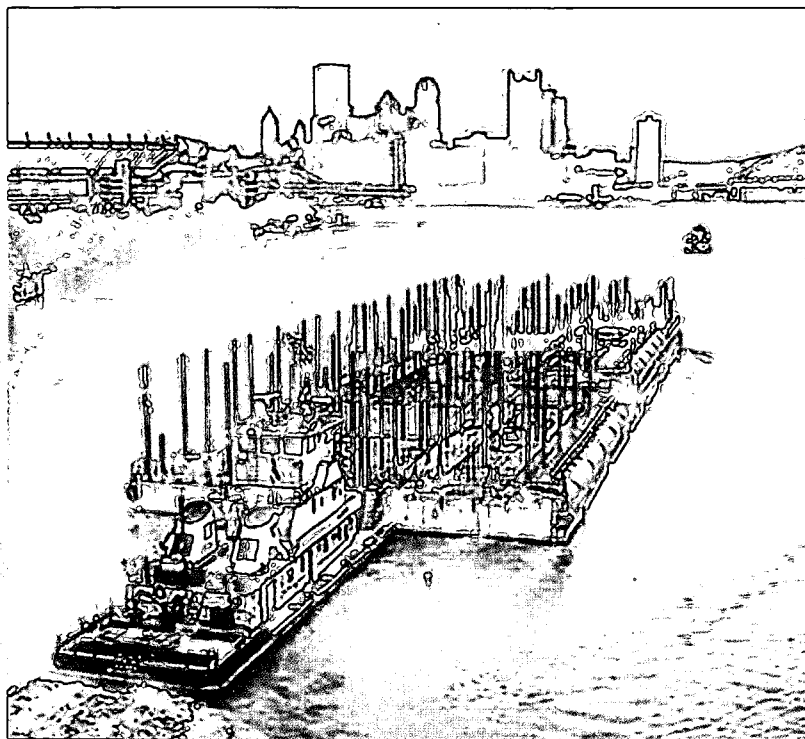


High-Performance *Structural Lightweight Concrete*



Editors:

John P. Ries

Thomas A. Holm



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High-Performance Structural Lightweight Concrete

Editors

John P. Ries

Thomas A. Holm



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PREFACE

This is a compilation of papers addressing “High-Performance Structural Lightweight Concrete” presented October 30, 2002 at the American Concrete Institute Fall Convention in Phoenix, Arizona. This symposium was sponsored by ACI Committee 213, Lightweight Aggregate and Concrete, to report on a wide range of global construction applications incorporating high-performance lightweight-aggregate concrete. This diverse symposium included papers that covered microstructural issues (autogenous shrinkage, internal curing), material and structural properties (transfer length, shear strength, seismic behavior), and applications in large civil structures (long-span balanced cantilever bridges, offshore platform, float-in navigational locks).

We and the members of ACI Committee 213 are grateful to the authors for their important contributions and dissemination of valuable state-of-the-art information.

A significant amount of data from this symposium was immediately incorporated into the revision of ACI 213R-03, “Guide for Structural Lightweight Aggregate Concrete.” We are extremely grateful for the authors’ timely contributions in updating this valuable reference document.

John P. Ries
Thomas A. Holm
Session Moderators and Editors

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First Use of Lightweight High-Performance Concrete Beams in Virginia

by **C. Ozyildirim, T. Cousins and J. Gomez**

Synopsis: An experimental program was planned and executed to demonstrate the feasibility of using lightweight high performance concrete (LWHPC) bridge beams and decks. The prestressed American Association of State Highway and Transportation Officials (AASHTO) beams were designed for a minimum 28-day compressive strength of 8,000 psi (55 MPa) and the deck concrete for 4,000 psi (27.6 MPa). The maximum permeability was 1500 coulombs for the beam and 2500 coulombs for the deck concrete. The density was less than 120 lb/ft³ (1920 kg/m³). This program was a necessary prelude to an LWHPC demonstration bridge built over the Chickahominy River on Route 106 at the Charles City County–New Kent County line in Virginia.

Keywords: development length; flexural strength; high-performance concrete; lightweight concrete; low-permeability concrete; prestressed concrete bridge beams; structural testing; transfer length

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Biographical Sketches:

Dr. Celik Ozyildirim is a Principal Research Scientist with the Virginia Transportation Research Council in Charlottesville. He has more than 30 years of research experience in concrete materials.

Dr. Tommy Cousins is an Associate Professor at the Virginia Polytechnic Institute & State University in Blacksburg. His area of expertise is prestressed concrete, with a particular emphasis in transportation structures.

Dr. Jose Gomez is an Associate Director at the Virginia Transportation Research Council in Charlottesville. His areas of interest are bridge design and behavior.

INTRODUCTION

High performance concretes (HPC) are characteristically higher in strength and more durable than their regular counterparts because of their denser cementitious matrix as a result of the lower water–cementitious materials ratios (w/cm) and mineral admixtures in their makeup. These attributes contribute directly to desired structural and economic efficiencies in the design and construction of highway bridges. Recently HPC was used successfully in bridge design and construction [1,2]; however, the bridges were built with normal weight concretes with a density of generally 145 lb/ft³ (2320 kg/m³). Encouraged by the successful implementation of HPC technology in demonstration bridges built in Virginia by the Virginia Department of Transportation (VDOT), the Virginia Transportation Research Council (VTRC) is seeking to employ this technology further by using lighter weight concretes. In this effort, lightweight aggregates (either lightweight coarse aggregates alone or lightweight coarse combined with lightweight fine aggregates) have been used. To produce lightweight HPC (LWHPC) with strengths exceeding 8,000 psi (55 MPa), high-quality lightweight coarse aggregates are needed. However, there has been uncertainty regarding the tensile strength, modulus of elasticity, shrinkage, and creep in both lightweight concrete (LWC) and LWHPC. These material properties directly affect the transfer length, development length, flexural strength, and prestress losses of prestressed beams.

To that end, an experimental program was planned and executed to demonstrate the feasibility of producing prestressed AASHTO beams made of LWHPC with a density of about 120 lb/ft³ (1920 kg/m³) and a 28-day compressive strength of 8,000 psi (55 MPa). Material properties were determined and structural testing of beams was conducted to evaluate the effects of using LWHPC on transfer length, development length, flexural strength, and prestress losses. This paper describes the experimental program conducted to determine the feasibility of using LWHPC on a bridge structure in Virginia. The planned result of this testing program was an LWHPC demonstration bridge to be built over the Chickahominy River on Route 106 in Virginia.

METHODS AND RESULTS

Five bridge beams were fabricated for the experimental program. Three were fabricated for structural testing and consisted of two AASHTO Type II beams fabricated with LWHPC and a third Type II beam fabricated with normal weight HPC (NWHPC). All three beams were later cast with a NWHPC composite deck section to ensure full composite action. The third beam was intended to serve as a control and comparison specimen. The other two beams were AASHTO Type IV beams made of LWHPC. They were identical in design with the bridge beams to be used in the bridge to be constructed on Route 106. The beams were tested for transfer length, development length, flexural strength, and prestress loss. Prestress losses were also calculated using the ACI and PCI models that incorporate lightweight structural concretes.

For the beams, the specified 28-day minimum compressive strength was 8,000 psi (55 MPa), with a release strength of 5,600 psi (39 MPa). Grade 270 low-relaxation prestressing strands 0.5 in (13 mm) in diameter were used. The maximum specified density of LWHPC for the beams was 120 lb/ft³ (1920 kg/m³), and the maximum permeability (ASTM C 1202) was 1500 coulombs. The beams were steam cured to obtain high early-release strengths. Specimens from the beams were kept in the recesses of the beam forms during steam curing and were then air-dried.

Concrete properties were determined for the freshly mixed and hardened states. Slump (ASTM C 143), air content (ASTM C 173), and density (ASTM C 138) were determined in the fresh state. Compressive strength (AASHTO T22), elastic modulus (ASTM C 69), and splitting tensile strength (ASTM C496) were determined in the hardened state.

Materials Evaluation

The mixture proportions for the lightweight and normal weight concretes are listed in Table 1. The cementitious material was a combination of finely ground Type II cement and Grade 120 slag. The coarse aggregate used for the control mixture was No. 68 granite with an absorption of 0.6 percent, a specific gravity of 2.98, and a bulk density of 108.4 lb/ft³ (1737 kg/m³). The lightweight coarse aggregate was ¾ in to No. 4 (19.0 mm to 4.75 mm) expanded slate with an absorption of 5 percent, a specific gravity of 1.47, and a bulk density of 49 lb/ft³ (785 kg/m³). The fine aggregate was natural sand with an absorption of 1.4 percent, a specific gravity of 2.61, and a fineness modulus of 2.60. The admixtures were a commercially available air-entraining admixture (AEA); a vinsol rosin complying with the requirements of ASTM C 260; a water-reducing admixture (WRA); a lignin complying with the requirements of ASTM C 494, Type A; and a high-range water-reducing admixture (HRWRA), which is a polycarboxylate complying with the requirements of ASTM C 494, Type F.

The characteristics of the freshly mixed concrete are given in Table 2. The air contents were all within the specified 5.5 ± 1.5 percent range. The slump values for the LWHPC were higher than those for the NWHPC, and both concretes were workable.

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The densities for the LWHPC were lower than expected. The lower density was attributed to the high air content and excess water in the mixture. The air contents for the hardened state given in Table 3 were higher than those for the freshly mixed concrete and the design air contents. The conditioning of the lightweight aggregate may have been the cause of the extra water; the aggregates were prewet using a sprinkler system. The sprinkler system may not be effective in controlling and maintaining uniform moisture content within and at the surface of the aggregate. The extra water would increase the w/cm, thus decreasing the density and compressive strength, as discussed later.

The hardened concrete properties are summarized in Table 4. At 28 days, the NWHPC was close to the specified compressive strength, but the strength for the LWHPC was much lower. At 6 months, the NWHPC exceeded the specified strength, but the LWHPC did not. Excess water could have caused the lower compressive strength. In fact, the highest average strength recorded at the time of testing (6 months from placement) was 6,910 psi (48 MPa), with an actual average density of 114 lb/ft³ (1830 kg/m³). At transfer, the strength of the LWHPC beams was 4,780 psi (33 MPa), 15 percent lower than the specified strength of 5,600 psi (39 MPa). It is suspected that an undesirably high w/cm for the mixture and high air contents resulted in its lower strength. The elastic modulus values of the LWHPC were lower than for the NWHPC, as expected.

Structural Evaluation

Two types of test beams were used. The first type was an AASHTO Type II beam with a normal weight composite concrete deck 48 in x 8 in (1220 mm x 203 mm). Each Type II beam was prestressed with eight strands. Six strands were straight throughout the beam, and two were harped 14 ft (4.27 m) from each end of the beam. These beams were 36 ft (11 m) in length. The second type of test beam was a Type IV beam identical with the beams to be used in the Chickahominy Bridge. These beams were prestressed with 38 strands, with 30 straight strands and 8 strands harped 32 ft 8 in (9.96 m) from each end of the beam. These beams were 84 ft (25.6 m) in length.

A composite test specimen (Type II beam with cast-in-place concrete deck) was chosen to provide insight into the behavior of the actual composite LWHPC prestressed beams to be used in the Chickahominy Bridge. All decks of the test beams were NWHPC having a design 28-day compressive strength of 4,000 psi (28 MPa).

The mixture proportions for the Type II and Type IV LWHPC beams were intended to develop a concrete with a compressive strength of 8,000 psi (55 MPa) at 28 days with a density of about 120 lb/ft³ (1920 kg/m³). As stated previously, however, this mixture design did not develop its 28-day design strength. In all succeeding calculations and representations of strength in this paper, an average 28-day compressive strength of 6,375 psi (44 MPa) is considered for both the Type II and Type IV LWHPC beams.

