

Attachment E – Literature Summary and Detailed Work Plan

Introduction

Use of high-strength prestressed concrete girders and high-performance bridge decks has become accepted practice by many state highway agencies because of their technical and economic benefits. These girders and decks are generally manufactured with concrete made with natural normal weight aggregates. Use of manufactured lightweight coarse aggregates, such as expanded shale, slate, and clay to produce lightweight concrete offers the benefit of reducing the weight of the superstructure, leading to reductions in the size of girders, substructure, and foundations. These size and weight reductions facilitate shipping, handling, and construction or replacement of bridge elements, and result in economic benefits.

Recent advances in high-performance/high-strength lightweight concrete have had limited application in bridge construction because of the lack of design and construction guidelines and concerns about material properties and their impact on performance. Research is needed to address the factors that significantly influence the design, constructability, and performance of high-strength prestressed concrete bridge girders and high-performance bridge decks and recommend changes to the AASHTO LRFD bridge specifications. These modified specifications will provide highway agencies with the information necessary for considering lightweight concrete mixtures that are expected to yield economic benefits.

Objective

The objectives of this research are to (1) develop guide specifications for the use of lightweight concrete in high-strength prestressed concrete girders and in high-performance bridge decks and (2) recommend changes to the AASHTO LRFD Bridge Design and Construction Specifications relevant to high-strength lightweight concrete girders and high-performance lightweight concrete decks. It will deal with mixtures made with normal weight fine aggregates and manufactured lightweight shale, clay, or slate coarse aggregates to produce concrete with equilibrium density, as determined according to ASTM C 567, of not more than 125 pcf.

Tasks

The tasks as originally laid out in the proposal were as follows:

- Phase I
 - Task 1 – Literature and Project Review
 - Task 2 – Review of AASHTO LRFD Bridge Design and Construction Specifications
 - Task 3 – Identify Items for Further Investigation
 - Task 4 – Develop a Work Plan
 - Task 5 – Interim Report
- Phase II
 - Task 6 - Execute the Research Plan experimental investigation
 - Task 7 – Comparative Deck and Girder Designs
 - Task 8 – Final Report

This attachment presents the results of the first four tasks of the research project.

Task 1- Background

Scope

Collect and review relevant literature, design and construction specifications, research findings, and current practices for structural lightweight concrete. The review shall focus on the use of lightweight concrete for high-strength and high-performance applications. Emphasis should be placed on constructability and structural and durability properties relevant to use in prestressed concrete girders and bridge decks. This information must be assembled from published and unpublished reports, contacts with state transportation agencies, industry organizations, and other domestic and foreign sources.

Literature Review

Material Properties of Lightweight Concrete. There are some unanswered questions regarding the material characteristics of lightweight high-performance concrete (LWHPC). In particular, equations in AASHTO may not adequately predict the tensile strength, modulus of elasticity, coefficient of thermal expansion, shrinkage and creep behavior of LWHPC. All of these material

properties have a significant impact on the structural design of lightweight concrete bridge girders and decks. Furthermore, there is not enough data available to fully assess the durability of this material. Several material properties have been shown to be good measures of durability (permeability, absorption, and freeze/thaw behavior)

The literature pertaining to the material properties of lightweight concrete is reviewed in Appendix C. In addition, for each material property discussed, recent advances with regards to high performance, lightweight concrete are discussed and specific research needs are presented.

Shear Behavior of Lightweight Concrete. Figure E-1 presents a summary of the test programs found in the literature and discussed in detail in Appendix C. Some of the sources provided enough information that the results can be reproduced and used in further evaluations of shear strength conducted in this research effort. Others did not provide all required information. The studies with and without adequate information are separated in the table. Analysis of this data was performed to finalize the shear testing program, and this analysis is presented in the section discussing and describing Sub-Task 6.4.

The previously performed research provides valuable information about the shear behavior and shear strength of lightweight concrete. Important conclusions are as follows:

Compared to normal weight concrete of similar compressive strength-

- Lightweight concrete has lower shear strength,
- Shear cracks in lightweight concrete have a different angle of inclination,
- Shear cracks in lightweight concrete exhibit greater slip,
- Lightweight concrete behaves differently in bi-axial stress states.

In a recently completed study, Russell (2007) reported the need for evaluating the applicability of the sectional design method to lightweight concrete beams. Therefore, he recommends testing full sized prestressed concrete beams made with lightweight concrete to evaluate this design method as well as minimum reinforcement and maximum spacing requirements. The test program described in Sub-Task 6.4 is consistent with these recommendations and endeavors to provide additional data to investigate and quantify the differences in shear behavior and shear strength of lightweight prestressed concrete beams.

Table E-1. Summary of Past Research on Shear in Lightweight Concrete Beams

| Author (Year) | No. of beams | Reinf. Type | Unit Weight (pcf) | Compr. Strength (ksi) | Effective Depth (in.) | a/d | Cross-Sectional Area (in ²) | Shear Reinf. Index (psi) | Composite deck (Yes/No) |
|--------------------------------|--------------|-------------|-------------------|-----------------------|-----------------------|---------|---|--------------------------|-------------------------|
| Ahmad et al. (1994) | 15 | mild steel | 115-129 | 4-13 | 7.8-8.5 | 1-4 | 50 | 0-317 | No |
| Funahashi et al. (2001) | 5 | mild steel | 82-113 | 4.8-6.5 | 13.4-39.4 | 3-3.5 | 124-830 | 0 | No |
| Hanson (1958) | 16 | mild steel | 91-113 | 3.1-8.6 | 10.5 | 2.48 | 72 | 0 | No |
| Hanson (1961) | 30 | mild steel | 90-112 | 3-8.2 | 10.5 | 2.5-5.0 | 72 | 0 | No |
| Ivey and Buth (1967) | 26 | mild steel | 94-126 | 2.8-4.7 | 7.4-15.6 | 2-5 | 38-160 | 0 | No |
| Kawaguchi et al. (2000) | 9 | mild steel | 74-107 | 2.7-6.8 | 11.8 | 3.5 | 109 | 80.1 | No |
| Murayama and Iwabuchi (1986) | 11 | mild steel | 119 | 6.3-7.4 | 9.8 | 2.5-4 | 93 | 0 | No |
| Nishibayashi et al. (1968) | 16 | mild steel | 98-99 | 3.1-6.3 | 6.7 | 2.1-2.9 | 31 | 0 | No |
| Ramirez et al. (2004) | 5 | mild steel | 106-127 | 6.2-10.5 | 12.0-12.1 | 2.1-2.2 | 196 | 174-261 | No |
| Salandra and Ahmad (1989) | 16 | mild steel | 122.5 | 8-10.5 | 6.75 | 0.5-3.6 | 32 | 456 | No |
| Walraven and Al-Zubi (1995) | 11 | mild steel | 108-131 | 2.9-7.0 | 28.9-30.1 | 2.2-2.3 | 279 | 360-1290 | No |
| Malone (1999) | 5 | mild steel | 106-127 | 6.2-10.5 | 12.1 | 2.1-2.2 | 196 | 157-236 | No |
| Malone (1999) | 4 | prestressed | 106-127 | 6.5-10.1 | 29.3 | 3 | 469 | 0-110 | Yes |
| Kahn et al. (2004) | 6 | prestressed | 118-119 | 8.9-11.0 | 40 | 1.5-2.4 | 369 | 570 | Yes |
| Dymond (2007) | 2 | prestressed | 118 | 10.5-10.7 | 58.5 | 1.3-3.1 | 1559 | 530-760 | Yes |
| Evans and Dongre (1963) | 20 | mild steel | 90-115 | 3.1-4.9 | n/a | n/a | n/a | n/a | No |
| Hamadi and Regan (1980) | 5 | mild steel | n/a | 4.0-4.6 | 13.8 | 3.6 | 14 | 90-470 | No |
| Hoff et al. (1984) | 14 | mild steel | n/a | n/a | n/a | n/a | 46.5 | n/a | No |
| Jindal (1966) | 6 | mild steel | 93-106 | 2.9-3.4 | 13 | n/a | n/a | 0 | No |
| Kirmair (1981) | 5 | mild steel | 93.6 | 4.2 | n/a | n/a | 174 | 91-1154 | No |
| Kobayahsi et al. (2000) | 9 | mild steel | 75-105 | 4.3-8.9 | 8.7 | 2 | 84 | 61 | No |
| Swamy and Bandyopadhyay (1979) | 16 | mild steel | n/a | 5.2-5.8 | 5 | 1.5-6 | 41 | 0 | No |
| Swamy and Lambert (1983) | 27 | mild steel | n/a | 3.4-7.0 | 7.1 | 1.5-6 | 62 | 0 | No |
| Brettle (1962) | 16 | prestressed | 105 | 4.8-6.4 | n/a | 1.5-9 | 12-36 | 0 | No |
| Watanabe et al. (2004) | 7 | prestressed | 106-119 | 8.7-9.6 | 21.7 | 3 | 194 | 66 | No |

Note: Research listed above will be discussed in description of Sub-Task 6.4

Prestress Losses. Prestress losses in lightweight and high-performance lightweight concrete beams have been investigated by several researchers in the past. Nassar (2002) measured changes in strain at the centroid of the prestress force in two AASHTO Type IV girders constructed with high-strength lightweight concrete. These were test girders for the Chickahominy River Bridge constructed in Virginia. Measurements were taken for over nine months, and compared to time-step and other methods for predicting prestress losses. The measured elastic shortening loss was higher than predicted using ACI 209 (2000) or PCI (1975). However, the time dependent loss was considerably smaller than predicted with either model.

Waldron (2004) tracked the changes in strain in three additional Chickahominy River Bridge girders for over 900 days. This allowed the changes to be tracked through the construction process including casting of the deck and barrier rails, and under service conditions. Waldron compared the measured changes in strain to predicted changes using a wide variety of methods. He calculated losses using the time-step method with 12 different creep and shrinkage models, and he also calculated losses with nine other methods. He evaluated the effectiveness of the models in predicting losses in high-strength, lightweight girders, and concluded that the Shams and Kahn (2000) model provided the best predictions using time-step methods but noted that the method for calculating prestress loss proposed in NCHRP 496 (Tadros, et al. (2003))(essentially the method given in AASHTO) gives a conservative estimate of losses in high performance, lightweight prestressed girders. However, he did recommend further investigation of losses in lightweight concrete. All evaluated methods for direct calculation of losses underestimated elastic shortening losses and overestimated long term losses.

