NCHRP Project 04-36

CHARACTERIZATION OF CEMENTITIOUSLY STABILIZED LAYERS FOR USE IN PAVEMENT DESIGN AND ANALYSIS

FINAL REPORT
APPENDIXES

Prepared for
National Cooperative Highway Research Program
Transportation Research Board
of
The National Academies

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CHAPTER A-1. PERFORMANCE ISSUES OF PAVEMENT WITH CEMENTITIOUSLY STABILIZED LAYERS (CSL)

Hot Mix Asphalt Pavement

The performance of hot mix asphalt (HMA) pavement is significantly affected by cementitiously stabilized layers (CSL), especially when CSL are located directly underneath HMA layers. Stabilized subbase and base layers can reduce the rutting of HMA pavement as a result of minimal rutting in the subgrade, subbase, and base (Von Quintus et al. 2005). The bottom-up fatigue cracking of HMA also can be reduced. However, there are performance issues related to CSL, mostly when they are used as the base course.

Block Cracking in HMA

Block cracking often is reported in the HMA surface when the pavement has a stabilized base. Block cracking is caused by shrinkage of the underlying stabilized base and often occurs when the HMA layer is thin, as for local roads (Figure A-1). Highways in many parts of the world that use stiff bases and thin HMA layers also have encountered this problem (Yue 2004, Zube 1969). The shrinkage, caused by a loss of moisture and temperature variation, typically initiates shortly after construction and continues thereafter. According to Zube (1969), high unconfined compressive strength (UCS) causes block cracking which is likely due to the high shrinkage of CSL with high cement content and high strength. In short, block cracking can be attributed to the shrinkage of CSL.

Figure A-1. Block Cracking in HMA with Stabilized Base (Scullion 2002)

Transverse Cracking Induced by Shrinkage of CSL

Transverse cracking in the surface layer that results from shrinkage of the stabilized base, as shown in Figure A-2, starts from the bottom of the surface layer and propagates through the surface layer. The cracking is due to the bond between the surface layer and stabilized base (George 2002). Transverse cracking is also a concern for pavements with a stabilized subbase

and granular base, but at a much later stage (Ramsey 1959). The shrinkage cracking of the subbase causes stress concentrations at the locations of the cracks and eventually affects the stress distribution in the surface layer.

Figure A-2. Transverse Cracking due to Shrinkage Cracking of CSL (Freeman and Little 2002)

Atkinson (1990) reports that shrinkage cracking in CSL causes transverse cracking in HMA and is prominent in thin HMA pavement. Chen (2007) reports that lack of mellowing for lime slurry stabilized base layers causes shrinkage cracking and then transverse cracking in the HMA surface. Little et al. (1995) found that a high modulus value causes wide shrinkage cracks and low load transfers across the crack.

George (2002) found that high-strength CSL are prone to shrinkage cracking, based on Long-Term Pavement Performance (LTPP) and other pavements. When the 7-day in-service strength is 300 psi or lower, no shrinkage cracking occurs. Increasing the fines content increases the cracking intensity. Bituminous curing of the CSL before the placement of the surface layers corresponds to 65% relative humidity (RH) for most specifications. In the laboratory, moist curing corresponds to 95% RH. Crack width is significantly affected by drying shrinkage. Crack spacing decreases with an increase in friction between the CSL and underlying layer. For wide shrinkage cracks, load transfer efficiency is between 35% and 55%, and 80% for fine cracks for coarse-grained aggregate. Cracks wider than 0.1 inch (measured on HMA surface) affect the pavement performance significantly. For fine-grained soil, the critical crack width is claimed to be 0.06 inch. Decreasing the strength of the CSL decreases the tensile stress in the CSL. There is an optimum shrinkage strain level: 525 microstrain for fine-grained soil and 310 microstrain for coarse-grained soils, respectively.

Therefore, as with block cracking, the shrinkage of CSL causes transverse cracking of asphalt pavement and thus is included in this study.

Longitudinal Cracking

(a) Longitudinal cracking in wheel path (top-down cracking)

CSL provide strong support for surface layer, which is beneficial in reducing the fatigue of surface layers that can occur as a result of tension at the bottom of the surface layer. Therefore, alligator cracking in HMA can be mitigated when a stabilized base is used, unless the pavement is under-designed or the CSL are fatigued. However, for asphalt pavements that use high-stiffness CSL as the base, the HMA surface layer is prone to top-down fatigue cracking (ARA 2004), as shown in Figures A-3, A-4, and A-5. This top-down fatigue cracking has been confirmed by other researchers (Meng et al. 2004 and Barstis et al. 2000); and the team's field projects that use stabilized base layers also exhibited this distress. Actually, transverse and longitudinal cracks in the wheel path are the two most major distresses for highway pavements constructed with CSL, as reported by George (2002).

Figure A-3. Top-Down Cracking in HMA with CSL as Base (Wen and Ramme 2008)

Figure A-4. Top-Down Cracking in HMA Layer on Stabilized Base (Wen and Ramme 2008)

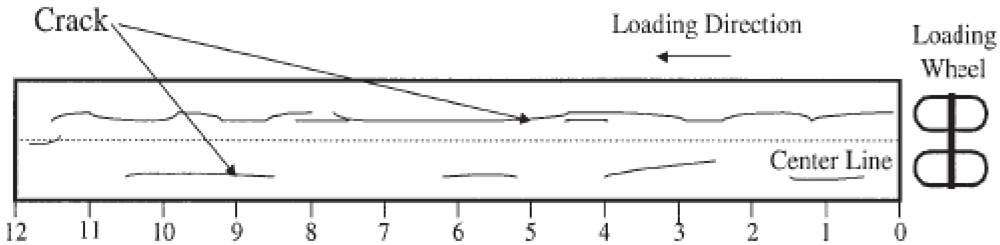


Figure A-5. Top-Down Cracking Based on Accelerated Pavement Testing (Meng et al. 2004)

Syed and Scullion (2001) report that a stabilized base with high stiffness causes longitudinal cracking in the wheel path. The high stiffness of CSL is due to an excessive amount of stabilizer. As a result, the authors recommend that a 7-day UCS of 200 psi should be targeted.

In another study, Scullion et al. (2003) also found that a high modulus value of cement-stabilized full-depth recycled base leads to more longitudinal cracking in the wheel path.

The literature supports that high stiffness or modulus values of CSL lead to top-down cracking in asphalt pavement. Stiffness and modulus values also are needed for the response model of a pavement structure to determine stress and strain.

(b) Longitudinal cracking (outside wheel path): dry-land cracking

Dry-land cracking occurs as a result of shrinkage of expansive soils. The shrinkage cracks reflect through the upper layers and appear in the HMA surface, as shown in Figure A-6. Luo and Prozzi (2008) report that longitudinal dry-land cracking initiates in untreated expansive soil and appears in the HMA surface, as shown in Figure A-7. Adding lime reduces the plasticity index (PI) value, suction, compression index value, and the swelling potential of expansive soils. Wise and Hudson (1971) also report that the subgrade beneath the pavement at the centerline has a high moisture content whereas the moisture content underneath the shoulder fluctuates. The shrink and swell caused by moisture change can lead to longitudinal dry-land cracking. Syed and Scullion (2001) indicate that the shrink-swell of subgrade comprised of expansive soil results in dry-land cracking. The shrinkage cracking in the subgrade reflects through the CSL and appears at the HMA surface. This phenomenon also is confirmed by forensic studies by Chen (2007) and Atkinson (1990) in which expansive soil causes dry-land cracking.

Figure A-6. Dry-Land Cracking (Scullion et al. 2003)

Figure A-7. Dry-Land Cracking Initiated in Subgrade (Luo and Prozzi 2008)

Bottom-Up Cracking (Alligator Cracking) of HMA Layer

Bottom-up cracking may be due to CSL surface raveling or fatigue of the CSL, as follows:

(a) Bottom-up cracking due to CSL surface raveling

Based on accelerated pavement testing, studies (Li et al. 1999, Meng et al. 2004, Thogersen 2005) show that the surface of a stabilized base layer can ravel, creating a layer of loose material between the HMA and base CSL (Figure A-8). Raveling of the base increases the strain level at the bottom of the HMA, which can result in alligator cracking. In addition, pumping was observed in these cases. The pumping is caused by the loss of fines in the loose material layer. This phenomenon may be linked to the erodibility of stabilized materials, which often happens when relatively fine raw materials are treated (De Beer 1985).

Li et al. (1999) reported the loose layer is about 0.8 in. thick and is believed to be due to shear failure within the CSL as a result of horizontal loading, which in turn causes alligator cracking.

Figure A-8. Surface Raveling of Stabilized Materials (Thogersen 2005)

(b) Fatigue of stabilized base (bottom-up tension)

The fatigue of a pavement is a form of structural failure. According to studies conducted in South Africa (De Beer 1990), there are two types of fatigue failure for CSL, bottom tension fatigue and top compression (top crushing).

Alligator cracking in thin HMA pavement, as shown in Figure A-9, also can be accelerated by fatigue cracking of the stabilized base or subbase caused by repeated traffic loads (Pretorius et al. 1972, Li et al. 1999). The fatigue resistance of CSL is reduced when subjected to freeze-thaw and/or wet-dry cycling (Naji and Zaman 2005). Bottom-up cracking in HMA could be due to bottom-up fatigue cracking of the CSL. Under repeated traffic loads, microcracking is initiated at the bottom of CSL due to tensile stress/strain, and propagates upwards.

Figure A-9. Alligator Cracking in Asphalt Pavement with CSL (Yeo 2008)

De Beer (1985) also indicates that when there are two or more lifts of CSL, the upper lift tends to crack first due to debonding and becomes more fractured than the bottom lift. Dry CSL densify whereas the wet, upper CSL loosen. A loose interlayer between the asphalt and CSL increases the potential for fatigue. Cement slurry, cement powder, or bitumen membrane are recommended for use between two lifts. Atkinson (1990) also found that debonding between lifts of CSL causes high stress levels at the bottom of the top layers and thus induces fatigue cracking.

Pretorius et al. (1972) report that after the formation of transverse shrinkage cracking, longitudinal fatigue cracking starts first in the wheel path, which results in corner loading and consequently ladder-type fatigue cracking, as shown in Figure A-10. Norling (1963) notes that ladder-type cracking indicates inadequate design.

Figure A-10. Ladder-Type Fatigue Failure (Scullion et al. 2003)

Jameson et al. (1992) found that with an increase in the number of load cycles, the modulus value decreases linearly on log(E) versus log(Cycles) plots. The modulus value reduces to one-tenth of its initial value when the first crack is visible in the asphalt layer. For asphalt layers that are less than 4-inch thick, the fatigue of the CSL is based on 50% of the initial modulus value of the CSL or 10% cracking in trafficked areas. For an asphalt layer that is thicker than 4 inches, the fatigue life of the CSL is defined as the number of cycles to 50% of the initial modulus value of the CSL.

Li et al. (1999) report that the cracking asphalt layer matches the cracking in the CSL, as shown in Figure A-11.

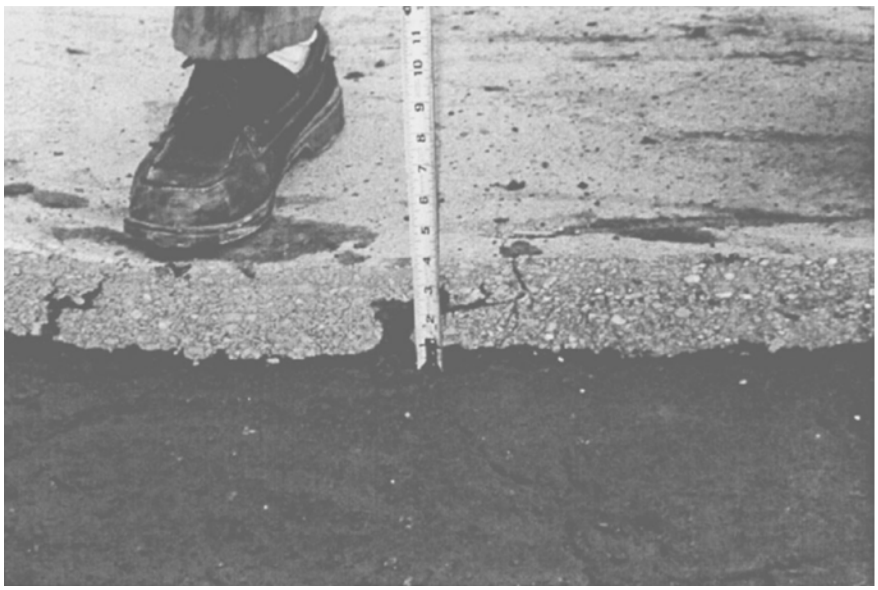


Figure A-11. Cracking of Asphalt Pavement with CSL (Li et al. 1999)

Rutting

Due to the high stiffness of CSL, rutting in the CSL and subgrade can be reduced substantially when compared to unbound materials (Von Quintus et al. 2005). However, the existence of CSL could affect the rutting in asphalt pavements in terms of three factors: high shear stress in the HMA layer, erosion, and failure of the CSL.

(a) Rutting induced by high shear stress in the HMA layer

The existence of CSL changes the stress/strain distribution and induces high shear strain in the HMA layer. As a result, there is high potential for rutting in the HMA layer (Bonnot 1991), as shown in Figure A-12. Meng et al. (2004) report that the high stiffness of CSL leads to deep rutting in HMA as well as top-down cracking due to the increased shear stress distribution in the HMA layer. In addition, once the CSL are cracked and water infiltrates into the pavement, rutting develops quickly at the interface of the base and HMA layers.

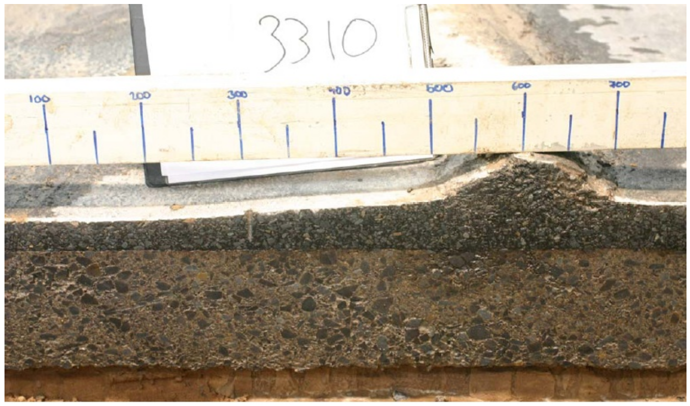


Figure A-12. Rutting in Asphalt Layer (Yeo 2008)

(b) Rutting induced by erosion of CSL

Rutting can occur when there is a loose layer between the HMA and CSL, which results from erosion and pumping of fines in the CSL. De Beer (1985) found that when lime is used to treat sand that is used as a base material, raveling, instead of fracturing, occurs, which causes rutting or increased tensile strain at the bottom of the HMA layer for alligator cracking.

Metcalf et al. (2001) found debonding between the asphalt and CSL, with free water and a soft layer at the interface. The erosion of CSL causes rutting. CSL that are thick and have low cement content perform best in terms of rutting and cracking. Li et al. (1992) also report that when CSL are dry, minimal rutting occurs. However, after CSL are cracked, the entrance of water causes rutting quickly.

(c) Rutting due to fatigue failure in CSL

Top compression/crushing results from fatigue that is due to repeated compression at the top of the CSL. For thick CSL, the tensile strain at the bottom is very small so that tensile fatigue is not an issue. According to De Beer (1990), thick CSL fail in compression (crushing) in the top 2 - 3 inches, as shown in Figure A-13. As a result, rutting occurs in the crushed materials. Increasing the UCS reduces the compression strain and thus increases the compression fatigue life. A compressive strain of 1% is stated to be the failure strain for compression fatigue. The compressive strain increases as the load repetitions increase.

Theyse et al. (1996) report that increasing the UCS increases the compression fatigue life. With regard to compression fatigue characterization, a 0.08-inch rut is criterion used for crushing initiation and a 0.4-inch rut is used for advanced rutting (Theyse et al. 1995). Jameson et al. (1992) also report that for thick CSL (>12 in.), crushing occurs in the top 2 inches of the CSL. Crushing does not cause cracking in the surface layer. However, Theyse et al. observed permanent deformation. The lower the cement content, the finer is the crushed CSL.

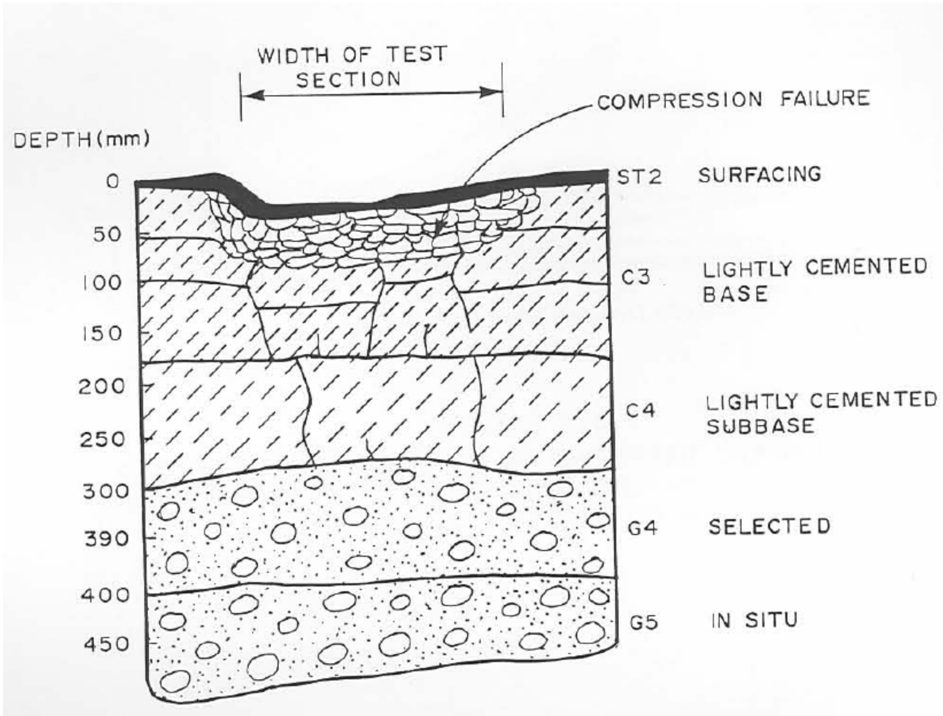


Figure A-13. Crushing Fatigue of CSL (De Beer 1990)

Heave

Expansive soils often are stabilized to mitigate swelling. However, heaving could still occur when sulfate-bearing soil is stabilized with calcium-based stabilizer in the presence of moisture. Chen et al. (2005) report that the formation of ettringite in lime-treated sulfate-bearing soil causes heaving in the pavement surface. Si (2008) reports that the swell of fly ash-stabilized sulfate-bearing soil causes heave in HMA, as shown in Figure A-14. Lime and fly ash are effective in reducing the swell of sulfate-bearing soils.

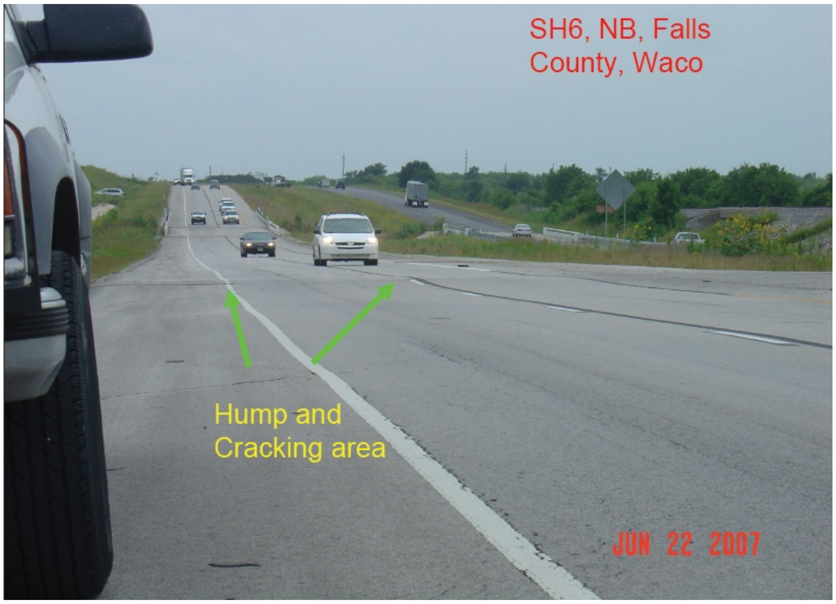


Figure A-14. Heave in Asphalt Pavement (Si 2008)

Concrete Pavement

For concrete pavement with stabilized base layers, most of the reports indicate positive effects, such as reduced faulting, pumping, and cracking (Neal and Woodstrom 1977, ARA 2004, Selezneva et al. 2000, Ruiz et al. 2005, Hall and Crovetti 2007). Nussbaum and Childs (1975) report that using CSL greatly increases the load-carrying capacity of concrete slabs. However, a study by Mallela et al. (2007) shows that the bond between the concrete layer and stabilized base might contribute to early-stage cracking in concrete pavement. The use of a bond breaker is recommended in such cases (Mallela et al. 2007, Ruiz et al. 2005). The erosion of CSL, especially weakly stabilized materials, also may cause pumping and resultant faulting and cracking, as shown in Figure A-15 (ARA 2004, Jung et al. 2009).

Figure A-15. Cracking of Concrete Pavement

In summary, performance issues related to asphalt pavements with CSL include block cracking, transverse cracking, top-down and bottom-up fatigue cracking, rutting, and heaving. For concrete pavements with CSL, the distresses related to CSL include pumping, faulting and cracking.

The properties of CSL that are related to performance issues include stiffness, strength, fatigue, erodibility, swelling, durability, and shrinkage. These properties should be included in the pavement design and analysis.

CHAPTER A-2. PROPERTIES OF CEMENTITIOUSLY STABILIZED MATERIALS LINKED TO PAVEMENT PERFORMANCE

Based on the previous discussion outlined in Chapter 1, the properties of CSL that are related to pavement performance are identified and described in detail in the following sections.

Shrinkage of CSL and Related Pavement Distresses

Shrinkage cracking of CSL as base layers, as shown in Figure A-16, can cause the cracking of the surface layer due to the bond between the surface layer and the CSL. Shrinkage of CSL includes autogenous shrinkage due to hydration, drying shrinkage due to loss of moisture and thermal shrinkage due to low temperature contraction (ACI 2008). Shrinkage of CSL, when restrained (e.g., bonding from underlying layer), causes the development of tensile stress in the CSL. When the tensile stress exceeds the tensile strength of the stabilized materials, shrinkage cracking occurs (George 1990). Shrinkage cracking could occur within a few days or over a couple of years, depending on the curing, shrinkage strain, and other factors. Shrinkage of CSL is affected by many factors, such as moisture, additive content, raw material characteristics, and curing after compaction.

Figure A-16. Shrinkage Cracking of CSL (George 2001)

Moisture

Kodikara and Chakrabarti (2001) report that shrinkage results from moisture loss, which leads to matric suction (capillary forces), osmotic suction, and thermal cooling. George (1990, 2001) reports that moisture content that is higher than the optimum moisture content (OMC) causes excessive shrinkage cracking. George (1990) also reports that shrinkage can be reduced by reducing molding moisture, increasing compaction density, avoiding montorillonite clay, and limiting the degree of saturation to 70 percent.

The moisture content in CSL is affected by the curing method. Sebesta (2005) reports that bituminous curing is minimally effective in reducing cracking problems. Bituminous curing and dry curing provide little difference in shrinkage cracking for 4% cement sections. Shrinkage cracking occurs within the first two days. However, moist curing works better in mitigating shrinkage cracking than dry curing and prime curing for 8% cement sections.

Pretorius et al. (1971) use the theory of viscoelasticity to analyze shrinkage cracking. Increasing the RH reduces the creep and shrinkage strain but increases the relaxation modulus. With an increase in curing time, the tensile strength continues to increase at 100% RH during curing. However, for other RH values, the strength value reaches a peak and then starts to decrease. Increasing the RH reduces shrinkage stress.

Binder Content

Sebesta (2005) reports that increasing the cement content increases the amount of shrinkage cracking. Matthew et al. found that low strength ensures narrow and closely spaced shrinkage cracks that do not reflect through the wearing course. This finding is in line with that of Van Blerk and Scullion (1995) who conclude that high-strength CSL have wide cracks at large spacing and low strength CSL have fine cracks at close spacing.

Little et al. (1995) report that shrinkage cracking spacing depends on tensile strength and the friction between the CSL and underlying layer. The width of the cracks depends on the tensile stiffness of the CSL. A maximum shrinkage strain of 250 microstrain is recommended.