The maximum load that could be applied during the flexural tests was limited by the strength of the load frame. The load frame (Figure 2) was designed to withstand the maximum anticipated load that could be carried by the test beams based on the beams' calculated flexural capacity. However, the actual loads applied to the beams exceeded the design loads by as much as 30 percent. As a result, two of the six tests were stopped at a maximum load of 285 kips (1,268 kN) before failure of the beam was reached, and an asterisk in Table 6 notes these tests. Table 6 also presents the embedment length results for each of the flexural strength tests.

Table 6 summarizes the findings of the development length and flexural strength testing and presents a comparison of the measured flexural strength and theoretical flexural strength. The theoretical flexural strength (M_{AASHTO}) was calculated in accordance with the AASHTO LRFD specifications, as was the theoretical development length, $L_d [(f_{ps} - 2/3f_{se}) d_b]$.

Table 6 clearly shows that all the specimens tested exceeded the design strength predicted by the AASHTO equation. This could be interpreted to indicate that each beam was fully developed at the tested strand embedment length. A more stringent criterion for evaluating the test results is to use the observed failure mode (given in the fifth column of the table) to determine if the strand was fully developed at a particular embedment length. Based on the observed failure mode, both LWHPC and NWHPC specimens were fully developed at embedment lengths of 72 in (1829 mm), which is less than the AASHTO calculated development length as shown in Table 6. Accordingly, it could be argued that estimation of the "development length" as defined by AASHTO to enable a beam to develop its "design strength" is conservative for both normal weight and lightweight prestressed concrete beams of this study.

Prestress Losses

Prestress losses in pretensioned concrete members are composed of instantaneous and long-term losses. Instantaneous losses are caused by elastic shortening of the concrete and by slip of strands at the transfer of prestress. Long-term losses are due to creep and shrinkage of concrete and relaxation of steel over time.

The two AASHTO Type IV beams were fitted with internal vibrating wire gages at midspan, which measured concrete strains at the center of gravity of the prestressing strands. Three vibrating wire gages were installed in each beam to ensure consistent measurements. A data logger recorded strain readings from the gages at 2-hour intervals. Prestress loss determined directly from the concrete strains throughout a 9-month period was compared to that derived from the ACI and PCI models.

Concrete strains at the level of the strands are a function of prestress, eccentricity of the strands, and the gravity loading at the critical section. Because of a low modulus of elasticity and relatively low strength at detensioning (4780 psi [33 MPa]), the elastic shortening loss was relatively high at 20.5 ksi (141.3 MPa), i.e., 10 percent of the initial jacking stress of 205 ksi (1413 MPa). The overall prestress loss

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estimated for the LWHPC Type IV girders over a 9-month period was 28.65 ksi (197.2 MPa) using the ACI model and 16.20 ksi (111.7 MPa) using the PCI model. Creep constituted 55 to 63 percent of the calculated losses, and shrinkage about 33 to 36 percent. The actual compressive strains measured at the level of the strands in the LWHPC Type IV girders were less than that estimated by using either the PCI or the ACI model as shown in Figure 3. The ACI model, which is the more conservative, predicts an effective prestress of about 143 ksi (986 MPa) at the end of service of the prestressed beams. That is to say, the final effective prestress would be about 53 percent of the ultimate strength of the strands. This implies that it is safe to use the AASHTO equations to determine the stress in the strands and subsequently the strength of the LWHPC prestressed bridge girders, since the retained prestress in the members can safely be assumed to be greater than 50 percent fpu as stipulated in AASHTO.

CONCLUSIONS

The measured transfer length (averaged for all beam ends measured) for the LWHPC prestressed beams was lower than the estimated value given in the AASHTO LRFD specification. The actual transfer length measured at any one end of a beam varied considerably; however, even the largest measured transfer length was less than the AASHTO estimate, indicating that the AASHTO estimate over predicts the transfer length for this LWHPC.

Testing of Type II beams to flexural failure at various embedment lengths indicated that the strand could be fully developed at an embedment length below the required development predicted by the AASHTO LRFD specification. This indicates that the AASHTO development length is conservative for this LWHPC mixture.

The elastic shortening loss of Type IV beams was relatively high. The actual compressive strains measured at the level of strands in the LWHPC Type IV beams over a 9-month period were less than that estimated by the PCI and ACI models, which indicates that the models are conservative in estimating the prestress losses in LWHPC. It is safe to use the AASHTO equations to determine the stress in the strands and subsequently the strength of the LWHPC prestressed bridge beams, since the retained prestress in the members can safely be assumed to be greater than 50 percent fpu as stipulated in AASHTO.

Upon completion of this testing, the Chickahominy River Bridge was constructed using AASHTO Type IV beams similar to those that were tested. The 28-day compressive strengths were near or above the target value of 8,000 psi (55 MPa). The bridge was instrumented to record prestress losses over the life of the bridge. The next phase of the current research with LWHPC is to compare these losses to those predicted by current models and make recommendations for determining the losses in LWHPC in Virginia.

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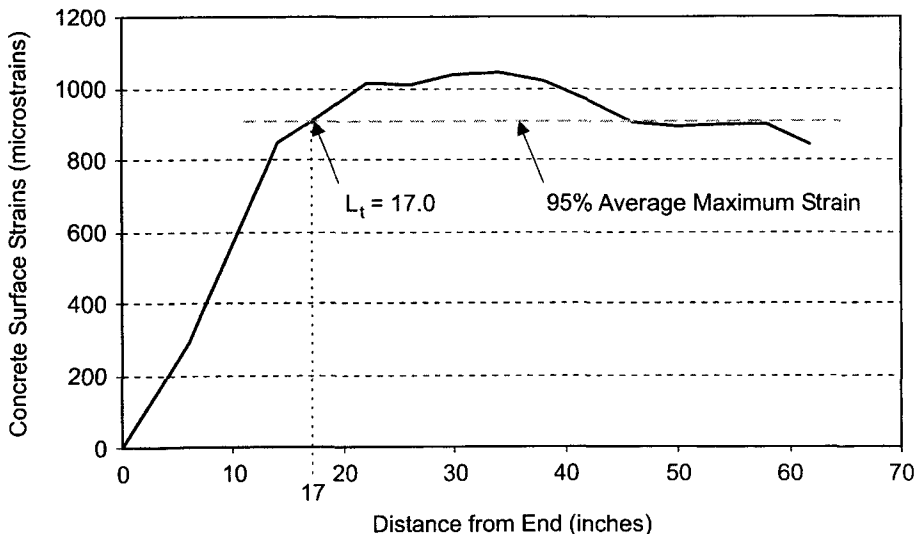


Figure 1. Average strain data profile for Type IV beams (1 in = 25.4 mm)

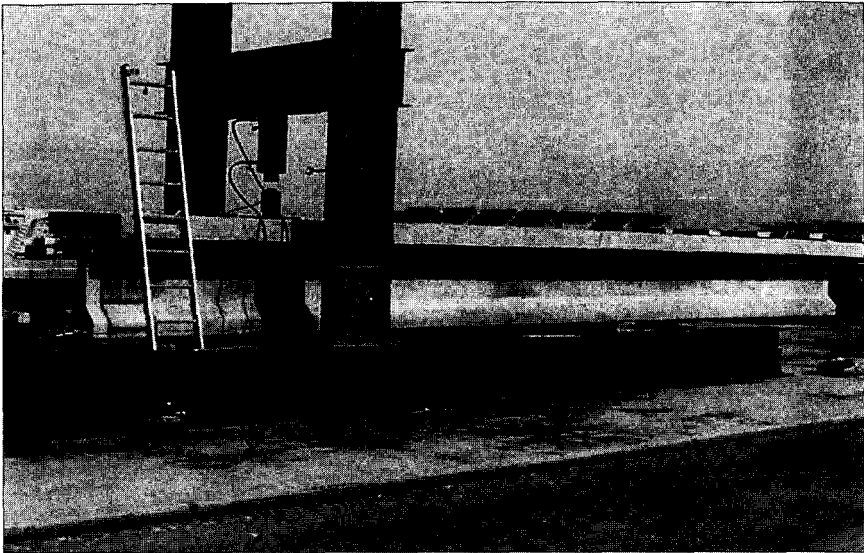


Figure 2. Load frame

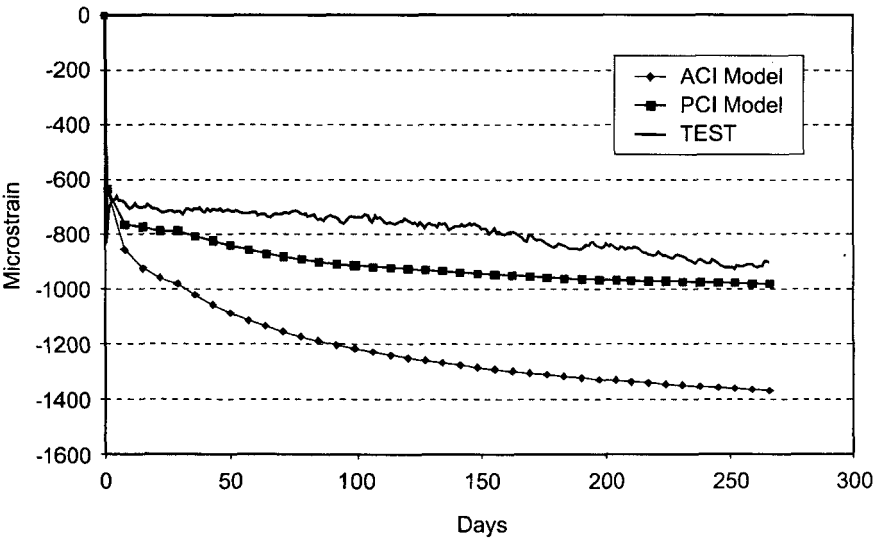


Figure 3. Concrete strains over time

Transfer and Development Length of 0.6-inch Strand in High Strength Lightweight Concrete

by K. F. Meyer and L. F. Kahn

Synopsis: This paper presents the findings of a research project conducted at Georgia Tech that tested six pretensioned AASHTO Type II girders constructed using expanded slate lightweight concrete with design strengths of 8,000 and 10,000 psi (55.2 and 68.9 MPa). Actual strengths ranged from 8,790 to 11,010 psi (60.6 to 75.9 MPa). Each was prestressed using 0.6-inch (15.2-mm) diameter low relaxation strands tensioned to 75% of strand ultimate stress. External strain measurements showed transfer lengths of 21.9 inches (556 mm) and 15.6 inches (396 mm) for the 8,000 and 10,000 psi (55.2 and 68.9 MPa) concretes; these were 73 percent and 52 percent of the design values given by AASHTO 16th Edition. Three-point bending tests were conducted on each beam to determine development length characteristics. The distance from the beam end to the load point was varied from between 70 and 100 percent of the AASHTO specified development length. Strand slip was measured for each test. Results indicated that the development lengths were 91 inches (2.31 m) and 67 inches (1.70 m) for the 8,000 and 10,000 psi (55.2 and 68.9 MPa) concretes; these were 95 percent and 70 percent of the design development lengths given by AASHTO 16th Edition.

Keywords: development length; direct pull-out; high-strength concrete; lightweight concrete; prestressed concrete; prestressing strand slip; pretensioned girders; silica fume; transfer length

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Lieutenant Colonel Karl F. Meyer is an Associate Professor in the Department of Civil and Mechanical Engineering at West Point. He earned his Doctorate in Civil Engineering at the Georgia Institute of Technology where he conducted research on pretensioned high strength lightweight concrete bridge girders. He is an associate member of the American Concrete Institute Committee 213, and a member of Committee E802, Teaching Methods and Educational Materials.

Lawrence F. Kahn, FACI, is an Associate Professor in the School of Civil and Environmental Engineering at the Georgia Institute of Technology. His research interests include prestressed concrete and structural retrofit techniques. He is currently a member of the American Concrete Institute Committees TRRC, CRC, 364 and 546.

INTRODUCTION

Advances in concrete quality and engineering practices have enabled the design and construction of precast prestressed concrete bridge girders over 150 feet (45.7 m) in length. High-strength lightweight concrete (HSLC) can be used to reduce member self-weight in order to facilitate easier road movement and erection.