Kahn and Lopez (2005) instrumented and monitored changes in strain at the level of the prestress in four AASHTO Type II girders made with high-strength, lightweight concrete. They compared measured losses to losses predicted with four prestress loss calculation methods. Similar to Waldron (2004) and Nassar (2002), the measured elastic shortening losses were higher than predicted and the long term losses were smaller than predicted.

Except for Waldron, the previously sighted works did not evaluate the current AASHTO method for predicting prestress loss, because they were published before the adoption of the current method into the specifications. The previously collected data should be compared against the current method. Since the method itself is highly rational, the efficacy of the method in predicting changes in strain is primarily dependent on the accuracy of the material models in

predicting modulus of elasticity and creep and shrinkage coefficients. Finally, Russell (2007) recommends investigation of prestress losses (and, concurrently, creep and shrinkage properties) in full size girders and comparing measured with predicted results.

Transfer and Development Lengths of Prestressing Strand. The end region of a member over which the effective prestressing stress, f_{se} , is transmitted from the strands to the concrete is referred to as the transfer length, L_t . A generally accepted model of the variation of prestress forces in a concentrically prestressed member is shown in Figure E-1. Prestressing force is transferred from the strand to the concrete near the end of a prestressed girder due to the swelling of the strand (Poisson's Effect). This swelling causes radial tension in the concrete around the strand. One way to determine if complete transfer of the prestressing force to the concrete is accomplished is by measuring the transfer length of the strand immediately after transfer. Also, if adequate spacing and cover of the strand is not provided, cracking of the concrete near the end of the member may result.

AASHTO defines development length as the "distance required to develop the specified strength of a reinforcing bar or prestressing strand" (AASHTO, 2007). The first bond region is the one characterized previously as the transfer length and the second bond region is termed the flexural bond length. This flexural bond region is the additional length required beyond the transfer length for the strands to develop their ultimate flexural stress, f_{ps} . Figure E-2 below, illustrates a qualitative variation of strand stress along the development length.

Neither ACI nor AASHTO accounts for differences in bond and development length in lightweight concrete. Research by Nasser (2002) on the transfer length in lightweight, high strength AASHTO girders indicated that the current equations were conservative for the prediction of transfer length. Transfer lengths were measured on each end of two Type IV girders. The release strength for the girders was 4780 psi. The 95% average maximum strain method was used to determine the transfer length for the four measured locations, and the average transfer length was 16.8 in. This was considerably less than the 50 bar diameter length of AASHTO and ACI, which for 0.5 in. diameter strands is 25 in.

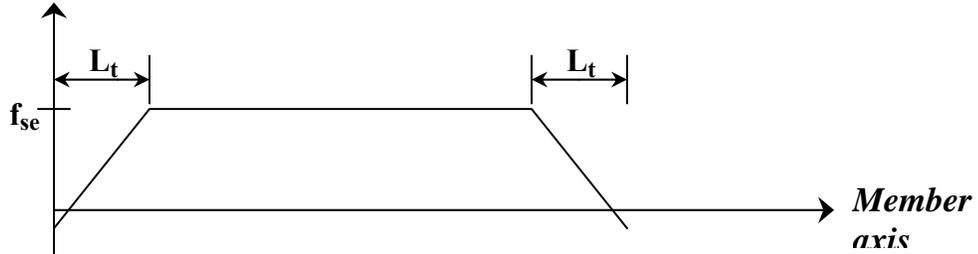


Figure E-1: Variation of stresses in a prestressed member

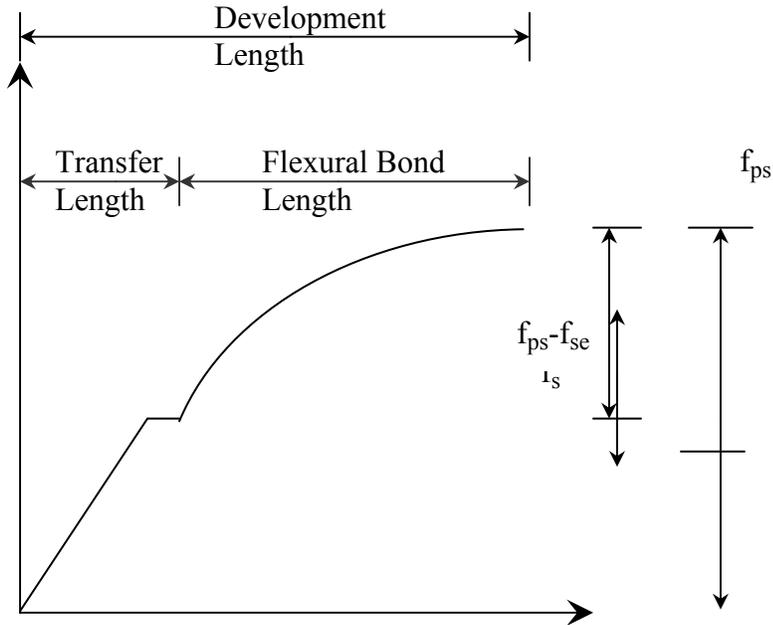


Figure E-2: Variation of steel stress with distance from free end of strand

Other researchers have investigated transfer length in lightweight concrete (Russell and Burns 1996) and Peterman et al., (1999)). Russell and Burns indicated 50 bar diameters was unconservative for lightweight concrete, while the research by Peterman et al. results indicated that 50 bar diameters was conservative. Also, a systematic laboratory investigation of transfer and development length of prestressing strand has been recommended by Russell (2007). Transfer of prestress force has a significant impact on shear design of prestressed concrete

(especially web shear strength). Development length is an important design consideration in states that use debonding of prestressing strands as opposed to draping of strands. This is an area which could benefit from additional data collection.

Another question related to design of transfer zones in lightweight concrete is whether 0.6 in. diameter strands can be used on a 2 in. grid without causing cracking or distress in the transfer region. The lower tensile strength of lightweight concrete could result in more cracking, longer transfer lengths, longer developments lengths, or the requirement for greater quantities of confinement reinforcing in the end zones of prestressed beams.

Development and Splice Lengths of Mild Reinforcement. With regards to mild steel reinforcement development and splice length, AASHTO (AASHTO, 2007) gives equations and factors in section 5.11.2 that account for the effect of the following on development length: bar size, bar yield strength, concrete compressive strength, top bar effect, epoxy coating, spacing, cover, and confinement. Factors are also included in AASHTO that account for the impact of the use of lightweight aggregates on development and splice length in reinforced concrete members. The factors given range from 1.0 (for lightweight concrete with sufficient splitting tensile strength) to 1.3 (for all lightweight aggregate concrete where splitting tensile strength is not specified). These factors are based on several research projects conducted between 1934 and 1993 (ACI, 2003) and primarily included testing of specimens with normal strength, lightweight concrete. The results of these projects indicate that bond strengths between reinforcement and lightweight concrete can range from approximately equal to that of normal weight concrete to about 65% less. In addition, several researchers in the 1990s found that for higher strength concrete ($f'_c \geq 8,000$ psi) the average bond strength at flexural failure decreases with an increase in compressive strength (when normalized to $\sqrt{f'_c}$). The term $\sqrt{f'_c}$ is used in the denominator of the AASHTO equations for development length to account for the effect of f'_c on development length. As stated in an American Concrete Institute report (ACI, 2003) there is a “paucity” of experimental testing and results of the bond strength of reinforcement in concrete containing lightweight aggregates. In addition, Russell (2007) recommends a systematic laboratory investigation of development of mild steel in lightweight concrete. When considering all this, it seems prudent to further investigate bond strength and development of reinforcement in lightweight concrete.

Bearing strength. Very little work has been done to evaluate the differences in bearing strength between lightweight and normal weight concrete. At Virginia Tech, Roberts-Wollmann et al. (2006) performed an experimental investigation into the bearing strength of lightweight concrete cylinders. Cylinders of both lightweight and normal weight concrete with compressive strengths of approximately 5000 psi and 8000 psi were tested. These cylinders were loaded with bearing plates of varying sizes. The ultimate loads and failure modes were noted. It was found that the equation for bearing strength presented in ACI 318, which is essentially the same as in Section 5.7.5 in AASHTO, can over predict strength for lightweight and high strength normal weight concrete at ratios of supporting area to bearing area (A_2/A_1) less than approximately 7. Two possible corrections were presented, one involves applying a lightweight concrete factor (λ) to the existing equation, and the other is a new equation based on a Mohr failure criterion for the concrete in the test prisms. Both provide safer predictions of the bearing strength of lightweight concrete than does the current ACI/AASHTO Equation.

Post-Tensioned and Pre-Tensioned Anchorage Zones. One critical region of post-tensioned structures, which has not been thoroughly investigated for lightweight concrete, is the local tendon anchorage zone. The local anchorage zone is the region immediately surrounding a post-tensioning anchorage device, which is subjected to very high local bearing stresses induced by the introduction of the tendon force into the concrete. Within the local zone, the high bearing stresses disperse and at the limits of the local zone the stresses must be spread sufficiently to be safely introduced into the surrounding structure. In normal weight concrete, if the introduced compressive stresses exceed bearing stresses which can be accommodated by unreinforced concrete, the anchorage system incorporates a spiral or other confining reinforcement to enhance the compressive strength of the concrete within the local zone.

The behavior and design of a local anchorage zone in lightweight concrete could be significantly different from that of normal weight concrete. This is because lightweight concrete is more susceptible to local crushing of the aggregates and has a lower splitting tensile strength (Heilmann 1983). Roberts-Wollmann et al.(2006) presented data which indicated that the bearing strength of unreinforced lightweight concrete was less than that of normal weight concrete with identical compressive strength. In addition, tests have shown that the compressive strength of

lightweight concrete is not increased as greatly as normal weight concrete by the presence of surrounding concrete or confining reinforcement (Heilmann 1983, Martinez et al. 1984, Basset and Uzeneri 1986, Shah et al. 1983).