Soil Properties

The characteristics of soils directly affect the shrinkage behavior of CSL. Norling (1973) found that increasing the clay content increases the occurrence of shrinkage. Van Blerk and Scullion (1995) also indicate that an increase in the plasticity index (PI) value increases the shrinkage potential. Smectite clay causes the most shrinkage, when compared to other types of clay. The linear shrinkage of the fine fraction of the aggregate is a good indicator of the ultimate drying shrinkage of the CSL. It is recommended that linear shrinkage of 1.5%, PI value of 4.0, passing #200 sieve of 7%, and shrinkage after 21 days of 250 microstrain are the maximum allowed.

Kodikara and Chakrabarti (2001) report that a clay size that is smaller than 0.08 mil is responsible for shrinking and swelling. Autogenous shrinkage accounts for only 5% of the total shrinkage. The shrinkage potential of cementiously stabilized materials (CSM) is between that of clay and cement paste. Adding cement to clay reduces shrinkage due to the reduction in matric suction. However, after reaching a low point, adding more cement will induce higher shrinkage strain due to more gel particles. The restrained shrinkage cracks were examined with a microscope in the Kodikara and Chakrabarti study.

Thermal Cooling

Bonnot (1991) reports that factors that affect shrinkage include water content, thermal shrinkage, and the strength of the materials. Shrinkage cracking can result from thermal contraction. Cooling by 9°F to 18°F can cause thermal cracking in the laboratory.

Fatigue of CSL and Related Pavement Distresses

There are two types of fatigue failure for CSL: bottom-up tensile-fatigue and top-down compressive-fatigue.

Bottom-Up Tensile-Fatigue of CSL

Bottom-up tensile-fatigue of CSL, as shown in Figure A-17, occurs as a result of tensile strain at the bottom of the CSL due to repeated traffic loads. Otte (1978) studied the fatigue cracking of CSL and reported that when the tensile stress exceeds 35% of the strength or a tensile strain level of more than 25% of the break strain in a flexural beam test, microcracking starts and the stress-strain relationship becomes nonlinear. Microcracks initiate at the bottom of the CSL and propagate upwards, as shown in Figure A-18. According to Theyse (1996), bottom tension fatigue consists of three phases, as shown in Figure A-19. De Beer (1990) reports that bottom-up cracking typically occurs in relatively thin CSL in which tensile strain could cause fatigue damage. The fatigue life in the laboratory is multiplied by a shift factor to account for traffic that occurs between the time of crack initiation and visible cracks. The bottom-up fatigue consists of three phases: (1) a pre-cracking phase (including shrinkage cracking) prior to fatigue initiation, (2) fatigue initiation and propagation, and (3) a post-cracking phase. The duration of the pre-cracking phase consumes about 20% of the life of the CSL. During fatigue propagation, the modulus of the CSL decreases due to fatigue damage. Permeability of the CSL increases as the number of loads increases due to cracks. The rate of degradation of the effective modulus is 52 ksi per 1 million loads for the wet state. In Phase 3, the CSL degrade into small pieces. The size of the degraded CSL depends on the strength of the CSM. After Phase 3, the disintegrated pieces could intrude into the underlying subgrade. During the post-cracking phase, the effective modulus value of the CSL is equivalent to that of the granular materials in terms of CSL thickness. When CSL is dry, the size of the equivalent granular materials is about 1.5 times of the CSL thickness and, after it is wet, the size is 0.3 times that of the CSL thickness. During the post-cracking phase, compressive strength and erodibility govern the rut depth.

Figure A-17. Fatigue Cracking in CSL (Yeo et al. 2002)

Figure A-18. Fatigue Cracking in CSL Trenching (Moisture in Cracks) (Yeo 2008)

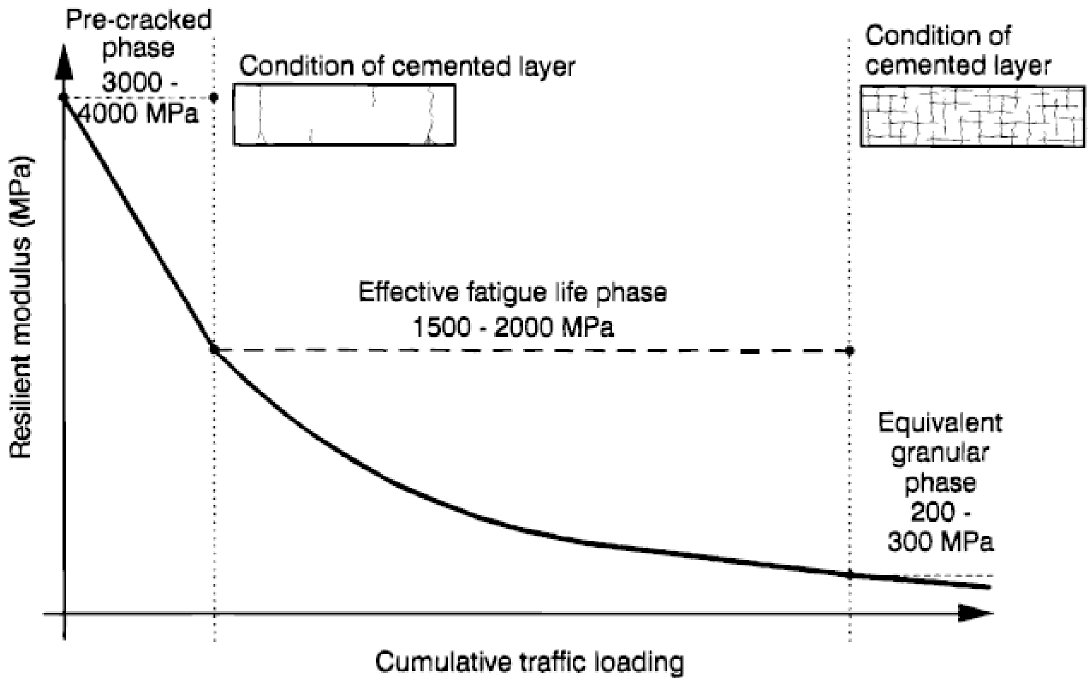


Figure A-19. Three Phases of CSL Fatigue (Theyse 1996)

According to Little et al. (1995), heavily stabilized base layers often fail in fatigue due to tension if the CSL are thin. A minimum thickness of 8 inches is recommended to limit the stress ratio, i.e., the tensile stress at the bottom of the CSL divided by the modulus of rupture (MOR), to 0.5. The crack propagation in the CSL follows the laws of fracture mechanics. The modulus value of the CSL is reduced as a result of fatigue. Yeo (2008) reports that a reduction in the backcalculated modulus is an indication of fatigue. Increasing the modulus improves fatigue resistance.

The fatigue of CSL is related directly to the strength of the CSM. For concrete pavement with CSL, Nussbaum and Childs (1975) found that higher MOR (also called flexural strength) values of CSM correspond to a longer fatigue life of the CSL. Hadley et al. (1972) investigated the indirect tensile (IDT) strength of CSM and found that increasing the IDT strength increases fatigue resistance. Theyse et al. (1996) report that an increase in the break strain of CSM increases the tensile fatigue life of CSL.

Crushing Fatigue of CSL

Crushing fatigue occurs due to the repeated compressive strain at the top of CSL (De Beer 1990). Crushing fatigue of CSL could cause rutting in asphalt pavements with CSL. Crushing typically happens in relatively thick, lightly stabilized CSL and is related to the compressive stress ratio, as shown in Figure A-20. Freeman and Little (2002) report that the

failure of CSL is due to debonding between the CSL, fatigue on top of the CSL, and pumping of fines.

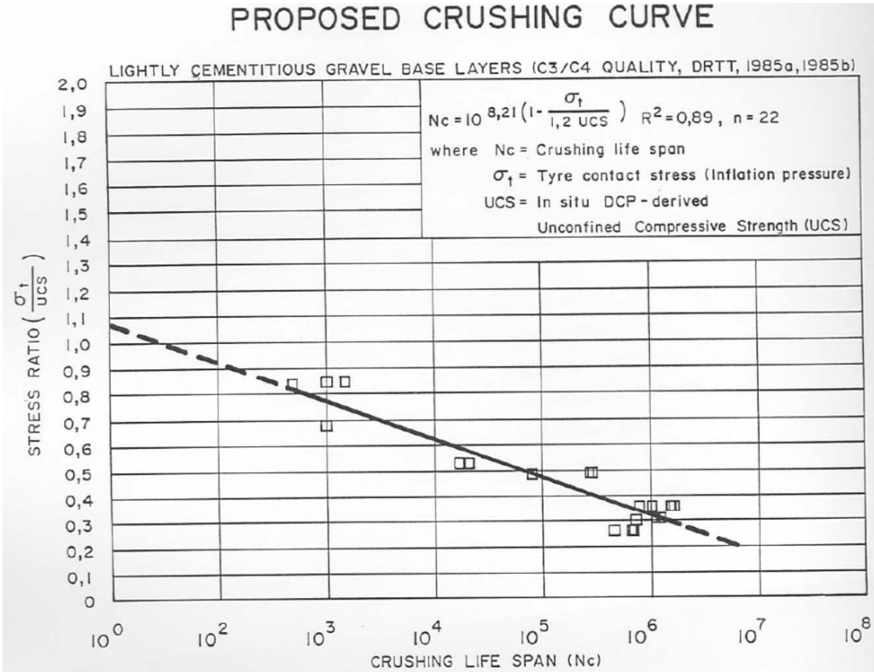


Figure A-20. Compressive Crushing Fatigue of CSL (De Beer 1990)

Durability of CSL and Related Pavement Performance

Stabilized material deteriorates as a result of environmental conditions such as freeze-thaw cycles, wet-dry cycles, and erosion. Under freeze-thaw and wet-dry cycles, the strength and stiffness values of the CSL are reduced. As a result, resistance to fatigue cracking might be compromised. As shown in Figure A-21, freeze-thaw cycles can cause significant damage to CSM.

Laboratory studies indicate that freeze-thaw cycles significantly reduce the UCS and MOR of CSL (Wen and Ramme 2008, Naji and Zaman 2005, Dempsey and Thompson 1973). A field study by the research team for a previous project shows that, after seven years of service, the UCS in the middle of the traffic lane of CSL is less than 10% of the original strength (Wen and Ramme 2008). The loss of strength in the middle of the traffic lane indicates that the deterioration of CSL strength comes primarily from climatic conditions instead of traffic loads. In addition, the reduction of stiffness and strength causes high deflections and high stress levels in the surface layer, which results in bottom-up fatigue cracking in the surface layer.

Dempsey et al. (1984) studied the effects of freeze-thaw cycles on the properties of CSM and found that cooling and heating rates are important to freeze-thaw durability. In Illinois, most freeze-thaw cycles occur between December and February, with January having the most cycles. The average number of cycles is four per year in Illinois. The cooling rate is 0.15°F/hr and the

heating rate is 0.30°F/hr. According to Bonnot (1991), durability can be evaluated in terms of expansion, loss of mass, residual strength or change of strength, or swelling.

Figure A-21. Material Degradation after Freeze-Thaw Cycles (Khoury 2005)

Swelling/Shrinkage of CSL and Related Pavement Performance

Expansive soil in any layer of a pavement system is detrimental to its performance, creating problems such as dry-land cracking (Luo and Prozzi 2008). Cementitious stabilization often is used to mitigate the swell tendencies of expansive soil.

Expansive Soils

The volume change of expansive soils can cause dry-land cracking in an asphalt surface layer. Therefore, expansive soils typically are stabilized. The U.S. Corp of Engineers (1994) define expansive soil as soil that swells more than 3 percent. The PI can be used to indicate the swell potential.

Wise and Hudson (1971) report that changes in moisture content in expansive soil cause swell and shrinkage. The study reports that moisture content below 6% is stable. Montmorillonite clay is expansive, and illite and kaolinite clay are not as expansive as montmorillonite. Lime treatment and pre-wetting mitigates swelling.

Petry and Jiang (2007) report that, after stabilization, total suction increases as a result of an increase in osmotic suction; however, swelling potential decreases. A WP4 Dewpoint potentiometer was used to measure total suction and osmotic suction. Beckham and Hopkins (2005) report that the stabilization of expansive soils by lime reduces swell. The heaving in HMA results from the expansion of untreated subgrade soil.

Sulfate-Bearing Soils

For sulfate-bearing soils, ettringite can form in the presence of moisture, calcium, sulfate, and alumina, and can cause heave in a pavement, as shown in Figure A-22 and A-23. A sulfate concentration of 3,000 ppm in the soil is the threshold for swelling in sulfate-bearing soils (TxDOT 2005). When the concentration is less than 3,000 ppm, a traditional stabilization method can be used. When the sulfate concentration is between 3,000 ppm and 8,000 ppm, a single lime application, mellowing and additional moisture can be used. When it is higher than 8,000 ppm, removal and replacement or blending of non-plastic soils is recommended (Texas DOT 2005). Sulfate-induced swell can occur overnight with a sufficient source of moisture.

Vasudev (2007) reports that sulfate-bearing soils stabilized with Type V cement and fly ash exhibit the best field performance in terms of heaving resistance, followed by ground blast furnace slag.

Figure A-22. Swell of Sulfate-Bearing Soil Stabilized with Calcium-Based Additive (Texas DOT 2005)

Figure A-23. Sulfate-Bearing Soil (Texas DOT 2005)

As mentioned previously, many agencies often replace expansive soils with other soils. The swelling issue is not investigated further in this study, as directed by the NCHRP panel.

Strength of Cementitiously Stabilized Materials and Related Pavement Performance

The strength of CSL directly controls their performance and thus affects overall pavement performance. Various strength measures of CSM are used to quantify their specific engineering behavior. The MOR is the key parameter in the fatigue failure of CSL. Tensile strength affects the development of shrinkage cracking in CSL. UCS is a key parameter for the top compression fatigue model. In addition, UCS tests often are used for the purpose of mix design.

Otte (1978) reports that the linear portion of the stress-strain curve in a flexural beam test reflects up to 35% strength of 25% break strain. The ratio between direct tensile strength and IDT strength is close to one. The Otte study also evaluates compressive strength, tensile strength, IDT strength, and bending strength. The bending test is recommended by Otte (1978) for fatigue study. Bonnot (1991) reports that in Europe, Spain uses the MOR, Italy uses IDT strength, and France uses tensile strength.

According to Theyse (1996), increasing the UCS reduces compression fatigue, and increasing the breaking strain decreases tension fatigue. The yield strength of damaged CSL is negatively related to the plastic strain of CSL. Thompson (1986) reports that an increase the

MOR mitigates fatigue cracking. George (2001) reports that a low strength or low modulus/strength ratio is beneficial in mitigating shrinkage cracking.

Pretorius et al. (1972) report that flexural testing simulates field conditions better than direct tension testing. High confinement leads to high strength of the CSM but low failure strain. A sustained load that is larger than the critical stress (75% strength) can cause microcracks and eventual failure. Pretorius et al. also found that tensile strength is about one-tenth of UCS and one-fifth of MOR.

Stiffness of CSL and Related Pavement Performance

The stiffness (or modulus) of CSL is critical to the analysis of pavement and performance prediction. Low stiffness of CSL may create high stress levels in the surface layer and, subsequently, fatigue cracking. However, HMA pavements with a very stiff base are prone to top-down cracking (ARA 2004). In general, high stiffness stems from high additive content, which also may cause high shrinkage rates. Therefore, the impact of stiffness (or modulus) must be studied to develop an appropriate stiffness range for pavement application. For stabilized subbase, high stiffness is generally not a concern. Stiffness of CSM refers to the resilient modulus, modulus of elasticity, flexural modulus, or IDT modulus, depending on the test mode.

Erodibility

Erosion can cause several issues in pavements with CSL, such as pumping of fines, creating a loose layer between the surface layer and CSL, and accelerating the degradation of the CSL. Water can infiltrate the pavement structure through the shoulder and surface cracks or from underground. High dynamic pore pressure builds up with traffic loading, which loosens fine particles, reduces densities, and creates voids (De Beer 1990).

Wjvdm et al. (2001) report that the introduction of water accelerates permanent deformation and erosion. Vorobieff (1997) reports that, in order to prevent erosion, a minimum of 4% binder content is needed. Jung et al. (2009) report that concrete pavement on top of weak CSL does not perform well due to pumping. High-strength CSL have better pumping resistance than low-strengh CSL. Jung et al. introduced and developed erosion models and tests.

The damage to CSL by weathering and traffic also tends to accelerate erosion and pumping issues (Li et al. 1999, Meng et al. 2004). Thus, the durability of CSL is an essential property that affects pavement performance.

Interface Bond

Even though it is not a material property, the interface bond between CSL and underlying material restrains the free movement of the CSL due to shrinkage or expansion. Interface bonding thereby affects shrinkage crack spacing and width.

Romanoschi and Metcalf (2001) report that the loss of a bond between HMA and CSL significantly increases the tensile strain at the bottom of the HMA layer. The bond between CSL and HMA is often lost due to the presence of water and erosion/crushing of the CSL surface. Shear failure occurs within the top of the CSL instead of at the interface of the CSL and HMA. The loss of the bond between the asphalt and CSL causes a shift in critical tension from the top of the subgrade to the bottom of the asphalt layer.

Wimsatt et al. (1987) report that increasing the interface bond strength between Portland concrete cement (PCC) and the base results in narrow crack spacing. The failure plane happens in CSL. The IDT strength of the CSL is correlated with the friction forces. Grogan et al. (1999) report that for concrete on top of CSL, asphalt emulsion does not work well as a bond breaker. Slippage and horizontal cracks are located below the interface of the concrete and CSL.

Wesevich et al. (1987) also report that for concrete pavement with CSL, friction at the interface results from adhesion, shearing and bearing. The soil cement base has the highest level of friction with a concrete surface, followed by the granular base, and asphalt and lime clay bases.

Based on the above discussion, the relationships between pavement performance and significant CSL properties are developed, as shown in Table A-1.

Table A-1. Matrix of the Relationship between Pavement Performance and Engineering Parameters

CSL Properties	**Distresses in Surface Layer**							
	Rutting in Asphalt Layer	Block Cracking in Asphalt Layer	Bottom-Up Alligator Cracking of Asphalt Layer	Transverse Cracking in Asphalt Layer	Top-Down Longitudinal Cracking in Wheel Path	Heave	Transverse Cracking of Concrete Pavement	Faulting of Concrete Pavement
Stiffness/Modulus	(+) CSL Base		(-) CSL Base/Subbase		(+) CSL Base/Subbase			
Strength	(+) CSL Base		(-) CSL Base/Subbase		(+) CSL Base			
Durability (freeze-thaw, wet-dry)			(-) CSL Base/Subbase					
Fatigue Resistance	(-) CSL Base		(-) CSL Base/Subbase					
Erodibility Resistance	(-) CSL Base		(-) CSL Base				(-) CSL Base	(-) CSL Base
Shrinkage Resistance		(-) CSL Base		(-) CSL Base			(-) CSL Base	
Swell Resistance						(-) CSL Base/Subbase		

Note: "+" indicates a positive relationship; and "-"a negative relationship. For instance, increasing the modulus also increases the rutting potential in the asphalt layer (not pavement).

CHAPTER A-3. LITERATURE REVIEW FOR PROPERTIES OF CSL

Strength and Modulus

The modulus of CSM is used in the pavement response model for the critical stress and strain analysis that is used in the performance models. The modulus of CSM can be obtained by conducting resilient modulus (M_r), modulus of elasticity (MOE), IDT modulus, flexural modulus (E_f), and seismic modulus tests. The current MEPDG recommends the use of the modulus of elasticity for heavily stabilized materials, such as cement-treated aggregate, and the resilient modulus for lightly stabilized materials, such as soil-lime, as the Level 1 input. For Level 2, the modulus is predicted from the UCS. The modulus of CSM is affected by many factors, as follows (AustROAD 2008, Foley 2002**,** Marais et al. 1973, Yeo 2008, Khoury 2005, Arora and Aydilek 2005):

- proportion of coarse angular aggregate
- density
- compaction moisture content up to OMC
- binder content
- age
- efficiency of mixing
- field moisture content
- size of aggregate

Figure A-24 illustrates the effects of density on the modulus of CSM (Carteret 2009).

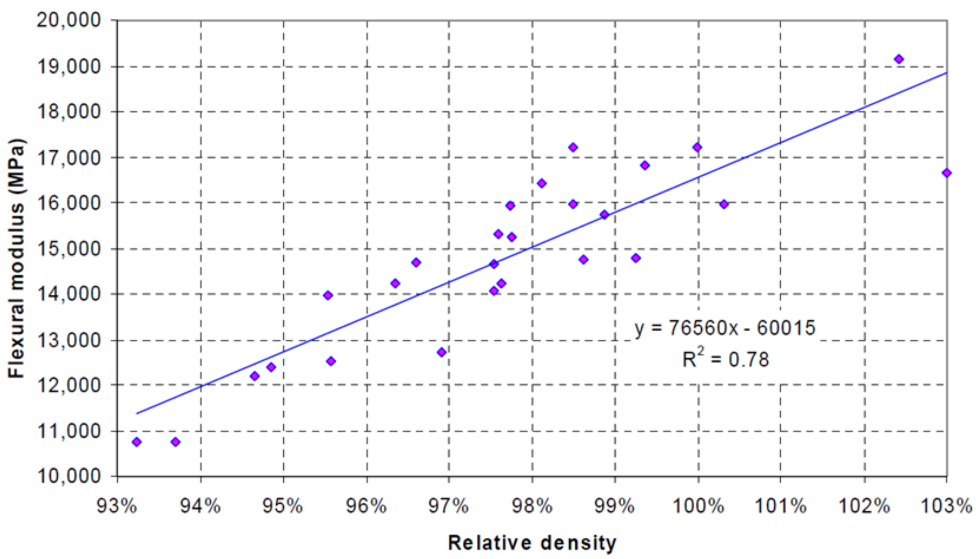


Figure A-24. Effect of Density on Flexural Modulus (Carteret et al. 2009)

The strength of CSM often is used in performance prediction models. The strength of CSM can be obtained from MOR, IDT strength, UCS, and direct tensile tests. The strength of CSM is affected by many factors. Factors that affect the tensile strength of soil-cement include: molding water content, curing time, aggregate gradation, type of curing, aggregate type, curing temperature, compaction efforts, type of compaction, and cement content (Jayawickrama et al. 1998). It is reported that delayed compaction can reduce the strength of CSM. High levels of compactive effort lead to high UCS values. Increasing the water content decreases the UCS (Bhattacharja and Bhatty 2003). Increasing the cement content increases the UCS and resilient modulus values (Arora and Aydilek 2005). Increasing the fines content up to 30% increases the resilient modulus of lightly stabilized soil and the UCS (Ashtiani et al. 2007).

Relationships between the modulus and strength of CSM are reported, as follows.

For soil-cement:

$M_r = 1245 \times UCS + 300$ (Foley 2002) Eq. (A-1)

$M_r = 0.2 \times UCS;\ UCS = 0.028 \times MOE - 1142.6\ (psi)$ (Miller et al. 2006) Eq. (A-2)

$M_r = 1245 \times UCS + 300$ (AustROAD 2008) Eq. (A-3)

$E_f\,(28d) = k \times UCS\ (28\text{-day})$, and k = 1000~1250 (AustROAD 2008) Eq. (A-4)

$M_r = 62.5 \times UCS(7\text{-day})^{0.5}$ (Scullion et al. 2008) Eq. (A-5)

$MOE(t) = 4.38 \times w^{1.5} \times UCS^{0.75}$ where w is water content (Lim and Zollinger 2003) Eq. (A-6)

$M_r = 5.2851 \times UCS(7\text{-day})^{0.5}$ (Scullion et al. 2008) Eq. (A-7)

For lime/fly ash:

$M_r = 696 \times UCS^2 - 222 \times UCS$ for lime/fly ash (1:3) (Foley 2002) Eq. (A-8)

$M_r = 574 \times UCS^2 + 564 \times UCS$ for lime/fly ash (1:1) (Foley 2002) Eq. (A-9)

For soil-lime:

$IDT\ strength = 0.13 \times UCS;\ M_r = 0.25 \times UCS$ (Little 1999) Eq. (A-10)

$M_r = 0.124 \times UCS + 9.98$ (ARA 2004) Eq. (A-11)

For full-depth reclaimed materials:

$M_r = 7.3 \times (E/1200)^{0.5}$ (Barnes 2008) Eq. (A-12)

Figure A-25 shows the relationship between UCS and IDT strength (Kennedy and Hudson 1973), and Figure A-26 shows the relationship between the MOR and the resilient modulus (M_r) (Sobhan and Krizek 1998).

