Attachment C. Detailed Literature Review

Material Properties

Compressive Strength. The common design compressive strengths required by the construction industry for cast-in-place, precast, and prestressed structures range from 3000 to 8000 psi. These design strengths are economically met with the use of lightweight aggregate. Some lightweight aggregate concretes can obtain strengths above 8000 psi; however, not all lightweight aggregates are capable of obtaining these strengths.

A common concept used to indicate the maximum compressive and/or splitting tensile strengths of concretes using lightweight aggregate is a "strength ceiling." A mixture reaches its strength ceiling when, using the same aggregate, it possesses only slightly higher strength with higher cement content. This property is predominantly influenced by the coarse aggregate fraction of the mixture. The strength ceiling can be increased by reducing the maximum size of the coarse aggregate.

As with normal weight concrete, water reducing and mineral admixtures can be used with lightweight concrete to improve the workability, placing, and finishing.

Kahn et al. (2004) investigated the development of 8,000 psi, 10,000 psi, and 12,000 psi compressive strengths for high-performance lightweight concretes for precast, prestressed bridge girders. A strength ceiling of about 11,600 psi was found using a ½-in. expanded slate aggregate and normal weight natural sand. Laboratory and field mixtures were developed that met the 8,000 psi and 10,000 psi design strength, with the field mixtures attaining higher strengths.

Ozyildirim et al. (2005) investigated 8,000 psi and 4,000 psi design strengths for lightweight concretes used for beams and decks, respectively. Test beams were prepared and tested for material properties. A test mixture was designed for normal weight and lightweight high-performance concretes. The average 28-day compressive strength for the normal weight mixture was close to the 8,000 psi design strength, however, the average 28-day compressive strength for the lightweight mixture was below the 8,000 psi design strength. After 1 year, the average compressive strength for the normal weight mixture was above the 8,000 psi design strength, and the average compressive strength for the lightweight mixture was still below the design strength. The low compressive strength was attributed to excess water in the mixture. Therefore, it was determined that better water control was needed during mixture production.

Testing was also performed on the actual mixtures used for the bridge beams and deck. For the bridge beams the average 28-day compressive strength was at or near the target value of 8,000 psi. The average 28-day compressive strength for the deck was above the specified 4,000 psi design strength. From these results, the importance of water control in mixture production is apparent.

Zhou et al. (1998) found that for the same mixtures using either normal weight aggregate or lightweight aggregate, there was a 22% reduction in the compressive strength for the lightweight aggregate concrete mixtures compared to the normal weight concrete.

Reaching the 8,000 psi to 10,000 psi 28 day compressive strength threshold consistently with lightweight concrete (which is necessary for lightweight concrete to be competitive with high performance, normal weight concrete) is presently a challenge. Better understanding and further development of high performance, lightweight concrete mixtures is necessary so that this level of compressive strength can be reached at precasting plants on a routine basis with adequate workability.

Splitting Tensile Strength.Under continuous moist curing, lightweight and normal weight concretes have similar splitting tensile strengths, though lightweight exhibits greater variability. The typical strengths are between 300 and 550 psi depending on the compressive strength. For normal weight and lightweight concretes of similar compressive strength, the tensile strength of a normal weight concrete is typically in the middle of the tensile strength range for a lightweight concrete.

Zhou et al (1998) investigated the spitting tensile strength for high-strength normal weight and high strength lightweight concrete mixtures. Test results showed that there was a 26% reduction in the splitting tensile strength of the lightweight mixtures as compared to the normal weight mixtures.

For air dried specimens, the splitting tensile strength of normal weight concretes tends to be higher than that of lightweight concretes and less variable. The splitting tensile strengths of lightweight concretes are approximately 70 to 100% that of normal weight concretes at the same compressive strength. AASHTO addresses this difference in section 5.4.2.6, which covers modulus of rupture. For sand lightweight concrete the modulus of rupture for lightweight concrete for which no data exists is considered to be 83% of the modulus for normal weight concrete of the same compressive strength.

Splitting tensile strength is an important (maybe the most important) material property when it comes to impact on structural behavior and design. Splitting tensile strength has a major impact on shear strength, crack control, cover and spacing of strands, and tensile allowable stresses. Therefore, it is imperative that the splitting tensile strength of all trial mix designs be carefully evaluated.

Modulus of Elasticity. The modulus of elasticity of a given concrete depends on the relative amounts of aggregate and paste and their individual moduli. The modulus of elasticity of normal weight concrete is typically higher because of the higher modulus of the normal weight aggregate as compared to that of the lightweight aggregate. Typically, the modulus of elasticity for normal weight concrete ranges from 3 to 4 $x10^6$ psi. Lightweight concretes usually have a modulus of elasticity of about ½ to ¾ that of a normal weight concrete.

AASHTO addresses the lower modulus of elasticity of lightweight concrete by specifying an equation which includes a term for the unit weight. This equation should be further investigated for typical high-performance, high-strength lightweight concretes, because the modulus term is an extremely important component of prestress loss and deflection calculations. If the modulus is not accurately predicted, the other calculations will also be in error.

Kahn et al. (2004) obtained modulus of elasticity values for high-performance lightweight concretes in the range of 2,980 ksi to 4,680 ksi. Ozyildirim et al.(2005) obtained modulus of elasticity values of about 3,000 ksi for high-performance lightweight concrete. Ozyildirim's investigation also showed that the modulus of elasticity for the lightweight concretes was, as expected, lower than the normal weight concrete mixtures. Modulus of elasticity was measured for the beam and deck mixtures used in the investigation and compared to theoretical equations used by ACI. The values for the beam mixtures were a close match; however, the measured values for the deck mixtures were lower than the predicted values of the theoretical equations.

Stiffey (2005) conducted an investigation to determine a new equation that more accurately predicts the modulus of elasticity of lightweight concrete. The current equations specified by ACI 318 (ACI, 2005) and ACI 363 – Guide for High Strength Concrete have been found to be inaccurate for lightweight concrete. A percent difference statistical analysis between the theoretical and measured values was used to develop the new equations presented below:

$$E_c = 50\, w_c^{1.7} \sqrt[3]{f'_c} \qquad (1)$$

$$E_c = \left(173{,}400 \sqrt[3]{f'_c} + 1.0 * 10^6\right)\left(\frac{w_c}{145}\right)^{1.7} \qquad (2)$$

Stiffey stated that further research is needed to verify the proposed equations for all types of lightweight aggregate.

Lightweight aggregates have a significant effect on modulus of elasticity as discussed above. Modulus of elasticity is vital to accurately predicting girder camber, girder deflections, and prestress losses. The verification or modification of existing E_c models for lightweight concrete with compressive strengths up to 10,000 psi is necessary if high performance, lightweight concrete is to receive widespread use.

Internal Curing. In general, in high-strength (HSC) and high-performance (HPC) concretes the cementitious material contents in the mixtures are higher than those of a normal-strength concrete. Higher cement contents, as well as, the use of blast furnace slag, fly ash and silica fume are used to refine the pore structure. This creates a denser more refined pore structure in the cement fraction of the concrete. The denser more refined pore structure aids in reducing the transport mechanism of the environment into the concrete. Another concern associated with high cementitious material contents is that of shrinkage. With a higher cementitious material content and lower water-to-cementitious material ratios (w/cm), the magnitude of autogenous and drying shrinkage is increased (Hoff and Eng 2002).

The use of light-weight aggregate (LWA) in concrete can aid in the curing process. LWA is more porous than normal weight aggregate, thus it has a higher absorptive capacity. The use of a saturated LWA can provide additional moisture during the curing process through the release of moisture in the saturated LWA. The release of this moisture from the LWA during curing is often referred to as internal curing. Traditionally, HSC and HPC are vulnerable to increased shrinkage and cracking at early ages. As stated previously, this additional moisture aids in the curing of the concrete, producing a denser more refined pore structure. The internal curing reduces the internal stresses created by shrinkage in the mixture, thus reducing the cracking in the concrete (ESCSI 2006).

While the denser, refined pore structure of concretes can aid in reducing shrinkage of the mixture, it can also improve other properties of the hardened concrete. By creating a denser concrete, properties such as compressive strength, splitting tensile strength, modulus of elasticity and flexural strength may be improved. Furthermore, this improved pore structure can also aid in reducing the permeability of the concrete, thus making the concrete more durable. The increased durability of the concrete enables a longer lasting concrete that is able to withstand environmental degradation over time.

