL PRESTRESSING

5.9.5 LOSS OF PRESTRESS

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.

Hanson (1964) 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 12.

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. 12. In general, the refined method overestimated the losses, whereas the lump sum losses underestimated the losses for one of the mixes.

ARTICLE 5.11 DEVELOPMENT AND SPLICES OF REINFORCEMENT

Article 5.11.2.1.2 includes modification factors for development length and splices of 1.3 and 1.2 for all-lightweight concrete and sand-lightweight concrete, respectively. 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

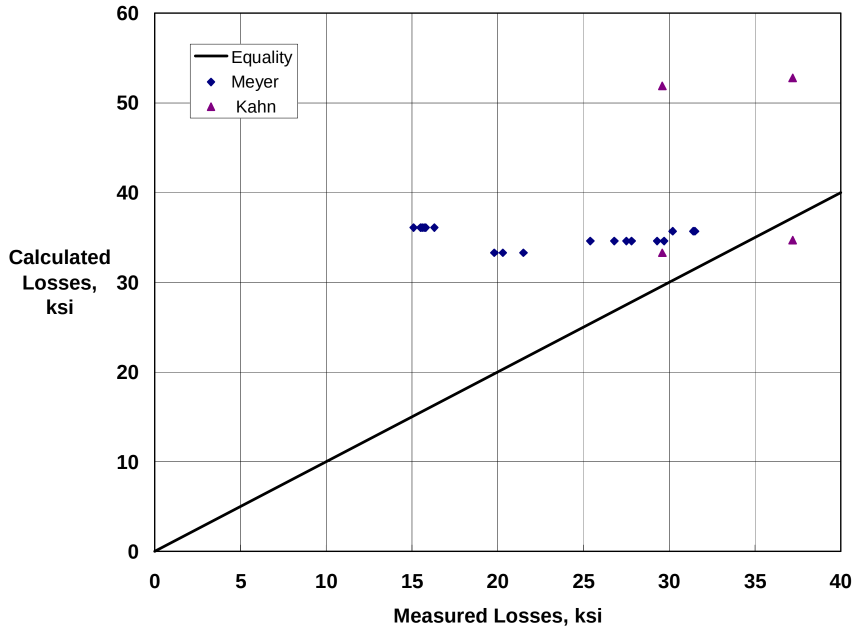

Fig. 12 Comparison of Calculated and Measured Prestress Losses

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).

5.11.4 DEVELOPMENT OF PRESTRESSING STRAND

Transfer Length

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, 2000) 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. 13. 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.

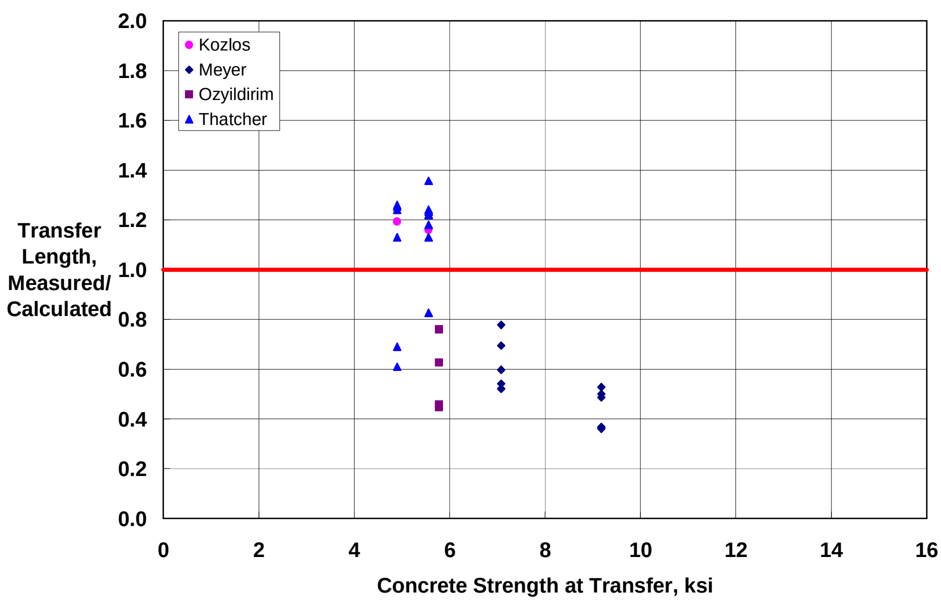

Fig. 13 Comparison of the Ratio of Measured to Calculated Transfer Length with Concrete Compressive Strength

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.

Development Length

According to Article 5.11.4.2, the development length of strand shall be calculated using Eq 5.11.4.2-1 as follows:

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

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.

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 length per Eq. 5.11.4.2-1 with κ = 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 κ = 1.0.

CONCLUSIONS

For the articles of the AASHTO LRFD Bridge Design Specifications discussed in this paper, the existing provisions are generally adequate for the design of lightweight concrete members with concrete compressive strength up to 10.0 ksi. Refinement of some provisions would improve their consistency and accuracy.

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