Previous research conducted relating to lightweight concrete (LWC) used in prestressed applications involved concrete with compressive strengths less than 7,500 psi (51.7 MPa). The research presented herein was designed to extend the application of lightweight concrete to design strengths up to 10,000 psi (68.9 MPa) and to verify the application of current design standards to the higher strength materials. An earlier analytical study showed that the use of HSLC would be beneficial for extending the lengths of bridge girders (1).

BACKGROUND

Transfer Length

Transfer length is defined as the distance required to transfer the effective prestressing force from the strand to the surrounding concrete. The transfer length is developed when the pretensioning strands are released by flame cutting or other method. Numerous factors are thought to contribute to determining the transfer length including size and surface condition of the prestressing strand, concrete strength and modulus of elasticity at time of strand release, level of prestressing in the strand, the amount of confining steel, and concrete consolidation in the transfer length region. Many experimental programs have focused on the above factors in determining an expression to predict transfer length (2-12).

Initial transfer length testing by Janney (2) in 1954 concluded that transfer length was attributable to diameter and surface condition of the prestressing wire and concrete strength.

Hanson and Kaar's (3) work in 1959 found an average transfer bond stress of 400 psi (2.75 MPa). Equation 1 was proposed as a way to determine the transfer bond stress.

$$u_t = \frac{A_{ps} f_{se}}{\Sigma_0 l_t} \quad (1)$$

A_{ps} is the area of prestressing strand, f_{se} is the effective prestressing stress at transfer, Σ_0 is the actual perimeter of the prestressing strand, l_t is the transfer length and u_t is the transfer bond stress in psi.

In 1962, Mattock (4) used Equation 1 with an assumed average transfer bond stress of 400 psi to derive the current ACI (13) equation for transfer length. The factors 0.725 and 4/3 were used to determine the surface area and perimeter, respectively.

$$l_t = \frac{\left(0.725 \frac{\pi d_b^2}{4}\right) f_{se}}{\left(\frac{4}{3} \pi d_b\right) 0.4} = \frac{1}{2.94} f_{se} d_b \quad (2)$$

Equation 2 was simplified to Equation 3, which is the current ACI expression.

$$l_t = \frac{f_{se} d_b}{3} \quad (3)$$

Mattock (4) assumed the effective stress, f_{se} , for 250 ksi (1.72 GPa) strand was 150 ksi (1.38 GPa) and further simplified Equation 3 to Equation 4, which was adopted by ACI in 1963 and AASHTO (14) in 1973.

$$l_t = 50 d_b \quad (4)$$

In 1963, Kaar et al. (5) concluded that transfer length varied directly based on strand diameter for 1/4-inch (6.35-mm) and 1/2-inch (12.7-mm) strands and found that concrete strength did not impact transfer length. Higher concrete strength allowed the concrete to pick up stress more quickly when compared to lower strength specimens. Kaar et al. also reported differences in transfer length between “dead” and “cut” ends.

AASHTO LRFD (15) has increased l_t to $60d_b$.

Numerous researchers (5-12) have suggested other equations for predicting transfer length that include terms to address characteristics of the concrete including strength and modulus of elasticity. To date, no equations have been suggested to predict transfer length that specifically address HSLC.

Development Length

Development length of prestressing strands is the sum of the transfer length and the flexural bond length. Development length can be defined as the minimum distance from the end of the member beyond which the application of a point load will result in a

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flexural failure rather than a bond failure. As in transfer length, many factors are thought to affect development length. Many experimental programs have addressed development length resulting in suggested equations for its prediction (6-8, 10, 11, 12, 16).

Initial development length testing by Janney (2) in 1954 was based on the use of 5/16-inch (7.94 mm) wire having two different surface conditions and prestressed to 0, 60, 120 and 165 ksi (0, 414, 818 and 1137 MPa) in beams made with normal weight concrete (NWC) having 28-day strengths from 4,500 to 4,800 psi (31.0 to 33.1 MPa). Janney reported a “wave of flexural bond stress concentration” and noted that once general bond slip occurred, the beam failed shortly thereafter. Janney concluded that bond in the transfer region was due predominantly to the Hoyer effect, which was the confining pressure applied to the wire by the concrete. Janney also reported improved bond characteristics for slightly rusted wire.

In 1959, Hanson and Kaar (3) reported a study in involving 47 small-scale concrete beams reinforced with various sizes of Grade 250 stress-relieved strand. The tests focused on five factors including strand diameter, embedment length, concrete strength, percentage of reinforcement, and strand surface condition. All beams were loaded to failure. Hanson and Kaar reported observations similar to those of Janney (2). The results of Hanson and Kaar were the basis for the current AASHTO development length expression developed by Mattock (4) and shown in Equation 5. It is important to note that Equation 5 is based on Grade 250 prestressing strand, not Grade 270 prestressing strand commonly used in present day.

$$l_d = (f_{su}^* - \frac{2}{3} f_{se}) d_b \quad (5)$$

Researchers have also suggested other equations for predicting development length that include terms to address concrete strength and prestressing strand stress values at various times (6-8, 10, 11, 12, 16). To date, there have been no equations suggested to specifically predict development length for HSLC.

EXPERIMENTAL PROGRAM

Six AASHTO Type II composite girders were constructed with ten 0.6-inch (15.2 mm) diameter 270 ksi (1.86 GPa) low relaxation strands as illustrated in Figures 1 and 2. Three girders were made with an 8,000 psi (55.2 MPa) design strength expanded slate HSLC and three girders were made with a 10,000 psi (68.9 MPa) design strength expanded slate HSLC mix. The 8,000 psi (55.2 MPa) HSLC was termed a Grade 1 mix while the 10,000 psi (68.9 MPa) HSLC was termed a Grade 2 mix. The eight bottom and two top strands were stressed to $0.75f_{pu}$ (203 ksi, 1.40 GPa) prior to release. The girders were cast in pairs on three different days as listed in Table 2. Note that girders G1C and G2C were cast on the same date.

The 39-foot (11.9-m) and 43-foot (13.1-m) long girders were designed so that point loads could be placed near each end for development length tests and so that the middle section could be subsequently tested to examine shear and flexural capacity.

A composite deck with a thickness of 11.5 inches (292 mm) and width of 19 inches (483 mm) was placed on each girder. The composite deck was designed to give the same

internal flexural moment arm “jd” as in a composite bridge with an 8-inch (20.3-cm) thick deck and an effective width of 93 inches (2.36 m), but to ease construction by having the deck with a width as close to the width of the top flange of the girder as possible. A 3,500 psi (24.1 MPa) normal weight bridge mix was used for the composite deck.

The test configurations are given in Table 1. Figure 3 illustrates the test set-up.

Shear reinforcement in the development length region of each girder end was designed to satisfy AASHTO and ACI provisions and was termed “single density.” In some girders, twice this required shear reinforcement was included, termed “double density,” in order to study the effect of shear reinforcement on strand development. Stirrups were two “C” shaped # 4 Grade 60 bars ($f_y = 62$ ksi, 427 MPa).

Expanded slate HSLC mix designs having design strengths of 8,000 psi (55.2 MPa) and 10,000 psi (68.9 MPa) developed in the laboratory and field-tested earlier in the research project were used to construct the girders. The coarse aggregate in both mix designs consisted of expanded slate lightweight aggregate from North Carolina. Table 2 provides details on the mix components for each girder, the date of casting, and the 1-day, 56-day, and “on day of test” compressive strengths. The G1 and G2 series girders refer to girders constructed with the 8,000 psi (55.2 MPa) and 10,000 psi (68.9 MPa) design strengths, respectively.

TRANSFER LENGTH

Measurement of Transfer Length

Transfer length data for the bottom prestressing strands were measured using the Concrete Surface Strain (CSS) method. As the prestressing strands transferred stress to the concrete, compressive stress and thus strain was induced in the concrete. Based on compatibility, the point at which the concrete strain reaches a maximum value indicates the transfer length.

To collect transfer length data for the bottom strands, detachable mechanical (DEMEC) gage embeddings were installed at 2-inch (50.8-mm) spacing at the level of the bottom strands over a 48-inch (1.22-m) distance from each girder end on each side of the girder. The 8-inch (203-mm) gage length device with its 0.0001-inch (0.0025-mm) accuracy produced a sensitivity of 12.5 microstrains in each strain measurement.

CSS measurements were taken on each of the six girders prior to strand release (2 readings), just after strand release, and at 1, 2, 3, 7, and 14 days after strand release.

Except for the “initial” and “just after release” readings, CSS readings were taken just prior to sunrise before the girders were exposed to direct sunlight to avoid surface temperature variations. Results in this research indicated stabilization of surface strains as soon as 7 days after release.

Determination of Transfer Length

Determination of the transfer length was a several step process. A spreadsheet was created which allowed entry of the raw transfer length data. The spreadsheet calculated raw strains on each side of the girder, averaged the raw strain values between the sides, and then smoothed the averaged raw strain values using a floating three point average. From the smoothed strain profile, the transfer length was determined using the 95 Percent

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Average Maximum Strain Method (95% AMS). Both Buckner (11) and Russell (9) recommended this method as a conservative and objective technique for determining transfer length. Figure 4 shows the smoothed 14-day CSS profile for girder end G1C-East with the transfer length determined using the 95% AMS technique.

The magnitude of the constant strain plateau was determined by taking 95 percent of the average of the strain values measured at greater than 30 inches (762 mm) from the girder end. Evaluation of the 12 girder end CSS plots showed that the constant strain plateau began at a point approximately 30 inches (762 mm) from the girder end in each case. The 30-inch (762-mm) distance was chosen for consistency and provided an excellent prediction overall of the strain plateau for all specimens.

The initial linear trend line was visually fitted to the initial sloped portion of the averaged, smoothed strain profile. The intersection of the initial linear trend line with the 95% AMS line was defined as the transfer length. Table 3 lists the measured transfer lengths from each girder end. The asterisk for three of the values indicates the girder ends having over 25 feet (7.62 m) of free strand between the girder end and the abutment. Measured transfer lengths on those free ends were substantially longer in comparison to ends having only a small amount of free strand. This finding has been reported by others (9).

Effect of Concrete Strength

A single variable statistical analysis produced the results shown in Table 4 for the two design strengths of concrete. The average transfer length for the G1 series girders was 6.3 inches (160 mm) longer than that of the G2 series girders indicating a relation between concrete strength and transfer length.

Concrete strength affected material properties and the amount of change in prestress from release until all losses had occurred. For the purpose of examining transfer length, this study focused on concrete strength and modulus of elasticity. Indirectly, however, the strength of concrete impacted the resulting initial level and effective level of prestress. Initial elastic shortening was affected by the modulus of elasticity of the concrete at release. Creep shortening over the period of time from release until the transfer length was measured and was directly related to the concrete strength. The G2 mixes, in general, underwent less creep. Surface strain and internal vibrating wire strain gage (VWSG) measurements at girder midspan were used to evaluate prestress loss.

Comparison of Transfer Length Results with Values Predicted by Code Equations

A comparison of current code provisions with the 12 HSLC transfer lengths obtained in this research showed the current ACI (13) and AASHTO (14) equations (Equations 3 and 4) to be conservative as presented in Table 3. The AASHTO equation overestimated transfer lengths by 37% on average and never underestimated transfer lengths. The ACI equation using VWSG data for strand stress overestimated transfer lengths by 41% on average and never underestimated transfer lengths. Use of either the ACI or AASHTO equations to predict transfer length for expanded slate HSLC was conservative. Based on the concrete strength range addressed in this research project, modification of the current code specifications for transfer lengths was not necessary for expanded slate HSLC.

DIRECT PULL-OUT TEST

During girder placement, 24-inch x 36-inch x 24-inch (610-mm x 914-mm x 610-mm) blocks were cast from the G1 and G2 series mixes for the prestressing strand direct pull-out test in order to assure good quality surface condition of the prestressing strands. Four 48-inch (1.22-m) segments of strand were embedded 20 inches (508 mm) into the block with the top 2 inches (25.4 mm) being encased by PVC pipe as a bond breaker. The total embedded length was 18 inches (457 mm). The minimum dimension between strands was 8 inches (203 mm).