Drying Shrinkage. Drying shrinkage is the reduction in concrete volume due to water loss, and is important because it can affect the extent of cracking, prestress loss and warping in concrete structures. For normally cured concretes, lightweight concretes exhibit greater drying shrinkage than normal weight concretes at lower strengths. At higher strengths, the drying shrinkage of lightweight concretes is similar to that of normal weight concretes. The use of partial replacement of lightweight sand with normal weight sand has been shown to reduce the drying shrinkage. Steam curing aids in reducing the drying shrinkage of lightweight concrete by approximately 10 to 40%. The lower ranges of these drying shrinkage values are similar to typical normal weight concretes.

Vincent (2003) investigated the creep and shrinkage of the lightweight, high strength concrete used in the Chickahominy River Bridge in Virginia. He tested both standard-cure and match-cured specimens, as well as concrete produced in the lab and concrete produced in a precast plant. He compared his results with creep and shrinkage results from Meyerson (2001) who had tested high-performance, high-strength normal weight concrete of similar strength. Vincent noted that the shrinkage strains of the lightweight concrete were 30% higher than the normal weight concrete.

Kahn et al (2004) showed that the 620-day drying shrinkage values for 8,000 psi and 10,000 psi design strengths were 820 and 610 microstrain, respectively. Ozyildirim et al. (2005) showed that the one-year drying shrinkage for lightweight concrete ranged from 555 to 615 microstrain, while the normal weight concrete tested had drying shrinkage of 505 microstrain at one year.

Currently the AASHTO models for creep and shrinkage do not address unit weight; however, this is an area in which uncertainty remains, and which would benefit from further research.

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. Further study of drying shrinkage of high performance, lightweight concrete is required so that modifications to existing equations that predict prestress loss due to drying shrinkage can be made.

Creep. Creep is the increase in strain over time at a constant stress. The effects of creep can be either beneficial or detrimental to concrete structures. Creep can be beneficial by reducing concentrations of stress over time. However, creep can be detrimental because it may lead to excessive long term deflection and prestress loss, or loss of camber.

For normal curing, lightweight concrete typically has a higher specific creep (creep per psi of sustained stress) than normal weight concrete. The range of typical creep values for lightweight concrete can be narrowed by using normal weight sand in the mixture. Also, increasing the compressive strength from 3000 to 5000 psi significantly reduces the creep.

Steam curing for lightweight concrete aids in reducing the creep by about 25 to 40% of the creep of similar concrete subjected only to moist curing. The steam cured lightweight concretes have slightly higher creep values compared to normal weight concretes with the same compressive strengths. These differences are reduced at higher compressive strengths.

Vincent (2003) indicated that at 90 days the lightweight concrete creep specimens had twice the total strain of the normal weight specimens tested by Meyerson (2001). Vincent also noted that the time-dependent deformations of the lightweight concrete were growing at a more rapid rate at 90 days than the normal weight concrete specimens whose strain growth was very slight at 90 days. Lastly, Vincent noted that the GL2000 model (Gardner and Lockman, 2001) was the best predictor of time-dependent strains for lightweight concrete.

Kahn et al. (2004) observed high-performance lightweight creep values that were less than normal weight concretes after 620 days. The creep strains for the lightweight concretes with design strengths of 8,000 psi and 10,000 psi were 1800 and 1400 microstrain, respectively.

The research discussed above 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. Additional research can confirm the results reviewed above and lead to better design practices.

Thermal Properties. The coefficient of thermal expansion for lightweight concretes typically ranges from 4 to 5 x 10^{-6} in. / in. / °F. This is dependent on the amount of natural sand used in the mixture. AASHTO specifies using 6.0 x 10^{-6} in. / in. / °F for normal weight concrete and 5.0 x 10^{-6} in. / in. / °F for lightweight concrete. The lower coefficient of thermal expansion can be an advantage in bridges because the self-equilibrating and restraint stresses from thermal gradients will be smaller.

Kahn et al. (2004) obtained coefficients of thermal expansion for 8,000 psi and 10,000 psi design strength mixtures that were consistent with AASHTO specifications. The coefficients of thermal expansion were 5.34 x 10^{-6} in. / in. / °F and 5.28 x 10^{-6} in. / in. / °F for the 8,000 psi and 10,000 psi design strength mixtures, respectively. This may be an area where a synthesis of existing data will be sufficient to evaluate the current AASHTO provisions.

Freezing and Thawing Resistance. Freezing and thawing resistance and permeability are important factors in the durability of concrete. For freezing and thawing resistance, properly proportioned and placed lightweight concrete has been found to perform as well or better than normal weight concrete. The permeability of lightweight concrete has been found to be equal to or less than that of normal weight concrete. This is attributed to the elastic compatibility of the constituents and the enhanced bond between the coarse aggregate and the cement paste. Reduced permeability and increased resistance to the effects of freezing and thawing lead to a more durable concrete that is better able to resist corrosion of steel in concrete.

Permeability. Ozyildirim et al. (2005) investigated the permeability of bridge beam and deck mixtures using lightweight high strength concrete. Normal weight and lightweight test beams were fabricated for initial testing before the actual lightweight beams were fabricated for construction. The normal weight mixture exhibited lower permeability values than the lightweight mixture; however, the lightweight beam and deck mixtures for the actual construction both exhibited coulomb values below the specified maximum values for the project of 1500 Coulombs and 2500 Coulombs, respectively.

Kahn et al (2004) observed low permeability's for high strength lightweight mixtures. The specimens were cured using both accelerated curing and a standard ASTM curing procedure. In both cases, the permeability values were low. This may also be an area where existing data will suffice in the evaluation of lightweight concrete performance.

Absorption. For structures that are to be exposed to severe environments, it is imperative that the concrete be of high quality and durability. When using lightweight concrete in these types of structures, it is important that a high quality lightweight aggregate be used. These high quality aggregates have been shown to absorb very little water. This is attributed to the enhanced bond between the cement paste and the aggregate. This enhanced bond reduces absorption and allows for a dense, durable concrete.

Alkali-Aggregate Reaction. ACI 201.1R (ACI,1992) reports no documented instance of in-service distress caused by alkali reactions with lightweight aggregate.

Shear and Torsion

The current approach in AASHTO (2007) to account for the lower splitting tensile strength of lightweight concrete and its effect on shear and torsional strength has two distinct aspects. The first is a modification to typical shear and torsional strength calculation terms, where 4.7 f_{ct} replaces each $\sqrt{f'_c}$ term (f_{ct} is the specified splitting tensile strength in ksi, f'_c is the specified compressive strength in ksi). If f_{ct} is not known, then $\sqrt{f'_c}$ is to be replaced with 0.75 $\sqrt{f'_c}$ for all-lightweight concrete and $0.85\sqrt{f'_c}$ for sand-lightweight concrete.

In addition to the modification of the calculation of nominal strength, AASHTO also stipulates a different strength reduction (ϕ) factor for shear and torsion of lightweight concrete. For shear and torsion in normal weight concrete, the ϕ factor is 0.90, while for lightweight concrete the ϕ factor is 0.70. This means that both the concrete and steel contributions to shear strength (V_c and V_s) are reduced due to the use of lightweight concrete. Also, torsional strength, which is calculated based on the assumption that the concrete does not contribute to strength (except in carrying torsion in diagonal compression struts), is penalized with a lower ϕ factor as well.

The ACI code (ACI, 2005) has similar requirements for the treatment of strength calculations that include $\sqrt{f'_c}$, but they make no distinction between the ϕ factor for torsion and shear for lightweight and normal weight concrete. ACI also makes some exceptions to the $\sqrt{f'_c}$ modification. Calculations in which the $\sqrt{f'_c}$ term is not to be modified include maximum steel contribution to shear strength ($V_{s,max}$), maximum allowed combined shear and torsional stress, maximum nominal punching shear strength and other limits on terms. AASHTO does not stipulate these exceptions.

This section presents research programs performed to investigate shear strength of mildly reinforced and prestressed lightweight concrete beams.