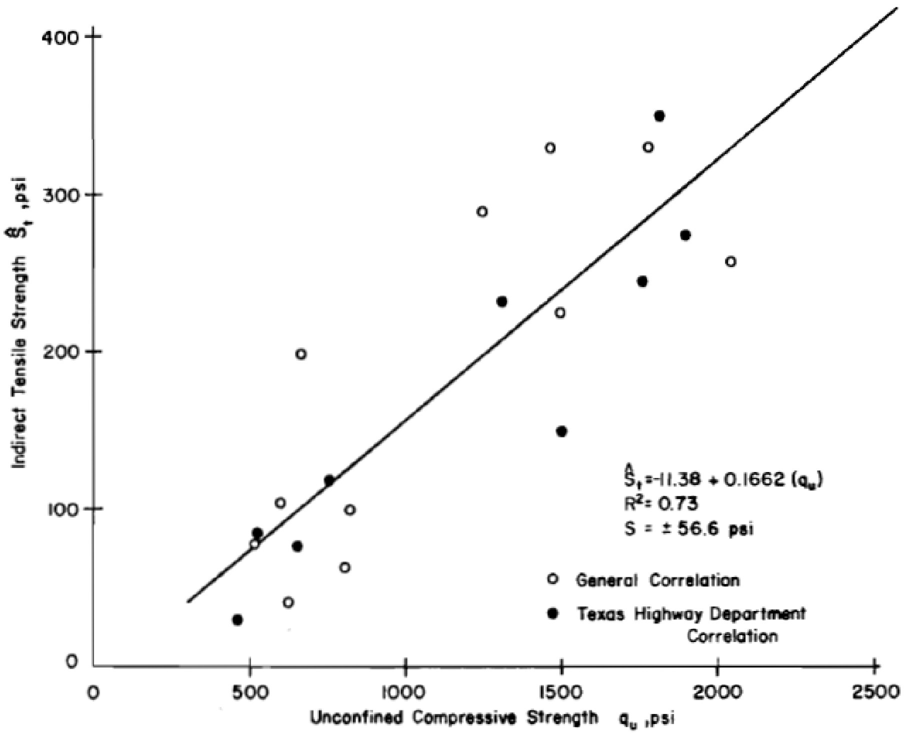


Figure A-25. Relationship between UCS and IDT Strength (Kennedy and Hudson 1973)

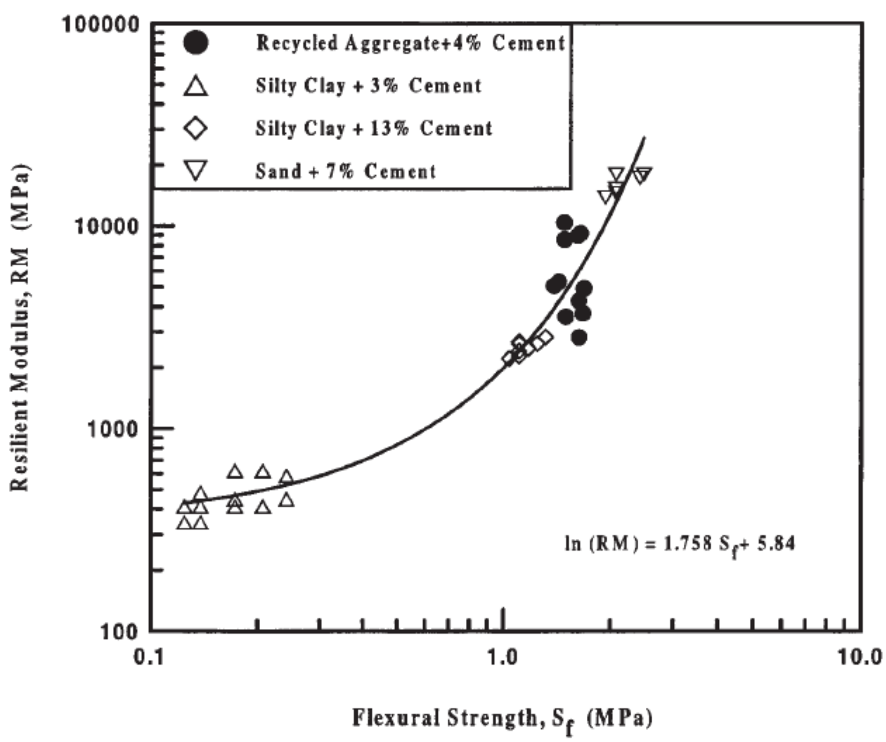


Figure A-26. Relationship between Flexural Strength (MOR) and Resilient Modulus (Sobhan and Krizek 1998)

In addition, good correlation is found between the seismic modulus and resilient modulus, as shown in Figure A-27.

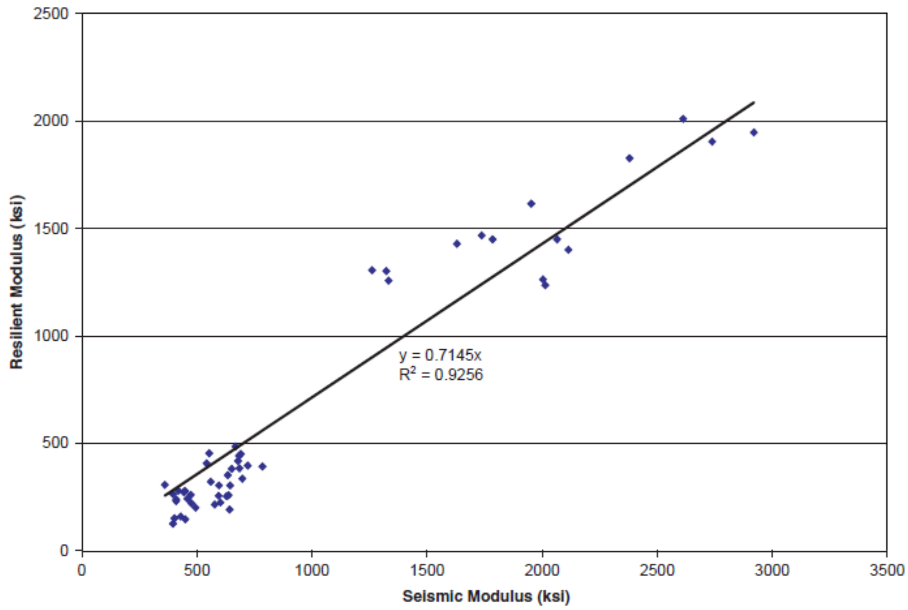


Figure A-27. Correlation between Seismic Modulus and Resilient Modulus (Hilbrich and Scullion 2007)

Strength values obtained from different test modes can be related, but the modulus and strength values can only be related for a specific material (Carteret 2009). In addition, the strength and modulus of CSM continue to grow as the materials age. Little and Nair (2007) report that for soil-lime, the UCS increases by 3.2 times from 28 days to 26 weeks and in another cases by 11 times from 60 days to 180 days.

Wang and Hudson (1972) use the following formula to predict the direct tensile strength of soil-cement.

$$Direct\ tensile\ strength = [0.5+60C^{1.33}/(32+C^{1.33})][1+(2log(t/92))^{2.67}+(log(t))^{2.67}] \quad \text{Eq. (A-13)}$$

where

C = cement content, %, and

t = curing time, days.

The U.S. Air Force also provides a prediction equation to estimate the growth of UCS of soil-cement, as follows.

$$UCS(t)=UCS(t_0)+Klog(t/t_0) \quad \text{Eq. (A-14)}$$

where

K = 70C for granular soils and 10C for fine-grained soils, *t* is the number of curing days, and *C* is the cement content (%).

Kim and Zollinger (2003) developed a prediction equation to estimate initial strength of concrete based on 28-day strength:

$$f(t)=f(28)(t/[2.5+0.9t] \quad \text{Eq. (A-15)}$$

where

$f(t)$ = UCS earlier than 28 days.

t = days since the compaction of CSM within 28 days.

The modulus of CSM reduces as a result of fatigue damage or weathering. After complete damage (i.e., failure), the modulus of CSM is equivalent to the modulus of granular materials. The U.S. Army Corps of Engineers developed a relationship between the deteriorated modulus of CSL and the original strength of CSM, as shown in Figure A-28.

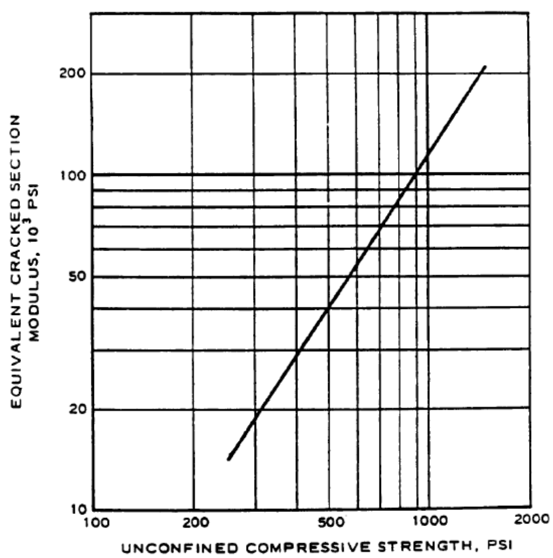


Figure A-28. Relationship between Equivalent Cracked Modulus of CSL and Original UCS of CSM (Department of the Army and the Air Force 1994)

The above relationship can be approximated by Eq. (A-16):

$$E_{csm}(min) = 10^{(1.5529\, log(UCS_{28})+0.4132\,)} \quad \text{Eq. (A-16)}$$

where

E_{CSM} = minimum CSL modulus, ksi

UCS_{28} = UCS after 28-day curing, psi

Durability

The CSL are subjected to weathering in the field, such as wetting and drying, freezing and thawing, moisture susceptibility and erosion, which must be characterized.

Wetting and Drying

Wetting and drying affect the strength and modulus values of CSM. Typically, the modulus and strength values reduce as the number of wetting and drying cycles increases (Paige-Green 1998, Zaman et al. 1999, Khoury and Zaman 2007, Ling et al. 2008, Scullion et al. 2008). However, after a certain number of cycles, the strength and modulus values reach their minimum values. Zaman et al. (1999) found that the resilient modulus reaches its minimum value, which is about 60% of the original modulus value, after 8 cycles of wetting and drying, and 60% of the original UCS after 4 cycles. In another study by Khoury and Zaman (2007), the minimum modulus value after wetting and drying cycles is about 30% of the original modulus value. In addition, wetting and drying can also lead to susceptibility of CSM to erosion (Van Wijk 1985, De Beer 1993, Ras and Visser 2004, Scullion 2005).

In summary, wetting and drying significantly affect the performance of CSL and should be considered in pavement design and analysis.

Freezing and Thawing

Freezing and thawing also affect the performance of CSL. It is reported that freeze-thaw cycles significantly reduce the resilient modulus, flexural modulus, MOR, and UCS values (Khoury 2005, Paige-Green 1998, Zaman et al. 1999). Figure A-29 shows the effects of freezing and thawing on the MOR (Khoury 2005). After a certain number of freeze-thaw cycles, the reduction of the strength and modulus is not pronounced (Esmer et al. 1969, Thompson and Dempsey 1977, Zaman et al. 1999) because freezing and thawing increase the pore size, reducing the damaging effects of later freeze-thaw cycles (Esmer et al. 1969). Freeze-thaw cycles also are used in the mix design of stabilized materials (Portland Cement Association 1992). The mass loss after 12 freeze-thaw cycles is one of the design criteria. Freeze-thaw durability is significantly affected by the original UCS of the CSM. According to the PCA, 95% of CSM samples with UCS values of 750 psi or higher can pass the freezing and thawing design criteria. Table A-2 lists the design criteria used by the U.S. Army Corps of Engineers.

Table A-2. Durability Criteria for CSM (Corp of Engineers 1994)

Type of Soil Stabilized	*Maximum Allowable Weight Loss After 12 Wet-Dry or Freeze-Thaw Cycles Percent of Initial Specimen Weight*
Granular, PI < 10	11
Granular, PI > 210	8
Silt	8
Clays	6

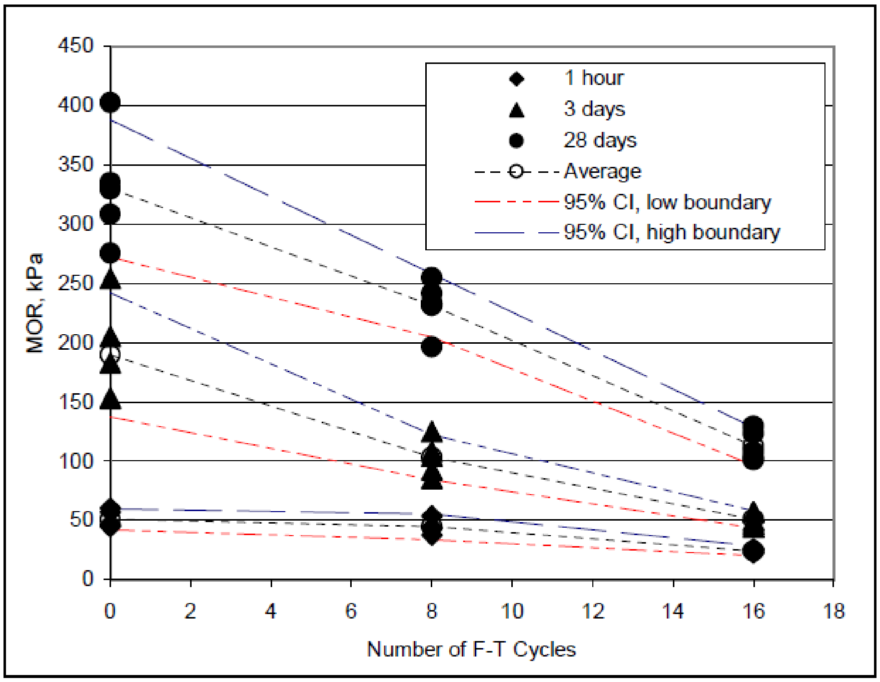


Figure A-29. Effects of Freeze-Thaw Cycles on the MOR (Khoury 2005)

Moisture Susceptibility

Moisture conditioning significantly affects the performance of CSL, especially for lightly stabilized materials (Celaya et al. 2009, Dempsey and Thompson 1973) in terms of strength and modulus. The moisture conditioning of CSM can be applied through soaking, backpressure saturation, vacuum saturation, or capillary rise. The retained UCS after moisture conditioning has been used in the mix design in Texas (Geiger et al. 2007). The moisture conditions used in Texas are drying at 104°F for 2 days followed by capillary saturation for 8 days or 4-day soaking. Recently, the dielectric value (DV) based on the tube suction method has been used as a rapid assessment of moisture susceptibility of stabilized materials after the CSM are subjected to capillary rise. An increase in the DV is found to correspond to a decrease in the residual UCS (Zhang and Tao 2006) and wetting-drying brushing results (Syed et al. 2003), as shown in Figure A-30. A DV lower than 10 indicates good durability of the CSM (Barbu 2004). The capillary rise is affected by pore distribution, initial moisture content, and the infiltration process (Foley 2002,

Zhang and Tao 2006). Si (2008) found that the DV of CSM corresponds to the retained strength for stabilized low-sulfate soils. However, for stabilized high-sulfate soils, no correlation was found between the DV and retained strength.

The moisture conditioning preparation method is reported to affect the evaluation of moisture susceptibility. Celaya et al. (2009) report that the current tube suction method is time-consuming and that moisture conditioning could be interrupted at the compaction lift interface. In addition, the capillary rise method causes non-uniform distribution of moisture. Celaya et al. used the backpressure method to saturate samples uniformly and recommend it to replace the capillary rise method.

Vacuum saturation is another method for the rapid assessment of moisture susceptibility of CSM. Dempsey and Thompson (1973) found good correlation for the residual strength and moisture content between vacuum saturation and five freeze-thaw cycles.

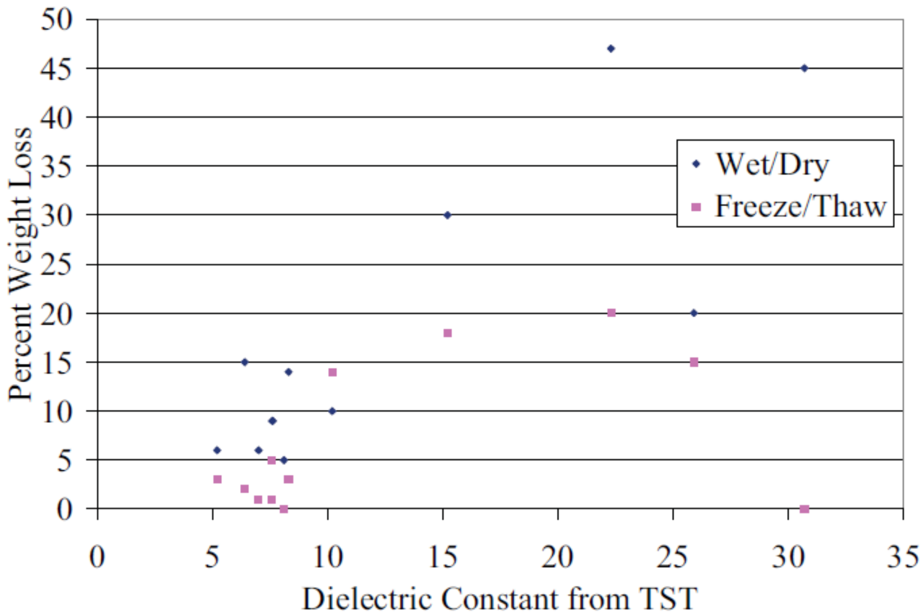


Figure A-30. Relationship between DVs obtained from Tube Suction Test Results and Weight Loss from Wet-Dry/Freeze-Thaw Test Results (Syed et al. 2002)

Erodibility

Erodibility refers to the capability of a soil to resist the detachment of fines under external loading or weathering. The erodibility of CSM reduces the erosion potential. The factors that are found to increase the resistance to erosion include an increase in the curing RH, increase in density, increase of cement content, and decrease of excess fines and/or excess coarse particles (Foley 2002, Gass et al. 1993, Harris et al. 2007). For erosion to occur, the following conditions are needed: heavy loads, sufficient moisture, and erodible materials (Foley 2002).

Van Wijk (1985) reports that material gradation and permeability affect the erodibility of materials. Material particles larger than 0.4 inch do not pump. For an open-graded subbase, the pore water pressure is less than 0.3 psi with velocity of 6.6 ft/sec or less. For dense-graded materials, the pressure is up to 3 psi with velocity of 22.6 ft/sec. Erodibility of stabilized materials depends on binder content, water content, and environmental effects (e.g., freezing and thawing). The water pressure also is affected by the speeds and loads of passing vehicles, deflections, and voids under the slab. Water pressure is higher at higher traffic speeds; water pressure increases with an increase in traffic speed up to 22 miles/hour and then decreases. Large deflections cause high levels of water pressure. Curled slabs exhibit high water pressure levels up to 5.8 psi. Pressure is found to be higher when load was on leave slab. The shear stress of water caused by a moving truck (42,000 lbs) is about 2 psi at 60 miles/hour. An automobile produces water pressure of 0.5 psi. Therefore, most erosion occurs due to moving truck traffic instead of automobile traffic. Van Wijk proposes a critical shear strength of 3.6 psi. It is seen that the water pressure level is relatively low compared to the strength of most CSM. Therefore, as reported in the literature (Foley 2002), the presence of impacts from heavy loads is a required condition for erosion to occur.

The UCS often is used to indicate the performance of CSL. It is reported that a strong correlation exists between UCS and erosion (Gass et al. 1993, Barbu 1994). A minimum 7-day UCS of 300 psi is deemed to be sufficient to resist erosion. In addition, wet/dry brush test results correlate with the critical shear stress of materials (Gass et al. 1993). The moisture susceptibility of CSM, i.e., the DV obtained from tube suction test results, also is found to correlate with the erosion potential of CSM (Barbu 1994), as indicated by wheel abrasion test results (Syed et al. 2000).

For erosion to occur, the shear stress caused by moving water must exceed the critical shear stress, as shown in Figure A-31. It is found that the critical shear stress is related to the saturated, undrained soil shear strength (Leonard and Richard 2004, Indraratna et al. 2008), which is typically half of the UCS. Adding binder increases the erosion resistance of CSM. There is a critical cement content below which erosion will occur. Curing time, soaking, delay of compaction, cement content and the number of freeze-thaw cycles are found to affect erosion (Howard 1988). Brushing after wetting and drying also is used to assess the erodibility of CSM in term of mass loss (Paige-Green 1998).

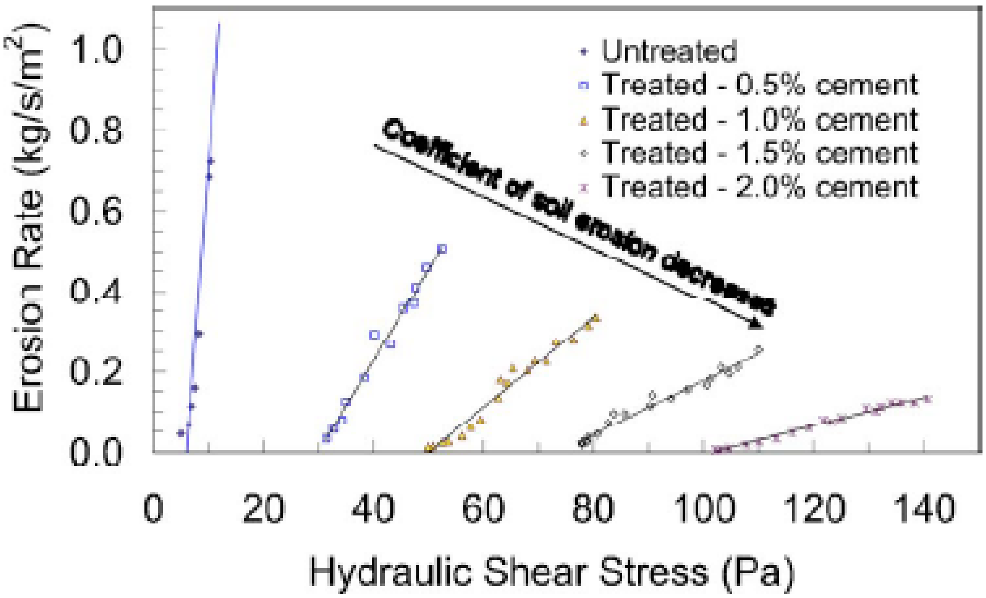


Figure A-31. Relationship between Hydraulic Shear Stress and Erosion Rate (Indraratna 2008)

Fatigue

Fatigue occurs as a result of repeated loads caused by traffic. According to AustROAD (2008), the fatigue resistance of CSM can be reduced by:

- a decrease in the modulus value
- a decrease in the density
- an increase in moisture content
- insufficient mixing.

The fatigue life of CSL typically is related to the stress ratio (applied tensile stress over MOR) or the strain ratio (applied tensile strain over breaking strain) (AustROAD 2008, ARA 2004, Otte 1978, Sobhan and Mashnad 2000, Yeo 2008). When beam tests are used to obtain the fatigue life of CSM, the number of cycles to 50% of the initial modulus value is considered to be the fatigue life (Yeo 2008).

The MOR of CSM is used to predict the fatigue performance of CSM (ARA 2004, Sobhan and Mashnad 2000). The flexural modulus is found not to be a significant variable for the fatigue model, whereas the breaking strain is a significant variable. The strain ratio, or strain level in the case of relatively constant breaking strain, is more applicable to different materials (Carteret 2009). Figure A-32 shows the relationship between fatigue life and strain level.

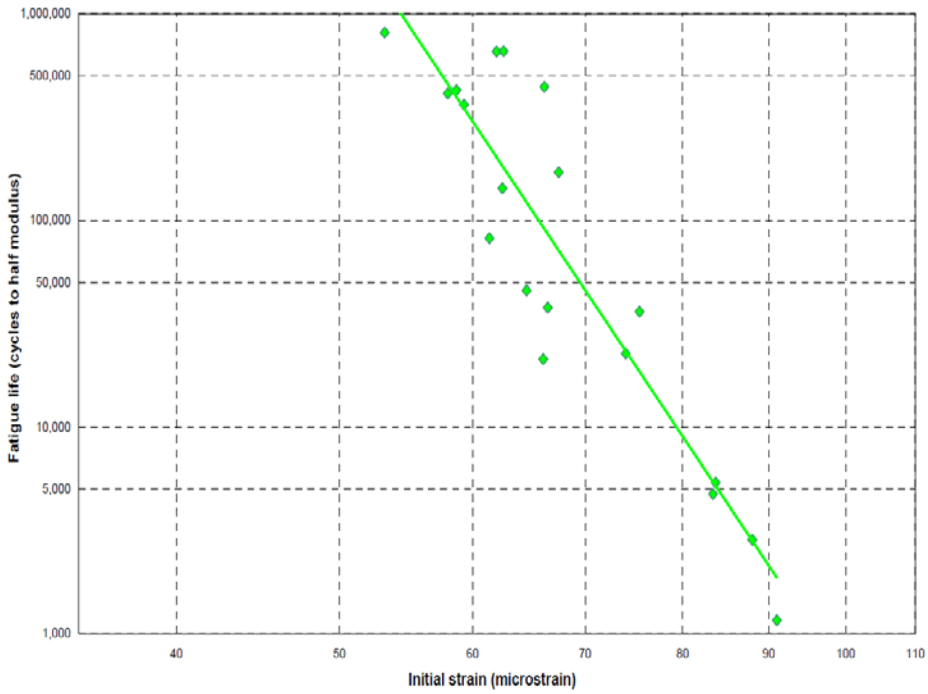


Figure A-32. Relationship between Fatigue Life and Initial Strain (Carteret 2009)

Shrinkage

Shrinkage cracking occurs as a result of contraction of CSM that are restrained by the underlying materials. Shrinkage cracking can reflect through the surface layer and allow the ingress of surface water. The shrinkage cracking of CSL can be reduced by (AustROAD 2008, George 1968):

- the immediate application of a curing coat
- a lower binder content
- the use of slow-setting binder
- a lower fine clay content (<20% 0.003 in fines and PI <20)
- pretreating with lime or lime+cement
- adding gravel or rock.