Pre-tensioned anchorage zones in lightweight girders are more prone to cracking than similar regions in normal weight girders. Case studies reported by Crispino and Davis (2006) indicate longer and wider cracking in lightweight girder end zones. The simple methods currently prescribed in AASHTO for the design of these regions are proving to be inadequate to control end zone cracking in lightweight girders.

Results of State DOT Survey

A survey was sent to all 50 State Bridge Engineers and the bridge engineers in the District of Columbia and Puerto Rico to determine if they have used lightweight aggregate concrete in bridge structures. Questions regarding usage in girders, usage in decks, typical mix designs, special design requirements, and numbers of projects were included in the survey. The survey also requested information about the engineer's satisfaction with lightweight concrete and any problems experienced with or benefits perceived from the use of lightweight concrete. A list of projects was compiled. Appendix D contains each question included in the survey with results received. Responses were received from 38 states with two having used lightweight concrete in prestressed girders (Virginia and Alaska) and 15 states having used lightweight concrete in bridge decks. Of the 52 agencies contacted, fourteen did not respond. Figure E-3 shows a map of the United States with the states that responded labeled.

- About half of the fifteen states that have used lightweight concrete report have experience with pumping lightweight concrete.
- About half of the fifteen states that have used lightweight concrete report using shale lightweight aggregates and one quarter slate.
- A variety of methods for pre-wetting lightweight aggregates were reported.

Summary of responses regarding Lightweight Girders. Respondents to the survey reported 33 bridges built with lightweight concrete girders (13 in Alaska and 20 in Virginia). Personal contact with Georgia DOT revealed the construction of a bridge utilizing lightweight concrete bridge girders. That bridge is still in design so was not reported in the survey. Also, bridges in California (Muruges, 2007), Ohio (Spaans, 1997), and Indiana (Wanders, 1993) have been constructed with lightweight concrete girders, but these three states did not respond to the survey. Design strengths used in Alaska and Virginia are split evenly between less than or equal to 6,000 psi and between 7,000 to 8,000 psi.

The survey results reveal that lightweight concrete is rarely used in bridge girders and most recent uses have been in one state (Virginia). Typically in Virginia a mixture of lightweight and normal weight aggregates is used at a ratio of 42 % to 58 % and at a water-to-cementitious material ratio of between 0.30 and 0.34.

From a design perspective neither state utilizes special provisions for the design of prestress girders containing lightweight concrete.

Summary of responses regarding Lightweight Decks. Of the 15 states that have used lightweight concrete decks, New York and Pennsylvania had the most. The states responding reported on a total of 228 bridges constructed with lightweight concrete decks.

About three quarters of the respondents reported using design compressive strengths of between 3,000 and 5,000 psi. Approximately three quarters of the bridge decks constructed with lightweight concrete were done with normal weight fine aggregate and with a water-to-cementitious material ratio of between 0.40 and 0.44. The responding states are split evenly with regards to the number of states that have or do not have special provisions for the design of lightweight concrete bridge decks.

Summary of Contacts with Lightweight Aggregate Manufacturers

To facilitate the selection of aggregate, lightweight aggregate manufacturers were contacted and data and other information materials have been acquired for each of the production facilities. It was determined that some facilities are no longer producing lightweight aggregate, some plants have totally closed, and others do not produce structural-grade lightweight aggregate. It should be noted that structural, in this context, refers to aggregate used in ready-mix or precast, prestressed concrete applications. Aggregate used primarily in concrete masonry units (CMUs) was not considered to be structural aggregate, for the purpose of this study.

Table E-2 lists the plants currently in operation and producing structural-grade lightweight concrete. This table includes the highest documented compressive strength using each respective aggregate based on data received, a corresponding unit weight for each concrete mixture, and the highest percentage aggregate absorption recorded in laboratory tests. Though high compressive strength mixtures may be desirable for certain prestressed girder applications, excessive strength may not always be preferable, or even desirable for bridge decks. Lightweight mixes used in bridge decks require strengths around 4,000 psi. Indeed, higher strengths in cast-in-place bridge deck applications are often accompanied by increased frequency of cracking and related durability issues. Therefore, extensive material testing of moderate strength mixtures is not proposed in the work plan, though they may provide greater durability in bridge deck applications.

Table E-2: Structural Grade Lightweight Aggregate Manufacturers

| Manufacturer | Max Compressive Strength | Corresponding Unit Weight | Max Absorption |
|--------------------------------|---------------------------------|----------------------------------|-----------------------|
| Big River Industries | 4,349 psi | 111.8 pcf | N/A |
| Buildex, Inc. | 6,065 psi | 114.3 pcf | 30% |
| Carolina Stalite Company | 12,945 psi | 113 pcf | 6% |
| DiGeronimo Aggregates LLC | No Data Received | | 10% |
| Hydraulic Press Brick Co. | 6,425 psi | 113 pcf | 10% |
| Norlite Corporation | No Data Received | | |
| Northeast Solite Corporation | No Data Received | | |
| Solite, LLC | 5,830 psi | 117 pcf | 16.7% |
| Texas Industries, Inc. – Clay | 6,470 psi | 125.6 pcf | 21.8% |
| Texas Industries, Inc. – Shale | 6,300 psi | 122.4 pcf | 22% |
| Utelite Corporation | 5,230 psi | 121.4 pcf | 16.8% |

Based on the data obtained from the various lightweight aggregates manufacturers, the research team concluded that the highest feasible design compressive strength for production lightweight aggregate concrete is 10,000 psi. The target average compressive strengths to meet a design strength of 10,000 psi would be between 11,000 psi, and 12,000 psi, depending upon variability in production. Though there have been documented results showing that an expanded lightweight aggregate is capable of strengths in the 12,000 psi range, a lightweight concrete mixture yielding compressive strengths consistently above 12,000 psi does not seem attainable, and would require cementitious content at proportions that would likely induce complications due to shrinkage and creep. It is for this reason the research team will focus on developing mixtures with design compressive strengths in the 8,000 and 10,000 psi range.

The research team recommends four aggregate sources to use in laboratory testing for this project. The four sources are as follows: an expanded clay source from California; an expanded shale source from Kansas; an expanded shale source from Indiana; and an expanded slate source from North Carolina. The selection process took into account the data acquired from the various manufacturers and the highest documented compressive strengths reported by those suppliers. The particular sources were chosen in order to optimize the geographic distribution, to assure at least one source from each of the three material types (clay, shale, slate), and to select sources believed to be capable of achieving the strength goals proposed in the work plan.

Task 2- Review of AASHTO LRFD Bridge Design and Construction Specifications

Scope

Review AASHTO LRFD Bridge Design and Construction Specifications. Identify and discuss the provisions that have limited the application of lightweight concrete in highway bridges and could be improved based on new research. Prepare a prioritized list of specific issues that are hindering the use of high-strength and high-performance lightweight concrete in bridges. Also, identify material properties and other factors relevant to the design, construction, and durability of bridge girders and decks (creep, shrinkage, permeability, heat of hydration, etc.) and discuss their impact on the use of lightweight concrete in bridge decks and girders.

Introduction

The research team have reviewed both AASHTO (4th edition) and AASHTOC (1998) to determine the articles that specifically address lightweight concrete. Articles that don't currently address lightweight concrete, but possibly should, have been identified as well. Some of these areas have been identified in the background section of this report.

AASHTO LRFD Bridge Design Specifications

The following two tables present the references within AASHTO that mention lightweight concrete or may need to. Table E-3 presents sections discussing material properties and Table E-4 presents sections discussing structural design. Included in both tables is a summary of findings regarding the specific code sections based on the literature review and a review of AASHTO.

In general, sections of AASHTO that deal with material properties of concrete do not mention lightweight concrete; and, if they do, need revision. Of special importance are equations for predicting tensile strength and modulus of elasticity of lightweight concrete as well as time dependent material properties (i.e., creep and shrinkage). All have a significant effect on design of prestressed concrete girders

There several areas of prestressed girder and deck design with lightweight concrete that need further investigation. For lightweight concrete decks, the modification factor for development length of mild steel in lightweight concrete needs further evaluation. For prestressed girders the following factors rate moderate or high and need further investigation: resistance factors, flexural resistance, deflection and camber, shear design, allowable stress limits, loss of prestress, strand spacing limits, and bond of strand to lightweight concrete.

Table E-3. Material Properties

| Section | Title | Is LWC mentioned? | Probability of Change | Summary |
|----------------|----------------------------------|--------------------------|------------------------------|--|
| 5.4.2.1 | Compressive Strength | Yes | Low | Is $f'_c = 10,000$ achievable? |
| 5.4.2.2 | Coefficient of Thermal Expansion | Yes | Low | This is an area where a synthesis of existing data will be sufficient to evaluate the current AASHTO provisions. |
| 5.4.2.3.2 | Creep | No | Moderate | Previous research shows conflicting conclusions regarding creep of lightweight concrete. Creep (as with shrinkage) has a major impact on the level of prestress loss in a prestressed bridge girder. |
| 5.4.2.3.3 | Shrinkage | No | Moderate | Researchers have generally found that lightweight concrete experiences more drying shrinkage than normal weight concrete. This appears to be especially true for high performance, lightweight concrete. |
| 5.4.2.4 | Modulus of Elasticity | No | High | Lightweight aggregate has a significant impact on E_c . The AASHTO equation for E_c may be unconservative for lightweight concretes. |
| 5.4.2.6 | Modulus of Rupture | Yes | Moderate | Lightweight aggregate can have a significant impact on f_r . Is AASHTO equation for f_r appropriate for use with lightweight concrete? |
| 5.4.2.7 | Tensile Strength | No | High | Equation in Commentary does not consider lightweight concrete. |