Mildly Reinforced Concrete

Early Research at the University of Texas at Austin: The earliest research into the shear behavior of lightweight concrete came out of the University of Texas, including work by Maltepe (1957), Eldridge (1958) and Aguirre (1959). These studies focused on small sets of small beams that were generally 9.375 in. deep by 5.5 in. deep and had a clear span of 6 ft to 7 ft. The concrete mix design included expanded shale for both the fine and coarse aggregate, with a nominal maximum aggregate size of $^3/_8$ in. The design compressive strength in these studies ranged from 2.5 ksi to 7 ksi. None of the beams had shear reinforcement, but there was both compressive and tensile longitudinal reinforcement that was anchored in order to prevent bond slippage. The beams were simply supported and loaded with two symmetric point loads.

Some of the conclusions from these test programs were as follows.

- As *a/d* became larger, the difference between lightweight and normal weight concrete in regards to the diagonal tension strength grew larger. In particular, $v/\sqrt[3]{f_c'}$ was lower in lightweight concrete than normal weight concrete when *a/d* was greater than 2.5.

- As f_c' increased, the difference between the diagonal tension strength of the lightweight concrete and the normal weight concrete became larger. In some cases, the diagonal tensile strength was about 30% below that of normal weight concrete.
- An increase in f_c' did result in an increase in the diagonal tension strength measured at ultimate load.
- With the longitudinal reinforcement ratio, ρ, of 3%, the results showed values for shear decreasing dramatically when *a/d* went from 2.5 to about 4 or 4.5, at which point, the shear tended to level off as *a/d* increased. This behavior was somewhat similar to tests performed on normal weight concrete specimens.
- There were no conclusive results regarding the angle of inclination of the cracks; some studies showed the crack angle in lightweight concrete to be shallower than their normal weight counterparts, while other tests resulted in steeper angles.

Hanson: Research at Texas was followed up by Hanson (1958, 1961), who expanded the types of aggregate under investigation. Hanson's work included 57 lightweight concrete beams that used expanded shale, expanded clay, expanded slate, expanded blast furnace slag and carbonaceous shale from coal processing for coarse aggregate. All beams had the same dimensions measuring 6 in. wide by 12 in. tall (with an effective depth of 10.5 in.) and either 6.5 ft or 10 ft long. The concrete compressive strength ranged from 3.0 ksi to 8.6 ksi, while the concrete density varied from 90 pcf to 113 pcf. None of the beams had shear reinforcement, although there were two different percentages of longitudinal steel reinforcement used. Each beam was simply-supported and loaded at the third points.

For some of the beams, the initial diagonal tension cracking load was the ultimate load; the remaining beams were able to recover and formed a second diagonal crack occurring on the opposite end of the beam. Of these beams, the failure was generally crushing at the load point, and in some cases the beams also had splitting along the longitudinal reinforcement. The ultimate load at this type of failure ranged from less than ten percent to one hundred percent more than the initial diagonal tension cracking load. Because of the random nature of the failure due to the initial diagonal tension cracking, Hanson recommended that the diagonal tension cracking load be considered as the nominal strength for the purposes of design.

Compared to normal weight concrete, the diagonal tension resistance of lightweight concrete varied from 60% to 100% of the shear resistance for a given compressive strength, with the variation dependent on the type of aggregate. All of the lightweight cylinders that were tested failed with the shear plane passing through the aggregate, indicating that all lightweight concrete is affected by the tensile strength of the aggregate. The proportionality ratio between f_{sp}' and f_c' decreased as f_c' increased, while the ratios for "wet" lightweight concrete cylinders tended to be 2-3% higher in split-tensile strength compared to "dry" cylinders; the reverse was true for normal weight concrete. The f_{sp}' of dry lightweight cylinders varied from 0 to 40% less than the wet cylinders, possibly due to drying shrinkage.

Computed beam deflections for lightweight concrete were 15% to 35% greater than those for normal weight concrete, although the modulus of elasticity for lightweight concrete can be 50% less than that of normal weight concrete. Therefore, the deflections are not necessarily inversely proportional to the modulus of elasticity.

Hanson developed a method for determining the shear strength of lightweight concrete that is similar in form to that for normal weight concrete, but has different constants to reflect the lower diagonal tension strength. He also gave a minimum formula should there be no tensile strength data available for a particular lightweight concrete mix. Although this method tends to be highly conservative, the calculation accounts for any shrinkage and continuity effects on diagonal tension resistance.

Ivey and Buth: In response to the work by Hanson, Ivey and Buth (1967) set out to determine which of several design methods provided the most accuracy relative to experimental shear tests. They tested 26 beams that used expanded slate and two different expanded shale aggregates. Generally, the beams were 6 in. wide by 12 in. deep, although some were 9 in. and 18 in. deep. The experimental program varied the *a/d* ratio, the percentage of steel and the beam cross-section. While f_c' was between 3 ksi and 4 ksi, f_{sp}' was between 300 psi and 450 psi, and concrete densities ranged from 94 pcf to 126 pcf.

The researchers noted that an *a/d* of 3.5 seemed to be the dividing line between shear compression failure and failure due to diagonal tension cracking. They also found that there was little if any effect of the beam depth on the shear resistance of the beams. The measured strengths of the beams in this study averaged 14% below the strengths predicted by Hanson

(1961). In comparison to ACI's Eq. 17-9 (ACI, 1963), the test results were 21% higher, when using the value for F_{sp} suggested by the manufacturer, where F_{sp} is the splitting ratio, defined as the ratio of f_{sp}' to $\sqrt{f_c'}$.

Ivey and Buth also examined another approach by applying a 0.85 (for sand-lightweight concrete) or 0.75 (for all-lightweight concrete) correction factor to the original normal weight concrete equation for shear. In this case, the average test results were 36% higher than strengths predicted using the correction-modified equation, which the research team deemed highly conservative. However, the previous studies performed at the University of Texas and by Hanson seemed to provide data from low-tensile strength aggregate that fell below even this conservative design method.

A third approach suggested by the researchers was to use f_{sp}' determined from a priori testing in substitution for $F_{sp} \cdot \sqrt{f_c'}$. In this case, the test results were only 18% higher than the predicted values. The researchers concluded that the f_{sp}' method would be acceptable for high-tensile strength lightweight aggregates, albeit at a reduced factor of safety.

Evans and Dongre: Outside of the United States, early work included experiments in Great Britain, India, Japan and Australia. Evans and Dongre (1963) investigated using a sintered clay material, called Aglite, which was crushed after it had been cooled into large slabs. The objective of this study was to determine the tensile and compressive strength of Aglite, its behavior at ultimate loads in shear and flexure, and its absorption and permeability characteristics. The beams measured 6.25 in. wide by 12 in. deep and 9 ft long. Concrete densities ranged from 90 pcf to 115 pcf, while the concrete compressive strength was between 3.1 ksi and 4.0 ksi. The reinforcement ratio for the tensile and compressive longitudinal steel were 5.7% and 1.9%, respectively, using ¾ in. square twisted bar. In this study, beams with and without shear stirrups were tested; beams having stirrups had a reinforcement ratio of 0.262%. The beams were tested on an eight-foot span with two concentrated loads spaced twelve inches apart, with an *a/d* ratio of about 4.0.

The researchers concluded that Aglite had a tensile strength that was about 75% of that of normal weight concrete. Furthermore, they recommended that the allowable shear stress in Aglite concrete be 60 psi provided that the concrete cube strength was at least 3 ksi. When shear reinforcement was present, Evans and Dongre suggested limiting the allowable stress in the concrete to 300 psi in order to limit the crack width in the concrete to 0.01 in. Results also showed that there was not much of an effect on the ultimate shear strength due to shrinkage cracking or shrinkage stresses in the lightweight concrete, probably because of the presence of the shear stirrups that inhibited failure due to shrinkage.

Jindal: In 1966, Jindal performed flexural and shear tests on lightweight concrete beams constructed with sintered fly ash and without shear reinforcement. The beams tested for shear were 9 in. wide with an effective depth of 13 in., and were 10.5 ft long. The lightweight coarse aggregate had a nominal size of ¾ in. to $^3/_8$ in., while the fine aggregate was either sand or sintered fly ash. The compressive strength for all of the beams was 4.3 to 4.7 ksi and the concrete density varied from 93 pcf to 106 pcf. During testing, the beams were simply supported over a span of 9 ft and loaded under third-point loading. Both of the lightweight concrete mixes followed the same cracking pattern as their normal weight control specimens. The shear strengths of the all-lightweight and sand-lightweight beams were 75.5 to 82.4% and 82.9 to 90.5%, respectively, of the strength of the normal weight beams.