The determination of pull-out capacity was based on the point at which the strand experienced increased slip with little or no increase in load or the point at which the strand underwent a sudden “pop” resulting in a dramatic increase in slip and drop in load. This “popping” occurrence was common for all 8 tests performed. Table 5 provides direct pull-out test results; the value listed is the load at which the sudden “pop” occurred.

Based on Logan’s (17) work, the recommended average pull-out capacity for 0.5-inch (12.7-mm) strand is 36 kips (160 kN) which corresponds to an average bond stress of 955 psi (6.58 MPa) over the 18-inch (457-mm) embedment of the strand. The same average bond stress applied to a 0.6-inch (15.2-mm) strand over the 18-inch (457-mm) embedment would require a minimum direct pull-out force of 43.2 kips (192 kN). The average pull-out strength for both series exceeded this minimum value. One G2 test was slightly below the 43.2-kip (192-kN) minimum. The pull-out tests showed that the surface condition of the 0.6-inch (15.2-mm) strands was good and that the transfer and development length results are reliable.

DEVELOPMENT LENGTH

Determination of Development Length

Each end of the six Type II girders was tested for development length producing 12 sets of data. Girder end designations were the same as used in transfer length testing. In order to determine development length, the position of the point load was varied from girder to girder as detailed in Tables 1 and 6. The precise development length would be the location at which the point load resulted in concurrent flexural and bond failure. If the failure mode were purely flexural, the tested embedment length was greater than the development length. If the failure mode were bond or bond/shear, the tested embedment length was less than the development length. The load was placed at several locations between 70 and 100 percent of the predicted development length to bracket the actual development length. The embedment length, l_e , was the shear span “a” plus the 6 inches (152 mm) from the center of bearing to the end of the girder.

Table 6 shows the embedment lengths tested as a percentage of the estimated development length values and also shows the embedment lengths tested as a percentage of the development length calculated using Equation 5 based on experimentally determined values of f_{su}^* and f_{sc} , which are listed for each girder.

Development Length Test Data and Failure Criteria

CSS data were collected during testing. The CSS data were used to track the change in the magnitude of strain in the strands beneath the load point and in the first 48 inches

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(1.22 m) of the girder. Perfect bond between the strand and concrete was assumed so that the concrete surface strains were equated to added strain in the strands. Strand yielding was assumed to occur if the total strain in the strands exceeded 1 percent. The strand's stress-strain relation was provided by the manufacturer.

Strand slip was measured continually through each test using linear potentiometers. Failure from strand slipping was defined to occur if the average slip of the 8 bottom strands during the test exceeded 0.01 inches (0.25 mm). After strand slip exceeded 0.01 inches (0.25 mm), loading was stopped. Typically the two exterior strands would slip more than interior strands if a slip failure occurred. Russell (9) and Dill (18) also used this 0.01-inch (0.25-mm) slip to define a slip condition. This type of failure was termed "shear-slip" because it was believed that the formation of a shear crack was the trigger for strand slip to begin.

The failure criteria used to classify the test results are listed below.

FL - The girder failed by "both" crushing of the deck and yielding of the strands.

SH - The girder failed purely in shear without strand slip. It did not meet the criteria for a flexural failure.

SH/FL - The girder failed predominantly in shear, but still yielded the strands and crushed the deck. The two failure states occurred almost simultaneously.

SH-SL - The girder underwent a shear failure and strand slipping.

FL/SH-SL or SH-SL/FL - The girder underwent both failure modes with one occurring more predominantly than the other. Both failures would result in yielding of the strands and crushing of the deck.

Development Length Test Results

Table 7 provides the results of development length testing including the failure modes. The failure mode of Girder G1A-East is listed as FL/SH-SL. This indicates that the girder exhibited both flexural and shear-slip modes with the flexural mode occurring most predominantly. The failure mode of Girder G1C-E is listed as SH-SL/FL. This also indicates the girder exhibited both flexural and shear-slip modes, but the shear-slip mode was predominant. Girders with a failure type FL exhibited only flexural failure characteristics. Shear cracking occurred in all tests.

In order to evaluate the development length test results, the level of strand end slip was compared with embedment length. Since only selected embedment lengths were tested, it was not possible to exactly pinpoint the development length. It was possible, however, to identify the region in which the failure transitioned from flexure to shear-slip. The upper end of the region was conservatively identified as the development length. For G1 girders with single and double density stirrups, the development lengths were 91 and 81 inches (2.31 and 2.06 m), respectively. For G2 girders, all failed in flexure; so, the tested embedment lengths were all greater than the actual development length. The minimum embedment length of 67 inches (1.70 m) was identified as the experimental development length for the G2 girders, for both single and double density stirrups.

Effect of Shear Cracking on Strand Slip

A comparison was made between the onset of shear cracking and strand slip. The comparison was accomplished by examining the plot of applied shear vs. average end slip to determine the applied shear at which slip initiated and comparing that shear value to crack locations on the girder. This comparison was made for all tests. Figure 5 shows a diagram of applied shear vs. average strand end slip for test G1A-East. The diagram has a well-defined slip initiation point.

Figure 5 also shows “unloading slip” which occurred on four of the 12 development length tests. At a maximum load condition, the support reaction applied a clamping action to the strand. As the support reaction became less during unloading, the clamping action also became less allowing additional strand slip beyond the amount reached at the point of maximum loading. There was no apparent correlation between strand slip failures and the occurrence of unloading slip.

Examination of girder end tests having single stirrup density revealed that in every case a shear crack passing through the level of the bottom strands, within the transfer length region, initiated or dramatically increased strand slip. Tests incorporating double stirrup density showed the resulting slips were much less than for the single stirrup density tests. For the G1 series girders, doubling the stirrup density dramatically reduced slip values and increased the load at which significant slip occurred, especially in tests of shorter embedment lengths. For the G2 series girders, the effect was much less pronounced; however, the increased stirrup density did reduce end slip and cause slight increases in the load at which slip occurred.

The results clearly indicated that shear cracking across the bottom strands within the transfer length region initiated large increases in strand end slip in girders using only the AASHTO or ACI specified shear reinforcement. Other researchers have reported this phenomenon in normal weight concrete (9). Despite several shear cracks in the transfer length region, tests G1B-West and G2B-West showed no slip because of the double stirrup density. Russell (9) reported that additional bond stresses can be developed beyond initial strand slip and that small slips are not always followed by complete anchorage failure. Russell (9) also reported that closely spaced cracks were an indication of good bond and wide crack spacing indicated poor bond. Both of Russell’s observations were confirmed in the present tests with expanded slate HSLC.

Effect of Concrete Strength on Strand Slip

Concrete strength also played a significant role in determining development length. The G1 series girders had compressive strengths on average approximately 1500 psi (10.3 MPa) less than the G2 series girders. The water cement ratios were approximately 0.28 for the G1 series girders and 0.23 for the G2 series girders. A significant difference between the two mix designs was the amount of silica fume. The G1 series girders had 2 percent silica fume by weight of the total cementitious materials, and the G2 series girders had 10 percent. Past researchers have stated that the addition of silica fume will significantly improve the tensile strength and bond of lightweight concrete to prestressing strand (19, 20, 21). The results of the present research confirm this observation. As well as increasing early and later compressive strength, the silica fume apparently created a much better bond between the prestressing strand and the concrete matrix.

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Comparison of Development Length Results with Values Predicted by Code Provisions

Development length test results from this research were compared with development lengths predicted by ACI (13) and AASHTO (14) code provisions to determine their adequacy for design with expanded slate HSLC. Table 8 compares experimental results with code predicted development length values. The comparison shows the current ACI (13) and AASHTO (14) provisions to be conservative. The code equations overestimated development lengths by 19 percent for expanded slate HSLC. Use of the current code provisions for design of development length for slate HSLC and 0.6-inch (15.2-mm) diameter strand was conservative. Based on the actual concrete strengths between 8,790 psi (60.6 MPa) and 11,010 psi (75.9 MPa) used in this research, modification of the current code specifications for development length was not necessary for expanded slate HSLC using 0.6-inch (15.2-mm) diameter strands.

CONCLUSIONS AND RECOMMENDATIONS

Current ACI (13) and AASHTO (14) provisions for transfer and development length of 0.6-inch (15.2-mm) prestressing strand are conservative for slate HSLC. The AASHTO (14) equation overestimated transfer lengths by 37% on average and never underestimated transfer lengths. The ACI (13) equation with f_{se} determined using VWSG data overestimated transfer lengths by 41% and never underestimated transfer lengths. Based on the HSLC strengths between 8,790 psi (60.6 MPa) and 11,010 psi (75.9 MPa) used in this research project, modification of the current code specifications for transfer lengths is not necessary for expanded slate HSLC.

The code equations overestimated development lengths by 19 percent for expanded slate HSLC and never underestimated them. Based on the HSLC strengths between 8,790 psi (60.6 MPa) and 11,010 psi (75.9 MPa) used in this research, modification of the current code specifications for development length was not necessary for expanded slate HSLC using 0.6-inch (15.2-mm) diameter strands.

Test results showed conclusively that shear cracking in the transfer length region across the bottom strands did not induce significant strand slip if stirrup density were doubled over the current AASHTO (14) specified stirrup spacing in the transfer length region. Further investigations and code committees should consider doubling the shear reinforcement requirement in the transfer length region in order to limit strand slip in the event of shear cracking.

The addition of silica fume into the mix design at a 10% replacement rate by weight of cementitious materials appeared to reduce development length. Further research should investigate the effectiveness of silica fume for increasing bond.

This research examined the use of expanded slate HSLC. Similar research is required for each type of structural lightweight aggregate to verify compliance or non-compliance with current ACI and AASHTO code provisions.

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LIST OF NOTATIONS

Report	ACI 318-99	AASHTO Standard	Units	Description
A_{ps}	A_{ps}	A_s^*	inch ²	Cross Sectional Area of Prestressing Strand
CSS	--	--		Concrete Surface Strain
d_b	d_b	D	inch	Diameter of prestressing strand
DEMEC	--	--		Detachable Mechanical Strain Gage
f_c'	f_c'	f_c'	psi	Concrete compressive strength at specified time
f_{ci}'	f_{ci}'	f_{ci}'	psi	Concrete compressive strength at strand release
f_{ps}	f_{ps}	f_{su}^*	ksi	Stress in prestressed reinforcement at nominal strength of member
f_{pu}	f_{pu}	f_s'	ksi	Specified tensile strength of prestressed reinforcement
f_{se}	f_{se}	f_{se}	ksi	Effective prestressing stress after losses
f_y	f_y	f_{sy}	ksi	Specified yield strength of non-prestressed reinforcement
h	h	h	inch	Overall depth of the composite girder
l_d	l_d	--	inch	Development length of prestressing strand (In AASHTO Standard, l_d refers to non-prestressed reinforcement development length)
l_e	--	--	inch	Embedment length of prestressing strand
l_t	--	--	inch	Transfer length of prestressing strand
VWSG	--	--		Vibrating Wire Strain Gage

Table 1 - Girder Test Configurations

Test Configuration	Stirrup Density	Stirrup Spacing (in)	Shear Span "a" (in)	Distance "L ₁ " (in)	Distance "L" (in)
1	Single	7	90	456	456
2	Double	3.5	61	316	456
3	Double	3.5	75	456	456
4	Single	7	85	504	504
5	Single	7	61	321	456
6	Single	7	75	369	504

Note: 1 inch = 25.4 mm

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Table 2 – Expanded Slate HSLC Mixes for Girder Placements (Quantities per Cubic Yard Mix)