Queensland in Australia uses the following specifications to reduce shrinkage (Scullion et al. 2005):

- Linear shrinkage of raw soil passing the #40 sieve: 2.5% maximum
- Plasticity index: 4% maximum
- Introduction of a fly-ash blend cement
- Percentage of fines passing #200 sieve: 7.0% maximum

- Linear shrinkage of cement-treated base material should not exceed 250 microstrain after 21 days.

Scullion et al. (2005) report that typical shrinkage cracking spacing is between 3 ft. and 60 ft. The crack width is affected by temperature. Little (1999) found that in summer, the backcalculated modulus value of CSL with a crack width less than 0.1 inch is about half of the backcalculated modulus value of CSL without cracks. However, in winter, the modulus value at a cracked area is one-third of the modulus value at an area without cracks.

CHAPTER A-4. TEST PROCEDURES FOR CSM PROPERTIES

The proper characterization of the properties of CSM is important for selecting appropriate CSM and predicting the field performance of CSL. Different test procedures are available to measure these properties and need to be evaluated for selection.

Strength Tests

The strength values of CSM can be obtained through UCS, IDT strength, MOR, direct tensile or direct shear tests.

Unconfined Compressive Test

The UCS test is the most common test performed on CSM to determine the suitability of the mixtures for uses such as in pavement bases and subbases, stabilized subgrades, and structural fills. Most state highway agencies use the UCS test for their mix designs and for quality assurance (QA) and quality control (QC) because of the simplicity of the test. The UCS test is well-established and meets all cost, practicality and availability requirements (Yeo et al. 2002). Tensile strength can be estimated conservatively as 10% of the UCS, and the MOR can be estimated conservatively to be twice the tensile strength or approximately 20% of the UCS (Little 1999). Several correlations have been developed between UCS and the resilient modulus (Camargo et al. 2009, Thompson 1970, Dallas et al.). Typically, AASHTO T22, ASTM D1633, AASHTO T220, and ASTM C593 are used to test lean concrete, soil cement, lime-stabilized materials, and fly ash-stabilized materials with or without lime, respectively.

The UCS value is the peak load divided by the cross-section of a cylindrical specimen, except for soil-lime for which the enlargement of the cross-section during loading is taken into account. The typical height-to-diameter ratio of a UCS specimen is two. Otherwise, a correction factor must be used to obtain a meaningful strength. Figure A-33 presents the UCS test setup.

- For lean concrete, the loading rate in AASHTO T22 is displacement-controlled, which corresponds to 35+/-7 psi per second. The ratio of height to diameter is 1.75 or higher. Otherwise, a correction factor should be used.
- For soil-cement, the specimen size is either 4 inches in diameter by 4.6 inches in height or 2.8 inches in diameter and 5.6 inches in height according to ASDTM D1633. For the specimen with a 4-inch diameter and 4.6-inch height, a correction factor of 0.909 is used to convert the strength to that of a specimen with a height-to-diameter ratio of 2.0. The preparation of soil-cement specimens is based on ASTM D1632. Capping is required for soil-cement specimens. The test is conducted using either displacement-controlled, 0.05 in./min, or stress-controlled, 20 psi per second. Prior to the testing, the specimen is subjected to 4-hour soaking.

- For soil-lime, AASHTO T220 specifies a specimen size of 6 inches in diameter and 8 inches in height. The loading rate is 0.13-0.15 in./min. Prior to compression testing, the specimen is subjected to 7-day curing at room temperature, 6-hour drying at 140°F, and 10-day capillarity with confining pressure. The UCS is determined as follows:

$$UCS = \frac{Pd}{Ah} \quad \text{Eq. (A-17)}$$

where

P = total vertical load
d = deformation after testing
A = cross-section of specimen before testing
H = height of specimen before testing.

- For high calcium fly ash, such as Class C fly ash, or low calcium fly ash and lime-stabilized materials, ASTM C593 recommends the use of ASTM C39/C39M, *Test Method for Compressive Strength of Cylindrical Concrete Specimens*, which is equivalent to AASHTO T22. Prior to the unconfined compression test, the specimen is subjected to 7-day curing at 100°F and 4-hour soaking. The specimen geometry is 4 inches in diameter and 4.6 inches in height.

Figure A-33. Unconfined Compressive Strength Test Setup

IDT Strength Test

The IDT strength test involves loading a cylindrical specimen with compressive loads distributed along two axial lines that are diametrically opposite, as shown in Figure A-34. This loading setup results in a relatively uniform tensile stress perpendicular to and along the diametric plane. Failure occurs by splitting along this loaded plane.

The IDT strength can be calculated as follows:

$$S(t) = \frac{2P}{\pi t d} \qquad \text{Eq. (A-18)}$$

where

S(t) = IDT strength
P = peak load
t = height of specimen
d = diameter of specimen.

From a review of the literature concerned with the evaluation and use of IDT strength, a number of advantages and disadvantages are found. The six major advantages attributed to the test are as follows (Hudson and Kennedy 1968):

- It is relatively simple.
- The type of specimen and equipment are the same as that used for compression testing.
- Failure is not seriously affected by sample end conditions.
- Failure is initiated in a region of relatively uniform tensile stress.
- The coefficient of variation (COV) of the test results is low.
- Mohr's theory is a satisfactory means of expressing failure conditions for brittle crystalline materials.

In addition, according to Yeo et al. (2002), the IDT strength test is practical, productive, economic, and user-friendly. Besides strength, the IDT strength test can also be used to measure the modulus, Poisson's ratio, fatigue, and permanent deformation (Gnanendran and Piratheepan 2008). The IDT test is also suitable for lightly stabilized materials (Yeo 2002).

The IDT strength values for concrete can be obtained using AASHTO T198, *Splitting Tensile Strength of Cylindrical Concrete Specimens*, or ASTM C496, *Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens*. The specimen geometry is 6 inches in diameter and 12 inches in height. The loading rate of the splitting tensile stress is 100-200 psi/min. The size and loading rate may not be appropriate, however, for soil-lime or soil-fly ash/lime, which is relatively weak material. The Australian method (Midgley and Yeo 2008) to test for IDT strength of stabilized materials uses specimens either 4 inches or 6 inches in diameter, which is dependent on the maximum particle size. The loading rate is 4500 ±450 lbs/min. For CSM, neither composition nor width of the loading strip make any difference to the test results (Hudson and Kennedy 1968).

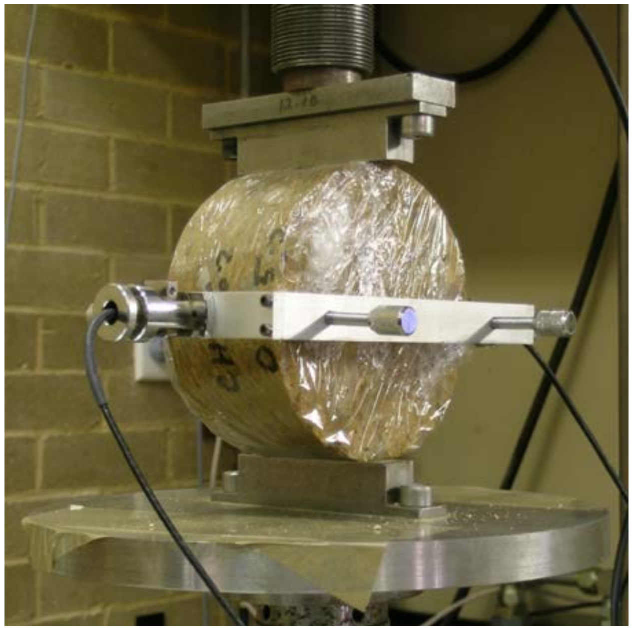


Figure A-34. IDT Test Setup (Yeo 2008)

Modulus of Rupture Test

The MOR is a direct measurement of the resistance of CSL to bending and cracking. In the MOR test, a beam specimen is subjected to a bending load at a constant stress rate until failure, as shown in Figure A-35. For Level 1 input in the MEPDG, the MOR from AASHTO T97, *Standard Method of Test for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)*, or ASTM D1635, *Standard Test Method for Flexural Strength of Soil-Cement Using Simple Beam with Third-Point Loading*, is required for heavily stabilized materials for flexible pavement design (ARA 2004). For Level 2, MOR can be estimated from the UCS. The MOR from a third-point beam test can be determined using the following formula:

$$MOR = \frac{3FL}{2bd^2} \quad \text{Eq. (A-19)}$$

where

F = the load (force) at the fracture point
L = the length of the support span
b = width
d = thickness.

The beam test is good for high-stiffness CSM. However, the moderate and low-stiffness specimens may break during handling. In addition, the beam sample is difficult to prepare (Yeo et al. 2002). In early Australian tests (Midgley and Yeo 2008), the flexural beam test was not planned to be used due to the perceived difficulty of extracting field beam samples from the road

bed. However, after some trials and improvements, a routine procedure was developed that enables beam samples to be readily obtained. For the MOR test, the sample is loaded with a seating force of 11 lbs for the first 6 seconds, after which the load is increased at a rate of 742 lbs/min until the sample fails, as described in Australian Standard AS1012.11–2000. The results from the MOR tests tend to overestimate the tensile strength of concrete by 50% to 100%, mainly because the flexure formula assumes a linear stress-strain relationship throughout the cross-section of the beam. Additionally, in direct tension tests the entire volume of the specimen is under applied stress, whereas in the flexure test only a small volume of the material near the bottom of the specimen is subjected to high stress levels. With low-strength concrete, the MOR can be as high as twice the strength in direct tension (Kumar et al. 2006). Nevertheless, the test is favored by many engineers because the loading conditions are similar to those in the field for pavement materials (Hudson and Kennedy 1968).

Figure A-35. MOR Test Setup (Yeo 2008)

Direct Tensile Strength Test

Direct tensile strength is used to characterize CSM (Raad 1977, Otte 1978). Direct tensile strength tests subject a specimen to a tensile force with constant movement or load rate. The direct tensile test simulates the field conditions when the material is subjected to pure tension, (Hudson and Kennedy 1968), such as the case of thermal contraction or drying shrinkage in the field. Direct tensile tests can be conducted in accordance with ASTM D2936–08, *Standard Test Method for Direct Tensile Strength of Intact Rock Core Specimens*. However, the test has many limitations, including a long preparation time (about 3 hours), misalignment for pure tension, and failure outside the central portion (Yeo et al. 2002). In addition, the method that is used to grip the specimen is also challenging (Gnanendran and Piratheepan 2008). All these factors contribute to lack of practicality and repeatability. Chakrabarti and Kodikara (2008) modified the

tensile strength test setup (Figure A-36). However, this configuration still is subjected to bending stress.

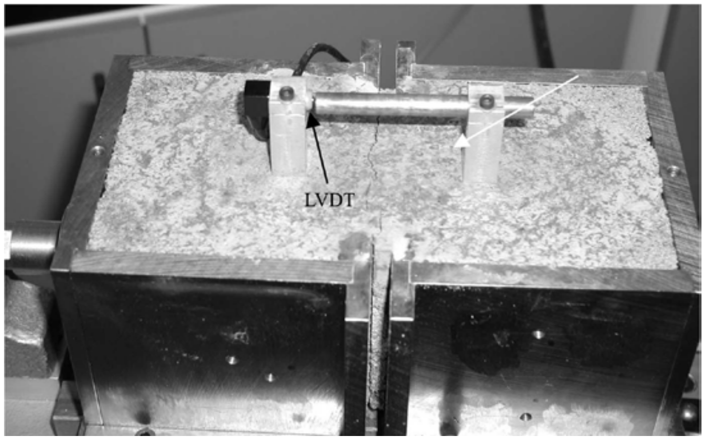


Figure A-36. Modified Tensile Strength Test (Chakrabarti and Kodikara 2007)

Modulus Tests

Currently, the MEPDG requires the resilient modulus for lightly stabilized materials (e.g., materials stabilized with fly ash or lime) and the modulus of elasticity for heavily stabilized materials, (e.g., lean concrete, soil-cement). For Level 2 input in the MEPDG, the resilient modulus and modulus of elasticity can be predicted from the UCS. Both the resilient modulus and modulus of elasticity are used in pavement analysis for stress/strain determination. The resilient modulus of lightly stabilized materials has some dependence on stress levels and is sensitive to the test procedure.

Resilient Modulus Test

The resilient modulus (M_r) is the ratio of applied cyclic deviator stress to recoverable strain. Laboratory resilient modulus testing of unbound aggregate bases or subgrade soils is performed by methods such as found in NCHRP 1-28A, AASHTO T-307, or the LTPP test protocol P46. The current MEPDG recommends the use of AASHTO T-307. The resilient modulus test is conducted in a load frame fitted with a hydraulic load actuator with a servo-hydraulic control system capable of delivering rapidly varying loads, as shown in Figure A-37.

$$M_r = \frac{\sigma_{dN}}{\varepsilon_r} \quad \text{Eq. (A-20)}$$

where

σ_{dN} = the cyclic deviator stress of the *N*th load pulse

ε_r = the resilient deformation from σ_{dN}.

The resilient modulus test simulates field conditions, such as stress level and traffic loading (Little and Nair 2007). However, the resilient modulus test setup is very time-consuming and tedious in nature. The mounting of sensors is difficult, and the noise level in the system makes it hard to run on a routine basis (Hilbrich and Scullion 2007). Also, the test equipment is expensive, and the test procedures are very difficult to run. The existing test procedures were developed for granular or low-stiffness materials. When testing stiff materials, it is critical to cap the samples. Even with very good control, problems are likely to be encountered with the displacements in the LVDTs (Hilbrich and Scullion 2007). The COV is reported to be 10% or higher (Scullion et al. 2008).

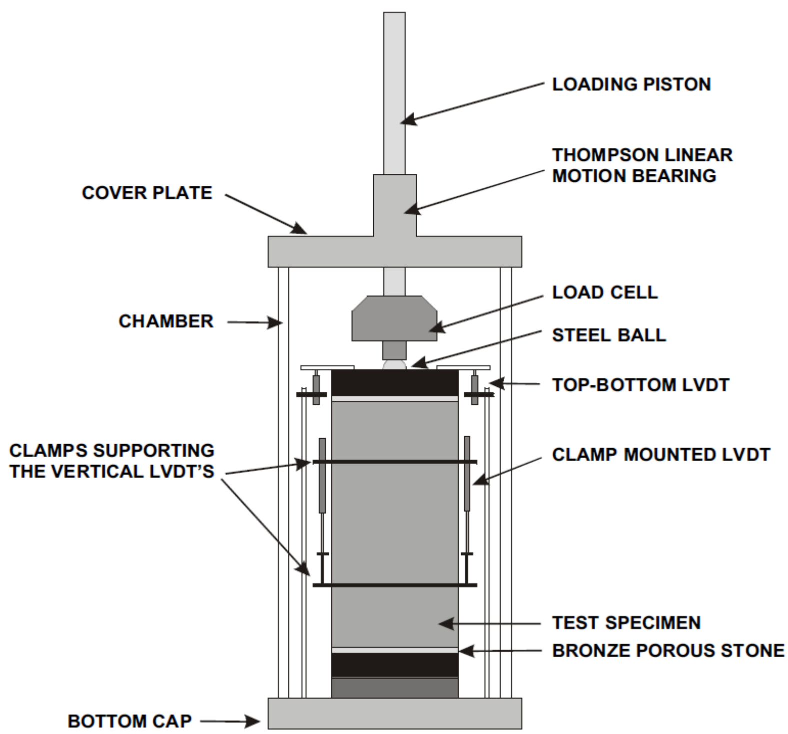


Figure A-37. Resilient Modulus Test Setup (AASHTO T307)

Modulus of Elasticity Test

For heavily stabilized materials, the modulus of elasticity (based on ASTM C 469) is specified in the MEPDG. The ASTM method consists of failing a cylindrical specimen using a constant stress rate, as shown in Figure A-38. The test method provides a stress-to-strain ratio and a lateral-to-longitudinal strain ratio. Because the test protocol is designed for concrete or heavily stabilized materials, some development is needed for lightly stabilized materials. Richardson (1996) conducted the modulus of elasticity test on soil-cement and reported the COV as 16.3 percent.

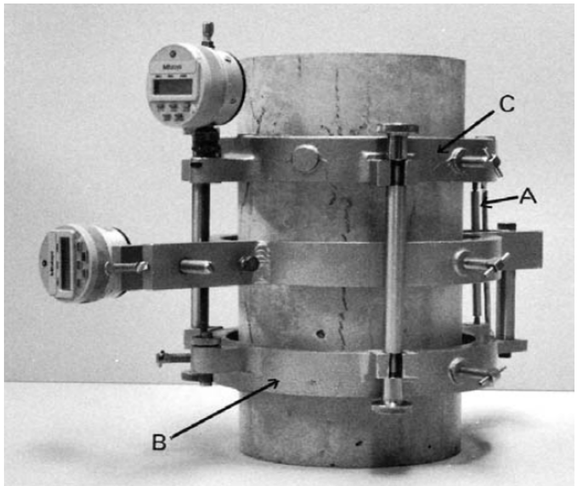


Figure A-38. Modulus of Elasticity Test Setup (ASTM C469)

Flexural Stiffness Test

Flexural stiffness is obtained by applying cyclic loads to a beam specimen without damaging the specimen. The flexural stiffness can be calculated as follows:

$$E = \frac{FL^3}{2wh^3d} \quad \text{Eq. (A-21)}$$

where

F = applied force
L = support span of beam specimen
w = width of beam specimen
h = height of beam specimen
d = deflection of specimen at mid-span.

Flexural stiffness is used extensively in Australia for CSM (Yeo 2008). It simulates the field bending of CSL (AustROAD 2008, Yeo 2008) and is suitable for high-stiffness CSM. However, the beam sample is difficult to prepare and handle, especially for lightly stabilized materials (Yeo 2002, Gnanendran and Piratheepan 2008). The results of flexural stiffness tests also show some variation with a COV typically below 20 percent (Yeo 2008, Carteret 2009).

IDT Modulus Test

The IDT modulus test involves placing a cylindrical sample horizontally along its axis with compression loading applied vertically across the diameter of the specimen. Induced tensile strains are measured horizontally across the diameter. The IDT modulus can be determined based on the following equation (Midgley and Yeo 2008):

$$E = P \times \frac{(\vartheta + 0.27)}{H \times h_c} \qquad \text{Eq. (A-22)}$$

where

P = peak load
ϑ= Poisson's ratio
H = recovered horizontal deformation of specimen after application of load
h_c = height of specimen.

The IDT modulus value is higher than the beam modulus value (Yeo 2008). Yeo et al. (2002) report that the Australian IDT test is suitable for determining the strength and stiffness of stabilized materials. Cylindrical test specimens are considered to be the most practical both in terms of laboratory compaction and field sampling via coring techniques. In addition, this test is practical, cost-effective, has good equipment availability, requires basic sample preparation and provides for the preferred tensile mode of failure for both the strength and fatigue tests. Currently, there is no test standard to determine the IDT modulus of CSM. The Australian test method for the IDT modulus of CSM has been evaluated (Midgley and Yeo 2008). To determine the specimen IDT modulus, 50% or less of the ultimate failure load determined using the IDT strength test is used such that the material remains within its elastic range. The displacement is measured by LVDTs.

The COV of the IDT modulus is reported to be less than 20 percent (Carteret 2009). The location of the LVDTs is found to affect the test results significantly. By using internal LVDTs, the COV can be reduced significantly (Gnanendran and Piratheepan 2008).

Seismic Modulus Test

The seismic modulus (or vibration) test (Tex-147-E or ASTM C 215) is a simple, nondestructive laboratory test for determining the moduli of pavement materials. The modulus measured using this method is the low-strain seismic modulus (SM). This method was developed originally for testing PCC specimens. With appropriate modifications in the hardware and software, it is now also applicable to specimens composed of stabilized base and subgrade materials. The procedure used in the seismic modulus test involves finding the Young's modulus by measuring the velocity of an elastic wave that propagates through a cylindrical specimen. Figure A-39 presents the test setup.

The seismic modulus test shows potential because it is relatively inexpensive and rapid. However, if the design program requires the use of resilient modulus values, then a modulus correction factor must be developed for unbound materials. It is reported that the seismic modulus test provides more repeatable data for estimating the resilient modulus of stabilized materials (Whitehurst and Yoder 1952, Si 2008). The seismic modulus is found to correlate with the resilient modulus of stabilized materials, and a COV of 7% is reported (Scullion et al. 2008).

Table A-3 provides a comparison of the seismic modulus and resilient modulus test procedures and equipment. The seismic modulus values are reported to be about 1.7 times those obtained from the FWD (Geiger et al. 2007).

Table A-3. Comparison of Resilient Modulus and Seismic Modulus Test Methods and Equipment (Hilbrich and Scullion 2007)

	Seismic Modulus Test	Resilient Modulus Test
Equipment cost	$4,000	$350,000
Test set-up time	2 min	1 h
Testing time	3 min	5 min*
Plaster capping required	No	Yes

*Although the source indicates 5 min, the M_r test typically takes 3-4 hours.

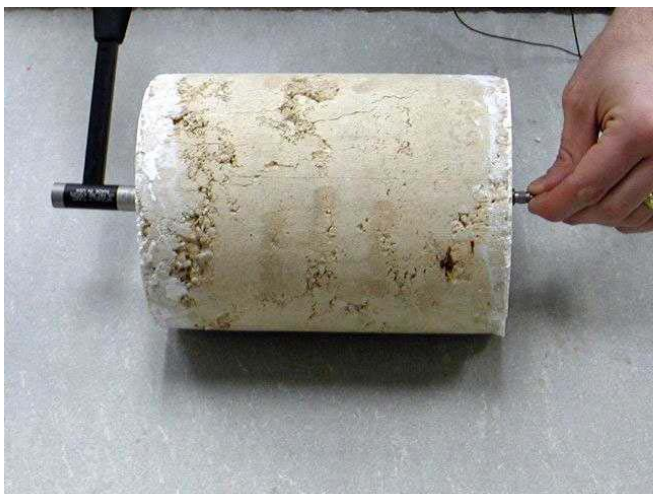


Figure A-39. Seismic Modulus Test Setup (Guthrie et al. 2001)

Ultrasonic Pulse Velocity Test

The ultrasonic pulse velocity test (ASTM C597), also called the acoustic modulus test, is a nondestructive testing technique that transmits sound waves ranging in frequency from 20 kHz to 1 GHz through the specimen. By measuring the travel time through the specimen, the p-wave or shear wave velocity and related dynamic properties of the material are determined. Since 1970s, ultrasonic pulse velocity has been developed and widely used as a simple and quick measurement for quality control and defect detection in civil infrastructure. The method is simple and quick and attractive for application to soils. Figure A-40 shows a schematic view of the pulse velocity test apparatus (ASTM C597).