Nishibayashi: Researchers in Japan took a strong interest in lightweight concrete. Nishibayashi et al. (1968) were one of the first groups in that country to study the shear behavior of concrete. Sixteen beams were tested for shear strength; the dimensions of these beams were 3.9 in. wide by 7.9 in. tall and 4.9 ft long. Pelletized expanded shale served as both the coarse and fine lightweight aggregate, with a nominal maximum aggregate size of 0.8 in. for the coarse aggregate. The two concrete densities used were 97.8 pcf and 98.6 pcf while the compressive strength was between 3.1 ksi and 6.3 ksi. The beams had no web reinforcement, but they had longitudinal reinforcement consisting of either two 0.5-in. or 0.6-in. round or deformed bars in the tension side and two 0.4-in round bars in the compression side of the beams.

The test results showed that the shear strength of the lightweight concrete was about 30% less than that of normal weight concrete. Plots of $V_c / bd\sqrt{\sigma_c}$ versus $pVd / M \sqrt{\sigma_c}$, showed that the results matched relatively closely with the formula proposed by Hanson (1961).

***Swamy and Bandyopadhyay*:** In 1979, Swamy and Bandyopadhyay continued with past experiments from Great Britain regarding sintered pulverized fuel ash and expanded slate. This study examined twenty-four lightweight T-beams and compared them with companion normal weight T-beams. All beams had the same cross-sectional dimensions of 11.8 in. for the flange width, a flange thickness of about 2 in., a web width measuring 3.9 in. and an effective depth of 5.0 in. The overall length of the members was 7.2 ft, while the span length was 6.2 ft. For the mix design, both lightweight concretes used lightweight coarse and fine aggregate, with a nominal maximum aggregate size of 0.4 in. for the fuel ash and 0.75 in. for the expanded slate. The concrete compressive strength was around 5.5 ksi. Tensile reinforcement used in the bottom of the members was either 0.63 in. or 0.79 in. diameter bars that had a tensile strength of 60 ksi.

The ratio of longitudinal reinforcement was the second variable in these tests, and this ratio ranged from 1.64% to 2.70%, which corresponded to 55%, 71% and 91%, respectively, of ρ_b. All of the steel had adequate anchorage in order to prevent bond slippage. None of the beams had any shear reinforcement. The beams were simply supported and loaded with two concentrated point loads, with the ratio of the shear span to the effective depth varying from 1.5 to 6.0.

All of the beams failed in shear. Flexural cracks in the beams formed independently of the type of concrete. Likewise, the research team confirmed that the mechanism for shear transfer through the compression zone was similar, regardless of the type of aggregate used. While the formation and propagation of diagonal tension cracks was the same in lightweight concrete as for normal weight concrete, the lightweight members generally had wider crack widths than their normal weight counterparts. Cracks in the lightweight concrete were characterized as passing through the aggregate, as opposed to around the aggregate like normal weight concrete. Therefore, there was a reduction in the interface shear transfer along the crack.

In this study, some tests of the expanded slate aggregate had comparable levels of shear cracking load as normal weight concrete, while other specimens actually had cracking strengths that were 10% to 17% greater than the normal weight specimens. On the other hand, the fuel ash aggregate had results that were 5% to 12% less than the normal weight concrete results. Regarding ultimate shear strength, the ranges were 75% to 95% of normal weight concrete for the expanded slate and 83% to 95% for the fuel ash.

This study also revealed the significant effect of the amount of longitudinal reinforcing; for lightweight concrete, the dowel action of the longitudinal steel could increase the shear resistance between 30% and 70%. However, this dowel effect appears to be insignificant in lightweight beams with relatively short shear spans, as opposed to there being a dowel influence in normal weight concrete regardless of the span length. The authors postulated that the reason for the difference between the two concretes was that shorter spans seem to undergo both beam action and arch action, resulting in a combined state of stress. Work by Hanson (1961) showed that lightweight concrete performs poorly compared to normal weight concrete under biaxial stresses.

***Hamadi and Regan*:** Hamadi and Regan (1980) also studied T-beams, this time considering expanded clay as the lightweight aggregate and employing shear reinforcement. This study covered five lightweight reinforced concrete T-beams that were 15.7 in. tall and 4.7 in. wide in the web; the top flange dimensions were 3.9 in. by 19.7 in. The cube strength of the concrete in the lightweight beams was around 3.5 ksi, while the primary variable under investigation was the shear reinforcing index, which ranged from 92 psi to 493 psi. All of the beams were designed to fail in shear. The beams were loaded with a single concentrated load at the beam centerline, giving the ratio of the shear span to the effective depth for all of the beams of 3.6.

The three lightly shear-reinforced lightweight beams failed with an inclined shear crack going through the flange, while the more heavily-reinforced beams exhibited bond-splitting at failure. Given the same amount of shear reinforcement, the lightweight beams had higher stresses in the stirrups compared to the normal weight beams. Furthermore, stirrup yielding occurred at loads that were well below the ultimate failure loads. Once the stirrups had yielded, there was a progressive decrease in the crack angle, along with a realignment of the truss system to resist the additional load. However, this realignment appeared to be limited to the aggregate interlock at the crack interface.

The reduced aggregate interlock in lightweight concrete resulted in the web compression strut being closer to the angle of inclination of the original shear crack. Thus, the angle of inclination was greater in lightweight beams than in normal weight beams. Within the beams, displacements at the cracks were greater parallel to the cracks in the lightweight beams compared to the normal weight counterparts. Overall, both the V_c and V_s components of V_u were lower in the lightweight concrete beams compared to the normal weight beams. Furthermore, the lightweight beams did not show as much of an increase in shear strength as the amount of shear reinforcement increased.

Kirmair: In 1981, Kirmair published a study of five beams that were designed to fail at a range of low, medium, and high shear stresses. However, all of the beams failed in flexure. Nevertheless, Kirmair made comparisons of the crack widths and crack patterns between the lightweight and normal weight concrete. The cross-sectional shapes were rectangular, T, and Bulb-T, either 19 or 26 in. tall and 14 or 17 ft long. The unit weight for the lightweight concrete was only 94 lb/ft^3. Due to lower tensile strength of the lightweight concrete, shear cracking started at a lower load and the cracks were wider and were spaced closer together compared to normal weight concrete. This was particularly the case for the beams designed for lower shear stresses, where the stirrups were spaced further apart. The author recommended either limiting the stirrup spacing or increasing the reinforcement.

Swam and Lambert: Similar to previous work by Swamy and Bandyopadhay (1979), Swamy and Lambert (1983) studied T-beams constructed with sintered fuel ash and without shear reinforcement. This time, the basic geometry was a 4-in. web, with 12in. wide by 3 in. thick flanges, an effective depth of 7.1 in. and a length of 9.84 ft. The concrete strength ranged from 3.4 ksi to 7.0 ksi, using a nominal maximum aggregate size of 0.55 in. The percentage of longitudinal reinforcement ranged from 0.29% to 3.01%, based on the flange width. The beams were loaded with two concentrated point loads with a variable *a/d* ratio that ranged from 1.5 to 6.0.

For $a/d \leq 3.0$, the T-beams failure by diagonal tension cracking occurred in T-beams where *a/d* was less than 3.0. There were failures exhibiting aspects of both an arching mechanism failure and a flexural-type failure for *a/d* between 3.0 and 4.5. The major factors found to influence shear strength were *a/d*, percentage of longitudinal steel and concrete compressive strength. Certainly, *a/d* had a strong effect and longitudinal steel had a lesser effect. On the other hand, f_c' had a much smaller effect, where an 80% increase in compressive strength only yielded a maximum 21% increase in ultimate shear strength, where there was a 10% variation in all of the results. The lightweight coarse aggregate combined with sand had about 75% to 85% of the strength of normal weight concrete.

To account for *a/d*, longitudinal reinforcement ratio and f_c', the research team proposed a design equation to be used for the ultimate shear stress in all lightweight concrete beams as:

$$v_u = 0.292 f_{sp} + 31.83 \frac{\rho V d}{M}$$

where *M* is the bending moment at the ultimate limit state, *V* is the shear force at the ultimate limit state and this equation does not include any safety factors. However, this equation does not work well for beams with $a/d \leq 2.0$.