Material / Girder #	G1A, G1B	G2A, G2B	G1C	G2C
Design Strength (psi)	8,000	10,000	8,000	10,000
Date of Casting	9 Jul 01	12 Jul 01	17 Jul 01	17 Jul 01
Lightweight Aggregate (lbs)	989	979	974	998
Normal Weight Sand (lbs)	1098	1087	1078	1083
Class F Fly Ash (lbs)	142	150	142	150
Silica Fume (lbs)	19	100	19	100
Type III Portland Cement (lbs)	785	740	785	740
Water Reducer (WRDA 35) (fl oz)	57	60	57	60
Air Entrainment (Daravair 1000) (fl oz)	12	10	12	10
HR Water Reducer (ADVA Flow) (fl oz)	48	129	48	129
Water (lbs)	165	153	186	138
Water/Cementitious Materials Ratio	0.28	0.23	0.28	0.23
1-day f_c' (average, accelerated curing) (psi)	7,470	9,640	6,320	8,260
28-day f_c' (average, ASTM curing) (psi)	8,840	10,120	7,600	9,810
56-day f_c' (average, ASTM curing) (psi)	9,350	10,820	8,460	10,510
f_c' on day of girder test (psi)	10,230	11,010	9,120	10,870
Girder age at test (days)	103	123	110	144

Note: 1000 psi = 1 ksi = 6.9 MPa, 1 lb = 4.45 N, 1 fluid oz = 29.6 ml

Table 3 – Comparison of Experimental Transfer Length with Code Predicted Values

Girder End	Experimental l_t (in)	AASHTO Predicted l_t (in)	Percent Diff. AASHTO vs. Exp.	f_{se} at 14-days ** (ksi)	ACI Predicted l_t (in)	Percent Diff. ACI vs. Exp.
G1A-East	19.50	30	35.0	154.7	30.9	37.0
G1A-West	18.75	30	37.5	154.7	30.9	39.4
G1B-East	25.00 *	30	16.7	153.3	30.7	18.5
G1B-West	18.75	30	37.5	153.3	30.7	38.8
G1C-East	28.00 *	30	6.7	153.7	30.7	8.9
G1C-West	21.50	30	28.3	153.7	30.7	30.1
G2A-East	17.50 *	30	41.7	176.5	35.3	50.4
G2A-West	13.25	30	55.8	176.5	35.3	62.5
G2B-East	13.00	30	56.7	176.5	35.3	63.2
G2B-West	13.00	30	56.7	176.5	35.3	63.2
G2C-East	19.00	30	36.7	159.8	32.0	40.6
G2C-West	18.00	30	40.0	159.8	32.0	43.7
		Average	37.4		Average	41.3

Note: 1 inch = 25.4 mm, 1 ksi = 6.9 MPa

* Free end of strand

** Based on VWG data

Table 4 – Statistical Analysis of Transfer Length Data

Girder Series	f_c' Design Strength (psi)	f_{ci}' Design Strength at Release (psi)	Average f_{ci}' Actual Strength at 24 Hours (Time of Release) (psi)	Average Transfer Length l_t (in)	Standard Deviation of l_t (in)
G1	8,000	6,000	7,080	21.9	3.81
G2	10,000	7,500	9,180	15.6	2.83

Note: 1000 psi = 1 ksi = 6.9 MPa, 1 inch = 25.4 mm

Table 5 – Direct Pull-out Test Results

Strand Test #	G1 Series (kips)	G2 Series (kips)
1	47.43	43.82
2	48.22	49.52
3	50.75	42.81
4	51.47	54.36
Average	49.47	47.63

Note: 1 kip = 4.45 kN

Table 6 – Embedment Lengths Tested as Percentages of AASHTO Predicted Development Length Values

Girder End	Configuration and Stirrup Density	l_e (in)	l_e / l_d Based on Estimated Values of f_{su}^* and f_{se}	Actual Values		l_d Based on Actual Values of f_{su}^* and f_{se} (in)	l_e / l_d Based on Actual Values of f_{su}^* and f_{se}
				f_{su}^* (ksi)	f_{se} (ksi)		
G1A-East	2 – Double	67	70 %	265.1	153.7	97.6	69 %
G1A-West	1 – Single	96	100 %	265.4	152.3	105.9	91 %
G1B-East	5 – Single	67	70 %	227.6	151.2	106.3	63 %
G1B-West	3 – Double	81	84 %	266.3	149.8	106.9	76 %
G1C-East	6 – Single	81	84 %	263.0	149.0	107.2	76 %
G1C-West	4 – Single	91	95 %	264.4	147.6	107.8	84 %
G2A-East	2 – Double	67	70 %	266.3	175.8	96.5	70 %
G2A-West	1 – Single	96	100 %	266.8	175.7	96.5	100 %
G2B-East	5 – Single	67	70 %	265.0	176.2	96.3	70 %
G2B-West	3 – Double	81	85 %	266.6	175.6	96.6	84 %
G2C-East	6 – Single	81	85 %	267.7	159.3	103.1	79 %
G2C-West	4 – Single	91	95 %	266.6	158.8	103.3	88 %

Note: 1 inch = 25.4 mm, 1 ksi = 6.9 MPa

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Table 7 – Development Length Test Results

Girder End	Stirrup Density	Strand Embedment (in)	Average Strand Slip at P_{max} (in)	Maximum Strand Strain ϵ_{ps} (in/in)	Maximum Deck Strain ϵ_{cu} (in/in)	Failure Mode FL-Flexure SH-Shear SL-Strand Slip
G1A-East	Double	67	0.0102	0.0158	0.0038	FL/SH-SL
G1A-West	Single	96	0.0000	0.0168	0.0042	FL
G1B-East	Single	67	0.7350	0.0078	0.0030	SH-SL
G1B-West	Double	81	0.0007	0.0190	0.0032	FL
G1C-East	Single	81	0.1988	0.0113	0.0032	SH-SL/FL
G1C-West	Single	91	0.0000	0.0140	0.0036	FL
G2A-East	Double	67	0.0033	0.0190	0.0036	FL
G2A-West	Single	96	0.0000	0.0204	0.0072	FL
G2B-East	Single	67	0.0065	0.0155	0.0045	FL
G2B-West	Double	81	0.0000	0.0199	0.0064	FL
G2C-East	Single	81	0.0041	0.0228	0.0045	FL
G2C-West	Single	91	0.0000	0.0198	0.0049	FL

Note: 1 inch = 25.4 mm

Table 8 – Comparison of Experimental Development Length with Code Predicted Values

Girder End	Experimental l_d (in)	AASHTO, ACI Predicted l_d (in)	Percent Difference AASHTO, ACI vs. Experimental
G1A-East	81	98.0	17.3
G1A-West	91	98.8	7.9
G1B-East	91	99.2	8.3
G1B-West	81	99.6	18.7
G1C-East	91	100.0	9.0
G1C-West	91	100.4	9.4
G2A-East	67	89.2	24.9
G2A-West	67	89.2	24.9
G2B-East	67	89.2	24.9
G2B-West	67	89.2	24.9
G2C-East	67	96.0	30.2
G2C-West	67	96.0	30.2
		Average	19.2

Note: 1 inch = 25.4 mm

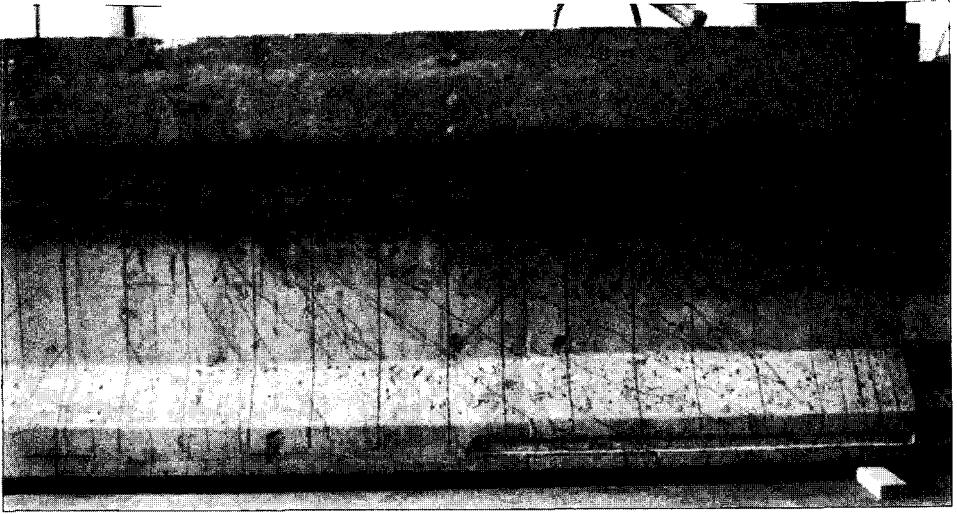
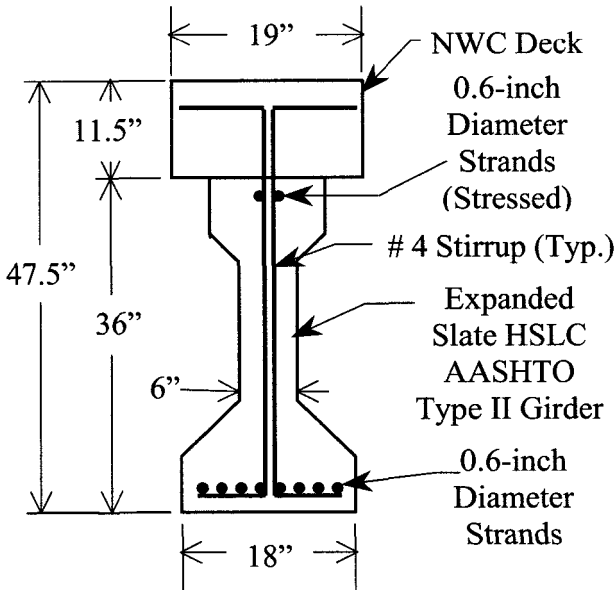


Figure 1 – East End of Girder G1C after Development Length Testing



Note: 1 inch = 25.4 mm

Figure 2 – Composite Girder Cross Section

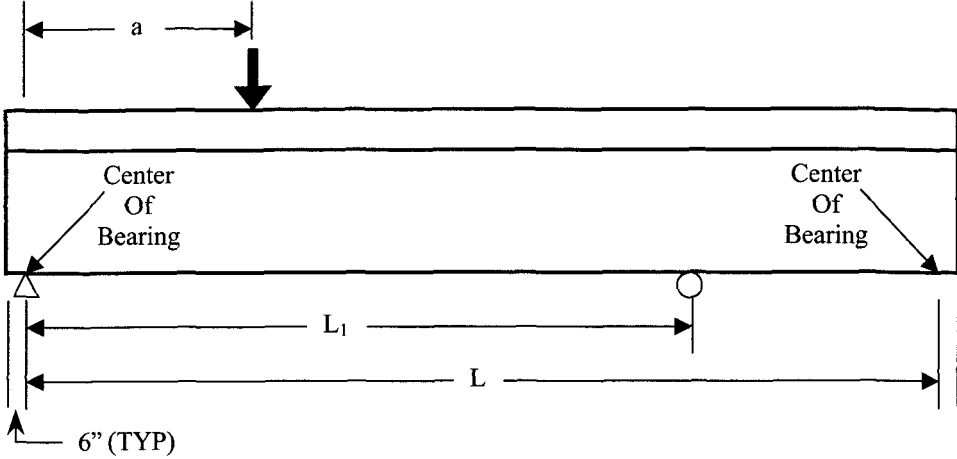
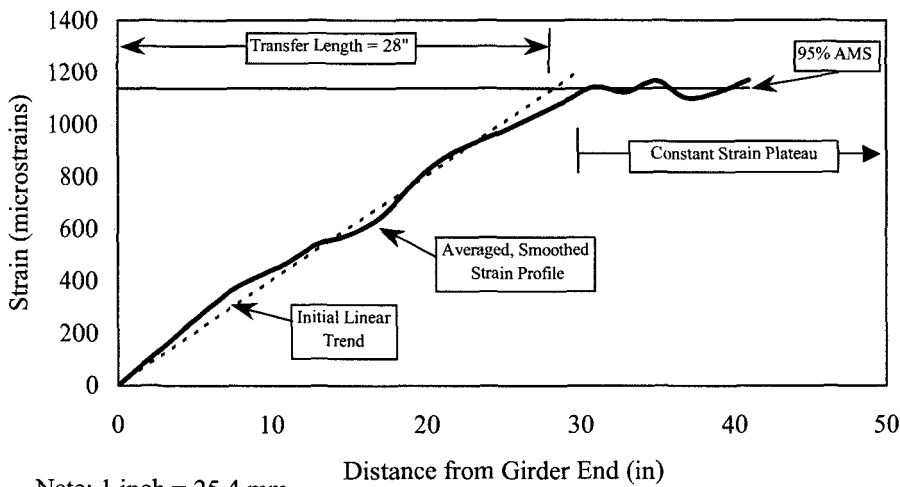