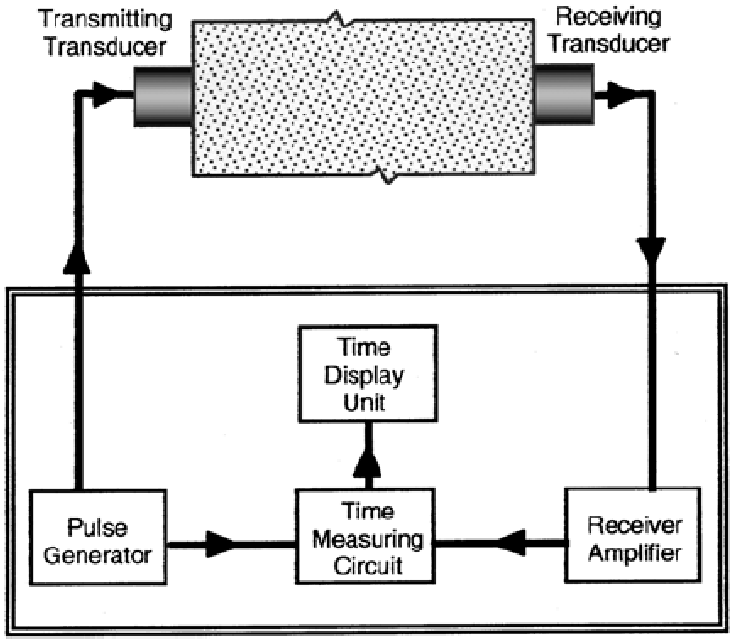


Figure A-40. Schematic of Pulse Velocity Test Apparatus (ASTM C597)

Yesiller et al. (2001) evaluated the feasibility of applying ultrasonic pulse velocity testing to stabilized materials. Their p-wave measurement system consisted of a 50-kHz p-wave transducer, receiver, and data acquisition system. They calculated the first arrival time as the difference between the time of application of the pulse by the transmitting transducer and the arrival time of the signal at the receiver. These researchers conducted tests on lime, cement, and fly ash-stabilized clays with high plasticity. They applied the ultrasonic pulse velocity test method to monitor the strength growth of the specimens in terms of curing time. In general, Yesiller et al. found that velocity increases with curing time, and the p-wave increases more in the first seven days than during the rest of the curing period. Cement mixtures show larger increasing rates of p-waves and higher velocities during curing. Additionally, Yesiller et al. investigated the effect of compaction moisture content on p-wave velocity. They measured the maximum p-wave velocity for specimens compacted at the optimum moisture content. For clay-cement and compacted clay specimens, the variation in p-wave velocities with compaction moisture content followed the same trend as the variation in dry density with compaction moisture content. However, this trend was not observed in the fly ash-stabilized clay. Good correlations were observed between velocity and strength for fly ash-stabilized soil. The p-wave velocity increased with the increasing modulus value of the mixture. Yesiller et al. (2001) conclude that p-wave velocity correlates directly with the stiffness of the stabilized mixtures, but not with the strength.

Durability Tests

Wetting and Drying Test

Among the existing laboratory procedures, AASHTO 135, *Standard Method of Test for Wetting-and-Drying Test of Compacted Soil-Cement Mixtures*, is the test protocol used to assess the wet and dry durability of soil cement. This test procedure consists of placing a soil–cement specimen cured for 7 days in a water bath at room temperature for 5 hours, then placing it in an oven at a temperature of 160°F for 42 hours. The effect of 12 wetting-drying cycles is determined by the degradation of the specimens and their weight loss. It is reported that evaluating durability based on weight loss, however, is overly severe (Khoury and Zaman 2007).

The quality of the results produced by this standard is dependent on the competence of the personnel performing it and the suitability of the equipment and facilities used (ASTM D559-03). The brushing is subjective and not repeatable due to the susceptibility of the brushing technique to operator variability and loss of single large aggregate particles (Ventura 2003, Scullion et al. 2005). In addition, the 12 cycles of wetting and drying takes more than one month to conduct, which is impractical compared to other methods. It is known that wetting and drying test results may also reflect the erodibility performance of CSL.

Freezing and Thawing Test

The freeze-thaw cycling procedures outlined in AASHTO T136, *Freezing-and-Thawing Tests of Compacted Soil-Cement*, or ASTM D560, *Standard Test Methods for Freezing and Thawing Compacted Soil-Cement Mixtures*, can be used to evaluate the durability of soil-cement. This protocol involves subjecting specimens cured for 7 days in a fog room to 12 cycles of freezing and thawing. The procedures require freezing for 24 hours at a temperature not warmer than -10°F and thawing for 23 hours at 70°F and 100% RH. Following the specified thawing period, the specimens are weighed and abraded with two firm strokes on all areas with a wire scratch brush. Although AASHTO T136 and ASTM D560 specify that specimen durability should then be measured in terms of percentage of mass loss, many agencies omit the brushing portion of the test due to the variability associated with the brushing process and replace it with UCS testing after completion of all 12 cycles (Guthrie et al. 2008). In addition to standard test procedures for wetting-drying and freezing-thawing, mechanical tests, such as stiffness or strength tests, are conducted before and after the cyclic action to determine the amount of deterioration in the mechanical properties and stiffness characteristics (Camargo et al. 2009).

Similar to the wetting-drying durability test, the freeze-thaw test is time-consuming and the use of mass loss is subjective.

Moisture Susceptibility Test

In order to assess the moisture susceptibility of granular base materials, the tube suction test is a commonly used test that is performed in accordance with the Texas Department of Transportation Test Method Tex-144-E, as shown in Figure A-41. This test is designed to evaluate the durability and moisture susceptibility of aggregate used as base materials in pavements. The test consists of measuring the surface DV of compacted specimens after 10 days of capillary soaking in the laboratory. The interpretation of the test results is based on an empirical relationship between the measured DVs and expected performance of the aggregate base materials in the field. It is noted that this methodology may be a much quicker and more cost-effective means of assessing base durability than the traditional ASTM D 559 and D 560 protocols, which require more than a month to perform. The final averaged DVs are used to rate the moisture susceptibility of the specimens. Materials with final DVs less than 10 are expected to provide good performance, whereas materials with DVs above 16 are expected to provide poor performance as base materials. Aggregate with final DVs between 10 and 16 are expected to be marginally moisture-susceptible (Miller et al. 2006). It is reported that the tube suction method can be linked to the field performance of the durability of granular materials (Scullion et al. 2005). This test also can be used to measure the moisture susceptibility of CSM.

Guthrie et al. (2001) also recommend that the optimum cement contents of aggregate base materials should be determined by the strength test and the tube suction test. According to the authors, sufficient quantities of cement should be added to tested samples to obtain a minimum UCS of 300 psi and maximum average surface DV of 10. The minimum amount of cement that satisfies both criteria should be recommended for construction.

The tube suction test is repeatable at high DVs, but not repeatable below a DV of 10 (Guthrie et al. 2001). The number of replications required in the test depends upon its reliability and tolerance, as well as specified maximum DVs. The test is intended to be used as a screening test. The DV of stabilized soil reduces after long-term curing, indicating that chemical reactions affect the DV (Si and Herrera 2007).

Zhang and Tao (2006) point out that the maximum DV criterion (less than 10 for good base materials) is more conservative than the mass loss criterion specified in wetting-drying durability tests. The DV criterion may be too strict and rejects many quality materials. Also, studies have found that the DV test is extremely sensitive to molding moisture content, and there is inconsistency between freeze-thaw and tube suction test results (Zhang and Tao 2006, Cho et al. 2006).

Figure A-41. Tube Suction Test Setup (Guthrie et al. 2001)

The vacuum saturation test, which is performed based on ASTM C593, *Standard Specification for Fly Ash and Other Pozzolans for Use with Lime,* also provides an evaluation of the durability of CSM. The test is carried out to evaluate the relative performance of cement-, fly ash-, and/or lime-stabilized soils. In this test, the soil samples are compacted at the optimum moisture content using standard Proctor compaction and cured under standard conditions. At the end of the curing period, the soil specimens are placed in a chamber and slowly evacuated over 45 minutes to reach a pressure of 24 inches of Hg. In order to 'de-air', the specimens are left at this pressure for 30 minutes. Upon de-airing, water is introduced into the chamber, the vacuum is released, and the entire sample is soaked for one hour. Subsequently, the UCS is measured (ASTM D2166) after draining the surface water for approximately two minutes (Bhattacharja and Bhatty 2003).

Guthrie et al. (2008) report that the UCS values after freeze-thaw cycles and vacuum saturation are strongly correlated. The UCS values after tube suction testing do not correlate with other UCS values. Vacuum saturation is a more severe process than freeze-thaw cycling. The moisture contents after the freeze-thaw, vacuum saturation, and tube suction tests are 5.4, 7.0, and 5.1, respectively.

Erodibility Tests

Many erosion tests were developed in the 1970s and 1980s using various test devices. However, none of those tests have become standard test methods (Jung et al. 2009).

In South Africa, wheel tracking is used to measure the erosion of CSL (Si and Herrera 2007). The wheel tracking test consists of running a wheel on a saturated beam specimen, as shown in Figure A-42. After 5000 cycles, the erosion/abrasion depth is considered to be the erosion index. Although the wheel tracking test simulates field conditions, it is time-consuming

(8 days) and the equipment is expensive; thus, the test is not suitable for routine testing. Another drawback is that it does not simulate realistic traffic loading, speeds, and consequent dynamic water pressure.

Wet-dry brushing is another available test method that consists of curing, soaking, conditioning, and brushing the specimens, followed by determination of material loss. There is a strong correlation between the tube suction test results and the brush test results when the sample size is 4 in. × 4.6 in. (Scullion et al. 2005). The wet-dry brush test is simple and inexpensive to perform, but difficult to control the force (Ras and Visser 2004). This test shows poor repeatability and reproducibility due to the manual brushing technique used (Gass et al. 1993), operator variability, and loss of single large aggregate particles (Ventura 2003).

The rotational shear device (RSD) involves a sample placed inside a cylinder that is surrounded by water in the annular space, as shown in Figure A-43. The cylinder, encapsulated by top and bottom discs, is then rotated. A 560-W motor generates rotational speeds between 300 and 3000 rpm. The eroded material is weighed after completion of the test. It should be noted that only cohesive materials can be tested by the RSD, and the test takes only water-induced shear stress into account. South African researchers used a RSD test method to simulate the movement of water and the resultant loss of material (Ras and Visser 2004). During the test, relatively large aggregate particles came loose, which may not happen within the pavement. As the erosion index is based on mass loss, an incorrect interpretation of erodibility potential could occur (Gass et al. 1993). The RSD test results correlate with the wet-dry test results. In addition, the RSD test is a method that can measure and control the shear stress induced by moving water.

Jetting is another method that is used to measure the erodibility of CSM. It uses 50 psi and a 20º angle between the water jet and sample surface, as shown in Figure A-44. Jetting is simple to conduct (Ras and Visser 2004). However, shear stress cannot be accurately determined, as the stress distribution is not uniform (De Beer 1994).

A vibrating table also can be used to measure the erodibility of CSM (Howard 1988). The vibration table test is similar to the RSD test, except that the whole cell containing the specimen and water is vibrated vertically to generate erosion. It is easy to operate. However, in the vibrating table method, it is difficult to measure shear stress accurately.

Van Wijk (1985) evaluated the effectiveness of various erosion tests and listed their advantages and disadvantages, as shown in Figure A-45.

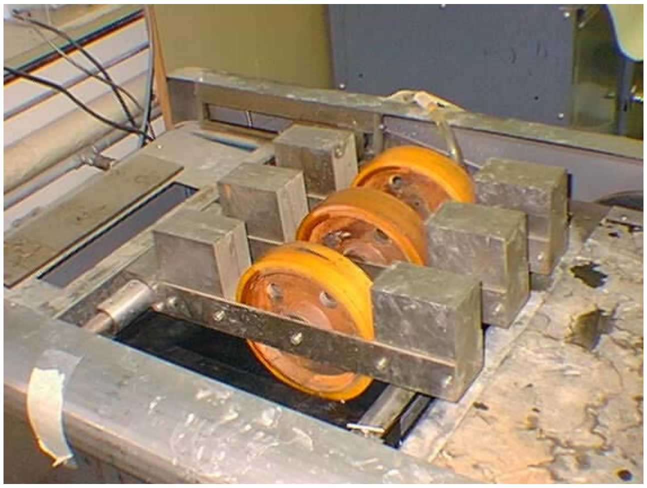


Figure A-42. South Africa Wheel Tracking Test

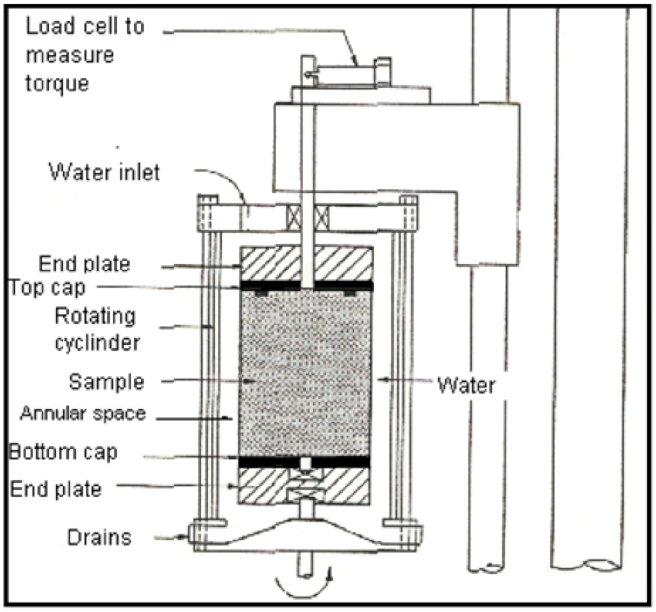


Figure A-43. Rotational Shear Device (Van Wijk 1985)

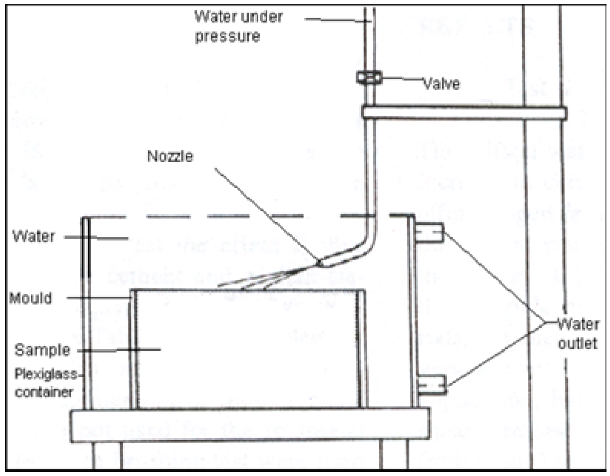


Figure A-44. Jetting Test (Van Wijk 1985)

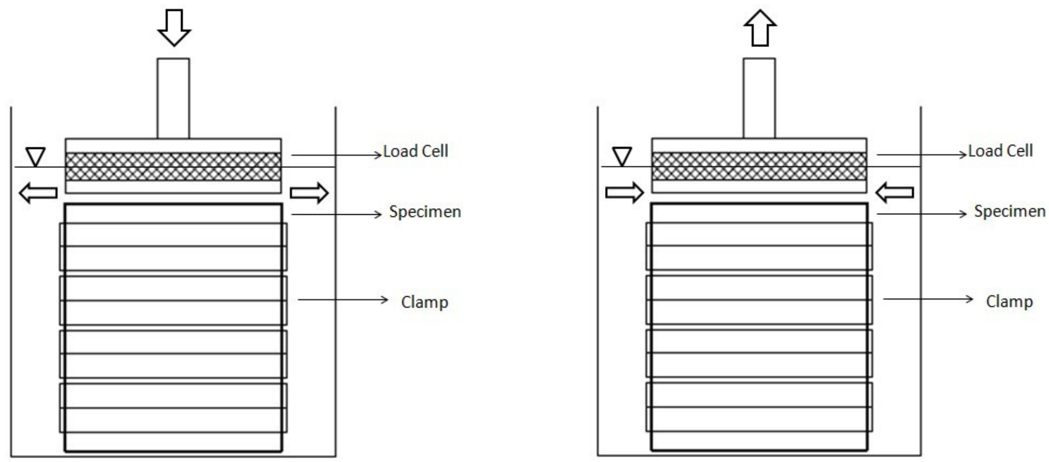
	Required Characteristics											
	1	2	3	4	5	6	7	8	9	10	11	12
1. Full scale		*	*					*	*	*		P/E
2. Scaled	*	*	*					*	*	*		P/E
3. Model	*		*	*		*		*	*	*	*	P/E
4. Brush test			*	*	*	*	*		*	*		E
5. Jetting test		*	*	*	*	*	*		*	*		E
6. Rotational shear		*	*	*	*	*	*		*	*	*	E
7. Vibrating table			*	*	*	*	*		*	*		E
8. Abrasion test			*	*	*	*	*		*	*		E

* : test does satisfy the requirements

1. Effect of the traffic included.
2. Water pressures, velocities, and shear stresses close to the actual.
3. Climate and environment included.
4. Standard size compacted soil samples used.
5. Easy to use.
6. Not expensive to build and operate.
7. Easily repeatable for a large number of samples.
8. Simulate actual conditions closely.
9. Able to test all types of materials.
10. Sensitive to detect differences in materials.
11. The erosion forces and eroded material can be measured accurately.
12. Mechanism of pumping:- E: erosion and P: pumping of fines from within.

Figure A-45. Evaluation of Various Erosion Tests (Van Wijk 1985)

The cyclic impact erosion (CIE) test has been used by other researchers to study the erosion of CSM (Sha and Hu 2002). This test consists of applying a cyclic load through a loading plate on a specimen submerged in water. The loading squeezes the water in and out during loading and unloading. The process creates erosion (Figure A-46) and simulates the erosion under the surface layer in a pavement that results from both the load impact caused by traffic and water movement.



(a) Loading *(b) Unloading*

Figure A-46. Flow of Water during Loading and Unloading Phases

Fatigue Tests

In the MEPDG, the fatigue behavior in the flexible pavement design of CSL is considered only for flexible pavements, not rigid pavements. When different stress levels are applied in the fatigue tests, a relationship between fatigue life and stress ratio can be developed for the fatigue model development.

Flexural Fatigue Test

The Australian flexural beam fatigue test method and AASHTO T321, *Determining the Fatigue Life of Compacted Hot-Mix Asphalt (HMA) Subjected to Repeated Flexural Bending*, are two available tests methods. Sobhan and Das (2007) used prismatic beam specimens with dimensions of 30 in. × 6 in. × 6 in. for the flexural fatigue tests. Beams were prepared by compacting the mix in detachable steel molds in three equal layers at a target unit weight. Each layer was scarified for proper bonding with the next layer. The beams were sealed and cured for 24 hours in the laboratory environment, and then were de-molded and carried to the 100% RH room for curing. The average curing period was 28 days. Beams were tested in bending under a third-point loading configuration over a span length of 27 inches. In a typical fatigue experiment, the repeated load is usually expressed in terms of a stress ratio, defined as applied tensile stress at the bottom of a beam over the MOR.

Large-Scale Model Experiment

Fatigue testing also can be conducted using the Large-Scale Model Experiment (LSME). The LSME is a prototype pavement structure testing system, which is constructed with the same materials employed in the field or in small-scale specimen tests (Kootstra et al. 2010, Benson et al. 2009). The pavement structure is instrumented to monitor stress and deformations, and is loaded cyclically using a plate load that simulates the wheel load. The boundary conditions and the applied cyclic loading are chosen carefully to replicate field conditions. The material being evaluated (in this case, CSL) is placed and tested (in different thicknesses if desired) covering the likely ranges that would be expected in actual field applications, or under the same conditions applied in a field test. The LSME requires large-scale mixing operations similar to those encountered in field construction and thus provides a realistic simulation of the fatigue of CSL. Figure A-47 shows the LSME setup.

Figure A-47. LSME Setup

Shrinkage Tests

Free Drying Shrinkage Test

AASHTO T160, *Length Change of Hardened Hydraulic Cement*, can be modified to characterize the drying shrinkage of CSM. Different RH values can be used to obtain the drying shrinkage strain. Long-term curing also is needed to measure the ultimate drying shrinkage strain.

Tex-107-E also allows users to determine the bar linear shrinkage of soils and is used for CSM (Si 2008, Guthrie et al. 2001).

Chakrabarti and Kodikara (2003) conducted drying shrinkage tests according to methods found in AS1012.13-1992. A known quantity of stabilized mix according to standard maximum dry unit weight was compacted in two layers into a rectangular steel mold measuring 3 in. × 3 in. × 11 in (l_o). A standard Proctor hammer with a maximum cross-sectional dimension of 2 inches (2.6 in., including the guide) was used to compact the material. Two gauge studs were placed at the middle of the end sections during compaction to facilitate shrinkage measurements. Specimens in duplicate per set were cured for 24 hours at 90% RH or above and at an air temperature of 70°F to 75°F. Subsequently, the specimens were dried in a controlled environment with 50% RH and at an air temperature of 71.6°F. Specimen lengths, l_i (in.), were recorded at gradually increasing intervals for up to 90 days while the specimens were kept in a controlled environment of 71.6°F and 50% RH. Shrinkage (ε_{sh}) at any time in microstrain can be determined as follows:

$$\varepsilon_{sh} = (l_o - l_i) \times 10^6 / l_o \qquad \text{Eq. (A-23)}$$

Figure A-48 shows a free drying shrinkage test setup. The unrestrained or free shrinkage represents the full shrinkage potential of a geomaterial under a given environmental condition. In free drying shrinkage, no shrinkage stress is generated.

Figure A-48. Free Drying Shrinkage Test

Restrained Shrinkage Test

Tensile stress is generated when the changes of moisture or temperature cause a CSL to contract, and the underlying friction or self-restraint keeps the base from contracting. Shrinkage cracking occurs when the tensile stress exceeds the tensile strength of the cement-treated pavement layer (Cauley and Kennedy 1972). Because the restraint is in place either externally by the subgrade/surface layer or internally by differential shrinkage within different depths of the

CSL due to the presence of humidity and temperature gradients in actual field conditions, the shrinkage cracking potential of each material in the CSL can be examined by restrained shrinkage testing.

In the concrete shrinkage cracking test, a ring of concrete is fabricated outside a steel ring. The internal steel ring hinders free drying and generates cracks. The onset of cracking and the development of the crack width are measured (Kodikara and Chakrabarti 2001).

The PCA recommends a drying shrinkage test on soil-cement beams restrained with a rough surface at their base (George 2002). This recommended procedure can be modified as a restrained shrinkage test for CSL. However, the bond condition recommended by the PCA, i.e., CSL on sand, does not generate shrinkage cracking within this 12-inch long beam. A typical shrinkage cracking spacing in the field is between 3 ft. and 60 ft. To generate shrinkage cracking in a laboratory-scale specimen, a strong bond must be created between the specimen and base. In addition, the length of the specimen must be increased.

Weiss et al. (1998) conducted restrained concrete slab tests to generate shrinkage cracking in a 39-inch long slab. The slab was fixed to the base, or to both ends and base, as shown in Figure A-49. By restraining the movement of the slab, the shrinkage cracking spacing is significantly shorter than the field cracking spacing. The concrete slab was bonded to a steel tube to prevent curling.

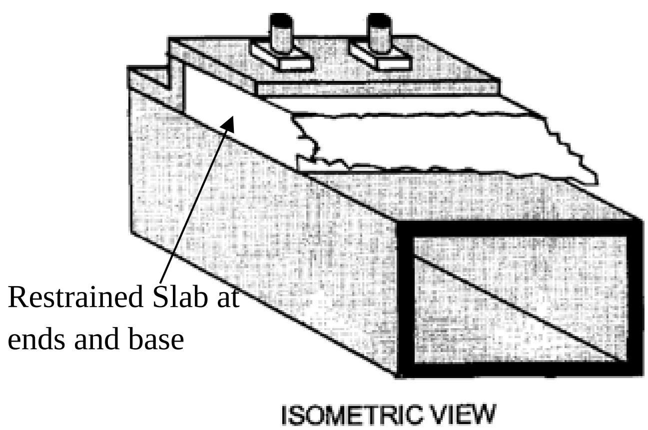


Figure A-49. Restrained Slab at Base and Ends (Weiss et al. 1998)

Thermal Shrinkage Test

The coefficient of thermal expansion (COTE) is important for thermal strain determination, which is a key parameter in the shrinkage cracking model. AASHTO TP60, *Standard Method of Test for Coefficient of Thermal Expansion of Hydraulic Cement Concrete,* is a test method to determine the COTE of concrete. This method determines the COTE of a cylindrical concrete specimen, maintained in a saturated condition, by measuring the length change of the specimen due to a specified temperature change, as shown in Figure A-50. The

measured length change is corrected for any change in length of the measuring apparatus (previously determined), and the COTE is then calculated by dividing the corrected strain by the temperature change between 50°F and 122°F.

Cusson and Hoogeveen (2006) also recommend a test method for measuring the COTE, especially for early-age concrete. In their test, a concrete beam specimen is subjected to temperature cycles between 77°F and 86°F, including a 15-min temperature ramp followed by a 4-hour constant temperature. The cyclic strain, which does not include the permanent autogenous shrinkage strain, is used to determine the COTE.