While the research team saw little effect in raising the compressive stress limit in the design codes, they thought that raising the reinforcing limit to 5% would provide adequately conservative predictions of shear strength.

Walraven and Al-Zubi: In 1995, Walraven and Al-Zubi considered lightweight concrete beams with stirrups in conjunction with the variable strut inclination method of analysis. The test program involved testing three different types of lightweight aggregates in I-beams. The beams had an effective depth between 29 in. and 30 in. and a cross-section area of 279 in^2, with a web thickness of 3.9 in. There were three series of lightweight beams cast using three types of lightweight aggregate: Lytag, Liapor and Aardelite. The actual compressive cube strengths and unit weights ranged from 2.9 ksi and 7.0 ksi and 108 pcf and 131 pcf, respectively. A fourth series was similar to the Lytag series, but had a design compressive strength of 8.7 ksi, while a fifth series was constructed with conventional aggregate.

All of the beams used natural sand as fine aggregate. Longitudinal reinforcement steel was present in both the top and bottom flanges of the I-beams; the amount and spacing of longitudinal and vertical steel varied depending on the type of desired shear failure and design compressive strength. Within each series of beams, three levels of shear reinforcement ratio varied between 0.43% and 0.89% and 1.45%. The medium ratio was chosen to have the stirrups yield just at the point of web crushing; the other two levels of reinforcement were designed to yield and not yield, respectively, prior to concrete crushing in the web.

The authors' conclusion from the results was that there was virtually no difference between the lightweight beams and their normal weight companions regarding strut rotation behavior. One possible explanation is that despite the reduced aggregate interlock from the lightweight aggregate, there is a sufficient amount of shear force transferred due to the irregular shape of the crack faces. This reasoning was evidenced by the fact that the more vertically-inclined cracked regions in the lightweight beams exhibited concrete crushing. Analytically, the two different types of concrete had the same redistribution capacity and the same degree of strut rotation. The researchers did note, however, that the lightweight concrete beams had about twice the shear displacement along the crack interface as that of the normal weight beams.

Hoff: Hoff et al. tested six beams constructed with high-strength lightweight concrete designed to fail in shear (1984). The beams were 6 in. by 8 in. by 9.35 ft. Concrete compressive strength was about 10 ksi, the average splitting tensile strength was 540 psi, and the average density was 119.6 pcf. These beams had no shear reinforcement.

Unfortunately, there was significant scatter in the results for ultimate shear strength. Nevertheless, the shear strength of lightweight concrete was definitely lower than normal weight concrete. However, the ratio of measured to calculated shear strength was similar for lightweight concrete as it was for normal weight concrete, where the calculated shear strength was according to the Norwegian Code.

Murayama and Iwabuchi: Researchers in Japan started their own program in high-strength lightweight concrete in the early 1980's. Murayama and Iwabuchi (1986) published a paper examining both the flexural and shear strength of reinforced high-strength lightweight concrete. Eleven beams had cross-sectional dimensions of 7.9 in. wide by 9 in. tall and were either 8.2 ft or 9.8 ft long, with span lengths being 6.6 ft and 8.2 ft, respectively. The three lightweight concrete mixtures used sand as the fine aggregate; all mixes had a density of about 119 pcf. A second parameter, longitudinal reinforcement ratio, ranged from 1.55% to 3.18%. No shear reinforcement was present in the beams. The beams were simply-supported and loaded with two concentrated point loads, where the shear span-to-effective depth varied from 2.5 to 4.0.

The splitting tensile strength of lightweight concrete was about 15% less than that of normal weight concrete, while the shear transfer results showed that lightweight concrete had 30% less resistance than normal weight concrete. This result was possibly due to the smoother shear planes along the crack. Therefore, the authors suggested that the shear transfer strength of high-strength lightweight concrete be limited to 70% of that of normal weight concrete. Although the researchers found that, for *a/d* equal to 4.0, all of the beams had shear failures after diagonal cracks formed, but that lightweight beams had shear cracking strengths that were 30% less than the strengths of the normal weight beams.

The Joint ASCE-ACI Task Committee (1973) summarized that the reason for this reduction may be a combination of the difference in the mode of bond failure, shear transfer strengths, strengths under biaxial stresses and of course, the tensile splitting strength. Murayama and Iwabuchi's research confirmed the role of lower tensile splitting strength and lower shear transfer strengths in the reduced cracking strength. For *a/d* equal to 3.0, the normal weight concrete beams failed immediately upon the formation of a diagonal tension crack, with the strengths agreeing with the predicted results.

On the other hand, the lightweight concrete beams actually exhibited *higher* reserve capacities after the first diagonal tension crack appeared, and had higher experimental strengths than that predicted by Niwa and Maeda (1983) for arch action in deep beams:

$$V_u = \frac{0.53\, b_w d\, {f_c'}^{\frac{2}{3}} \left(1+\sqrt{\rho_w}\right)\left(1+3.33\frac{r}{d}\right)}{\left[1+\left(\frac{a}{d}\right)^2\right]}$$

where *r* is the width of the load applied to the beam. There was a difference in the way the two different types of aggregate developed their cracking patterns. Diagonal cracks in the normal weight beams started in the upper half of the beam member and progressed to the top of the beam on the outside of the two loading points. On the other hand, cracks in the lightweight beams reached the top of the section in between the two load points. For beams with *a/d* = 2.5, both the normal weight and lightweight aggregate beams failed in shear compression.

Salandra and Ahmad: In the U. S., researchers started examining high-strength, reinforced lightweight concrete beams with the publication by Salandra and Ahmad in 1989. Sixteen beams were tested; eight had stirrups, while eight did not. All beams were 4 in. wide and 8 in. deep, with an effective depth of 6.75 in. The compression strength ranged in between 8 ksi and 10.5 ksi, and the average unit weight was 122.5 pcf. The lightweight aggregate was expanded slate, with a nominal maximum aggregate size of ½ in. All beams had the same longitudinal and web reinforcement ratio of 1.45 and 0.78%, respectively. The shear-to-effective depth ratio varied from 0.52, 1.56, 2.59, to 3.63.

ACI Eq. 11-6 (1983) proved unconservative for beams with *a/d* of 2.59 and 3.63 and f_c' of 10 ksi. ACI Eq. 11-3 proved unconservative for beams with *a/d* of 3.63 and f_c' of 10 ksi. Results also suggested that the maximum limit of 100 psi for f_c' used in calculating the shear contribution from V_c may be too unconservative. The code also overestimated the diagonal tension parameter

$$\frac{V_c}{b_w d\sqrt{f_c'}}$$

when *a/d* is greater than or equal to 2.59. The beams that had web reinforcement and *a/d* greater than 0.52 failed in flexure. The stirrups were widely spaced and hence did not become effective after cracking, because the failure crack did not cross any stirrups. Failure occurred without redistribution of the concrete stresses to the web reinforcement.

Results for beams with reinforcement were similar to beams without reinforcement. Beams with *a/d* of 2.59 and 3.63 exhibited ductile behavior. Both beams with and without reinforcement had a decrease in the angle of critical cracking as *a/d* increases, although the angle is greater for beams with web reinforcement than for beams without reinforcement, except for beams with *a/d* of 0.52, which was the same. Beams with reinforcement had more inclined cracks than beams without web reinforcement. Beams with a smaller *a/d* had more reserved capacity after cracking compared to beams with larger *a/d*.

Ahmad et al.: Ahmad et al. followed up the previous reference with a comparison between normal and high-strength concrete in 1994. Fifteen beams, some with and some without web reinforcement, were tested to study the effects of f_c', *a/d* and the web reinforcement ratio. All beams were 10 in. deep by 5 in. wide in cross-section, although there were two different effective depths used. The normal compressive strength ranged from 4.4 to 6.5 ksi, while the high compressive strength ranged from 11.9 to 13 ksi. The lightweight aggregate was expanded slate with ½ in. maximum size. Longitudinal reinforcement consisted of Grade 60 steel, whereas the shear stirrups were fabricated from Grade 40 steel.

One set of normal-strength beams had a shear reinforcement ratio of 0.49%; one set of high-strength beams had a shear reinforcement ratio of 0.51%, while two other high-strength beams had ratios of 0.65% and 0.78%. The *a/d* ratio was typically 1, 2, or 3, although one high-strength beam had an *a/d* of 4.