Figure 3 – Girder Layout Dimensions



Note: 1 inch = 25.4 mm

— Smoothed 14-Day CSS - - - 95% AMS Initial Linear Trend

Figure 4 – Transfer Length Determined Using 95 Percent AMS Method for G1C-East

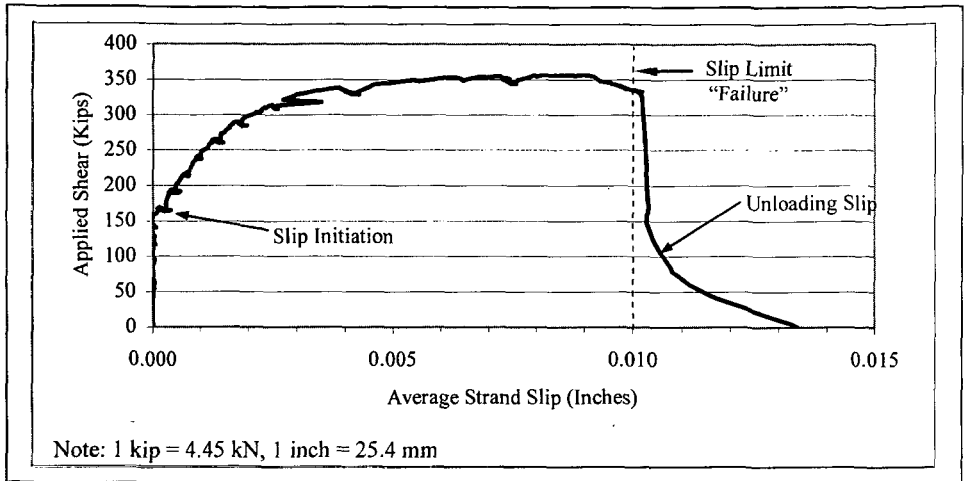


Figure 5 – Applied Shear vs. Average Strand Slip (G1A-East) Showing Slip Initiation

Review of Parameters Influencing the Seismic Design of Lightweight Concrete Structures

by M. J. Kowalsky and H. M. Dwairi

Synopsis: This paper discusses four aspects related to the seismic behavior of structures composed of lightweight concrete. Following a brief discussion of the merits of lightweight concrete, the past seismic performance of the material is discussed. The second portion of the paper discusses research that has been conducted on the seismic behavior of lightweight concrete structures and research on parameters that influence the seismic behavior of lightweight concrete. This is followed by a discussion of various existing code-based provisions that have an effect on the seismic design of lightweight concrete structures. Lastly, the paper concludes with a discussion of future research needs.

Keywords: ductility; flexure; lightweight concrete; seismic; shear

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Mervyn J. Kowalsky is an assistant professor in the Department of Civil Engineering at North Carolina State University. His research interests are in the area of earthquake engineering and seismic design of concrete and masonry structure. Dr. Kowalsky is a member of ACI Committees 213 (Lightweight Concrete) and 374 (Performance-Based Design).

Hazim M. Dwairi is a Ph.D. student in the Department of Civil Engineering at North Carolina State University. His research is focused on the development of displacement-based design methods for bridge structures.

INTRODUCTION

The motivation for reducing weight of constructed systems dates back several thousand years. In the case of lightweight concrete, early applications include portions of the Pantheon which were constructed with pumice aggregate concrete [1]. With the advent of artificially made lightweight aggregate early in the 20th century, it became possible to obtain lightweight aggregate concrete with compressive strengths similar to normal weight concrete. This type of lightweight concrete has been used recently for the design of building structures as well as for roadways surfaces and bridge decks, and in a more limited role for entire bridge superstructures.

The primary advantage that lightweight concrete accords the engineer is a reduction in weight. In bridge design, this means reduced gravity load and seismic inertial mass, resulting in reduced member sizes and foundation forces. Generally, only superstructures are constructed of lightweight concrete with the substructure in normal weight concrete. For building structures, the reduction in storey mass has a significant effect on the seismic response of the structure. The largest weight savings comes from the use of lightweight concrete in the flooring system. Additional benefits, however, can be achieved by utilizing lightweight concrete for beam and column members.

It is felt that the advantages that lightweight concrete provides the engineer can be further enhanced by using the material for the design of bridge substructure elements such as cap-beams, joints, and columns. While the mass of the superstructure is generally much more significant than the column masses, there are cases such as deep narrow valley crossings or river crossings with navigation channels where the pier height results in the column mass becoming significant when compared to the superstructure mass. Also, where span lengths are comparatively small, the column mass can become significant. Lightweight concrete can also be used in retrofit applications where a concrete column jacket is desired due to architectural reasons over other methods such as steel or advanced composite jackets. In that situation, a conventional weight concrete jacket might result in foundation forces that would require expensive foundation retrofit whereas a lightweight concrete jacket may not require a footing retrofit.

SEISMIC BEHAVIOR OF LW CONCRETE STRUCTURES IN PAST EARTHQUAKES

In assessing the behavior of any type of material or construction technique, its performance in an earthquake can provide valuable insight. In the case of lightweight concrete, there are some well documented case histories of performance in earthquakes. This section will focus on LW concrete structures that were somewhat or severely damaged during the 1971 San Fernando and 1989 Loma Prieta earthquake.

In the 1971 San Fernando earthquake, several structures of the Olive View Hospital were damaged, some of which were constructed from lightweight concrete. A report by Jennings [2] discusses the damage to the structures in detail. All of the structures at the Olive View hospital complex were constructed between 1964 and 1970 [2]. The structure that exhibited the most significant damage was the two-story Psychiatric Day-Care Center. This building suffered total destruction as the first story columns collapsed causing the second story to fall onto the first [2]. All structural columns and beams were constructed from 17 MPa concrete. Photos from the collapsed structure indicated that the first story columns in this structure suffered a brittle shear failure, which in turn caused a loss of axial load capacity and collapse of the second story onto the first [2]. This failure is likely to be attributed to insufficient transverse reinforcement rather than properties of the concrete.

Another building at the Olive View Complex that suffered damage was a single story assembly hall [2]. This structure consisted of a lightweight waffle grid roof supported on eight lightweight concrete columns. Although the structure did not collapse, it did suffer a large residual displacement that rendered it useless. Jennings [2] suggests that the large displacement is a result of insufficient lateral strength. A third lightweight concrete structure at the Olive View Complex that suffered damage was a free-standing canopy [2]. It was suggested that the failure at the connection of the cantilever beams was a result of torsional response.

The amount of lightweight concrete bridge sub-structures is relatively few, so the available data on performance in earthquakes is small. However, bent 48 of the Central Viaduct in San Francisco was constructed from lightweight concrete and did sustain some damage as a result of the 1989 Loma Prieta earthquake [3]. The lower columns and cap beam of bent 48 of the central viaduct were constructed from lightweight concrete, while the upper columns and cap were normal weight concrete. The lower columns of this bent were the only lower columns of any bent in the viaduct to sustain any damage [3]. The cracks in bent 48 were nearly vertical, indicating shear-dominated behavior.

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PAST RESEARCH ON SEISMIC BEHAVIOR OF LIGHTWEIGHT CONCRETE

Overview

The discussion in this section is limited to structural testing on beams, columns, joints, and slabs that has aimed to investigate parameters that are important for seismic design. The parameters of interest include: (1) Shear strength, (2) Flexural strength, (3) Ductility, (4) Energy dissipation, and (5) Initial stiffness. Past research has included (1) Static monotonic tests, (2) Cyclic tests, and (3) Dynamic tests. Where possible, the type of lightweight aggregate used is stated such that comparisons can be drawn with knowledge of possible material effects. The research described below is divided into two broad categories: (1) Shear, and (2) Flexure. When reviewing this section, the reader is referred to Table 1 which summarizes the past research.

Shear

Monotonic Behavior of Beams – Early US Research: Hanson; Ivey

In the United States, Hanson [4] studied twenty-one beams, without shear reinforcement, with compressive strengths of 3000, 4500, 7000 and 9000 psi. A wide range of aggregates were used in this study: (1) expanded shale, (2) expanded clay, (3) expanded slate, (4) expanded blast furnace slag and (5) Elgin sand. Two types of failures were identified throughout these tests: one set of the beams failed when the diagonal cracking load was reached, while the other set failed at ultimate loads greater than the diagonal cracking load due to crushing at the load points. Hanson reported from his tests that the nominal unit shear strength of lightweight concrete was less than that of normal weight concrete in restrained beams, but still greater than ACI allowable stress (ACI 318-56) with a safety factor of two to four. Also all the nominal shear strength values were found to be similar for almost all types of aggregates.

Ivey and Buth [5] investigated twenty-six lightweight concrete beams for shear capacity under monotonic loading, with expanded shale and expanded slate aggregates being considered. The results were compared with the first specific recommendations for lightweight concrete provided by the ACI standard building code (ACI 318-63):

$$\text{Lightweight: } v_c = \phi(0.28F_{sp}\sqrt{f'_c} + 2500\frac{\rho Vd}{M}) \quad (\text{Eq. 1})$$

$$\text{Normalweight: } v_c = \phi(1.90\sqrt{f'_c} + 2500\frac{\rho Vd}{M}) \quad (\text{Eq. 2})$$

In the above equations, F_{sp} is the average ratio of splitting tensile strength to the square root of the compressive strength. Ivey and Buth showed that using the ACI 318-63 shear capacity equation for normal weight concrete multiplied by a factor of 0.75 for all lightweight concrete and a factor of 0.85 for sand-lightweight concrete is 36% conservative when neglecting the ϕ factor and 60% conservative when including it. In

addition, they showed that using the lightweight equation with $F_{sp} \sqrt{f'_c}$ replaced by the splitting strength is 7% and 26% conservative, with and without ϕ , respectively.

Monotonic Behavior of Beams - Europe Research: Hamadi; Berge; Lambert; Clarke

In the United Kingdom, Hamadi and Regan [6] tested ten T-beams with vertical stirrups loaded monotonically. The beams were divided into two groups, the first group had natural gravel coarse aggregate and sand fine aggregate, and the second group had both coarse and fine aggregate as expanded clay ('Leca'). Stresses in the stirrups were found to be higher in lightweight concrete beams and yielding occurred at loads well below the ultimate load. Hamadi and Regan adopted a concrete shear capacity equation developed by earlier researchers and derived two expressions for stirrup contribution using the truss analogy method. Their analysis indicated good agreement with the test results. $V_{u(calc)}/V_{u(test)}$ average was 1.0 and 1.02, and the standard deviation was 0.09 and 0.11 for both expressions.

Clarke [9] in his paper summarizes the work done by Berge [7] and Lambert [8]. A set of rectangular lightweight aggregate concrete beams with and without stirrups were tested monotonically by Berge [7]. By comparing the results with the British code approach [33], it was determined that the code equation was adequately conservative with the ratios of the actual to predicted shear strength ranging from 1.16 to 1.35 with a mean of 1.23. A series of tests were carried out by Lambert [8] using T-beams made with Lytag coarse aggregate and natural sand fines, without web reinforcement. After comparing the test results done by Lambert with the British code [33], the lowest ratio of measured-to-code shear strength ratio was 1.08, neglecting the material safety factor, which indicates that the 0.8 factor used by the British standards is conservative. The beam tests performed by Clarke [9] used three different lightweight aggregates: (1) Lytag, (2) Pellite and (3) Pumice. The beams were tested under a monotonically increasing load to failure. Clarke uses the British code [33] as a baseline and concludes that the design equation is overly conservative as it calls for a 20% reduction in shear strength for lightweight concrete.