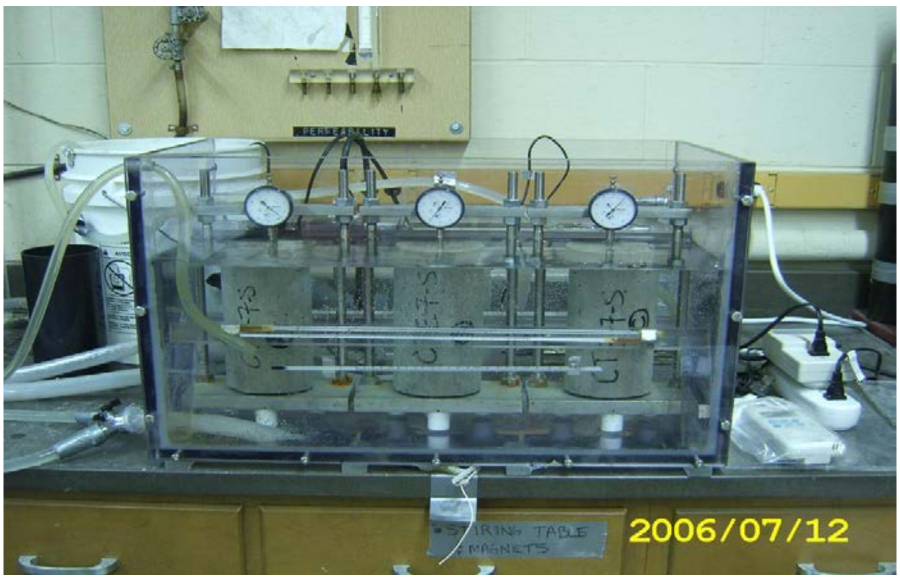


Figure A-50. Test Setup of Coefficient of Thermal Expansion (Naik et al. 2006)

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FINDINGS OF SURVEY AND REVIEW OF SPECIFICATIONS

1. **Survey Results**

2. **Summary of State Agencies' Specifications**

NCHRP Project 04-36
Characterization of Cementitiously Stabilized Layers for Use in Pavement Design and Analysis

Results from Survey of State Highway Agencies

The research team distributed questionnaires to collect information about cementitiously stabilized layers from state highway agencies. In total, the team received 40 responses from state DOTs. The goal of the survey was to collect feedback from state DOTs regarding the characterization of cementitiously stabilized layers in pavements. The survey covers specifications for types of cementitious additives, the extent of their use, engineering parameters, pavement performance, quality control methods, and construction practices.

Statistics of the Results from Questionnaire

The following 8 questions were listed in the questionnaire. The number of occurrences per answer relative to the total responses is shown in parentheses or in graphs.

1) Does your agency use cementitiously stabilized materials for subgrade, subbase or base of pavement construction?

[**28 of 40-**out of 40 responses, 28 states use it] Yes; if so, what is the extent of use?
Subgrade modification: (**10 of 28**) Little; (**6 of 28**) Moderate; (**7 of 28**) Extensive
Subbase modification: (**12 of 28**) Little; (**2 of 28**) Moderate; (**3 of 28**) Extensive
Base modification: (**9 of 28**) Little; (**9 of 28**) Moderate; (**5 of 28**) Extensive

[**12 of 40**] No; if so, please provide details for non-usage (please use additional page as needed and there is no need for providing input for the items below):
Wyoming chooses "No" for Q1, but it still answers the other questions.

Summary: Most of the agencies (28 of 40) use cementitiously stabilized materials. Little subgrade and subbase modifications are used, and most of stabilization focuses on base.

2) If Yes, what type of cementitious additives are used:

	Clay	Silt	Sand	Gravel	RAP	Expansive Soil
Cement	**9/28**	**19/28**	**17/28**	**14/28**	**5/28**	**4/28**
Lime	**18/28**	**4/28**		**1/28**	**2/28**	**11/28**
Class C fly ash	**3/28**	**7/28**	**5/28**	**4/28**	**5/28**	**2/28**
Lime+Class F fly ash	**2/28**	**2/28**	**1/28**	**1/28**	**1/28**	**1/28**
Lime+cement	**3/28**	**1/28**	**1/28**	**1/28**		**2/28**
Cement kiln dust	**2/28**	**2/28**	**1/28**	**2/28**	**1/28**	

Note: ____

Summary: Cement and lime are the two major additives. Cement is usually used to stabilize cohesionless soils, such as silt (19/28), sand (17/28), and gravel (14/28). Lime is applied to cohesive soils, such as clay and expansive soil. Other stabilizers include class C fly ash, a mixture of lime and class F fly ash, and cement kiln dust.

3) How are the cementitiously stabilized materials incorporated in your agency's pavement design?

[**18/27**] Layer Coefficient
[**8/27**] Modulus
[**8/27**] Others ____

Summary: Most of the agencies use a layer coefficient (18/27) to incorporate cementitiously stabilized materials in pavement design, followed by the modulus (8/27). In addition, there are some other situations, a) Gravel Equivalent; b) when allowed; Mr (Mr<3000psi); c) no contribution; and d) soil support value or K value.

4) In your opinion, what are the important engineering parameters for cementitiously stabilized materials that should be considered? Please place a rank order of importance with 1 being the most important.

[ ] Strength; Test Method: ______________________________
[ ] Stiffness; Test Method: ______________________________
[ ] Durability; Test Method: ______________________________
[ ] Erodibility; Test Method: ______________________________
[ ] Swelling; Test Method: ______________________________
[ ] Shrinkage; Test Method: ______________________________
[ ] Fatigue; Test Method: ______________________________
[ ] Others: ______________________________

Summary: 28 states responded to this question. Strength (17 of rank 1), durability (8 of rank 2; 7 of rank 3) and stiffness (8 of rank 2; 4 of rank 3) are the three most significant engineering

parameters for cementitiously stabilized materials. The agencies also consider erodibility, swelling, shrinkage, fatigue, and some other engineering parameters.

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5) List the key performance issues for pavements with cementitiously stabilized layers based on your experience and knowledge? Please place a rank order of importance with 1 being the most important for each type of pavement.

Asphalt Pavement:

[ ] Transverse Cracking; Reasons: ______

[ ] Longitudinal Cracking in Wheelpatch; Reasons: ______

[ ] Longitudinal Cracking Outside Wheel path; Reasons: ______

[ ] Alligator Cracking; Reasons: ______

[ ] Block Cracking; Reasons: ______

[ ] Others; Reasons: ______

Summary: For asphalt pavement, 23 states responded to this question. Transverse cracking receives the most attention (13 of rank 1), followed by alligator cracking, longitudinal cracking in the wheel path, block cracking, and longitudinal cracking outside the wheel path.

Concrete Pavement:

[ ] Transverse Cracking; Reasons: ______

[ ] Others; Reasons: ______

Summary: For concrete pavements, 12 states answered this question. As for asphalt pavement, transverse cracking is also the most crucial pavement issue. In addition, there are some other issues reported: faulting due to CTB erosion, large-area block cracking, improvements needed in uniformity, freeze and thaw and load transfer, joint spalling, base failure, and longitudinal cracking.

6) What is the field quality control method adopted by your agency? Check all apply.

[**27/27**] Density; Method & Specifications: ______

[**13/27**] Moisture content; Method & Specifications: ______

[**10/27**] Uniformity of mixing; Method & Specifications: ______

[**12/27**] Strength; Method & Specifications: ______

[**5/27**] Others; Method & Specifications: ______

Summary: The results indicate the importance of a field quality control method from high to low density, moisture content, strength and uniformity of mixing. Other methods include uniform interval thickness, immediate bearing value, gradation, and deflection.

7) What are your agency's construction practices for cementitious stabilization?

Mixing: [27/27] In-situ Mixing [12/27] Plant Mixing
Curing Method: [18/24] Asphalt emulsion; [14/24] Moisture;
[4/24] others: wax; plasting sheeting; contractor's choice; cutback asphalt
Construction Cut-off Date/temperature: ____________

Summary: For mixing, all agencies that responded to this question allow the use of *in situ* mixing (27/27), and almost half of them (12/27) specify plant mixing. In general *in situ* mixing is applicable to lime as a stabilizer, whereas plant mixing, which could provide higher quality, is used when cement is the stabilizer.

With regard to curing method, 24 agencies responded. 18 of 24 agencies specify curing with asphalt emulsion; 14 of 24 use a moisture curing method. Other curing methods are wax, plastic sheeting, cutback asphalt and contractor's choice.

8) Did your agency conduct any field study/experiment in connection with cementitious stabilization of pavement materials?
[**14/27**] Yes; [**13/27**] No;
If yes, please provide information for lab test results and field distress survey data, or provide contacts for these studies/experiments?
(Please see the "Survey Results")

Summary: Out of 27 state DOTs, almost half of them (14/27) use field study/experiments; the others (13/27) do not. The information for lab tests or contacts are in the document "Survey Results.

Complete Listing of Responses Received for the NCHRP 04-36 Survey

	1			2	3	4	5		6	7			8
	Extent of use			Stabilized materials	Pavement design	CSL Properties	Ranking of performance of pavement (1-most important)		Field quality control	Construction practices			Field study/experiment
State	Subgrade	Subbase	Base	additives-soil		Ranking (1-most important)	Flexible	Rigid	Method	Mixing	Curing	Cut-off Date / temp	Information or contact
Alabama	moderate	little	little	•cement-sand •lime-clay, expansive soil	layer coeff.	1-strength (lime stabilization AASHTO T 208, soil-cement AASHTO T 22) 2-reduction in Plasticity Index (lime stabilization, AASHTO T 90)	1-transverse (reflective cracking and ride quality)	1-transverse (ride quality, water infiltration (potential pumping))	•density (AASHTO T 310) •moisture content (lime, AASHTO T 265; soil-cement, AASHTO T 134) •strength (lime, AASHTO T 208; soil-cement, AASHTO T 22)	•in-situ-lime •plant-soil-cement	•asphalt •moisture	Apr. 1 •soil-cement-air, 40F; soil,	No.
Alaska			little	•cement-gravel	modulus	1-shrinkage 2-durability 3-stiffness 4-fatigue	1-alligator cracking (shringkage cracking) 2-block cracking (shringkage cracking)		density	in-situ			
Arizona	no cementitious bases due to cracking												

	1			2	3	4	5		6	7			8
	Extent of use			**Stabilized materials**	**Pavement design**	**CSL Properties**	**Ranking of performance of pavement (1-most important)**		**Field quality control**	**Construction practices**			**Field study/experiment**
State	**Subgrade**	**Subbase**	**Base**	**additives-soil**		**Ranking (1-most important)**	**Flexible**	**Rigid**	**Method**	**Mixing**	**Curing**	**Cut-off Date / temp**	**Information or contact**
California	little	little	moderate	•cement-silt, sand, RAP, expansive soil •lime-RAP •class C fly ash RAP •lime+cement-expansive soil •cement kiln dust-RAP	Gravel Equivalent	1-strength(CT 312, CT 373 (ASTM C 977)) 2-stiffness 3-shrinkage 4-durability 5-swelling	*-transverse (reflective cracking) *longitudinal outside (reflective cracking) *-block (shrinkage cracking in CTN)	*-others (faulting due to CTB erosion)	•density(CT231,CT312) •moisture(CT226) •uniformity (CT338(ASTM M 92)) •strength(CT 312, CT 373 (ASTM C 977)) •others(CT 217 (sand equivalent), CT 202 (AASHTO M 92))	•in-situ •plant	•asphalt •wax	40F	Pulverized Asphalt Concrete and FDR-Foamed Asphalt data at: www.its.berkeley.edu/pavementresearch/ •Contact Dave Jones: djjones@ucdavis.edu
Colorado	little	little		•cement-gravel •lime-clay, expansive soil •class C fly ash -silt, sand, gravel	layer coeff.	1-strength 2-stiffness 3-swelling 4-durability 5-shrinkage 6-fatigue 7-erodibility	1-transverse 2-alligator 3-block 4-longitudinal in 5-longitudinal out	1-transverse	•density •moisture •uniformity •strength	in-situ	moisture		
Connecticut	Soils provide excellent support.												

	1			2	3	4	5		6	7			8
	Extent of use			Stabilized materials	Pavement design	CSL Properties	Ranking of performance of pavement (1-most important)		Field quality control	Construction practices			Field study/experiment
State	Subgrade	Subbase	Base	additives-soil		Ranking (1-most important)	Flexible	Rigid	Method	Mixing	Curing	Cut-off Date / temp	Information or contact
Delaware			moderate	•cement-silt, sand	layer coeff.	1-strength 2-stiffness 3-durability 4-fatigue 5-erodibility 6-swelling 7-shrinkage	1-transverse (reflective cracking) 2-alligator 3-longitudinal in 4-block 5-longitudinal out		•density •moisture •strength	in-situ	asphalt		
Florida	little		little	•cement-clay, silt, sand	•layer coeff. •when allowed	*-strength (Florida Test Method FM 5-520)	*-transverse (reflective)		•density (In archived specification 270, Soil – Cement Base)	•in-situ •plant	•asphalt •moisture	7 days	Yes
Georgia	little	little	moderate	•cement-clay •lime-clay	Layer coeff.	1-strength (CBR, Density) 2-stiffness (CBR) 3-durability	1-transverse (reflective joint cracking of a composite pavement because of similar appearance) 2-block cracking (same to transverse)	*- Large area block cracking on a high truck volume facility had a detrimental effect with time.	•density (Theoretical density; % of theoretical) •strength (Core breaks; 300 psi) •Uniform interval thickness (within one-half inch of design)	•in-situ •plant	Cutback asphalt	40F	Yes. Dwane Lewis 404-694-6685

	1			2	3	4	5		6	7			8
	Extent of use			**Stabilized materials**	**Pavement design**	**CSL Properties**	**Ranking of performance of pavement (1-most important)**		**Field quality control**	**Construction practices**			**Field study/experiment**
State	**Subgrade**	**Subbase**	**Base**	**additives-soil**		**Ranking (1-most important)**	**Flexible**	**Rigid**	**Method**	**Mixing**	**Curing**	**Cut-off Date / temp**	**Information or contact**
Illinois	extensive	little	little	•cement-silt, sand, gravel •lime-clay	modulus	*strength (AASHTO T 208) *stiffness (AASHTO T 208) *swelling (IL Modified AASHTO T 193)	*others (12"modification or full-depth HMA and PCC)	same to asphalt	•density (AASHTO T 99) •moisture content (AASHTO T 99) •uniformity (ILDOT's Standard Specifications) •strength (AASHTO T 208) •IBV (IL Test Procedure 501)	•in-situ •plant	•asphalt •moisture •plasting sheeting	45F	Riyad Wahab at 217-782-2704 or Greg Heckel at 217-782-6709
Indiana	extensive			•cement-silt, sand, expansive soil •lime-clay, silt, expansive soil •class C fly ash-silt, sand •lime+class F fly ash- clay, silt •lime+cement-clay, silt, sand •CKD-clay,silt	•modulus •Mr (Mr<3000psi)	1-strength 2-stiffness 3-swelling 4-durability 5-erodibility 6-shrinkage 7-fatigue *-leathing *-freeze & thaw	1-transverse 2-longitudinal in 3-longitudinal out 4-block 5-alligator *-rutting *-freeze and thaw *drainage	*-Improvements in uniformity, freeze and thaw and load transfer are obtained	•density •moisture •uniformity •strength	in-situ			

	1			2	3	4	5		6	7			8
	Extent of use			Stabilized materials	Pavement design	CSL Properties	Ranking of performance of pavement (1-most important)		Field quality control	Construction practices			Field study/experiment
State	Subgrade	Subbase	Base	additives-soil		Ranking (1-most important)	Flexible	Rigid	Method	Mixing	Curing	Cut-off Date / temp	Information or contact
Kansas	extensive		extensive	•cement-silt, sand •lime-clay, expansive soil •class C fly ash-silt, sand, RAP	Layer coeff.	1-swelling 2-shrinkage 3-durability 4-fatigue 5-erodibility 6-strength 7-stiffness	1-transeverse (thermal cracking) 2-alligator (Poor bond between HMA layers, insufficient structure) 3-longitudinal in (poor bond between HMA layers, insufficient structure) 4-longitudinal out (Construction joints)	1-joint spalling (inadequate air void system in concrete, also durability factors due to poor aggregates – “D”-cracking) 2-transverse (non-uniform subgrade or base support, inadequate mixing of cementitiously stabilized subgrades)	•density (Kansas Test Method KT-51, Nuclear Gauge) •moisture content (Gas Pressure Method, Kansas Test Method KT-11, or KT-51, Nuclear Gauge) •uniformity (Spec, test application rate of material, gradation requirements on mixed material	•in-situ •plant	•asphalt •moisture	40°F or subgrade frozen	Yes. Contact Greg Schieber, at gregs@ksdot.org or 785-296-1198 and will send reports
Kentucky	little			•cement-silt, sand •lime-clay	layer coeff.	1-strength (proof rool) 2-durability (pavement life)	1-longitudinal in (truck loads)		•density (nuclear density machine) •moisture (nuclear density machine) •uniformity (field observation)	in-situ	asphalt	Oct/ 40F	attachment
Maine	little	little	moderate	•cement-clay, silt, sand, gravel, RAP, expansive soil	layer coeff.	1-stiffness 2-strength 3-fatigue 4-durability 5-shrinkage 6-swelling 7-erodibility	1-transverse (low temp) 2-longitudinal in (heavy truck) 3-longitudinal out(bond)		•density (Nuclear method–98% of test strip)	in-situ	moisture	Sept.15/ 50F	Rick Bradbury, Maine DOT 207-941-4597 richard.bradbury @maine.gov

	1			2	3	4	5		6	7			8
	Extent of use			Stabilized materials	Pavement design	CSL Properties	Ranking of performance of pavement (1-most important)		Field quality control	Construction practices			Field study/experiment
State	Subgrade	Subbase	Base	additives-soil		Ranking (1-most important)	Flexible	Rigid	Method	Mixing	Curing	Cut-off Date / temp	Information or contact
Maryland	moderate	little	little	•cement-silt, sand, gravel •lime-clay, silt	•layer coeff. (base or subbase) •modulus (subgrade)	(For all ASTM or AASHTO) 1-strength 2-stiffness 3-durability 4-fatigue 5-shrinkage 6-swelling 7-erodibility	1-transverse (reflective cracking)	no use cementitious stabilized layers in concrete pavement	•density (MDSHA Standard Specifications) •moisture (MDSHA Standard Specifications) •strength(Core Sampling → Unconf. Compressive Testing, FWD)	in-situ	•asphalt •moisture	7 day cure period	
Massachusetts	just HMA												
Minnesota	No use due to cracking												

	1			2	3	4	5		6	7			8
	Extent of use			Stabilized materials	Pavement design	CSL Properties	Ranking of performance of pavement (1-most important)		Field quality control	Construction practices			Field study/experiment
State	Subgrade	Subbase	Base	additives-soil		Ranking (1-most important)	Flexible	Rigid	Method	Mixing	Curing	Cut-off Date / temp	Information or contact
Montana			extensive	•cement-gravel, RAP •class C fly ash-gravel, RAP (typically use 3% portland cement and 2% class C fly ash, for total of 5% cementitious material.)	Layer Coefficient 0.20 in/in	*-strength (unconfined compression of standard Proctor samples)	*-No performance issues have been identified.	One portland cement concrete pavement failure (may have been caused by erosion and/or durability of the lean concrete base.)	•density (nuclear density, 96% of standard Proctor) •moisture content (+/- optimum moisture from standard Proctor) •strength (unconfined compression of proctor sample, 400 psi minimum) •gradation	•in-situ •plant	•asphalt	•35F(ground) •40F(air)	No.
Nebraska	extensive	extensive	extensive	•cement-silt •lime-clay, expansive soil •class C fly ash-clay, silt, RAP •CKD-clay	•layer coeff. •modulus	*-strength (compression) *-stiffness (modulus)			•density (rolling) •moisture (mix design) •uniformity (specs)	in-situ	•asphalt •moisture	60F	Mick Syslo 402-479-4791 or mick.syslo@nebraska.gov
Nevada	moderate	moderate	moderate	•cement-clay, silt, RAP, expansive soil •lime-clay, RAP, expansive soil •lime+cement-clay, expansive soil	•layer coeff. •modulus	1-stiffness (ASTM D4609, D 6276) 2-durability(ASTM D560) 3-erodibility(ASTM D559) 4-strength(ASTM D1633)	1-transverse (reflective)	1-transverse (reflective) 2-base failure	•density (NEV. T101, T102, AND T103) •moisture (NEV. T236)	•in-situ •plant	•asphalt •moisture	35F	

	1			2	3	4	5		6	7			8
	Extent of use			Stabilized materials	Pavement design	CSL Properties	Ranking of performance of pavement (1-most important)		Field quality control	Construction practices			Field study/experiment
State	Subgrade	Subbase	Base	additives-soil		Ranking (1-most important)	Flexible	Rigid	Method	Mixing	Curing	Cut-off Date / temp	Information or contact
New Hampshire	NH DOT does not use Cementitiously Stabilized base materials												
New Jersey	no use												
New Mexico	moderate	moderate	moderate	•cement-sand, gravel •lime-clay, silt	layer coeff. (based on compressive strength)	1-stiffness 2-strength 3-durability 4-swelling 5-shrinkage 6-erodibility			•density •moisture •uniformity	in-situ	moisture		
New York	no use for cost												
North Dakota	little	little	moderate	•cement-gravel •class C fly ash-expansive soil •lime+class F fly ash- expansive soil	layer coeff.	1-strength 2-durability 3-swelling	1-thinner base section 2-alligator 3-block		•density •moisture •strength	in-situ	asphalt		

	1			2	3	4	5		6	7			8
	Extent of use			**Stabilized materials**	**Pavement design**	**CSL Properties**	**Ranking of performance of pavement (1-most important)**		**Field quality control**	**Construction practices**			**Field study/experiment**
State	**Subgrade**	**Subbase**	**Base**	**additives-soil**		**Ranking (1-most important)**	**Flexible**	**Rigid**	**Method**	**Mixing**	**Curing**	**Cut-off Date / temp**	**Information or contact**
Ohio	moderate			•cement-clay, silt •lime-clay	in design no stiffness, but constructability has	*-strength *-stiffness *-durability *-erodibility	no issues in Ohio	no issues in Ohio	•density •moisture	in-situ	•asphalt •others	40F	http://www.dot.state.oh.us/Divisions/TransSysDev/Research/reportsandplans/Reports/2004/Pavements/14746-FR.pdf
Oklahoma	extensive	little	little	•cement-silt, sand, gravel •lime-clay, expansive soil •class C fly ash-clay, silt, sand, gravel •lime+class F fly ash-clay •CKD-silt, sand, gravel	•layer coeff. •modulus	1-durability 2-stiffness 3-strength 4-erodibility 5-shrinkage 6-fatigue 7-swelling			•density (AASHTO T310) •moisture content (AASHTO T310)	in situ	moisture		

	1			2	3	4	5		6	7			8
	Extent of use			Stabilized materials	Pavement design	CSL Properties	Ranking of performance of pavement (1-most important)		Field quality control	Construction practices			Field study/experiment
State	Subgrade	Subbase	Base	additives-soil		Ranking (1-most important)	Flexible	Rigid	Method	Mixing	Curing	Cut-off Date / temp	Information or contact
Oregon	little		little	•cement-clay, silt, gravel •lime-clay, silt, expansive soil	non-structural for subgrade, a construction platform	1-strength (Unconfined Compressive Strength) 2-durability(Freeze-thaw performance)	1-block(reflective) 2-transverse(reflective) 3-longitudinal in (more in HMA failure) 4-longitudinal out(reflective) 5-alligator(more in HMA failure)	1- ODOT uses CRCP – longitudinal cracking has occurred in CRCP over CTB, likely due to a non-effective or non-existent bond breaker	•density-ODOTSpec 00344&00330 •moisture-ODOTSpec 00344&00330 •uniformity-ODOTSpec 00344 •deflection-ODOT TM 158	in-situ	contractor's choice	Not specified, must meet compaction / deflection	Limited Dynamic Cone Penetrometer (DCP) data collection on Cement Modified Subgrade (silt soil) in 2007 Contact is Rene' Renteria, 503-986-3122
Puerto Rico	not using cement treated bases,but lean concrete base												
Rhode Island	has very good aggregate that is used for subgrades												

	1			2	3	4	5		6	7			8
	Extent of use			**Stabilized materials**	**Pavement design**	**CSL Properties**	**Ranking of performance of pavement (1-most important)**		**Field quality control**	**Construction practices**			**Field study/experiment**
State	**Subgrade**	**Subbase**	**Base**	**additives-soil**		**Ranking (1-most important)**	**Flexible**	**Rigid**	**Method**	**Mixing**	**Curing**	**Cut-off Date / temp**	**Information or contact**
South Carolina	moderate	little	moderate	Cement-clay, silt, sand, gravel	•layer coeff.	1-strength (SC-T-38) 2-shrinkage (No test) 3-durability (No test)	1-transverse (shrinkage from excess cement) 2-block cracking (shrinkage from excess cement) 3-alligator (not in cement stabilized materials, but in asphalt above cement stabilized layers)		•density (95% of AASHTO T-134 by the nuclear density gauge) •moisture content (between optimum and optimum + 2 % by Speedy Moisture gauge for soil materials, nuclear density gauge for aggregate materials) •uniformity of mixing (mixed to a homogeneous mixture by visual inspection)	•in-situ (Cement modified subgrades and recycled bases) •plant (all other materials)	asphalt	40 F in shade and rising	No. Contact Mike Lockman, Geotechnical Materials Engineer, (803)737-6692, lockmangm@scdot.org if you have any questions
South Dakota	little			•class C fly ash-silt, expansive soil	no cement treated material in design	1-strength (initial strength)							
Tennessee	little		little	•cement-clay, silt, sand •lime-clay •lime+cement-gravel	layer coeff.	1-strength 2-stiffness	1-alligator (In compatability of flexible pavement and stabilized material)		•density •moisture	•in-situ •plant	asphalt		