Table E-4. Structural Design

| Section | Title | Is LWC mentioned? | Probability of Change | Comment |
|----------------|--|--------------------------|------------------------------|---|
| 5.5.3 | Fatigue Limit State | No | Low | Usually does not control bridge design. |
| 5.5.4.2 | Resistance Factors | Yes | High | Lightweight concrete has a reduced factor for shear |
| 5.7.2.2 | Rectangular Stress Distribution | No | Moderate | 0.85 may or may not be justified for use w/lightweight concrete |
| 5.7.3.2 | Flexural Resistance | No | Moderate | f_{ps} equation was not based on lightweight concrete behavior |
| 5.7.3.5 | Moment Redistribution | No | Low | |
| 5.7.3.6.2 | Deflection and Camber | No | High | Proper deflection and camber predictions are a function of reasonable equations for E_c (5.4.2.4), f_r (5.4.2.6), creep (5.4.2.3.2), and shrinkage (5.4.2.3.3) |
| 5.7.5 | Bearing | No | Low | A lightweight reduction factor may be in order |
| 5.8.2.2 | Modifications for Lightweight Concrete (Shear and Torsion) | Yes | High | The adjustment made for lightweight concrete in shear calculations are on one term in V_{cw} equation and have little effect on overall shear strength of a member. |
| 5.8.2.5 | Minimum Transverse Reinforcement | No | High | Lightweight modifications result in greater amounts of reinforcement |
| 5.8.3.3 | Nominal Shear Resistance | No | High | Applicability of equations to lightweight concrete should be addressed |
| 5.8.4 | Interface Shear Transfer--Shear Friction | No | Low | |
| 5.9.4 | Stress Limits for Concrete (Prestressing) | No | Moderate | Effect of potential lower tensile strength of lightweight concrete may lead lower allowable tensile stresses |
| 5.9.5 | Loss of Prestress | No | Moderate | Little prestress loss data available for lightweight members |
| 5.10.3.3 | Minimum Spacing of Prestressing Tendons and Ducts | No | Moderate | Minimum spacing of prestressing strands in pretensioned members set for normal weight concrete |
| 5.11.2.1 | Deformed Bars and Deformed Wires in Tension | Yes | High | modification factor for lightweight concrete should be evaluated |
| 5.11.4.2 | Bonded Strand (Development of Prestressing Strand) | No | High | Transfer and development length equations developed for normal weight concrete |

AASHTO LRFD Bridge Construction Specifications

AASHTOC does not directly address the use of lightweight or high performance, lightweight concrete in bridge girders or decks. However, there are several sections of this specification that cover areas that are critical to the proper construction of bridges containing lightweight concrete.

- Control of water content in lightweight concrete mixes is critical. This typically requires saturated lightweight aggregate stock piles (AASHTOC 8.5.1) and trial batches when changing the mix design or using a new mixing plant (AASHTOC 8.4.1.2)
- When determining the percent air content of fresh lightweight concrete only a roll-a-meter should be used (AASHTOC 8.5.6).
- Pumping lightweight concrete for bridge decks can be problematic because the concrete tends to loose slump if not proportioned properly (AASHTOC 8.7.3.2).
- Intentional roughening (or raking) of the top flange of prestressed girders is not recommended because of the natural tendency of the aggregate to rise to the top girder surface. Also, raking may lead to segregation of the aggregate and paste (AASHTOC 8.13.3).
- Extreme care should be taken when shipping and handling lightweight prestressed girders. Lightweight concrete generally has a smaller E_c than normal weight concrete girders which leads to potential lateral stability issues during handling and construction (AASHTOC 8.13.5).

These and other issues in fabricating girders and constructing bridge decks with lightweight concrete should be investigated further.

Task 3 – Items for Further Investigation

Scope

Based on the information gathered in Tasks 1 and 2, recommend for further investigation in Phase II, those issues, properties, and factors relevant to the use of lightweight concrete in prestressed concrete girders and decks that would support changes to the provisions of the AASHTO LRFD Bridge Design and Construction Specifications identified in Task 2.

Findings

An evaluation of issues, properties, and factors that need investigation is presented in Table 5. The issues listed as high or medium importance will be investigated as described in the Work Plan described in Task 4.

The background information presented previously illustrates many of the areas in which there is considerable uncertainty as to the correct treatment of lightweight concrete for prestressed bridge girders and bridge decks. Based on the literature and project review, discussions with bridge engineers, discussion with members of the lightweight concrete industry, and the review of AASHTO, those aspects of AASHTO that could be improved to better reflect the behavior of lightweight concrete have been identified. Table E-5 below provides the research teams' conclusions regarding what are the most important issues (see column labeled Importance Factor) and the corresponding test specimens that will be used to address these issues. Included in the table are the material properties that are important to a specific issue (for instance accurately predicting E_c of the lightweight concrete is critical to properly estimating girder cambers and deflections). Further details of the tests to be performed can be found in subsequent sections of this report.

Table E-5. Issues Identified for Further Investigation

| Issue | Importance Factor | Tests | | | |
|-------------------------------|-------------------|------------------------------|----------------|-------------------|-----------|
| | | Material Testing | Lab Cast Beams | Full size girders | Beam ends |
| Durability | High | * | | | |
| Shear & Torsional Strength | High | *(f_{ct}) | * | * | |
| Allowable stresses | Medium | *(f_{ct}, f'_c) | * | * | |
| Crack Control | Medium | *(f_{ct}) | | | |
| Deflection & Camber | High | *(E_c) | * | * | |
| Prestress Losses | High | *(E_c , Creep, Shrinkage) | * | * | |
| Transfer & Development of P/S | Medium | *(f_{ct}) | * | * | |
| Bond Strength of RC | Medium | | | | * |
| Punching Shear | Low | | | | |
| Bearing Strength | Low | | | | |
| Horz. Shear Strength | Low | | | | |
| Anchorage Zones | Low | | | | |

Task 4 – PHASE II Work Plan

Scope

Prepare an updated, detailed work plan for Task 6 that includes an experimental investigation of the properties and factors recommended in Task 3. It is expected that Task 6 will include design, construction, and testing of full-scale girder-deck assemblies and associated analysis to validate laboratory test results and structural design criteria.

Detailed Work Plan

The following describes the work plan for Phase II of the project. Phase II is divided into three tasks (Tasks 6, 7, and 8) with Task 6 having six subtasks.

The six subtasks for the Task 6 are:

- Subtask 6.1 Material Properties Testing
- Subtask 6.2 Material Property Verification Testing
- Subtask 6.3 Lab Cast Beams
- Subtask 6.4 Full Scale Beams
- Subtask 6.5 Interface Shear Transfer

- Subtask 6.6 Recommend changes to AASHTO Bridge Specifications

Subtask 6.1 Material Properties Testing

During Task 1 of this project, a detailed literature review was conducted to assess the depth of knowledge of the material properties of commonly available lightweight concrete. There are sources of lightweight aggregate throughout the country, and the material properties of aggregate from these different sources are already well documented. The research team compiled a database of national, as well as, regional sources. This database includes the source and any documented material properties for each individual source, such as elastic modulus, unit weight, specific gravity, absorption rates and angularity of the aggregate. Most of the sources have been properly characterized with regards to material properties. The research team used the completed database to select six candidate aggregates for testing in this project:

Source A - expanded clay from California (TXI-Hydrolite)

Source B - expanded shale from Colorado (TXI-Realite)

Source C - expanded shale from Indiana (HBPC-Haydite)

Source D - expanded slate from North Carolina (CSC-Stalite)

Source E - expanded clay from Louisiana (BRI-Gravelite)

Source F - expanded shale from New York (NSC-Solite)

The properties in the database will aid in the mixture design and proportioning of the lightweight concrete mixtures used for this project.

Based upon the findings of the literature search, detailed screening tests will be executed on concrete mixtures from the six aggregate sources (designated A through F). The selected sources are from the West, Midwest, South, East and Northeast regions of the country.

Testing will consider two nominal coarse aggregate sizes, $\frac{3}{4}$ in. and $\frac{1}{2}$ in. However, mix designs may vary depending upon availability of the aggregate sizes. Mixtures proportions will consider water-to-cementitious (w/cm) ratios of 0.30, representing high-strength, low permeability beam mixtures, and 0.40, representing moderate strength and permeability beam mixtures as well as good quality concrete deck mixtures. Alternative mineral admixtures to ordinary portland cement may include fly ash (FA), ground granulated blast furnace slag

(GGBFS), and silica fume (SF), each dosed at a typical rate to produce high-performance concrete mixtures. For deck concrete mix designs, Type 1/2 portland cement (Type 1/2 PC) will be used and, for girder concrete mix designs, Type 2 modified portland cement (Type 2 Mod PC) will be used. These materials will be representative of nationwide use and will be proportioned accordingly to achieve several different desired strength levels. A full evaluation of all the potential combinations of variables is not feasible. Therefore, Table E-6 presents the anticipated number of batches of each mixture to be assessed. The decisions inherent therein are intended to balance time, resources and predicted influence of each variable, while endeavoring to render statistically valid results.

Table E-6. Proposed Materials Screening Test Matrix

| Paste | | Coarse Aggregate Types | | | | | |
|-------|---------------------------|------------------------|---|---|----|---|---|
| | | A | B | C | D* | E | F |
| w/cem | Binder | Number of Batches | | | | | |
| 0.3 | Type 2 Mod PC | 2 | 2 | 2 | 2 | 2 | 2 |
| | Type 2 Mod PC + 20%FA** | 2 | 2 | 2 | 2 | 2 | 2 |
| | Type 2 Mod PC + 40% GGBFS | 2 | 2 | 2 | 2 | 2 | 2 |
| | Type 2 Mod PC + 7.5%SF | 2 | 2 | 2 | 2 | 2 | 2 |
| 0.4 | Type 1/2 PC | 2 | 2 | 2 | 2 | 2 | 2 |
| | Type 1/2 PC + 20%FA** | 2 | 2 | 2 | 2 | 2 | 2 |
| | Type 1/2 PC + 40% GGBFS | 2 | 2 | 2 | 2 | 2 | 2 |
| | | Total = 84 | | | | | |

* Source D represents aggregate source with highest strength potential

** Type and dosage of fly ash may vary based on standard practice at aggregate source

Each of the batches mixed during materials screening will undergo a battery of tests as presented in Table E-7. These tests will characterize the relative mixture designs and enable the research team to select final LWHPC mixtures to be used in structural testing.