Monotonic Studies of the 1990's: Salandra; Ahmad; Kawaguchi; Funahashi; Thorenfeldt

The research done by Salandra and Ahmad [10] centered around the evaluation of the shear strength of high strength lightweight concrete beams by static testing and are similar to the tests conducted by Clarke [9]. The aggregate used in the specimens was expanded slate. In all, 16 specimens were tested - 8 with web reinforcement and 8 without. Salandra and Ahmad concluded that the ACI equations yielded non-conservative results for extreme cases where the strength of the concrete is above 70 MPa. A comparison against similar tests done in normal weight concrete would yield more insight into the relative performance. Instead, the authors presented results in terms of how the actual strength compared with that predicted by the ACI equation.

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Ahmad et al. [11] investigated fifteen beams of normal and high strength lightweight concrete with and without shear reinforcement. They studied the effect of concrete compressive strength, shear span to depth ratio and amount of shear reinforcement on the shear ductility index. They indicated the shear ductility would increase as the concrete compressive strength increases.

The shear capacity of super lightweight concrete beams was studied by Kawaguchi et al. [12]. They used super lightweight artificial aggregates (SLA) developed in Japan to prepare the specimens. It was found that the shear reduction factor of 0.70, used in the Japanese code [28], for lightweight concrete is not suitable for this concrete because it is dependent on the concrete specific gravity. The beams tested by Masashi et al. [13] shares the same aggregate type used by Kawaguchi, with no shear reinforcement. The authors studied the effect of concrete density on the shear failure mode, shear capacity and mechanical properties of SLA concrete.

A set of sixteen lightweight beams without shear reinforcement were tested in Norway by Thorenfeldt and Stemland [14], the main parameters considered in the study were the shear span ratio, beam size and longitudinal reinforcement. The authors compared their shear capacity results with those predicted by the Norwegian code, and found that the average ratio of experimentally recorded to calculated strength was 2.03 and the shear strength ratios decreased almost linearly with the increase of the shear span above two.

Monotonic Behavior of Slabs: Hognestad; Nishimoto

Hognestad et al. [15] studied six lightweight concrete slabs and compared the results with three normal weight concrete slabs tested earlier. Two types of aggregates were considered in this study: (1) Expanded clay and (2) Expanded shale. The authors recommended that the design procedures developed by ACI-ASCE Committee 426 for slab shear strength could be modified for lightweight concrete slabs by relating ultimate shear stresses to the splitting tensile strength of lightweight concrete rather than the compressive strength.

In Japan, Nishimoto et al. [16] studied the effect of axial force on the shear resistance of lightweight aggregate concrete slabs. They used the artificial aggregate developed in Japan and concluded that shear strength increases proportional to the square root of axial force.

Cyclic Behavior of Slabs: Marzouk

Marzouk et al. [17] tested six slab-column connections under cyclic loading condition where all the specimens were subjected to a constant gravity load equal to the dead load plus 30% of the live load. The specimens were divided into three categories: (1) High strength lightweight (HSLW), (2) Normal strength lightweight (NSLW) and (3) Normal strength normal weight (NSNW). The authors reported that using HSLW concrete enhances the ductility and gives higher displacement ductility than NSLW and NSNW concretes. Based on the test results compared to the ACI specifications, the

authors recommended that the 0.85 factor used by the ACI for NSLW concrete can be replaced by a factor of 0.9 for HSLW concrete.

Cyclic Behavior of Columns: Kowalsky

Research by Kowalsky et al. [18] on the cyclic behavior of shear critical lightweight concrete columns is discussed in this section. Two large scale tests were performed: One was designed to achieve a shear failure at low levels of ductility, while the other was designed to achieve a shear failure at a higher level of ductility. The concrete strengths at the day of testing were calculated as 48MPa and 51MPa. Based on the test results, the shear transfer mechanism initially developed by Priestley et al. [34] and modified by Kowalsky et al. [35] was further revised for lightweight concrete [18]. The revision involved reductions of the ductility dependent concrete contribution to shear capacity ranging from 15% to 30% depending on the level of inelastic action expected in the column.

The model is comprised of three separate shear strength components [35]: One due to the concrete, V_c , the second due to the transverse steel, V_s , and the third due to the axial load, V_p . The nominal shear capacity, V_n , is then defined as the sum of three components as shown in Eq. 3. The concrete contribution to shear strength is a function of column aspect ratio, longitudinal steel ratio, and displacement ductility and is obtained with Eqs. 4-7. Eq. 7 includes the reduction for lightweight concrete based on the test results. The steel truss mechanism is obtained with Eq. 8 or the simplified Eq. 9 [18], while the axial load contribution is obtained with Eq. 10 where L' is the length of column from the critical section to the point of inflection.

$$V_n = V_c + V_p + V_s \quad (\text{Eq. 3})$$

$$V_c = 0.8 A_g \alpha \beta K \sqrt{f'_c} \quad (\text{MPa}) \quad (\text{Eq. 4})$$

$$1 \leq \alpha = \left(3 - \frac{M}{VD} \right) \leq 1.5 \quad (\text{Eq. 5})$$

$$\beta = (0.5 + 20\rho_l) \leq 1 \quad (\text{Eq. 6})$$

$$0.035 \leq K = 0.25 - 0.0358(\mu_\Delta - 2) \leq 0.25 \quad (\text{uniaxial bend}) \quad (\text{Eq. 7a})$$

$$0.035 \leq K = 0.25 - 0.0358(\mu_\Delta - 1) \leq 0.25 \quad (\text{biaxial bend}) \quad (\text{Eq. 7b})$$

$$V_s = \frac{\int_{x=-(r-c)}^r 2A_{sp}f_y \frac{\sqrt{r^2 - x^2}}{r} dx}{D - c - \text{cov}} \frac{D - c - \text{cov}}{s} \cot(\theta) \quad (\text{Eq. 8})$$

$$V_s = \frac{\pi}{2} A_{sp} f_y \frac{D - c - \text{cov}}{s} \cot(\theta) \quad (\text{Eq. 9})$$

$$V_p = \frac{P(D - c)}{2L'} \quad (\text{Eq. 10})$$

Dynamic Behavior: Kowalsky

A series of three reduced scale two column bent tests were conducted that aimed to investigate the effect of dynamic response on the shear and flexural behavior of lightweight concrete [19]. The research was part of a larger effort on the seismic response of lightweight concrete bridge structures and aimed to validate previous design recommendations made based on cyclic testing. Since no shear failure was observed, it is difficult to estimate the strength of the lightweight concrete shear resisting mechanism from these tests; however, following the design procedures outlined in the previous section resulted in a structure with adequate shear strength to ensure a ductile mode of response under seismic excitation.

Flexure

Monotonic Behavior of Beams: Ahmad; Birjandi

Ahmad and Barker [20] investigated the flexural behavior of six single reinforced high strength lightweight concrete beams. Test variables were concrete compressive strength and longitudinal reinforcement ratio. The researchers showed that the deflection ductility is less for lightweight concrete than that for normal weight concrete due to the fact that the lightweight concrete materials are less ductile as compared to normal weight concrete with the same strength. Also they reported that the ACI rectangular stress block provides accurate and conservative estimation of the ultimate flexural strength of singly reinforced lightweight beams having a compressive strength not exceeding 75.9 MPa and ρ/ρ_b not exceeding 0.54.

A series of beam tests by Ahmad and Batts [21] in 1991 were performed with the objective of verifying the validity of ACI normal weight guidelines as applied to lightweight concrete. Specifically, the authors were interested in determining flexural strength and ductility. The experimental program consisted of six doubly reinforced beams tested in three-point bending where two point loads were applied at 1/3 points along the span length. Longitudinal reinforcement was varied while transverse reinforcement was kept constant and consisted of #2 smooth stirrups spaced at half the section depth.

Their results concluded that actual test results in terms of flexural strength exceeded the ACI predicted rectangular stress block strength by 20% to 46%. Also concluded were that the value of a maximum useable concrete strain of 0.003 is conservative, thus resulting in adequate levels of ductility. A comparison with similar normal weight beams was not made. Load versus deflection curves were compared against an analytical model which does not consider the effects of confinement. Their test results thus generally under estimated the actual strength of the beams, although displacement capacity was generally overestimated which was explained to be a result of stopping the tests before failure.

In order to further shed light on these results, a series of moment curvature analysis were performed on their test specimens (assuming normal weight concrete behavior) in order to compare actual versus predicted strengths when confinement is considered. The results are summarized in Tables 2 and 3 [36]. The analysis was conducted with a computer program developed by King et al. [39] incorporating the stress strain model for concrete developed by Mander et al [37].

From these results we note that the predicted moment at failure is much larger from the moment-curvature analysis than from the ACI stress block, which is as expected. However, the fact that these moments exceed the experimental moments is somewhat troubling. On average, the experimental moments are 14% to 23% less than the analytical moments at failure.

Predicted maximum displacements from the moment-curvature analysis also diverge from the results presented by Ahmad in that experimental results always exceed the analytical prediction by 8% to 40%. This indicates that the experimental displacements recorded are the true maximum displacements and not those as a result of stopping the test before failure.

To summarize, in terms of flexural strength, it appears that a significant overprediction of strength is occurring; however, tests on equivalent normal weight specimens should be done to substantiate this result. In terms of displacement capacity, the results are good in that experimental displacements always exceed the predicted values from a moment-curvature analysis, which is consistent with the conservatism in the Mander energy balance approach [37].

Birjandi et al. [22] conducted a series of six tests on lightweight concrete beams under constant sustained load in an effort to assess the influence of long term loading on deflections. Their specimens were 3m long, 100mm wide and 200mm deep. They considered three different aggregates in their study, two of which were lightweight (Lytag and Leca), and one conventional normal weight. In all cases, the initial static deflections due to loading were greater (by approximately 20%) for the lightweight concrete due to the lower elastic modulus. There was no difference in rate of increase of deflection with time for the lightweight concrete beams when compared to the normal weight concrete members.

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Cyclic Behavior of Beam Column Joints: Bertero; Rabbat; Monti

In 1980, Bertero et al. published results [23] describing experimental results of 2 lightweight concrete beam-column sub-assemblages. Their large scale testing provides a unique opportunity for comparison with normal weight concrete as they had previously conducted tests using the same configuration with normal weight concrete. Both of their test units were identical and differed only in the way in which they were tested. The objective of their work was to investigate the bond between lightweight concrete and reinforcement bars, as well as to characterize the flexural behavior of lightweight concrete in general. The concrete strengths of the test specimens were measured as 29MPa and 32MPa.

The first test unit was tested monotonically in both directions of loading, while the second test unit was tested in cyclic manner. Results from the first test unit compare very favorable with an earlier test unit constructed from normal weight concrete, as displacement capacity, strength, and energy absorption were essentially identical. There was a significant difference in initial stiffness (52%). Results from the cyclic test were considerably less favorable. From that test, a comparison with normal weight concrete indicated lower strength and energy absorption. Displacement capacity was essentially the same. Bertero et al. [23] contributed this difference due to bond deterioration in earlier cycles as a result of the weaker aggregate.

A series of 16 tests (13 of which were lightweight concrete) were conducted by Rabbat et al. [24] on the cyclic behavior beam-column joint subassemblies. Their results indicated little difference with regards to strength, displacement capacity, or energy dissipation for lightweight concrete when compared to normal weight concrete. Their tests were conducted using lightweight concrete as strong as 41MPa.