In addition to shear capacity results for expanded slate, the authors present results for other lightweight aggregates published by Clark (1987). Results showed that shear capacity decreased with increasing *a/d*. Additionally, the ACI code (1983) did not adequately predict the shear capacity or give sufficient margin of safety for beams with *a/d* equal to 3, regardless of whether or not there were stirrups. The study also made comparisons to the British building code, and concluded that this code could allow lightweight concrete with f_c' greater than 5.8 ksi as well as allow for a 0.85 factor instead of the prescribed 0.80 factor and still have an adequate margin of safety. However, one must note that the British building code apparently applies the 0.80 factor to the contribution of both the concrete and the steel, whereas ACI only applies the 0.85 factor to concrete contribution to the shear strength.

Other results show that as *a/d* increases, the effectiveness of the shear reinforcement in improving the shear capacity increases and the slope of the post-peak deformation decreases. However, the post-peak slope of the load-deflection curves for the high-strength concrete is steeper than for normal-strength concrete. The researchers were able to achieve elasto-plastic behavior in normal-strength beams when the shear reinforcement was five times the minimum amount required by ACI.

Ramirez, Olek and Malone: In 2004, Ramirez, Olek and Malone presented the results that served as the basis of Malone's dissertation (1999) on high-strength lightweight concrete. The program at Purdue University tested a total of twelve beams, five of which were lightweight. The beams were 11.6 ft long and 14 in. by 14 in. cross-section in the test region while being 17 in. by 14 in. in cross-section outside of the test region. One of those lightweight beams had a high strength of 10.5 ksi, while the rest were regular strength. The lightweight concrete beams used expanded shale with a nominal maximum aggregate size of 0.37 in., which was half the size of the normal weight aggregate. The authors noted that the smaller aggregate resulted in larger shear stresses, which reduced the stress in the transverse reinforcement and contributed to the shear carrying mechanism (Bhide and Collins, 1989). The unit weight of the beams were either

106 pcf or 127 pcf. Both tension and compression reinforcement ratios were kept constant at 2.35%, although one variable was the distribution of the longitudinal reinforcement across the width of the beam. The shear reinforcing index varied between 157 psi and 236 psi. All of the reinforcing steel was uncoated deformed bars. Beams were simply supported with one end of the beam overhanging the support. Load was applied with a spreader beam straddling the support with the overhang; this condition provided a constant shear force and reverse bending across the test region, which was a more critical shear condition than that created with a simply-supported beam loaded at midspan.

All specimens had shear-compression failures and yielding in the web reinforcement. While both the AASHTO (1998) and ACI 318 (2002) specifications limited the magnitude of $\sqrt{f_c'}$ in calculating shear capacity, all specimens were able to exceed this maximum limit since the provided amount of transverse reinforcement exceeded the minimum amount that was required. Although the AASHTO-calculated values were 10% less than ACI-calculated values for shear strength, both codes yielded more conservative calculations for normal weight concrete versus lightweight concrete. The experimental shear capacity of the lightweight concrete was 82% of the normal weight concrete when normalized with $\sqrt{f_c'}$. This was less than the 0.85 factor used in the codes, but was within the variation of previous studies. As compressive strength increased, the difference in shear capacity between lightweight concrete and normal weight concrete decreased. In fact, the high-strength (10 ksi) beams had virtually the same shear strength, regardless of aggregate type. Nevertheless, both codes proved quite conservative compared to the measured shear strengths of high-strength concrete, again, regardless of aggregate type. In his dissertation, Malone stated that the high-strength, normal weight beams had 38% to 44% more shear capacity than predicted, while the lightweight beams had 48% to 59% more shear strength. On the other hand, the ratio of experimental to predicted shear strength using the AASHTO LRFD General Method of the moderate-strength (6 ksi) beams was 1.29 and 1.10 for the normal weight and lightweight beams, respectively, while, the ACI /AASHTO Simplified Method provided ratios of 1.36 and 1.18 for normal weight and lightweight beams, respectively.

Results showed that distributing the longitudinal steel across the width of the beam versus grouping the steel at the corners of the ties resulted in a decrease in shear strength for normal weight concrete but an 8% decrease for lightweight concrete. When looking at all twelve of the beams collectively, Malone determined that half of the beams had steel contributions to shear strength that were less than what was calculated using AASHTO, averaging 15% less than the calculated value. Nevertheless, the total experimental shear capacity was greater than the total calculated shear strength, regardless of which specification was used in the calculations. The reason was that the experimental concrete contribution was greater than the predicted results. For the 6 ksi beams, the experimental concrete contribution was 70% and 35% greater than the calculated contribution using normal weight and lightweight concrete, respectively. Likewise, the 10 ksi concrete had experimental results that were 78% and 68% greater for normal weight and lightweight concrete, respectively. Using a three-legged transverse tie versus a double-legged tie, while and keeping the reinforcement ratio the same, netted an 11 and 13% increase in shear capacity for normal weight concrete and lightweight concrete, respectively. The researchers noted that previous studies found that the shear strength of beams with different types of lightweight aggregate could vary by 100% (Ivey and Buth, 1967).

Super Lightweight Concrete

Kawaguchi et al.: Japanese researchers delved into the realm of super lightweight concrete (SLC) with Kawaguchi et al. publishing their work in 2000. The Kawaguchi team experimented with beams that had cross-sections that were 7.9 in. wide by 13.8 in. tall, with an effective depth of 11.8 in. and a span length of 7.9 ft. Densities of the lightweight concrete ranged from 74 pcf to 107 pcf due to the use of a combination of sand, conventional lightweight aggregate, and SLA for the fine aggregate. The longitudinal reinforcement ratio was a constant 1.77%. Apart from the beams that were reinforced with short fibers, there were nine beams that had either no shear reinforcement or had a steel reinforcement ratio of 0.15%, which was the minimum ratio under the JSCE code. All of the beams were simply-supported and loaded with two concentrated loads that were about six inches away from the beam centerline. The shear span-to-effective depth ratio was 3.5.

The beams with steel shear reinforcement failed in a relatively ductile manner, and exceeded the predicted strength for beams with minimum shear reinforcement under the JSCE code. On the other hand, the two types of beams without steel shear reinforcement had diagonal tension failures. Of the two beam types without reinforcement, the beams with fibers had fewer shear cracks and greater shear capacity than the beam type without fibers. The results showed that as the compressive strength increased, the ratio of the experimental to predicted shear strength decreased; the same is true for the ratio of f_t / f_c'. Furthermore, the researchers found that shear capacity of lightweight concrete is dependent on the unit weight of the concrete, and therefore, the concrete contribution to shear, V_c, should not be reduced to a constant factor of 70% of normal weight concrete beams.

Kobayashi et al.: Kobayashi et al. also published an article in 2000 that reported the development of ultra lightweight concrete which had concrete densities of 75 pcf and 105 pcf. The former concrete had a compressive strength of 4.4 ksi, while the latter had a compressive strength of 8.7 ksi. Nine lightweight concrete beams were tested along with four normal weight concrete beams, although three of the lightweight beams and one of the normal weight beams failed in flexure. All of the beams had a dog-bone shape, with the tested region being 7.9 in. wide by 10.6 in. tall, while the dimensions outside of the tested region were 7.9 in. wide by 15.7 in. tall. The total length of each beam was 10.2 ft. The beams were reinforced longitudinal in both the compression and tensile faces of the beams, while the percentage of shear reinforcing varied between 0.13% and 1.20%. The shear span-to-effective depth ratio was 2.0, and since the loading was cyclic causing double bending moment, there was longitudinal reinforcement in both the top and bottom of the beam.

Shear capacities of the 4.4 ksi beams were 20% to 30% below what was predicted, while the 8.7 ksi beams were 10% below. In comparison, the normal weight beams exceeded their predicted strengths by 0% to 15%. Furthermore, the shear force carried via arching mechanism in the lightweight concrete beams was less than the normal weight concrete beams. Relative to the normal weight concrete, the lightweight concrete had a lower elastic modulus, lower splitting tensile strength, and larger angle of inclination in the cracks. Consequently, the researchers modified the formula for estimating the arching mechanism by multiplying the formula by the ratio of the elastic moduli of the two different concretes, reducing the compressive strength by the ratio of the splitting tensile strength of the two different concretes, and using the crack angle observed during experimentation. Making these modifications to the equation for traditional concrete provided an adequate margin of safety.