	1			2	3	4	5		6	7			8
	Extent of use			**Stabilized materials**	**Pavement design**	**CSL Properties**	**Ranking of performance of pavement (1-most important)**		**Field quality control**	**Construction practices**			**Field study/experiment**
State	**Subgrade**	**Subbase**	**Base**	**additives-soil**		**Ranking (1-most important)**	**Flexible**	**Rigid**	**Method**	**Mixing**	**Curing**	**Cut-off Date / temp**	**Information or contact**
Texas	extensive	extensive	extensive	•cement-silt, sand, gravel, RAP •lime-clay, gravel, expansive soil •class C fly ash-silt, sand, gravel, RAP •lime+class F fly ash- silt, sand, gravel, RAP •lime+cement-clay •CKD-gravel	•modulus •shear strength	1-durability (Tex120E, Tex121E) 2-strength(Tex120E, Tex121E & Tex117E) 3-stiffness(NA, FWD) 4-Fatigue 5-shrinkage (Tex 107E) 6-erodibility 7-swelling(Tex124E)	1-transverse (shrinkage from excessive placement moisture) 2-longitudinal in (poor subgrade) 3-alligator (structure inadequate) 4-longitudinal out (moisture loss) 5-block (not due to stabilization or has been subject of environmental material shrinkage)	1-longitudinal cracking (inadequate sawing) 2-transverse (inadequate sawing requirements)	•density •moisture •strength (rarely used)	•in-situ •plant	•asphalt •moisture	35F and rising or 40F	•Proj. 4182 •Proj 4502 •Proj 4920
Utah	Specs. don't provide it, but have some application												
Vermont		little	moderate	•cement-silt, sand, gravel		1-strength 2-durability 3-stiffness 4-swelling 5-fatigue 6-shrinkage 7-erodibility	1-transverse (freeze-thaw, thermal constraction)		•density •moisture •strength	in-situ	asphalt	Oct,15	Jennifer Fitch Research Engineer 802-828-2553

	1			2	3	4	5		6	7			8
	Extent of use			Stabilized materials	Pavement design	CSL Properties	Ranking of performance of pavement (1-most important)		Field quality control	Construction practices			Field study/experiment
State	**Subgrade**	**Subbase**	**Base**	**additives-soil**		**Ranking (1-most important)**	**Flexible**	**Rigid**	**Method**	**Mixing**	**Curing**	**Cut-off Date / temp**	**Information or contact**
Virginia	extensive	extensive	extensive	•cement-silt, sand, gravel •lime-clay, expansive soil	layer coeff.	1-sthrength 2-erodibility 3-durability 4-fatigue	1-transverse (reflective)		•density •moisture	•in-situ •plant	•asphalt •moisture	40F	
Washington	In the past we have used cement modified base. When we used it we were prone to reflective cracking of the HMA layer. So we discontinued use.												
Wisconsin	little	little	little	•cement-clay •lime-clay •class C fly ash-clay	soil support value or K value	1-strength (Mr or plate bearing) 2-durability 3-stiffness 4-erodibility	Stabilization is not used very often	Stabilization is not used very often	•density (AASHTO T-99) •moisture	in-situ		•35F(ground) •50F(air)	See whrp.org for study reports

	1			2	3	4	5		6	7			8
	Extent of use			**Stabilized materials**	**Pavement design**	**CSL Properties**	**Ranking of performance of pavement (1-most important)**		**Field quality control**	**Construction practices**			**Field study/experiment**
State	**Subgrade**	**Subbase**	**Base**	**additives-soil**		**Ranking (1-most important)**	**Flexible**	**Rigid**	**Method**	**Mixing**	**Curing**	**Cut-off Date / temp**	**Information or contact**
Wyoming	don't use cement treated base now			•cement-gravel •class C fly ash-gravel	layer coeff.	*-strength *-durability (PCA mixture design)	1-transverse 2-longitudinal out 3-block		•density (AASHTO T 99 and AASHTO T191) •moisture •unformity (pug mill)	plant	cutback asphalt	40F	Several of our LTPP-GPS sites have cement treated base. Rick Harvey – State Materials Engineer- 307-777-4476 rick.harvey@dot.state.wy.us

NCHRP Project 04-36

Characterization of Cementitiously Stabilized Layers for Use in Pavement Design and Analysis

Results from Standard Specifications of State DOTs

The research team reviewed the standard specifications for construction posted on the 50 agencies' websites to complement the information about cementitiously stabilized layers. In total, 32 state agencies have pertinent specifications. From the standard specifications, the team obtained information about cementitiously stabilized materials, including the type of stabilizers and soils, the *in situ*/pugmil mixing process, temperature during the time of construction, and compaction density and moisture.

Statistics of the Results from Standard Specifications

1) Type of cementitious additives

Out of the 32 states, lime (22/32) and cement (27/32) are the two most important kinds of cementitiously stabilized materials; the mixture of lime and fly ash (5/32) also occupies a certain proportion. Other stabilizers include borrowed materials, pozzolan, soil binder, chloride, mixture of lime and fly ash and cement, mixture of fly ash and cement, and mixture of fly ash and cement and cement kiln dust.

2) What types of cementitious additives are used to stabilize different layers?

Lime is often used to stabilize subgrade (7/29); cement is usually applied to the base (14/29). Other additives are fly ash/cement, lime/fly ash, etc.

3) Mixing location

For lime, most of the mixing locations are *in situ* (11/18), followed by plants (6/18) and yard (1/18). For cement, the order from high to low is plants (19/36), *in situ* (14/36), transit (2/36), and yard (1/36). (The total number of 36 is greater than 32, as some state agencies have specifications for several mixing locations.)

4) Additive placement

More than half of the states (20/37) require lime in slurry condition; the others (17/20) use dry lime. More states prefer dry application of cement (8/10) than cement slurry (2/10). For lime and fly ash, slurry placement (4/6) is preferred over dry placement (2/6).

5) Mix design

Only a few state agencies have specifications for mix design. For example, California specifies California Test 338 as the mix design method; Illinois's mix design is based on "the Department's Geotechnical Manual Procedure;" Oklahoma's mix design is "OHD L 50/51." Texas has different mix design methods: for lime the mix design is TX-121-E; for cement it is TX-120-E; and for the mixture of fly ash and lime it is TX-127-E.

6) Cut-off date/temperature

All the agencies have specifications about cut-off date/temperature. The typical cut-off temperature is around 40°F and cut-off date between April1st and the end of October. The temperature has to be above the freezing point during construction.

7) Placement after slurry

When additive is applied for the slurry condition, it is specified to be mixed with the natural soil within a time window since the production of slurry. Illinois and Mississippi require that the placement of lime slurry is less than 6 hours after the production of slurry. Virginia also requires that the placement of mixture of lime and fly ash should be less than 6 hours. The placement of cement slurry should be less than 30 minutes in Nevada and Virginia, and less than 2 hours in Texas.

8) Mixing delay

The time of mixing delay after placement of an additive is different depending on the additive. Lime has a longer time (4-6 hours) than cement (less than 1 hour).

9) Moisture tolerance during mixing

Because density can reach the maximum value under the optimum moisture content, the moisture during mixing should be around the optimum value. For soil-lime, the moisture content is specified above the optimum value to account for the evaporation during the longer mixing delay. For cement, the moisture content is specified to be in the range of -2% to +2% of the optimum value.

10) Mellowing time

Mellowing time is needed when lime is used as the stabilizer. From the standard specifications, mellowing for more than two days is typical.

11) Compaction delay

For soil-lime, the compaction is allowed to finish within 6-24 hours; 2 hours is typical for cement-stabilized materials, and 4 hours for fly ash-stabilized materials.

12) Compaction moisture

The results are similar to 8) Moisture tolerance during mixing.

13) Density target

Most of the standard specifications require that the minimum value is 95% of the maximum dry density based on either standard or modified proctor.

14) Time between compaction and curing

After compaction, the layer should be cured under some special conditions. The time between compaction and curing is specified to be less than 24 hours (8/11).

15) Curing method

Asphalt curing is the most common curing method for any stabilized material. However, when cement is the stabilizer, there are some other choices in several states. Kansas allows the use of a wax-based liquid membrane-forming compound, Pennsylvania the use of a white membrane forming curing compound, Tennessee the use of transparent or white polyethylene sheeting, and West Virginia the use of white polyethylene sheeting.

16) Minimum curing time

Different states have different requirements; seven-day curing is the most common for various cementitiously stabilized materials.

17) Curing temperature

The temperature-specified curing process is the same as for the cut-off temperature. The temperature is specified to be above the freezing point during curing. The typical minimum curing temperature is 40°F.

Complete Listing of Standard Specifications for the NCHRP 04-36 Survey

State	Additive	Layer	Mixing location	Additive placement	Mix design	Cut off temp (°F)	Placement after slurry production	Mixing Delay	Moisture tolerance during mixing	Mellowing time	Compaction Delay	Compaction moisture	Density Target	Time between compaction and curing	Curing method	Minimum curing time	Minimum Curing temp (F)
Alabama	Lime		In situ	•dry •slurry		Apr1 to Oct1		6 hr	±2% optimum			±2% optimum	±5% of AASHTO T 180				
			yard						±2% optimum			±2% optimum					
			plant			40F			±2% optimum			±2% optimum	±5%				
	cement		In situ			35F↑ or 40F		6 hr	±2% optimum			±2% optimum	±5% of AASHTO T 180				
			yard						±2% optimum			±2% optimum					
			plant			40F			±2% optimum		3 hr	±2% optimum	±5%				
Arizona	Lime	subgrade	In situ	dry		40F			±2% optimum	24~48 hr			100% of maximum density		Bituminous	3 d	
	cement	subgrade	In situ	dry		40F			±2% optimum		2.5 hr		100% of maximum density		Bituminous	3 d	

State	Additive	Layer	Mixing location	Additive placement	Mix design	Cut off temp (°F)	Placement after slurry production	Mixing Delay	Moisture tolerance during mixing	Mellowing time	Compaction Delay	Compaction moisture	Density Target	Time between compaction and curing	Curing method	Minimum curing time	Minimum Curing temp (F)
	cement	base	plant	slurry		40F			±2% optimum		2.5 hr		100% of maximum density		Bituminous	3 d	
Arkansas	lime	subgrade	In situ	•dry •slurry		50F / Apr1 to Oct31			±2% optimum	3 d			»95% maximum density				
	cement	base	plant	dry		40F		60 min	±5% optimum		2 hr		»95% maximum density		asphalt		
	cement	crushed stone base	plant	dry		40F			±1% optimum						asphalt	72 hr	
California	lime		In situ	•dry •slurry		35F		7 d	above optimum	« 24 hr				«48 hr	asphalt	3 d	40F
	cement		In situ		California Test 338	35F			»99% optimum		2.5 hr	optimum			asphalt		35F
			plant		California Test 338	35F			»99% optimum		2.5 hr	optimum			asphalt		35F

State	Additive	Layer	Mixing location	Additive placement	Mix design	Cut off temp (°F)	Placement after slurry production	Mixing Delay	Moisture tolerance during mixing	Mellowing time	Compaction Delay	Compaction moisture	Density Target	Time between compaction and curing	Curing method	Minimum curing time	Minimum Curing temp (F)
Colorado	Lime		In situ	slurry		35F			» optimum	» 48 hr		2 ± 1% percent above the optimum	»95% maximum density		asphalt	»7d	35F
Delaware	Borrow material			•dry •slurry								±2% optimum	»100% maximum density				
Florida	cement	permeable base	plant	slurry		40							No requirements			3~4 d	
Georgia	lime			•dry •slurry		•45F •Apr1 to Oct15			±5% optimum	3 ~14 d		100~102% optimum		«24hr	bituminous	7d	
	cement		In situ			40F		45 min	100~120 % optimum		2h	100~120 % optimum	»98% maximum density				
			plant			40F		45 min	100~120 % optimum		2h	100~120 % optimum	»98% maximum density		bituminous	7d	
Illinois	lime			•dry •slurry		45F	6hr		100~ 103% optimum	48 hr			»95% lab dry density		asphalt	7d	

State	Additive	Layer	Mixing location	Additive placement	Mix design	Cut off temp (°F)	Placement after slurry production	Mixing Delay	Moisture tolerance during mixing	Mellowing time	Compaction Delay	Compaction moisture	Density Target	Time between compaction and curing	Curing method	Minimum curing time	Minimum Curing temp (F)
	cement		plant	dry	Department's Geotechnical Manual procedure	40F			80~ 110% optimum		2hr	slightly » optimum	100% standard dry density (AASHTO T 134)		bituminous	7d	
	pozzolanic		plant	dry		40F		90 min	80~ 110% optimum			slightly » optimum	1st lift, » 97% max density; 2nd,100% max density	«24hr	asphalt	lime fly ash,14hr; cement fly ash,12hr	
Indiana	fly ash/lime/cement			Hydrated Lime and Quicklime		45F					•cement- 3hr •fly ash- 4hr •lime-24hr	100~102% optimum	100% maximum density			72hr	
Kansas	lime	subgrade	In situ	slurry		40F			»108% optimum	»24hr					asphalt	7d	
			plant	slurry		40F			»108% optimum	»24hr					asphalt	7d	
	cement/fly ash	subgrade				40F		30 min	±3% optimum		2hr		»95% dry density		asphalt	7d	

State	Additive	Layer	Mixing location	Additive placement	Mix design	Cut off temp (°F)	Placement after slurry production	Mixing Delay	Moisture tolerance during mixing	Mellowing time	Compaction Delay	Compaction moisture	Density Target	Time between compaction and curing	Curing method	Minimum curing time	Minimum Curing temp (F)
	cement	base	plant								2hr		»95% standard density		wax-based liquid membrane-forming compound	7d	
Kentucky	lime	roadway		•dry •slurry		40F		4 hr	»optimum	48 hr			»95% maximum density		asphalt	7d	
Kentucky	cement	roadway		dry		40F		6 hr	100~ 102% optimum				»95% maximum density		asphalt	7d	
Louisiana	cement	base	In situ			35F			±2% optimum		3hr		»95% maximum density		asphalt	72hr	
Louisiana	lime		plant	slurry		35F				»48hr	6hr		»95% maximum density		asphalt	72 hr	
Mississippi	lime		In situ	•dry •slurry		40F	6hr			5~ 20d			»95% standard density	«24hr	asphalt	7d	35F
Mississippi	cement		In situ			40F		3 hr	« 102% optimum		4hr		»98% standard density	«24hr	asphalt	7d	» freezing

State	Additive	Layer	Mixing location	Additive placement	Mix design	Cut off temp (°F)	Placement after slurry production	Mixing Delay	Moisture tolerance during mixing	Mellowing time	Compaction Delay	Compaction moisture	Density Target	Time between compaction and curing	Curing method	Minimum curing time	Minimum Curing temp (F)
			plant			40F		60 min	«102% optimum		4hr		»98% standard density	«24hr	asphalt	7d	» freezing
	lime/fly ash			•dry •slurry		40F				No				«24hr	asphalt	7d	
Missouri	cement															48 hr	
Montana	cement		In situ			40F			±2% optimum		2hr	» optimum	»96% maximum dry density		bituminous	7d	
Nebraska	soil binder												» maximum density				
Nevada	cement	base	In situ			35F	«30 min		«101% optimum		3hr		»92% maximum density	24hr	asphalt	3d	
			plant			35F	«30 min		« optimum		3hr		»92% maximum density	24hr	asphalt	3d	
New Mexico	lime			•dry •slurry		40F			103~105% optimum	24 hr			100% maximum density		asphalt		

State	Additive	Layer	Mixing location	Additive placement	Mix design	Cut off temp (°F)	Placement after slurry production	Mixing Delay	Moisture tolerance during mixing	Mellowing time	Compaction Delay	Compaction moisture	Density Target	Time between compaction and curing	Curing method	Minimum curing time	Minimum Curing temp (F)
	cement					40F			100~105% optimum		4hr		100% maximum density		asphalt	7d	
New York	lime		In situ	•dry •slurry		40F		4 hr		24 hr	8hr			2d	asphalt	14d	
New York	cement		In situ			40F			100~102% optimum			optimum	»95% maximum density	48hr	bituminous	5d	
New York	cement		plant			40F		30 min	100~102% optimum			optimum	»95% maximum density	48hr	bituminous	•5d •freezing- 7d	
North Carolina	lime			•dry •slurry		45F		2 hr	100~103% optimum	1~4d	4d	100~102% optimum	»97% maximum density		asphalt	7d	
North Carolina	cement	base	plant			40F			100~101.5 % optimum		3hr	100~101.5 % optimum	»97% maximum density		asphalt	7d	
North Carolina	cement	base	In situ						« optimum		3hr	100~101.5 % optimum	»97% maximum density		asphalt	7d	

State	Additive	Layer	Mixing location	Additive placement	Mix design	Cut off temp (°F)	Placement after slurry production	Mixing Delay	Moisture tolerance during mixing	Mellowing time	Compaction Delay	Compaction moisture	Density Target	Time between compaction and curing	Curing method	Minimum curing time	Minimum Curing temp (F)
North Dakota	lime	subgrade		•dry •slurry		40F			»optimum			optimum			bituminous		
	Lime /fly ash	subgrade		•dry •slurry		40F			»optimum	24~48 hr		optimum			bituminous		
	cement	base	plant													48 hr	
Ohio	lime			Hydrated Lime and Quicklime		40F			»optimum	1~7d		»optimum	maximum dry density		asphalt	5d	» freezing
	cement			dry		40F		2 hr	100~103% optimum			»optimum	maximum dry density		asphalt	5d	» freezing
Oklahoma	cement/ flyash/ CKD	subgrade		dry	OHD L 50/51	40F			102~105% optimum			±2% optimum	»95% target density				

State	Additive	Layer	Mixing location	Additive placement	Mix design	Cut off temp (°F)	Placement after slurry production	Mixing Delay	Moisture tolerance during mixing	Mellowing time	Compaction Delay	Compaction moisture	Density Target	Time between compaction and curing	Curing method	Minimum curing time	Minimum Curing temp (F)
	lime	sugrade	slurry:plant/ transit	•dry •slurry	OHD L 50/51	40F			102~105% optimum	•hydrated: 72hr •quick:48hr		±2% optimum	»95% target density				
	cement	base				40F					2hr		»95% standard density			7d	
Oregon	lime	subgrade							optimum		12hr	96~102% optimum	»95% maximum density			7d	
Oregon	chloride	subgrade							optimum		12hr	96~102% optimum	»95% maximum density			7d	
Oregon	cement	subgrade							optimum		12hr	96~102% optimum	»95% maximum density			7d	
Pennsylvan ia	cement	base	•plant •truck			40F					1hr				white membrane forming curing compound	•40F-24hr •32~40F- 48hr	32F
South Carolin a	cement	modified subbase	In situ	dry		40F		3 hr	«optimum		2hr	±2% optimum	»95% maximum density		asphalt	3d	

State	Additive	Layer	Mixing location	Additive placement	Mix design	Cut off temp (°F)	Placement after slurry production	Mixing Delay	Moisture tolerance during mixing	Mellowing time	Compaction Delay	Compaction moisture	Density Target	Time between compaction and curing	Curing method	Minimum curing time	Minimum Curing temp (F)
	cement	modified recycled base	In situ			40F			«optimum		2hr				asphalt	3d	
	cement	stabilzied earth base	In situ			40F			±2% optimum		2hr	±2% optimum	»95% maximum density	«12hr	asphalt	7d	
			plant			40F		60 min	«102% optimum		2hr	±2% optimum	»95% maximum density	«12hr	asphalt	7d	
	cement	stabilzied aggregate base															
Tennessee	lime	subgrade	In situ	•dry •slurry		40F			102~108% optimum	2~7d		±3% optimum	»95% maximum density		bituminous		
	lime/ fly ash	base	In situ	hydrated Lime		40F			±2% optimum		8hr	99~103% optimum	»100% maximum density	«24hr	bituminous	7d	
			plant	hydrated Lime		40F			±2% optimum		8hr	99~103% optimum	»100% maximum density	«24hr	bituminous	7d	

State	Additive	Layer	Mixing location	Additive placement	Mix design	Cut off temp (°F)	Placement after slurry production	Mixing Delay	Moisture tolerance during mixing	Mellowing time	Compaction Delay	Compaction moisture	Density Target	Time between compaction and curing	Curing method	Minimum curing time	Minimum Curing temp (F)
	cement	permeable base	In situ												transparent or white polyethylene sheeting	7d	
Texas	Lime		In situ	Lime slurry	TX-121-E	35F↑or 40F		6 hr		•1-4 Days •2-4 Days (Pebble Quicklime)			95%		asphalt	•PI«35,2d •PI>35,5d	
			Plant	Lime slurry	TX-121-E	35F↑or 40F					6 hr		95%		asphalt	7d	
	Cement		In situ	cement slurry	Tex-120-E	35F↑or 40F	«2hr		±2% optimum		«2 hr	±2% optimum	95%		asphalt	»3d	
			Plant	cement slurry	Tex-120-E	35F↑or 40F	«2hr		±2% optimum		«2 hr	±2% optimum	95%		asphalt	»3d	

State	Additive	Layer	Mixing location	Additive placement	Mix design	Cut off temp (°F)	Placement after slurry production	Mixing Delay	Moisture tolerance during mixing	Mellowing time	Compaction Delay	Compaction moisture	Density Target	Time between compaction and curing	Curing method	Minimum curing time	Minimum Curing temp (F)
	Fly Ash/ Lime		In situ	Lime slurry	Tex-127-E	40F		6 hr		•1-4 Days •2-4 Days (Pebble Quicklime)	6hr		95%		asphalt	•LFA:7d-14d •FA:24hr	
Virginia	lime/ lime-fly ash		•In situ •plant	•dry •slurry			«6hr				12hr	100~120 optimum	»95% maximum density		asphalt	7d	
	cement		In situ			40F	«30 min		optimum		4hr	100~120 optimum	»100% maximum density		asphalt	7d	
			plant					60 min			4hr	100~120 optimum	»100% maximum density		asphalt	7d	
West Virginia	cement	open graded free draining base	plant			40F		45 min							white polyethylene sheeting		

State	Additive	Layer	Mixing location	Additive placement	Mix design	Cut off temp (°F)	Placement after slurry production	Mixing Delay	Moisture tolerance during mixing	Mellowing time	Compaction Delay	Compaction moisture	Density Target	Time between compaction and curing	Curing method	Minimum curing time	Minimum Curing temp (F)
Wyoming	lime		plant	•dry •slurry		45F		48 hr				±3% optimum	»95% maximum density			24 hr	

Appendix B. Mix Design and Test Procedure Evaluation

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CHAPTER B-1. MIX DESIGN

Mix designs were conducted to determine the appropriate binder contents. In this study, the methods developed by the National Lime Association (NLA) (2006) for soil-lime, the Portland Cement Association (PCA) (1992) for soil-cement, and the Federal Highway Administration (FHWA) (Veisi et al. 2010) guidelines for soil-fly ash were followed. Three replicates were used to measure unconfined compressive strength (UCS). It is noted that these mix design methods may be different from the methods used by other agencies. However, it is not the objective of this study to develop mix design methods. Instead, the mix designs were conducted to obtain representative mixes for test procedure evaluation and model development. In addition, depending on the location of a mix in a pavement, i.e., in the base or subbase, the binder content could be different for one type of soil.

Soil-Cement

Based on the soil designations, three trial cement contents were used to fabricate the soil-cement specimens, in accordance with the PCA method (PCA 1992). Among the trial contents, the minimum cement content that resulted in UCS values larger than 300 psi after 7-day curing was selected for that soil. It was found that a cement content of 3% is suitable for stabilizing gravel, 6% for sand, 8% for silt, and 12% for clay, as shown in Figure B-1.