Research done in Italy by Monti & Nuti [25] centered on the design and testing of 14 beam column subassemblages that were designed according to EuroCode 8 specifications. Each of 7 test units were constructed in normal weight concrete and lightweight concrete for purposes of direct comparison. The test program tested various reinforcement configurations while keeping the dimensions of the members constant. Column dimensions were 30 cm square, and the beams were 40 cm deep and 20 cm wide. In terms of ultimate flexural strength, analysis of results indicates that the lightweight concrete test units achieved a flexural strength that was 98% of the equivalent normal weight test unit. Since this falls within the reported margin of error, it can be concluded that no difference in response was observed. However, it is noted that these were beam tests where axial load was not a factor in flexural strength. The lightweight concrete units achieved a displacement capacity that was equivalent to the normal weight concrete units (98% of the normal weight displacement capacity). No qualitative assessment of energy absorption was made in the report, however examination of the force-displacement hysteresis indicate no discernible difference in response.

Cyclic Behavior of Columns: Kowalsky

A series of three tests were conducted on large scale columns (3.066m tall, 460mm diameter) subjected to single bending reversed cyclic loading. Two of the

columns were constructed from lightweight concrete with strengths ranging between 36MPa and 40MPa, with the sole difference between the two being the amount of transverse reinforcement. The third test unit was constructed from normal weight concrete while being identical to one of the lightweight concrete test units. Discussed in this section are the results of those tests with regards to flexural strength, stiffness, deformation capacity, and energy dissipation.

As a result of the tests performed, it was determined that the flexural strength of lightweight concrete should be lower than that for normal weight concrete. It was determined that the reduction in strength is entirely attributable to the reduced compression strength enhancement afforded by confinement. Past research on compression behavior of confined lightweight concrete cylinders [38,21,36] has indicated only marginal increases in the confined concrete compression strength on the order of 5% or less of well detailed members. Given this lack of compression strength enhancement, calculations were conducted which indicated that difference in strength between the normal weight and lightweight concrete tests specimens is attributable to this effect. Fig. 1 illustrates the hysteretic and envelope responses of the two comparative tests conducted as part of this research. Based on the test observations, recommendations for flexural strength reduction were made as a function of axial load ratio and longitudinal steel ratio as shown in Fig. 2

With regards to elastic stiffness, the test results indicated that a stiffness reduction of approximately 20% would be appropriate for calculating the stiffness of a lightweight concrete member. This modification recognizes the reduced elastic modulus expected for lightweight concrete.

Maximum deformation capacity is directly related to maximum strain capacity which can be governed by: (1) Buckling of the longitudinal reinforcement over a series of transverse reinforcement bars and subsequent rupture. (2) Buckling of the longitudinal reinforcement between layers of transverse reinforcement or over the gap length of a steel jacket and subsequent rupture. (3) Confinement failure due to rupture of transverse reinforcement. (4) Rupture of longitudinal reinforcement due to exceedance of tensile strain capacity. The results of the research indicated no significant difference in deformation capacity between lightweight and normal weight concrete.

Fig. 3 represents a plot of the hysteretic damping as a function of ductility for the test specimens conducted as part of the research [36]. From these results, it is clear that no significant differences exist between lightweight and normal weight concrete with regards to energy dissipation.

Dynamic Behavior: Kowalsky

Based on the shake table tests conducted as part of that research [19], it was determined that no additional modifications to behavior with regards to flexure are needed over what was identified as a result of the research done on cyclic tests [18].

Design Parameters Considered

To the knowledge of the authors, there are no explicit lightweight concrete seismic design provisions in any of the major building codes. However, general design provisions related to shear and flexural response are of obvious importance and many building codes provide recommendations on the influence of lightweight concrete on the various design expressions. In this section, lightweight concrete provisions of several building codes are discussed with regards to parameters that have a direct effect on seismic behavior.

ACI Guidelines

With regards to shear capacity, the ACI guidelines state that the overall shear strength is the sum of a concrete component and a steel component. The concrete component is given by Eq. 11 for axial compression and Eq. 12 for axial tension.

$$V_c = 0.166 \left(1 + 0.0725 \frac{P_u}{A_g} \right) \sqrt{f'_c} b_w d \quad (\text{MPa}) \quad (\text{Eq. 11})$$

$$V_c = 0.166 \left(1 + 0.29 \frac{P_u}{A_g} \right) \sqrt{f'_c} b_w d \quad (\text{MPa}) \quad (\text{Eq. 12})$$

The ACI code [26] provides two alternative forms of design shear strength. The first approach is based on the splitting strength of the concrete. The code contains the provision that all value of $\sqrt{f'_c}$ may be replaced with $f_{ct}/6.7$.

If the splitting strength is not specified, then the code specifies that all values of $\sqrt{f'_c}$ that affect the concrete shear capacity, tension capacity, and cracking moment be multiplied by 0.75 for all-lightweight concrete and 0.85 for sand-lightweight concrete. The splitting strength of concrete is determined by loading a test cylinder on its side and measuring the total applied load. The splitting strength is then defined as:

$$f_{ct} = \frac{2P}{\pi DL} \quad (\text{Eq. 13})$$

With regards to flexural strength, the ACI code [26] place an upper limit on compression strength equal to 34.5 MPa for structures in seismic regions (Section

21.2.4). The reason for this limitation is the limited data available on high strength lightweight concrete subjected to load reversals. Indeed, a review of the research discussed earlier in this paper identifies only 4 cyclic column tests by Kowalsky [18] with concrete strength greater than 35MPa, and the 13 beam to column joint assemblies by Rabbat [24]. The ACI code does stipulate that the limitation can be removed if sufficient experimental evidence is available that indicates that the ductility capacity of lightweight concrete with strengths beyond 34.5 is the same as comparable normal weight concrete. Aside from this limitation, there are no other specific requirements with regards to flexural behavior, energy dissipation or deformation capacity in the ACI code.

With regards to elastic stiffness, the ACI code [26] states the following criteria in section 8.6.1: "Use of any set of reasonable assumptions shall be permitted for computing relative flexural and torsional stiffnesses of columns..." Also stated is that due to the practicalities of design, an estimate of actual section flexural stiffness equal to 50% gross section stiffness is appropriate for inelastic members, while the full gross stiffness should be used for elastic members (commentary 8.6.1) [26]. A parameter that has a direct effect on calculation of elastic stiffness is the elastic modulus, and the ACI code utilizes Eq. 14 for that calculation. If a lightweight concrete has a density 30% lower than a similar strength normal weight concrete, a reduction in elastic modulus of 40% would be obtained. Based on past research [22, 18], this level of reduction appears to be too conservative as test results have indicated that a reduction on the order of 20% is more likely.

$$E_c = 0.04272\gamma^{1.5} \sqrt{f'_c} \quad (\text{MPa}, \gamma \text{ unit weight in Kg/m}^3) \quad (\text{Eq. 14})$$

EuroCode Approach

The EuroCode 8 [27] approach does not provide for any reduction in the shear strength of lightweight concrete. The design equations are based on the tensile strength of concrete, which EuroCode 8 bases on the compressive strength of the concrete. The basic EuroCode relation for allowable concrete shear stress is shown as Eq. 15.

$$v_c = 0.035\sqrt[3]{f'_c}^2 \quad (\text{MPa}) \quad (\text{Eq. 15})$$

Japanese Approach

According to the Japanese code under JIS A 5002, lightweight aggregate concrete has the characteristic bending, tensile, shear, bond and bearing strengths equal to 70% of those of normal weight concrete, where those are determined from the compressive strength of concrete. Also Young's moduli of lightweight concretes are 1.3, 1.5, 1.6 and 1.9 X 10⁵ MPa for concrete having the compressive strengths of 18, 24, 30 and 40 MPa, respectively [28], [29].

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Norwegian Approach

The Norwegian approach calculates the shear capacity based on the tension strength of the concrete [14, 30]. The values for the tensile strength of lightweight concrete are obtained by multiplying the values for normal weight concrete by a factor that includes the unit weight of the concrete. According to the Norwegian code NS3473, the reduction factor for lightweight concrete tensile strength is given as $(0.3 + 0.7 \rho/\rho_1)$, where ρ is the lightweight concrete density and $\rho_1=2400 \text{ kg/m}^3$ [14, 30].

Australian and New Zealand Approaches

No modification for the lightweight concrete is required as it is not discussed in the Australian code [31]. The 1982 New Zealand code (NZS 3101) [32] provides provisions very similar to ACI for shear. In the event that the tensile strength of the concrete is known, a value of $1.8f_{ct}$ replaces $\sqrt{f'_c}$ (MPa) in any equation that calculates the shear capacity. If the tensile strength is not provided, then the NZS follows the ACI approach.

British Approach

The shear requirements of the British code, BS 8110, are to multiply the allowable shear stress as calculated by Eq. 16 by a multiplier of 0.80 for lightweight concrete [33]. In Eq. 16, A_s is the area of longitudinal reinforcement, f'_c is the concrete cube strength and γ_m is a safety factor taken as 1.25.

$$v_c = \frac{0.79 \left(100 \frac{A_s}{b_w d} \right)^{\frac{1}{3}} \left(\frac{400}{d} \right)^{\frac{1}{4}} \left(\frac{f'_c}{25} \right)^{\frac{1}{3}}}{\gamma_m} \quad (\text{MPa}) \quad (\text{Eq. 16})$$

DISCUSSION AND FUTURE RESEARCH NEEDS

A case can be made that much of the past research on shear capacity of lightweight concrete members does not apply to structures in seismic regions. This is largely due to the fact that in many cases, only monotonic loading was considered. Furthermore, many of the tests used to develop code provisions consisted of beams without transverse reinforcement. There are only two known cyclic shear tests of lightweight concrete members [18], and those results did indicate a slight decrease in the strength of the concrete shear resisting mechanism, particularly at higher levels of ductility. The majority of the design codes suggest some form of reduction of shear strength, yet the number varies from 0% to 30%. Similarly, different researchers have suggested reduction values ranging from 10% to 25% for the lightweight concrete shear resisting component.

With regards to flexure, several monotonic tests and a smaller amount of cyclic tests seem to indicate a minor reduction in flexural strength, typically less than 10%, and a reduction in elastic stiffness on the order of 20%. With regards to energy dissipation, results are mixed with some researchers reporting no reduction for lightweight concrete while others suggest a minor reduction. No researchers reported any significant reduction in deformation capacity. With regards to building codes, there are no known reductions associated with cyclic flexural response, with the exception of the ACI code requirement on maximum compression strength, which indirectly effects flexural strength and stiffness.

It is clear that uniform agreement across all countries and researchers does not exist at this time. The lightweight concrete provisions of the ACI code are based on a comprehensive set of tests on beams subjected to monotonic loading. Those tests, conducted in the 1950's, utilized beams without any shear reinforcement in an effort to isolate the concrete contribution to shear capacity. Although such a test procedure will provide a lower bound estimate to shear strength, it is likely to be unrealistic when applied to actual beam members with transverse reinforcement. Since the early tests by Hanson [4], limited tests under cyclic loading have indicated that a reduction to the shear resisting mechanism may be appropriate; however, a more comprehensive evaluation should be undertaken. If the reduction factors applied to the shear capacity of lightweight concrete can be shown to be unnecessary, then there would be a significant benefit to the more widespread use of lightweight concrete in structural members. Conversely, if the reduction factors are too low for seismic regions, then that should be accounted for in design as well. With regards to the ACI code, current seismic design requirements for plastic hinge regions assume that the concrete does not contribute to the shear capacity. Clearly, this is a conservative requirement, and as such, it will not matter whether a plastic hinge region of member is constructed from lightweight or normal weight concrete as in both cases the contribution to shear strength is zero. It will, however, have an impact for regions outside of the plastic hinge regions or members subjected to low levels of ductility. Similar arguments can be made for flexural behavior as well where a comprehensive test program considering a wide variety of aggregates, confinement levels, and longitudinal reinforcement is warranted to provide more confidence for the use of lightweight concrete in seismic regions while providing more accurate estimates of reduction factors to ensure safe design.

REFERENCES

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