Table E-7. Tests for Material Screening

| Test Method | AASHTO (ASTM) Reference | Age | | | |
|--|----------------------------|-------------------|------------|------------|------------|
| | | 7 days | 28 days | 56 days | 90 days |
| | | Number of Samples | | | |
| Compressive Strength / Modulus of Elasticity | T 22 (C39/C469) | 2 | 3 | 3 | 2 |
| Modulus of Rupture | T 97 (C78) | - | 3 | - | - |
| Split Tensile Strength | T 198 (C496) | 2 | 3 | 3 | 2 |
| Shrinkage | T 160 (C157) | 3 | - | - | - |
| Permeability / Absorption | T 277 (C1202/C1585) | - | 2 | 2 | - |

Creep testing on each mix design from Table E-5 is not possible due to time and budget constraints. However, a supplemental creep testing program will be conducted in order to determine creep characteristics on a select number of lightweight concrete mix designs. The creep testing will be conducted using three creep frames with a capacity to test nine cylinders per cycle (hence three mix designs at a time). The testing for each cycle will be conducted for a period of 90 to 180 days, until the creep response becomes stable. The priority for creep testing will be on the high performance/high strength girder mixtures because these mix designs are needed first for the lab cast specimens (SubTask 6.3). Creep testing of the moderate strength girder mixtures will follow, and then the deck mixtures later. Table E-8 presents the test matrix for creep testing. Note that creep testing will be conducted on three lightweight concrete girder concrete mix designs. It is anticipated that only two of the lightweight aggregate sources will have aggregate properties adequate to produce lightweight concrete which can achieve these strength levels. However, it is expected that several aggregate sources can be used to reach the compressive strength needed for bridge decks (4 ksi). Assuming screening test results are sufficient, four bridge deck mix designs will be used in the creep testing. Only one of the four lightweight mix designs developed for use in decks will be utilized in the structural testing portion of the Work Plan.

Table E-8. Creep Test Matrix

| Mix Designs for Creep Testing | Use | First Test Group | Second Test Group | Third Test group |
|-------------------------------|--------|------------------|-------------------|------------------|
| NWHPC1 (8 ksi) | Girder | | * | |
| LWHPC 1 (8 ksi) | | * | | |
| LWHPC 2 (8 ksi) | | * | | |
| LWHPC 3 (10 ksi) | | * | | |
| NWHPC2 (4 ksi) | Deck | | * | |
| LWHPC 4 (4 ksi) | | | * | |
| LWHPC 5 (4 ksi) | | | | * |
| LWHPC 6 (4 ksi) | | | | * |
| LWHPC 7 (4 ksi) | | | | * |

The developed mix designs for deck concrete will have typical target compressive strengths for that use ($f'_c = 4,000$ psi) and will include variations of tensile strength. The developed mix designs for girder concrete will include one that is typical of present practice ($f'_c = 8,000$ psi) with two different aggregates, with differing tensile strengths, and one that is higher than typically used in bridges ($f'_c = 10,000$ psi). As stated earlier in the report, the research team views the maximum design strength achievable on a production basis as 10,000 psi. The research team will also use air entrainment admixture for the purposes of improved freezing and thawing durability. Lastly, a typical NWHPC mixture will also be used to cast similar specimens and used as a control for structural test comparisons. Table E-9 shows the types of concrete mixes that will be developed for and used in the structural testing portion of this project.

Table E-9. Concrete Mixtures for Structural testing

| Type (Compressive Strength) | Tensile strength | Specimens | Bridge Use |
|-----------------------------|------------------|--------------------------------------|------------|
| NWHPC1 (8 ksi) | Normal | Lab cast, full size, interface shear | girders |
| LWHPC 1 (8 ksi) | High | Lab cast, full size, interface shear | girders |
| LWHPC 2 (8 ksi) | Low | Lab cast, full size | girders |
| LWHPC 3 (10 ksi) | Average | Lab cast & full size | girders |
| NWHPC2 (4 ksi) | Normal | Full size & interface shear | deck |
| LWHPC 4 (4 ksi) | Average | Full size & interface shear | deck |

Subtask 6.2 Material Property Verification Testing

During casting of the structural specimens, the research team will obtain a number of samples of the production concretes to complete a comprehensive battery of material tests. In addition to the material properties assessed in the screening phase, tests will be performed to characterize shrinkage performance of the mixtures, which may affect prestress losses, as well as the bulk diffusion characteristics of the concrete, which influence durability. During fabrication at the precast plant, specimens will be obtained that are the same mixture proportions as the mixtures used in the laboratory. This will give an indication of the variability between the lab and field mixtures. The test matrix is presented in Table E-10.

Table E-10. Battery of Material Tests for Full-scale Specimens (Number of Specimens)

| Method | AASHTO (ASTM) Reference | Age (days) | | | | |
|--|----------------------------|------------|---|----|----|----|
| | | 1 | 7 | 28 | 56 | 90 |
| Compressive Strength / Modulus of Elasticity | T 22 (C39/C469) | - | 2 | 3 | 3 | 2 |
| Modulus of Rupture | T 97 (C78) | - | | 3 | | |
| Split Tensile Strength | T 198 (C496) | - | 2 | 3 | 3 | 2 |
| Shrinkage | T 160 (C157) | 3 | - | - | - | - |
| Permeability / Absorption | T 277 (C1202/C1585) | - | - | 2 | 2 | - |
| Bulk Diffusion | None (C1556) | - | - | 2 | - | - |
| Freeze-thaw | T 161 (C666) | - | - | 2 | - | - |

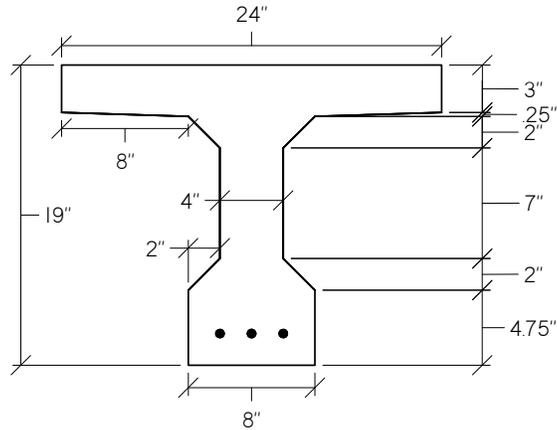
Subtask 6.3 Lab Cast Beams

The test beams have been designed to optimize the data that can be collected from testing these beams. Of particular interest will be data related to transfer length, development length, flexural strength, prestress losses, allowable stresses, and initial and long term camber. In addition, even though it is not within the scope of this portion of the project, some information with regards to shear strength of LWHPC will be obtained. The cross sections for beams containing ½ in. and 0.6 in. diameter strand are shown in Figure E-4. The beam cross section for 0.6 in. diameter strand is larger than the one for 0.5 in. diameter strand so that the state of stress in the two cross sections due to prestressing will be similar. The mild steel reinforcement shown in the cross section used with the 0.6 in. diameter is the same as what will be used in the specimens containing 0.5 in. diameter strand.

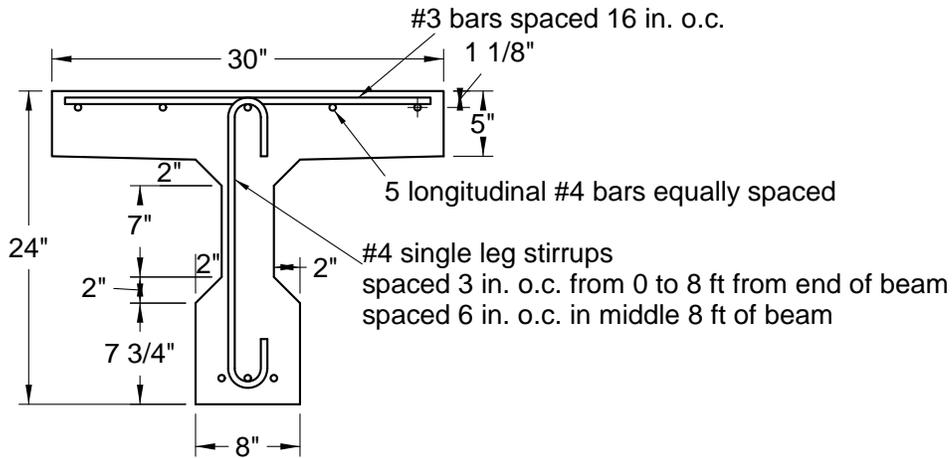
The capability exists within the Virginia Tech Structures and Materials Laboratory to cast pretensioned, prestressed concrete beams. The prestressing buttresses can anchor up to nine 0.6-in. diameter prestressing strands, and the buttresses can be set approximately 52 ft apart. With two prestressing lines, this allows for the manufacture of four- 24 ft long beams at a time. The lab cast beams will be made using this facility.

Since the lab cast beams are meant to be a smaller model of full size prestressed girders, concrete mix designs for girder concrete will be used. The mix design matrix is shown in Table 8 and the girder concrete mix designs are NWHPC1, LWHPC1, LWHPC2, and LWHPC3. These mixes have either a 8,000 psi or 10,000 psi design compressive strength. Both 0.5 in. and 0.6 in. diameter strands will be investigated. Due to the potentially lower tensile strength of lightweight concrete the minimum spacing requirements for prestressing strand given in AASHTO may not be applicable to lightweight concrete prestressed beams. On all of the lab cast beams the strands will be placed at a 2 in. center-to-center spacing which is the typical spacing used in prestressed girders.

Each beam will be instrumented with Demec gage points so that transfer length of the strand can be measured. Transfer length will be determined by measuring concrete surface strain in the end region of the members immediately after transfer. Also, every beam cast will be equipped with vibrating wire gauges to measure changes in strain in the concrete over time. Companion cylinders will be made and tested regularly to ensure that the concrete delivered from the ready-mix plant is similar in quality and behavior to the test batches made during the material properties characterization testing. Compressive strength, tensile strength, and modulus of elasticity tests will be repeated. In addition, the NASP (National Association of Strand Producers) test for documenting strand surface condition will be performed on samples of all strand to insure that the strand is acceptable. Long term material properties (creep and shrinkage) of the mixtures used in the structural testing will be investigated as well (as discussed earlier). The lab cast beams will be allowed to sit undisturbed for at least three months. During this time the internal strain gages will be monitored to collect data for the investigation of prestress loss and camber growth.



a) Cross Section for 1/2 in. Diameter Strand



b) Cross Section for 0.6 in. Diameter Strand

Figure E-4. Lab Cast Beams

After the beams have been stored for a minimum of three months, flexural testing will begin. The beams will be made long enough so that two tests per beam can be achieved. A schematic of a typical test set-up is shown in Figure E-5. The primary test configuration will be simply supported with a concentrated load applied at varying shear spans. The load point will be adjusted to precipitate bond and flexural failures, and, therefore, iteratively determine development length. The primary purpose for this testing is to determine development length of the strand in lightweight concrete, however, these tests will also provide important information about flexural strength and shear strength. The test matrix is shown in Table E-11.