Funahashi et al.: In 2001, Funahashi et al. tested four smaller beams measuring 7.9 in. wide, 15.7 in. tall and 11.2 ft long beams were tested with a range of concrete densities from 81.8 pcf to 107.4 pcf. Additionally, two larger-scale beams measuring 19.7 in. wide, 42.1 in. tall and 32.8 ft long had densities of 97.4 pcf and 113.0 pcf. The concrete mixes employed a combination of SLA, conventional lightweight aggregate and sand. The compressive strength of the smaller beams ranged from 3.3 ksi to 6.5 ksi, while the compressive strength of the larger beams was about 5 ksi. None of the beams had shear reinforcement. The beams were simply-supported, with a span length of 9.8 ft and 31.2 ft for the small-scale and large-scale beams, respectively. The loading consisted of two concentrated loads that were 4.1 ft away from the beam centerline, giving a shear span-to-effective depth ratio was 2.94.

Three of the smaller beams survived diagonal tension cracking on both sides of the beam before failing completely in shear compression. The normal weight beam failed at twice the load of the lightweight beams, but failed completely immediately at the formation of the first diagonal tension crack, as did the large-scale lightweight specimen with a density of 113 pcf. Although the 97.4 pcf large-scale beam also had a diagonal tension crack, this specimen formed a tied-arch mechanism before completely failing by concrete crushing. The authors stated that in general, normal weight concrete transitions from diagonal tension failure to shear compression failure when a/d is around 2.0. However, in the case of lightweight concrete, that value appeared to be about 3.5. For the SLA beams, the shear forces were 44% to 59% of the normal weight beams when diagonal tension cracking occurred. However, the ultimate shear capacity of the beams that had concretes densities of about 113.0 pcf were the same as what was predicted. On the other hand, the results for lower-density concretes were lower than what was predicted. Therefore, the methods used to calculate the shear capacity of normal weight concrete may only work for the higher range of lightweight concrete densities.

Furthermore, the shear capacity decreased as the concrete density decreased. The authors did find that the bond strength between the rebar and concrete decreased as the concrete density decreased. The Japanese Society of Civil Engineers code specifies that the shear strength of lightweight concrete should be calculated as 70% of that of normal weight concrete. In this research, the authors proposed a conversion factor in calculating the shear strength of lightweight concrete:

$$\eta_s = (\rho/2300)^{3/2}$$

where ρ is the density of the lightweight concrete and 2300 is the density of normal weight concrete (in kg/m^3). However, this factor would only work for values of *a/d* that are greater than or equal to 3.5. When *a/d* is equal to 3.0, i.e., the transition region between diagonal tension failure and the tied-arch mechanism, the conversion factor would be $\eta_s \cdot [3.5/(a/d)]^2$.

Prestressed Concrete

Brettle: While most of the research in lightweight concrete has focused on mild steel reinforcing, there has been some attention paid to prestressed concrete. The first study was done in Australia. Brettle (1962) published a paper that gave an overview of a study using crushed shale for both fine and coarse aggregate. The density of this concrete was about 105 pcf and the compressive strength ranged from 4.8 ksi to 6.4 ksi. While the beam width was held constant at 2 in., the depth varied from 6 in. to 18 in. and the length was in between 1.75 ft and 9 ft. There was a single 0.276 in.-diameter prestressing wire located at the bottom third point of the beam cross-section; the tensile strength of this prestressing wire was 220 ksi. Given the different cross-sectional dimensions, the percentage of longitudinal reinforcement steel varied from 0.010 to 0.015. The beams were simply-supported and loaded symmetrically with two concentrated loads. In order to cause both flexural and shear failures, the shear span varied relative to the type of failure desired, with the shear span-to-overall depth ratio ranging from 1.0 to 8.5.

Overall, sixteen beams failed in shear, and there were no bond failures. Despite lacking quantifiable data at the time, the author concluded the quality of failure in prestressed lightweight concrete beams is similar to that of normal weight concrete.

Malone: In addition to his dissertation on reinforced concrete, Malone (1999) was the first to provide substantial information regarding prestressing and lightweight concrete. The test program included four AASHTO Type I girders that had 4-in. thick by 4-ft wide cast-in-place decks, giving a composite area of concrete of 469 in^2. Total length of each girder was 22 ft, while the span length was 16 ft. The mix design included expanded shale for the coarse aggregate, with a $^3/_8$ in. nominal maximum aggregate size, and natural sand fine aggregate. The concrete in two of the beams weighed 106 pcf and had compressive strength measuring about 6.75 ksi. The concrete in the other two beams weighed 127 pcf and had a compressive strength of 10.1 ksi. Each beam had ten ½ in.-special, Grade 270 steel strands providing the main longitudinal reinforcement, with two of those strands located in the top flange. Also, the beams had either No. 7 or No. 8 Grade 60 uncoated rebar in the middle portion of the beam for extra reinforcement against flexural failure. For each of the two compressive strengths, one beam had no shear reinforcement, while the other had double-legged, Grade 60, No. 3 uncoated rebar stirrups spaced at 20 in. This 20-in. spacing represented the minimum shear reinforcement and maximum spacing allowed by AASHTO (1994). Deck reinforcement consisted of Grade 60, No. 5 rebar that ran longitudinally and transversely at a 5-in. spacing. The beams were simply supported and loaded with a single concentrated load at midspan.

All of the specimens suffered web-shear failures with smooth crack surfaces. At failure, there was also concrete crushing at the load point and a splitting crack along the tensile reinforcement. When comparing the 6.75 ksi and 10.1 ksi beams without stirrups, a 44% increase in compressive strength resulted in a 28% increase in shear capacity. However, a similar comparison of the beams with shear reinforcement yielded only a 3% increase in shear strength for a 56% increase in compressive strength. Thus, the author concluded that the minimum amount of shear reinforcement required by both the AASHTO General Method (AASHTO, 1994) and the ACI/AASHTO Simplified Method (ACI-ASCE, 1973) should be increased in order to provide sufficient post-cracking reserve strength.

Comparisons between the measured shear strength and those calculated using AASHTO or the ACI 318-95 specifications are in Table 1 below. These comparisons use the design yield strength of steel specified in AASHTO, with the measured strength ranging from 8% to 44% greater than the predicted results.

Table 1. Comparative Results between Experimental and Predicted Shear Tests in Malone's Study

Design Strength (ksi)	Stirrups?	$\frac{V_{exp}}{V_{AASHTO\ LRFD}}$	$\frac{V_{exp}}{V_{ACI\ 318\text{-}95}}$
6	No	1.18	1.13
6	Yes	1.19	1.32
10	No	1.44	1.32
10	Yes	1.08	1.25

Table 2. lists the angle of the inclined crack that ultimately led to failure in the beam along with the predicted angle using the AASHTO General Method. There is no apparent pattern as to whether the predicted angle was less than or greater than the predicted result.

Table 2. Comparative Results between Experimental and Predicted Crack Angles in Malone's Study

Design Strength (ksi)	Stirrups?	Failure Crack Angle (deg)	Predicted Crack Angle (deg)
6	No	36	30
6	Yes	34	26
10	No	23	32
10	Yes	33	27

In order to calculate the steel contribution to the shear strength, the author employed a combination of strain gages on the stirrups and Whittemore measurements on the concrete surface to determine how many shear stirrups crossing a crack had yielded. Based on these calculations, the experimental steel contribution to shear strength was about 30% less than the predicted result using the AASHTO method, but was 37% greater than the result predicted using the guidelines in ACI 318-95. The difference between the experimental and AASHTO results is presumably due to the lower crack angle predicted by AASHTO.