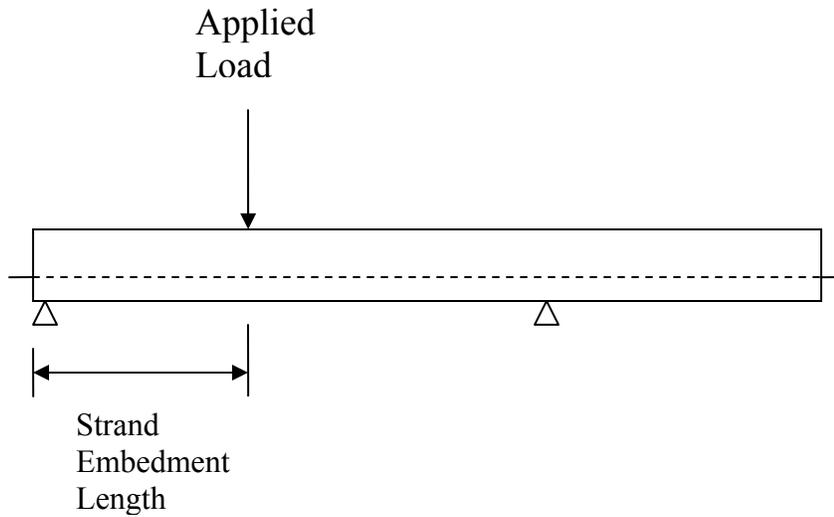


Figure E-5. Development Test Set-up

Table E-11. Proposed Test Matrix for Lab Cast Beams

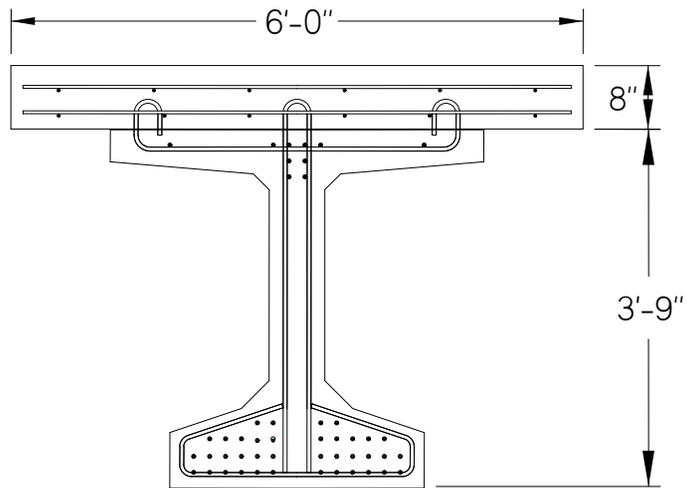
| Concrete Strength (ksi) and Designation | Strand Size In. | Number of Beams | Measured Transfer Lengths | Flexural/Development Length Tests |
|---|-----------------|-----------------|---------------------------|-----------------------------------|
| 8 – LWHPC 1 | 0.5 | 2 | 4 | 4 |
| 8 – LWHPC 2 | 0.5 | 2 | 4 | 4 |
| 10 – LWHPC 3 | 0.5 | 2 | 4 | 4 |
| | 0.6 | 2 | 4 | 4 |
| 8 – NWHPC 1 | 0.5 | 2 | 4 | 4 |
| | 0.6 | 2 | 4 | 4 |

Subtask 6.4 Full Scale Beams

A total of six full scale prestressed girders will be cast at a precasting plant in the region and tested in the Virginia Tech Structures Laboratory. A steam cure will be used on these girders during the first 24 hours of concrete maturity for increasing strength gain. Each will have a composite reinforced concrete deck added before testing which will be cured using the traditional moist curing regimen. Four of the girders will be 45 in. deep, 60 ft long. Three of these beams will be fabricated with lightweight concrete (one with LWHPC1 and two with LWHPC3, mix designs). One will be cast using normal weight concrete ($f'_c = 8$ ksi) for comparison purposes. Figure E-6 shows a cross section of the PCBT-45 girder. In addition, two

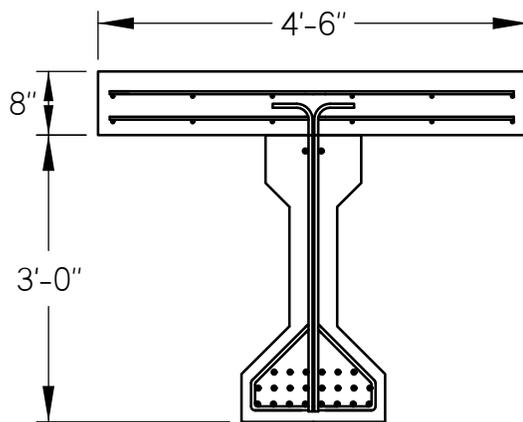
AASHTO Type II girders will be fabricated with the LWHPC1 mix, and Figure 7 shows a cross section of the AASHTO Type II. The strand pattern for the PCBT-45 and AASHTO Type II beam is based on 44-ft wide bridge with girders spaced at 8 ft and spanning 100 ft. and 60 ft., respectively. In the precast yard these beams will be instrumented to measure transfer length and internal strains and temperatures. Strain gages will be placed on selected shear stirrups. The beams will be allowed to age for approximately three months while camber changes and strain changes are monitored.

Then, the beams will be delivered to the Virginia Tech Structures and Materials Lab for testing. At the lab, a cast-in-place/composite deck will be placed on each beam. In research conducted at the University of Texas at Austin (Sylva, Breen, and Burns 2002) it was shown that the combination of a normal weight concrete deck with a lightweight concrete girder is not as structurally efficient as lightweight concrete used in both. Therefore, the deck will also be lightweight concrete. Curing of the decks will be done following common industry practice. The tests performed will investigate high shear and a combination of high moment and high shear in the lightweight concrete girders. The girders will be long enough for two tests per beam (one high shear and one high shear and moment).



PCBT-45

Figure E-6. Cross Section of PCBT-45 Girder



AASHTO Type II

Figure E-7. AASHTO Type II girder with cast-in-place deck

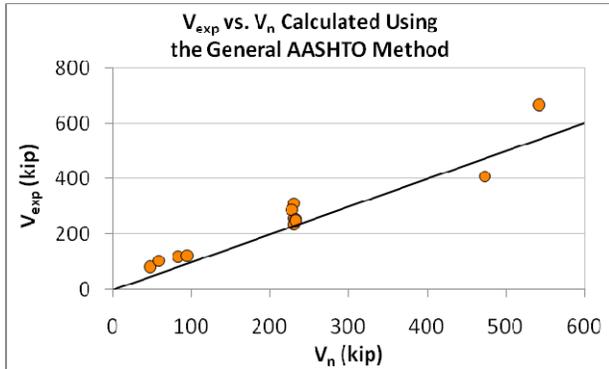
Background. Figures E-8 and E-9 show only the prestressed concrete beam test results. Figure 8 a) is a comparison of tested versus predicted strength for all of the results found. There were twelve test results found for lightweight, prestressed members and only two of them were for composite girders. It should be noted that most of the test results are far relatively small, non-composite test specimens.

The analysis of the previously reported data indicates that the proposed testing program should address several gaps in the data presented in Figures 5 and 6. Of most significance are the three following parameters:

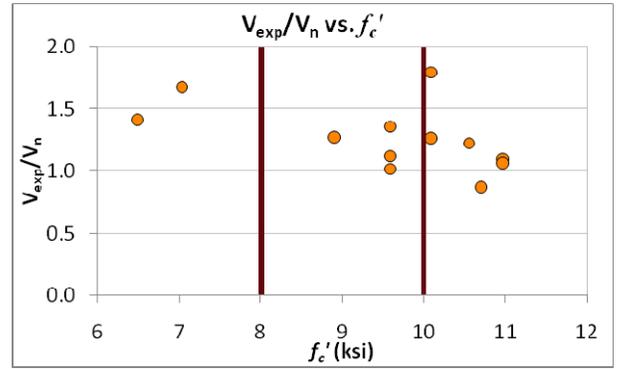
1. a/d ratio: Figure 5 d) shows a general lack of data at small a/d ratios and a concentration of data at an a/d ratio of 3.0. The girders to be tested will be loaded in such a fashion to produce a/d ratios that fall within the two shaded areas in the figure. Specific loading arrangements are shown later in Figure 7.
2. Cross-sectional area of composite girder: Figure 5 e) shows a concentration of test results at a small composite girder area (about 450 in²) with two test results at a more realistic area (about 1550 in²). The larger area is for a 53 inch deep prestressed girder typical of the size used in 100 ft span bridges while the smaller area is below that typically used in bridge design presently. It is proposed to test three different girder sizes to fill out the gaps in this plot. The sizes are represented by the colored vertical lines in the figure and are approximately 180 in², 800 in², and 1350 in². These are the lab cast girders, AASHTO Type II girder with added deck, and PCBT-45 with added deck.
3. Shear reinforcement ratio (ρ_v): Figure 6 b) shows a small amount of test results at small amounts of shear reinforcement and a fair amount of data at typical amounts ($0.5 \leq \rho_v \leq 0.75$). The plan for this testing program is to focus testing on the two areas in the figure that are indicated (vertical bar near ρ_v of 0.1 and shaded area).

Figures 5 b), 5 c), and 5 d) also are provided to show where the planned tests fall in relation to girder concrete compressive strength, girder cross sectional area, and effective girder depth, respectively. The vertical bars on these three plots represent the planned test specimen. In addition, Figure 6 a) shows the test results as a function of unit weight of the lightweight

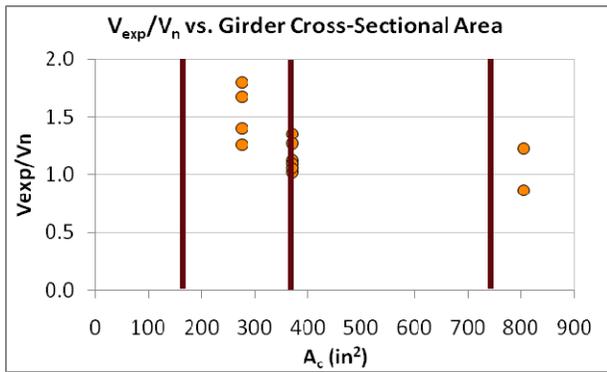
concrete used. The shaded area in this figure shows the planned unit weight for the full scale beams.



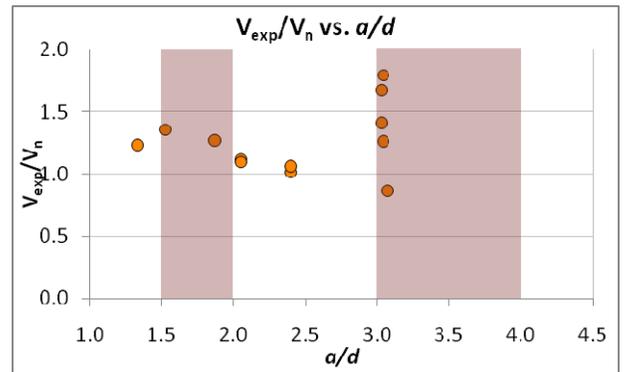
a)



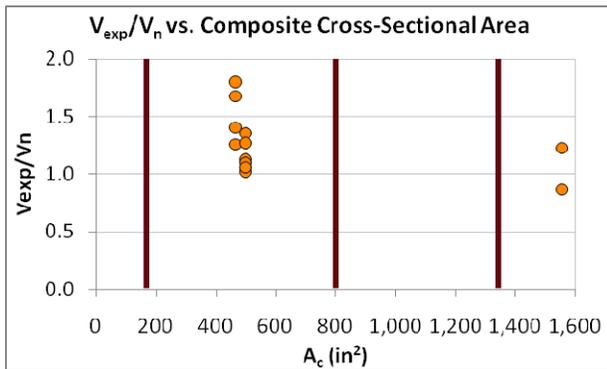
b)



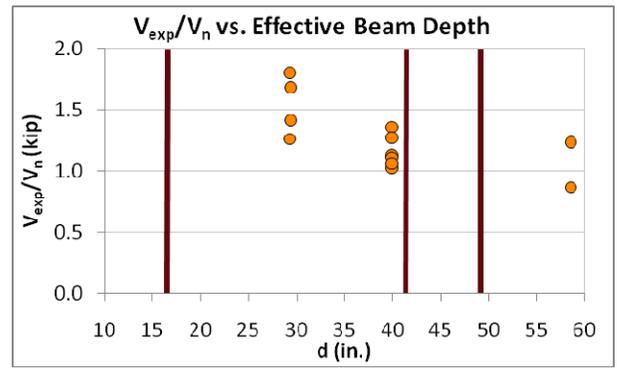
c)



d)

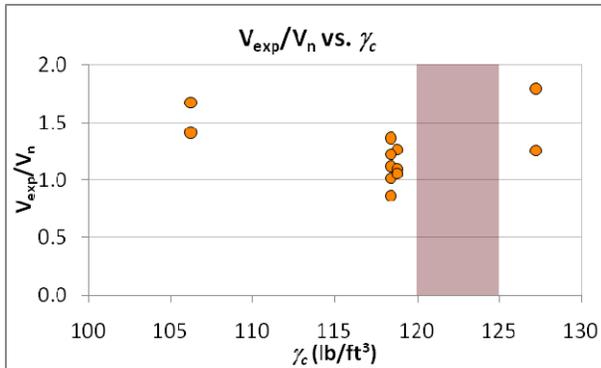


e)

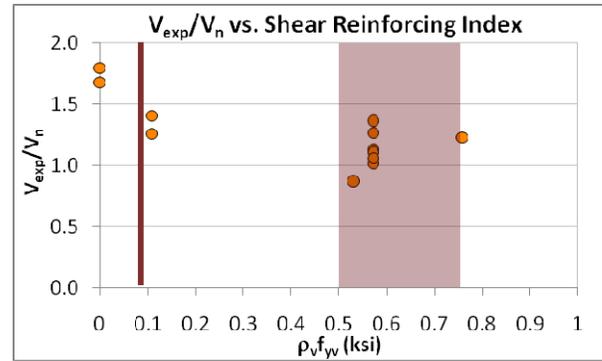


f)

Figure E-8. Lightweight Prestressed Concrete Test Beam Database compared to proposed test program.



a)



b)

Figure E-9. Lightweight Prestressed Concrete Test Beam Database compared to proposed test program.

The final testing program is presented in Figure E-10. Beams NW8, LW8 and LW10a will have identical prestressing layouts and shear reinforcing layouts. One end of the beam will be tested with a short shear span and the other a longer shear span. A comparison of the results of these tests will provide information on the differences in concrete contribution to shear strength between lightweight and normal weight concrete.

Specimen LW10b will have the AASHTO specified minimum shear reinforcement. Questions have been raised in the past about the safety of beams designed with this small amount of reinforcement. Results from these tests will help determine if the specified minimum shear reinforcement is appropriate for beams with lightweight concrete.

Specimens LW8a and LW8b are both AASHTO Type II beams with a composite lightweight concrete deck. LW8a will have a typical amount of shear reinforcement and LW8b will have the AASHTO minimum amount of shear reinforcement. These tests will provide information similar to that gained with the PCBT-45 test beams, but the data will fill different gaps in the current data base.

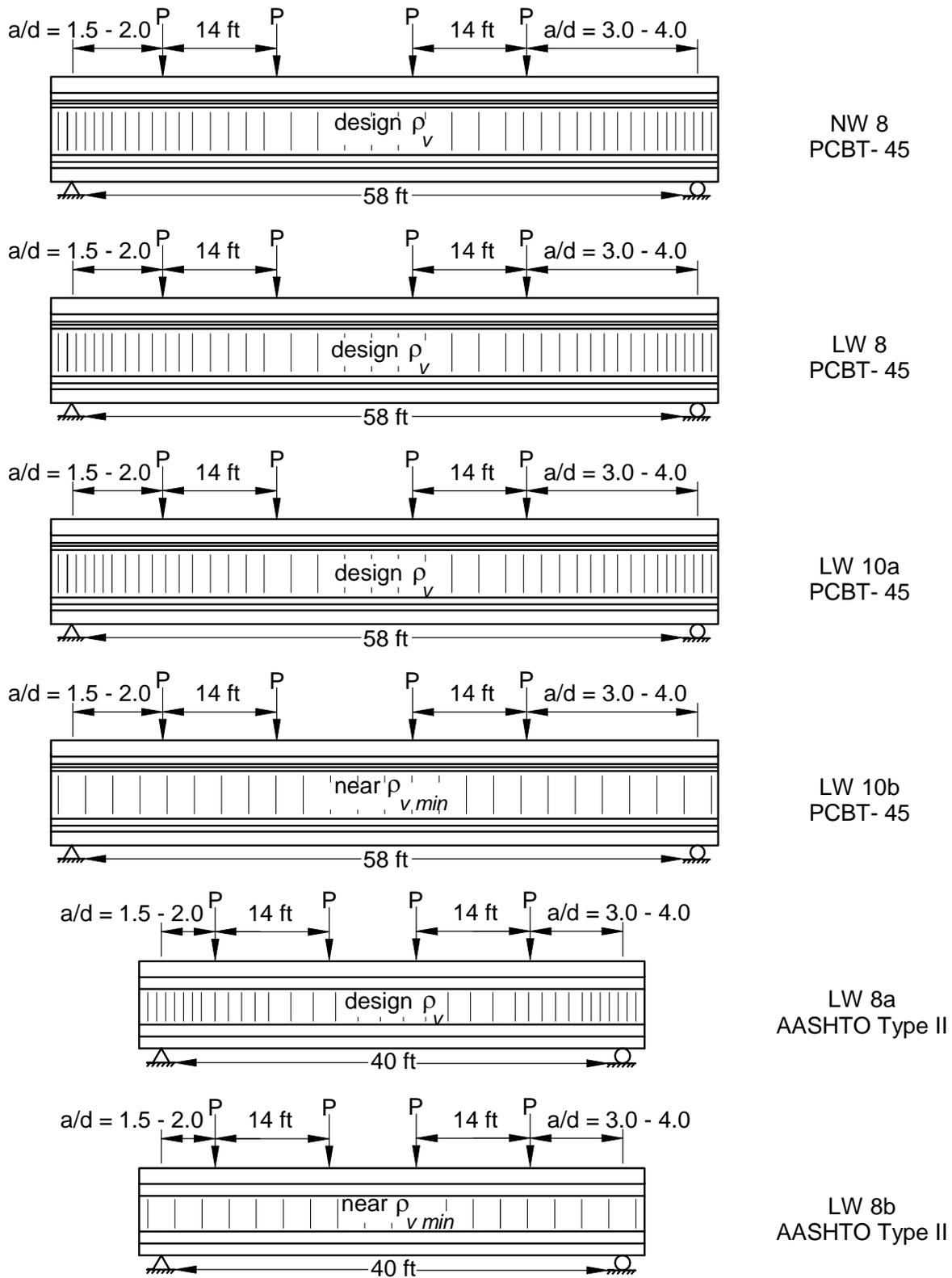


Figure E-10. Shear Testing Plan

In NCHRP Report 579, Kuchma and Hawkins presented a method to separate the contributions of concrete and steel to shear strength. The method required that a large number of strain gages be placed on the shear reinforcing. After completion of a test, the failure crack was determined, and the strain gages in closest proximity were identified. Interpolation was used to approximate the strains in stirrups which had not been gaged. Interpolation was also used when the failure crack passed between gage locations on a stirrup. A free body diagram between the end of the beam and the critical crack was drawn. At each load step, the applied shear was known and the total force in the stirrups crossing the crack was estimated. The concrete contribution to strength could then be calculated.

This method will be used with the results from the tests performed in this research program. It is critical to determine if only the V_c component of strength is different between normal weight and lightweight concrete, or if the V_s term is also different. If the angle of the crack varies with unit weight, then both V_c and V_s will vary. The comparison of the results of tests on specimens NW8, LW8 and LW10a will help to clarify these areas of uncertainty.