Kahn et al.: As part of a larger study on the performance of precast, prestress bridge girders, the researchers at Georgia Tech constructed six beams to examine the transfer and development length, the shear strength, the effect of shear reinforcement spacing on strand slippage, development length and shear capacity of prestressed high-performance, lightweight concrete (HPLC) girders (Kahn et al., 2004). There were two series of girders, with three girders in each series. The first series had a design compressive strength of 8 ksi, while the second series was designed for 10 ksi. All of the girders were AASHTO Type II beams with a cast-in-place deck measuring 7 in. thick and 19 in. wide, giving a total composite area of 502 in^2. The concrete was made with ½-in. expanded slate coarse aggregate and natural sand fine aggregate, resulting in a unit weight between 118 pcf and 119 pcf. The 8 ksi girders actually had a compressive strength ranging from 8.7 ksi to 10.6 ksi, while the actual compressive strength of the 10 ksi girders was between 9.9 ksi and 11.4 ksi. For the deck, the compressive strength ranged from 4.8 ksi to 6.1 ksi. Longitudinal reinforcement consisted of ten 0.6-in, Grade 270 low-relaxation strands, with eight strands in the bottom flange and two strands in the top. For shear reinforcement, there were two No. 4, Grade 60 stirrups spaced at either 3.5 in., 7 in., or 24 in. During testing, the beams were simply supported and loaded using a single point load. However, each beam was tested three times, with the location of the supports changing each time to avoid damaged sections. Although there were 18 tests, only seven of those test resulted in shear failures. Of these seven tests, the ratio of shear span-to-effect depth ranged from 1.5 to 2.4.

With one exception, the force to cause initial experimental cracking was greater than the force predicted using either the ACI specifications (1999), AASHTO Standard Specification (1996), or AASHTO (1998). The ACI calculations were the least conservative, with the shear strength being nearly 9% less than the predicted value for one beam and from 2% to 17% greater than predicted for the remaining beams. The AASHTO Standard Specification calculations provided more conservative results, with the experimental cracking load being 9% to 39% greater than the calculated values. On the other hand, AASHTO LRFD methodology grossly underestimated shear cracking strength, with the experimental results being 377% to 488% greater than the estimated strengths. As for the ultimate shear strength, again both the AASHTO

Standard and LRFD codes produced conservative results, with the ratio of experimental to predicted ultimate strengths ranging from 21% to 86% when using the Standard specifications and 13% to 211% when using the AASHTO LRFD specifications. The ACI-based calculations netted several results that overestimated the ultimate shear strength. Given these results, the authors recommended additional studies to determine the concrete contribution to the shear strength for beams constructed with concrete having a compressive strength greater than 10 ksi.

Watanabe et al.: Watanabe et al. (2004) focused on studying the shear strength of lightweight prestressed, precast members with a compressive strength of about 8.7 ksi, as well as the influence of prestressing on shear cracking strength and the behavior of lightweight concrete beyond shear cracking. Seven of the test beams had some type of shear failure. There were three different combinations of aggregate: lightweight fine aggregate with normal weight coarse aggregate, lightweight coarse aggregate with sand fine aggregate, and lightweight fine aggregate with lightweight fine aggregate. The lightweight aggregate was granulated expansive shale, with a nominal maximum aggregate size of 0.31 in. The density of the lightweight concrete ranged from 106 pcf to 119 pcf, while the concrete compressive strength varied from 8.6 ksi to 9.6 ksi. Dimensions of the beams were 7.9 in. wide by 24.6 in. tall and 20.7 ft long, although the span length was 14.1 ft. There were six 0.6-in. diameter prestressing strands arranged in two layers in the bottom of the members, with two additional strands at the top of the section. Prestressing resulted in three levels of stress: zero prestressing, and then enough prestressing to result in the top concrete fiber having zero stress and the bottom concrete fiber having 1.2 ksi and 2.3 ksi of compressive stress. Shear reinforcement consisted of No.4 rebar spaced at 7.9 in. All of the beams were subjected to two concentrated loads that were spaced about 19.7 in. away from the beam centerline, resulting in a shear span-to-effective depth ratio of 3.0.

For all of the beams with 2.3 ksi of compressive stress in the bottom fiber, there was concrete crushing prior to the shear reinforcement yielding, and within this set of beams, the beam with either coarse lightweight aggregate only or fine lightweight aggregate only failed in bending compression. As the concrete density decreased, the shear cracking strength declined. Furthermore, the concrete mix having both coarse and fine lightweight aggregate had a shear cracking strength that was much less than 70% of normal weight concrete, where 70% is a typical factor used in design of lightweight concrete. Thus, the researchers examined using the reduction factor formulated as:

$$0.4 + 0.6\frac{\rho}{147.33}$$

where 147.33 and ρ are the unit weights (in pcf) of normal weight and lightweight concrete, respectively. However, the results from the 106 pcf specimens had a greater reduction than suggested by this formulation, with one reason being the rather low tensile strength, which was about 50% of that of normal weight concrete. This concrete had a high brittleness factor, defined as the ratio of the compressive strength to the tensile strength. Thus, the conclusion was that concretes with a high brittleness factor will have a much lower shear cracking strength than calculated.

Given that the stress distribution through the depth of the cross-section was linear, researchers found no relationship between the type of aggregate and the degree of increase in shear cracking strength due to prestressing. After cracking, however, the results showed that the shear force carried by the normal weight concrete remained at the shear cracking force, where as the force carried by the lightweight concrete decreased. Since the shear force carried by the concrete, V_c, did not necessarily remain constant after cracking and tended to vary depending on type of aggregate that was used, the authors concluded that the modified truss method of analysis should not be used for design. The researchers noted that the concrete mix combining lightweight fine aggregate and normal weight coarse aggregate had about the same value for V_{cu} as normal weight concrete, while the other two combinations of aggregates had less.

Dymond: Dymond's research on lightweight self-consolidating concrete included web-shear and flexural-shear strength of a PCBT-53 girder (2007). The height of this beam was 53 in. and the cross-sectional area was 802.7 in^2, while the span length was 63 ft. Combined with a 9-in. thick by 7-ft wide cast-in-place deck, the composite section measured 1558.7 in^2. Both the girder concrete and the deck concrete consisted of expanded slate coarse aggregate, with a ¾-in. nominal maximum aggregate size, and natural sand for the fine aggregate. At the time of testing, the girder concrete had about 10.6 ksi compressive strength, while the concrete in the deck was about 8 ksi. Unit weights were 118 pcf and 125 pcf for the girder and the deck, respectively. 32

pretensioned, ½-in. diameter, Grade 270 ksi, 7-wire low relaxation strands provided the longitudinal reinforcement. Six of the strands were harped 30 in. from each side of the center line of the girder. Shear reinforcement consisted of No. 5, Grade 60, double-legged stirrups. The stirrup spacing ranged from 3.75 in. at the ends of the beam to 16 in. at midspan. The beam was simply supported and subjected to a simulated truck load with wheel patches spaced 14 ft apart for the flexural shear test. Due to the concrete compressive strength being much higher than the design strength, loading for the web-shear testing required that the two loads be 3 ft apart in order achieve the desired failure. Nevertheless, *a/d* was 3.1 for the flexural shear test and was 1.3 for the web-shear test. There was no strand slip during the testing. The researcher was only able to get close to failure during the web-shear testing, evidenced by signs of concrete flaking in the web. The flexural shear failure was evidenced by concrete crushing near the load point at the top of the girder and concrete splitting along the tension reinforcement.

The angle of inclination of the cracks was 39^o, while the predicted angle was 33.3^o using the design properties with the LRFD code. Strut and Tie Modeling and Mohr's Circle calculations came closer to the experimental value at 39.6^o and 37.2^o, respectively. In terms of shear strength, the experimental web-shear strength exceeded all code predictions when using the actual material properties. The closest comparisons were with the AASHTO Standard (2004) calculations and the simplified methods from NCHRP Report 549 (2005), where the ratio of experimental to predicted shear strengths were 1.09 and 1.07, respectively. On the other hand, AASHTO (2006) underestimated the shear strength by 25% when using the design properties to calculate θ, the angle of inclination. When using the design material properties instead to calculate the predicted web-shear strength, the AASHTO Standard and LRFD specifications had values that were 20% and 34% conservative, while the NCHRP method proved to be 11% unconservative. Juxtaposed to the web-shear results, the flexural shear strength was generally underpredicted when using the actual material properties for all of the calculation methods, with the exception for AASHTO. However, even the LRFD method produced a predicted strength that was only 3% greater than the experimental result. Even when using the design material properties, all of the other methods proved to be unconservative, with the NCHRP-suggested Simplified Method being the most unconservative due to a smaller predicted crack angle that led to overestimating the strength provided by the shear reinforcement. Additional analysis also showed that the nominal maximum shear strength specified in AASHTO for web-shear cracking was far too high, being 33% to 50% unconservative. Thus, the author concluded that the 0.25 multiplication factor being used to the limit the shear strength was too high.