Preliminary designs of the tests beams have shown that the lightweight reduction factor (λ) has only a small influence on the overall calculated strength of the beam. This was also shown in analysis of Dymond's test results (Dymond 2007), which indicated that using the AASHTO Sectional Model with λ the predicted shear strength of the first test was 600 kips, while the prediction without λ was 620 kips. The shear at failure was 660 kips. It is extremely difficult to conclude if the λ factor is or is not appropriate because the difference in calculated strength is so small compared to the total failure load. Using the method presented by Kuchma and Hawkins should help somewhat to isolate V_c so better conclusions can be drawn about the λ factor.

Subtask 6.5 Interface Shear Transfer

The AASHTO LRFD formulation to calculate nominal shear resistance of the interface plane between a precast girder and a cast-in-place deck is as follows:

$$V_{ni} = cA_{cv} + \mu[A_{vf}f_y + P_c] \quad (1)$$

The term “c” represents the cohesion between the two concretes at the interface. The cohesion term is 0.24 ksi for lightweight concrete placed monolithically, or non-monolithically, against a clean concrete surface, free of laitance with surface intentionally roughened to an amplitude of 0.25 in. The term “μ” represents the friction coefficient along the crack. This term is 1.0 for the previously outlined conditions. In the commentary to this section in the LRFD Specifications, it is stated that there is a paucity of data on non-monolithic lightweight concrete, so the coefficients that were derived from monolithic tests are used. As with vertical shear, the strength reduction factor for interface shear for lightweight concrete is smaller than for normal weight concrete ($\phi = 0.7$ for lightweight concrete and $\phi = 0.9$ for normal weight).

The specification also stipulates that the nominal shear resistance used in design shall not be greater than the lesser of:

$$V_{ni} \leq K_1 f'_c A_{cv}$$

or

$$V_{ni} \leq K_2 A_{cv}$$

For lightweight concrete $K_1 = 0.25$ and $K_2 = 1.5$ ksi.

Also: A_{vf} = area of reinforcement crossing interface,

A_{cv} = area of interface,

f_y = yield stress of reinforcement,

P_c = permanent net compressive force normal to the shear plane.

f'_c = specified 28-day compressive strength of the weaker concrete on either side of the interface

In this task, push-off tests will be performed to investigate the interface shear strength of cast-in-place concrete decks on precast concrete girders. A typical push-off test is shown in Figure E-11.

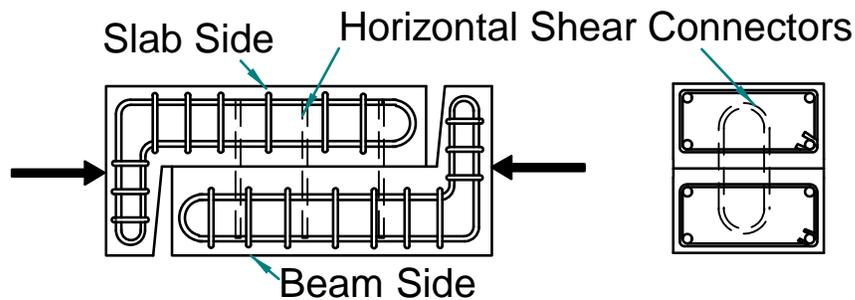


Figure E-11. Typical Push-off Test Specimen

Four different configurations of slab and girder concrete will be investigated. The four combinations are as follows:

1. Normal weight beam with normal weight deck,
2. Normal weight beam with lightweight deck,
3. Lightweight beam with normal weight deck,
4. Lightweight beam with light weight deck.

For each combination of deck and girder concrete, four different levels of horizontal shear reinforcement will be tested. The four levels will be:

1. No horizontal shear reinforcement,
2. Minimum horizontal shear reinforcement (approximately 1 ea. #4 stirrup (two legs across interface) on a 16 in. by 24 in. interface),
3. Maximum horizontal shear reinforcement (approximately 5 ea. #6 stirrups (10 legs) on a 16 in. by 24 in. interface),
4. Somewhere between minimum and maximum shear reinforcement (approximately 3 ea. #5 stirrups (six legs) on a 16 in. by 24 in. interface.)

For each configuration, three repetitions will be performed. This will provide an indication of the variability of the interface shear strength for normal weight and lightweight concretes. A test matrix is presented in Table E-12.

Table E-12. Test Matrix for Interface Shear

| Girder Concrete | Deck Concrete | $\rho_{iv} = A_{vf}/A_{cv}$ | Number of Repetitions |
|-------------------------------------|-------------------------------------|-----------------------------|------------------------------|
| Normal Weight 8000 psi NWHPC1 | Normal Weight 4000 psi NWHPC2 | 0.0 | 3 |
| | | 0.001 | 3 |
| | | 0.005 | 3 |
| | | 0.012 | 3 |
| | Lightweight 4000 psi LWHPC5 | 0.0 | 3 |
| | | 0.001 | 3 |
| | | 0.005 | 3 |
| | | 0.012 | 3 |
| Lightweight 8000 psi LWHPC1 | Lightweight 4000 psi LWHPC5 | 0.0 | 3 |
| | | 0.001 | 3 |
| | | 0.005 | 3 |
| | | 0.012 | 3 |
| Total Tests | | | 36 |

The top of the beam specimens will be roughened according to typical practice, with an amplitude of 0.25 in.

Subtask 6.6 Recommend changes to AASHTO Bridge Specifications

The research team will review both AASHTO (4th edition) and AASHTOC (1998) to determine the articles that specifically address lightweight concrete. Articles that don't currently address lightweight concrete, but possibly should, will be identified as well. Recommendations for modifications to both specifications will be made at the conclusion of this project.

Task 6 – Experimental Investigation and Analysis

Scope

Execute the plan approved in Task 5. Based on the results of this work, recommend changes to AASHTO LRFD Bridge Design and Construction Specifications to include provisions relevant to the use of lightweight concrete in prestressed concrete girders and in bridge decks. The recommended changes shall be prepared in a format suitable for consideration and adoption by

AASHTO. Also, develop guide specifications for the use of lightweight concrete in bridge girders and decks.

After review of the interim report by NCHRP, the details to be evaluated and the work plan will be finalized, the testing will then commence. Work on Phase II (Tasks 6, 7 and 8) will not commence until all the Tasks in Phase I have been completed and the work plan is approved.

TASK 7 – Comparative Deck and Girder Designs

Scope

Using the changes recommended in Task 6, develop comparative girder and deck designs with normal and lightweight concrete for selected conditions (e.g., foundation, span). Estimate relative costs. Identify the differences and the conditions that are likely to warrant use of lightweight concrete.

The goal of the comparisons will be to identify the conditions that are likely to warrant the use of lightweight concrete. This will be tied directly to the economic advantages that might be gained by fewer strands, wider girder spacings or longer span lengths that can be achieved with lightweight concrete.

Previous studies by Sylva, Breen and Burns (2002) and by Castrodale and Harmon (2006) have evaluated the advantages of lightweight concrete in bridge girders and decks through comparative designs. The results of these studies will be compared against the new comparative designs as final conclusions are drawn.

Girder Types

Longer span bridges are more likely to benefit from the reduced dead loads afforded by lightweight concrete than shorter span bridges. This is due to the higher percentage of dead load compared to total load in long span concrete bridges. Therefore, the focus of the comparative deck designs will be on medium to long span precast I-girder bridges. The comparative designs will be performed on the following girder types:

- Medium span – 90-110 ft, PCI-AASHTO BT-63
- Long span – 120-140 ft, VDOT-PCBT-85

All example problems will be performed for simple span configurations and for typical interior girders.

Combinations of Deck and Girder Concrete

For each type of girder four combinations of concrete types in the girder and in the deck will be investigated as follows:

- Normal weight girder, normal weight deck
- Normal weight girder, lightweight deck
- Lightweight girder, normal weight deck
- Lightweight girder, lightweight deck

Possible Comparative Studies

The benefits from the use of lightweight concrete may be realized in various ways, such as wider girder spacings for a given beam size and span length or longer span lengths for a given beam size and girder spacing. The economic impact will then be due to fewer required beams on a project or fewer bents. There is an overall economic benefit if these cost savings are greater than the premium paid for lightweight concrete. To investigate these possible benefits a variety of comparative designs will be performed in which some parameters are held constant and other parameters are maximized or minimized. Table E-13 presents the types of comparative studies to be performed in this task.

Reporting of Results

The final report will include one full design example of a lightweight concrete deck on lightweight prestressed I-girders. The example will highlight the differences between design with lightweight and normal weight concrete. The example will also reflect any changes to the LRFD Specifications recommended by the research team following the physical tests and evaluation of the results.

The results of the other comparative designs will be presented in a graphical or tabular format to best illustrate the differences in the outcomes of the designs with the various combinations of deck and girder concrete.

Table E-13. Comparative Deck and Girder Designs

| Given Parameters | Results to Compare |
|--|---|
| <ul style="list-style-type: none"> • Span Length • Girder Spacing • Release Concrete Strength • 28 day Concrete Strength | <ul style="list-style-type: none"> • Number of Strands • Required Shear Reinforcing • Deflections |
| <ul style="list-style-type: none"> • Span Length • Girder Spacing | <ul style="list-style-type: none"> • Lowest Feasible Release and 28 Day Concrete Strengths • Corresponding Number of Strands • Required Shear Reinforcing • Deflections |
| <ul style="list-style-type: none"> • Span Length • Release Concrete Strength • 28 Day Concrete Strength | <ul style="list-style-type: none"> • Maximum Feasible Girder Spacing • Corresponding Number of Strands • Required Shear Reinforcing • Deflections |
| <ul style="list-style-type: none"> • Girder Spacing • Release Concrete Strength • 28 Day Concrete Strength | <ul style="list-style-type: none"> • Maximum Feasible Span Length • Corresponding Number of Strands • Required Shear Reinforcing • Deflections |

Task 8 –Final Report

Scope

Submit a final report that documents the entire research effort. The report shall include an implementation plan for moving the results of this research into practice.

Based on all work performed a final report will be written and submitted. The specifications and example problems will be included as appendices to the report. A separate appendix will include all recommended specification and commentary changes.