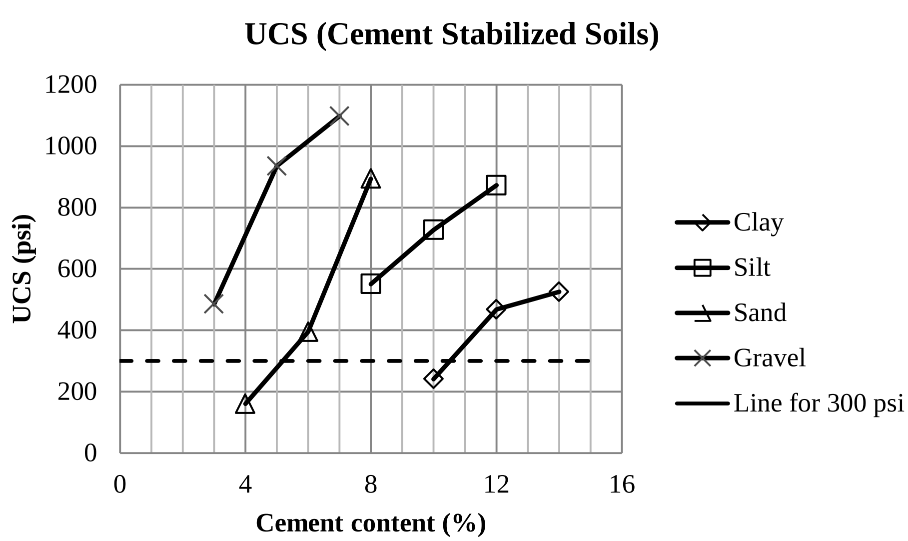


Figure B-1. Results for UCS after 7-day Curing for Cement-Stabilized Soils

Soil-Fly Ash (Class C)

Three fly ash contents, 10%, 13% and 16% based on the dry weight of the soils, were used as the trial contents for silt, sand, and gravel, respectively. The 7-day UCS of 400 psi

specified by the FHWA (Veisi et al. 2010) cannot be achieved for the gravel mixes or silt mixes, as shown in Figure B-2. Increasing the fly ash content did not significantly increase the strength of the gravel or silt mixes. Instead, 13% fly ash was used for stabilizing the silt, sand, and gravel, because this percentage has been used successfully in past studies and is consistent with the content used by some agencies, such as the Oklahoma DOT.

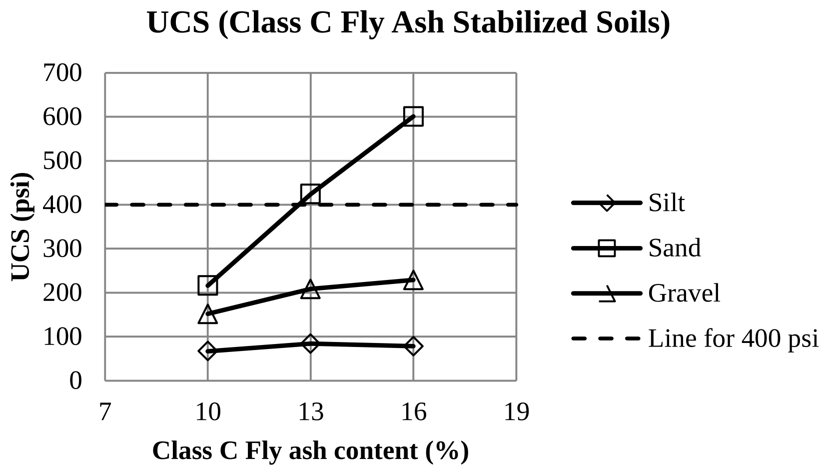


Figure B-2. Results for UCS after 7-day Curing for Class C Fly Ash-Stabilized Soils

Soil-Lime

According to the design method developed by the NLA (2006), the initial lime content was determined based on the Eade and Grim method. Different lime contents were added to a soil and water solution. The lime content that led to a pH of 12.49 was the initial lime content used for further laboratory evaluation in terms of strength. For clay-lime, 2% lime, based on the dry mass of clay, is the lime content needed to reach a pH of 12.49 (Figure B-3). Therefore, 2%, 4%, and 6% lime were used to fabricate specimens for strength testing to determine the minimum lime content for the clay-lime mix. It was found that 6% lime can lead to the UCS of 70 psi, which is specified by the NLA (2006) (Figure B-4).

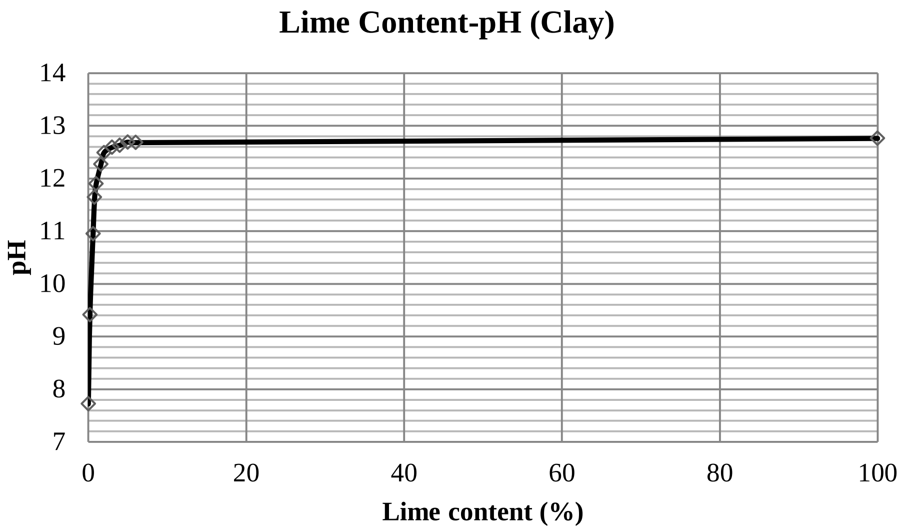


Figure B-3. pH Value vs. Lime Content

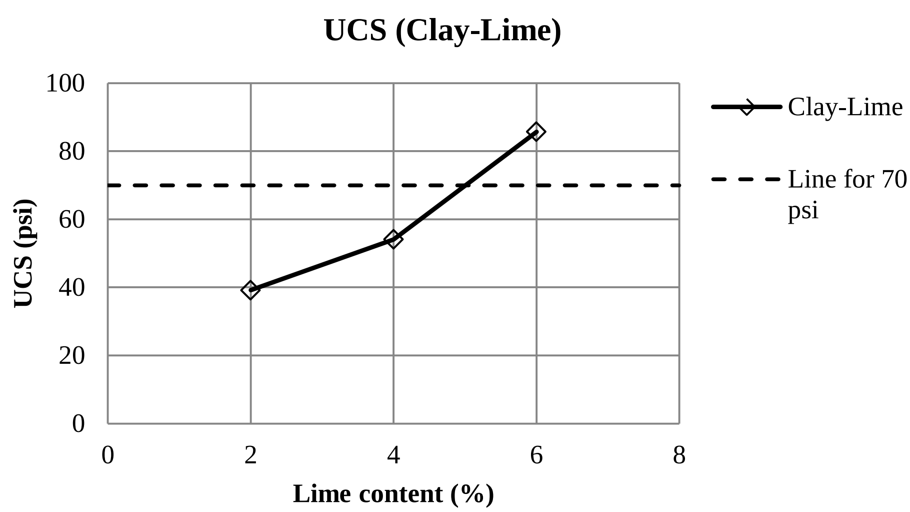


Figure B-4. Results for UCS after 7-day Curing for Clay-Lime Mix

Because the plasticity index (PI) of the silt is less than 10, this silt is not suitable for lime stabilization, in accordance with the NLA design method (2006). The team used lime and Class F fly ash to stabilize the silt. In accordance with the Mechanistic-Empirical Pavement Design Guide (MEPDG) (ARA 2004), the UCS should be at a minimum of 200 psi after 7-day curing and 4-hour soaking in water. This requirement can be achieved with 4% lime and 12% Class F fly ash, based on the dry mass of silt, as indicated in Figure B-5.

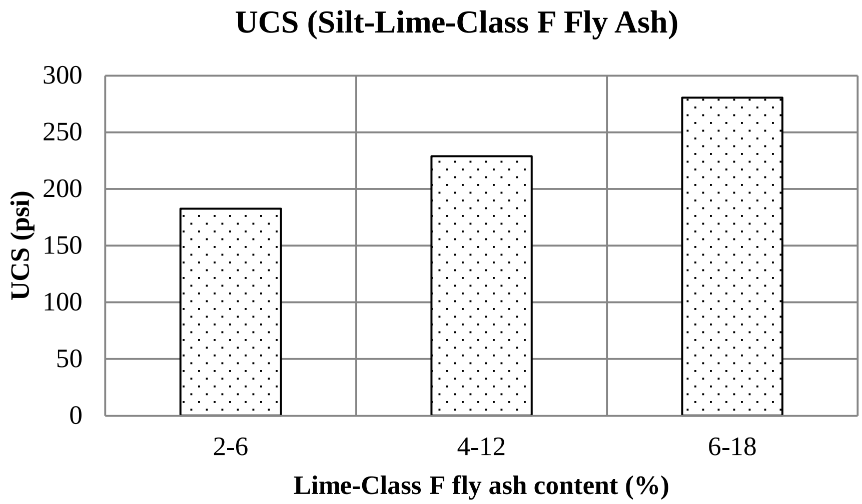


Figure B-5. Results for UCS after 7-day Curing for Silt-Lime-Class F Fly Ash

Summary of Mix Design

Table B-1 presents the final mix designs and Table B-2 presents the maximum dry density (MDD) and optimum moisture contents (OMC).

Table B-1. Final Mix Design of Stabilized Mixtures

	Clay	Silt	Sand	Gravel
Cement	12%	8%	6%	3%
Lime	6%	4% +12%*	Not applicable	Not applicable
Fly ash	Not applicable	13%	13%	13%

**Note: 12% Class F Fly Ash*

Table B-2. Maximum Dry Density and Optimum Moisture Content

	Clay		Silt		Sand		Gravel	
	OMC (%)	MDD (lb/ft^3)	OMC (%)	MDD (lb/ft^3)	OMC (%)	MDD (lb/ft^3)	OMC (%)	MDD (lb/ft^3)
No additive	19.1	107.4	10.4	123.7	7.2	115.4	7.7	138.5
Cement	18.0	103.2	11.1	119.9	8.7	122.6	6.2	140.1
Lime	19.4	104.8	12.4*	118.6*	N/A		N/A	
Class C fly ash	N/A		10.0	121.2	7.3	135.2	7.4	139.3

**Lime+Class F Fly Ash*

CHAPTER B-2. APPRAISAL AND LABORATORY EVALUATION OF TEST PROCEDURES

The test procedures needed to characterize the material properties are appraised in the following sections. These appraisals are based either on the literature review, or the team's experience or understanding. These test procedures were then evaluated in the laboratory.

Strength Tests

The types of strength tests for cementitiously stabilized materials (CSM) include UCS, indirect tensile (IDT) strength, tensile strength, modulus of rupture (MOR), and direct shear strength tests. Table B-3 presents the appraisal and recommendations of test procedures that are relevant to the strength of CSM.

Unconfined Compressive Strength Test: UCS is recommended as a Level 1 material property, which is a key parameter in the top-down compressive fatigue model. UCS testing also correlates well with other strength and modulus tests for Level 2 input. Most state highway agencies use UCS tests for mix design and for quality assurance/quality control (QA/QC) because of the simplicity of the tests. UCS testing is well-established and meets all cost, practicality and availability requirements (Yeo et al. 2002). UCS also is used in CSM mix design procedures. AASHTO T22, ASTM D1633, ASTM D5102, and ASTM C593 are test methods used for lean concrete, soil cement, lime-stabilized materials, and fly ash-stabilized materials with or without lime, respectively.

Modulus of Rupture Test: MOR, or flexural strength, is recommended as a Level 1 input for fatigue models of heavily stabilized materials because the MOR test simulates the resistance to cracking due to bending for CSL in the field. The MEPDG (ARA 2004) specifies AASHTO T97, *Standard Method of Test for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)*, or ASTM D1635, *Standard Test Method for Flexural Strength of Soil-Cement Using Simple Beam with Third-Point Loading*.

Indirect Tensile Strength Test: IDT strength testing is recommended for measuring tensile strength for Level I input to the shrinkage cracking model for both heavily and lightly stabilized materials, because direct tensile strength is difficult to measure. The IDT strength value is close to that of tensile strength and the test correlates well with other strength tests. Furthermore, the IDT test procedure is practical and repeatable.

The IDT strength can be obtained in accordance with the splitting test: AASHTO T198, *Splitting Tensile Strength of Cylindrical Concrete Specimens*, which specifies the specimen geometry of 6 inches in diameter and 12 inches in height. The specified loading rate of the

splitting tensile stress by AASHTO T198 is 100-200 psi/min. The size and loading rate may not be appropriate for soil-lime or soil-fly ash/lime, which are relatively weak materials. The Australian method (Midgley and Yeo 2008) used to determine the IDT strength of stabilized materials is recommended. The specimen sizes are either 4 inches in diameter and 2.4 inches in height or 6 inches in diameter and 3.3 inches in height, depending on the size of the soil/aggregate particles. The loading rate is 4500 lbs/min.

Modulus Tests

Modulus tests of CSM include resilient modulus, IDT modulus, flexural modulus, modulus of elasticity, seismic modulus, and acoustic modulus tests. Table B-4 presents the appraisal and recommendations of these test procedures to characterize the modulus of CSM.

Flexural Modulus Test: The flexural modulus test is recommended for characterizing heavily stabilized material, because the flexural modulus simulates bending conditions in the field. It is a dynamic and small-strain modulus, instead of a static and large-strain modulus, such as the modulus of elasticity. The use of the flexural modulus test also corresponds with the MOR tests recommended previously for different strength tests that share the same specimen. Currently, there is no standard for the flexural modulus test. The Australian test method used to determine the flexural modulus (Midgley and Yeo 2008) for CSM is evaluated in this study.

Resilient Modulus Test: Even though resilient modulus testing is time-consuming, the resilient modulus test is recommended for lightly stabilized materials, such as soil-lime, because it takes into account the effects of confining pressure on the resilient modulus. The compressive modulus is suitable when the lightly stabilized material is located deep in the pavement structure, such as in the subbase. AASHTO T307 is recommended by the current MEPDG (ARA 2004) to measure the resilient modulus of lightly stabilized materials.

Seismic Modulus Test: Both UCS and seismic modulus tests are recommended for modulus prediction as the Level 2 correlation for both heavily and lightly stabilized materials. Unlike UCS testing, seismic modulus testing is nondestructive and can be used to determine the modulus growth using the same specimen. The seismic modulus test is simple, repeatable, and inexpensive. Also, it has been shown to correlate very well with other modulus tests. The seismic modulus test can be based either on ASTM C215-08, *Standard Test Method for Fundamental Transverse, Longitudinal, and Torsional Frequencies of Concrete Specimens*, or the Texas DOT free-free resonant column (FFRC) test, Tex-147-E. Tex-147-E is evaluated in this study to determine its correlation with other modulus tests because it has been used extensively for CSM in the past.

Table B-3. Appraisal of Material Strength Tests

	Unconfined Compressive Strength (UCS)	Indirect Tensile Strength (IDT)	Direct Tensile Strength (DT)	Modulus of Rupture (MOR)	Direct Shear Strength
Performance Predictability	**Good** (a key parameter in the top-down compressive fatigue model; correlates well with modulus and other strength tests)	**Good** (predicts direct tensile strength well, which correlates with shrinkage cracking potential)	**Good** (predicts shrinkage cracking potential well, which is a key parameter in prediction model)	**Good** (predicts fatigue resistance well, which is a key parameter in prediction model)	**Good** (predicts erosion resistance well, which is a key parameter in prediction model)
Precision	**Average** (COV typically below 20%) (White 2005)	**Good** (COV typically below 10%) (White 2005, Hudson and Kennedy 1968)	**Unknown**	**Average** (COV around 20%)	**Poor** *(*Kaniraj and Havanagi 2001)
Accuracy	**Good** (correlates well with modulus and strength tests)	**Good** (0.9-1.1 of direct tensile strength) (Bonnot 1991)	**Good** (direct measurement of tensile strength) (Hudson and Kennedy 1968)	**Good** (direct measurement of modulus of rupture)	**Poor** (failure plane difficult to control for stiff materials)
Practicality	**Good** (well established strength test, and meets cost, practicality, and availability requirements; not for tensile strength and fatigue) (Yeo et al. 2002)	**Good** (practical, productive, economic, and user friendly (Yeo et al. 2002); same type of specimen and equipment as that used for compression testing) (Hudson and Kennedy 1968)	**Poor** (misalignment for pure tension, failure outside central portion (Yeo et al. 2002); gripping specimen is difficult (Hudson and Kennedy 1968)	**Average** (difficult to obtain field samples (Yeo 2008); good for high stiffness materials; beam is hard to prepare and handle (Gnanendran and Piratheepan 2008)	**Average** (difficult for stiff and cemented materials)
Machine Costs	**Low**	**Low**	**Average**	**Low**	**Low**
Time	**Short**	**Short**	**Average** (3 hr preparation of specimens)	**Short**	**Short**
Recommendation	**Recommended** for compressive fatigue model as Level 1 input and Level 2 for prediction of modulus and other strength tests	**Recommended** for shrinkage cracking as Level 1	**Not recommended** due to operational difficulties	**Recommended** for fatigue model as Level 1	**Not recommended** due to impracticality

Note: Cost Ratings: High: >$50,000; Medium: $20,000 to $50,000; Low: <$20,000; Very Low: <$5,000

Table B-4. Appraisal of Stiffness Tests

	Resilient Modulus (M_r)	**IDT Modulus**	**Flexural Modulus**	**Modulus of Elasticity**	**Seismic Modulus (SM)**	**Acoustic Modulus (AM)**
Performance Predictability	**Good** (simulates compression in the field and accounts for confining pressure)	**Average** (simulates shrinkage/tensile behavior of CSL)	**Good** (simulates field bending conditions; dynamic loads)	**Poor** (fundamental properties; but test does not simulate field bending conditions)	**Average** (does not simulate field bending conditions)	**Average** (does not simulate field bending conditions)
Precision	**Average** (mixed results) (Scullion et al. 2008)	**Average** (mixed results) (White 2005)	**Average** (COV below 20%) (Yeo 2008, Carteret 2009)	**Average** (COV below 20%) (Richardson 1996)	**Good** (repeatable and COV typically below 7%) (Hilbrich and Scullion 2007)	**Good** (repeatable) (Yesiller et al. 2001)
Accuracy	**Good** (compressive modulus for lightly stabilized materials used in subbase)	**Good** (direct measurement of tensile behavior)	**Good** (direct measurement)	**Average** (compressive and static moduli, instead of bending or tensile moduli)	**Good** (correlates well with other moduli) (Hilbrich and Scullion 2007)	**Good** (correlates well with stiffness) (Yesiller et al. 2001)
Practicality	**Poor** (has some dependence on stress levels and is very sensitive to the test procedure; test is complex and very time-consuming) (Hilbrich and Scullion 2007)	**Average** (relatively practical and user friendly; stress/strain control is difficult with brittle stabilized materials; suitable for lightly stabilized materials) (Yeo 2002)	**Average** (beam samples are difficult to manufacture in the laboratory; can use the same beam specimen for modulus of rupture) (Yeo 2002)	**Good** (simple and practical; the types of specimen and equipment are the same as those used for compression testing)	**Good** (inexpensive and rapid; nondestructive; plaster capping is not required.) (Hilbrich and Scullion 2007)	**Good** (inexpensive and rapid; nondestructive; plaster capping is not required.) (Yesiller et al. 2001)
Machine Costs	**High**	**High**	**High**	**Low**	**Very low**	**Very low**
Time	**Long**	**Long**	**Long**	**Short**	**Short**	**Short**
Recommendation	**Recommended** for evaluation of lightly stabilized materials	**Recommended** for shrinkage cracking modeling	**Recommended** for heavily stabilized materials	**Not Recommended** because of poor performance predictability	**Recommended** for Level 2 prediction	**Recommended** for Level 2 prediction

Note: Cost Ratings: High: >$50,000; Medium: $20,000 to $50,000; Low: <$20,000; Very Low: <$5,000

Acoustic Modulus Test: The acoustic modulus test, also called the ultrasonic pulse velocity test, also is assessed as an alternative method to the seismic modulus test. It is also a nondestructive test technique and is simple to set up. ASTM C597 can be followed for the ultrasonic pulse test. By measuring the travel time through the specimen, the p-wave or shear wave velocity and related dynamic modulus are determined.

Durability Tests

The durability of CSM can be evaluated in terms of resistance to wetting-drying, freezing-thawing and moisture conditioning (tube suction or vacuum saturation). Table B-5 presents the appraisal and recommendations for test procedures relevant to durability.

Wetting-Drying Test: Wet-dry cycling is recommended for wet-dry model development because it simulates the field conditions to which CSM are subjected. The test procedures follow AASHTO135, *Standard Method of Test for Wetting-and-Drying Test of Compacted Soil-Cement Mixtures*. Instead of brushing and measuring mass loss, the seismic modulus test and/or UCS test is conducted after each cycle of wetting-drying to determine the effects of wetting-drying on the strength/modulus. However, the applicability of this test to lightly stabilized materials needs to be evaluated.

Freezing-Thawing Test: Freeze-thaw cycling is recommended for freeze-thaw model development. The conditioning is conducted in accordance with AASHTO T136, *Freezing-and-Thawing Tests of Compacted Soil-Cement*. Instead of brushing and measuring mass loss, UCS tests are conducted after each cycle of freezing-thawing to determine the effects of freezing-thawing on the strength/modulus. However, the applicability of this test to lightly stabilized materials needs to be evaluated.

Vacuum Saturation Test: Vacuum saturation is recommended as a Level 2 test for durability. The residual strength values of specimens after vacuum saturation are found to correlate well with the residual strength values of specimens after freeze-thaw and wet-dry cycles. Residual strength, the UCS after vacuum saturation, is determined. The ASTM C539 test standard is followed for this test.

Table B-5. Appraisal of Durability Tests

	Wetting and Drying	**Freezing and Thawing**	**Tube Suction**	**Vacuum Saturation**
Performance Predictability	**Good** (simulates field conditions)	**Good** (simulates field conditions)	**Average** (strong correlation with brush test (Syed and Scullion 2001) and links to field performance of durability (Scullion et al. 2005))	**Good** (correlation with residual strength and moisture content after vacuum and 5 F-Z cycles)
Precision	**Poor** (brushing test has problems of poor repeatability and reproducibility (Scullion et al. 2005), due to the susceptibility of brushing technique to operator variability and loss of single large aggregate particles (Ventura 2003))	**Poor** (not repeatable) (Ventura 2003)	**Average** (repeatable at high DV, but not repeatable below DV of 10; the number of replicates required depends upon the selection of reliability and tolerance, as well as specified maximum dielectric values) (Guthrie et al. 2001)	**Average (**16.8% COV**)** (Guthrie et al. 2008)
Accuracy	**Good** (direct measurement; evaluates durability based on weight loss, which is overly severe)	**Good** (direct measurement; mass loss only may cause misleading assessment of the results)	**Average** (correlates well with W-D and F-Z results; the maximum dielectric value criterion, less than 10 for good base materials, seems to be more conservative than the soil-cement loss criterion specified in W-D durability tests; can be used as screening tool for mix design) (Guthrie et al. 2001)	**Good** (significant correlation between vacuum saturation strength and cyclic freeze-thaw strength.) (Dempsey and Thompson 1973)
Practicality	**Poor** (length – 1 month)	**Poor** (length –1 month)	**Good** (easy to perform and nondestructive)	**Good** (rapid and economical method) (Guthrie et al. 2008)
Machine Costs	**Low**	**Low**	**Very low**	**Very low**
Time	**Long** (approximately 1 month)	**Long** (approximately 1 month)	**Long** (10 to 14 days)	**Short** (a few hours)
Recommendation	**Recommended** for model development and Level 1	**Recommended** for model development and Level 1	**Not recommended** because it only provides an index and cannot be used for modeling	**Recommended** for Level 2

Note: Cost Ratings: High: >$50,000; Medium: $20,000 to $50,000; Low: <$20,000; Very Low: <$5,000

Erodibility Tests

The erodibility of CSM can be assessed using the rotational shear device (RSD), manual wet-dry brushing, wheel tracking, jetting, a vibration table, and/or cyclic impact erosion. Table B-6 presents the appraisal and recommendations for the erodibility tests.

Rotation Shear Device (RSD) Test: The RSD is recommended for laboratory evaluation for erosion model development because it can accurately measure shear stress.

Cyclic Impact Erosion (CIE) Test: The CIE test is recommended for erosion model development. It can simulate field conditions by combining dynamic traffic loads and erosion by water. It is simple to set up, and the load level can be adjusted easily.

Fatigue Tests

The fatigue of CSM can be characterized by flexural beam fatigue, IDT fatigue or large-scale model experiment. Table B-7 presents the appraisal of the fatigue tests.

Flexural Beam Fatigue Test: Flexural beam fatigue testing is recommended for the fatigue model development, due to the bending of CSL in the field. There is no standard for the flexural beam fatigue test; however, the Australian flexural beam fatigue test (Yeo 2008) can be modified for this test. This test is evaluated in terms of the load level used to induce fatigue damage for different materials.

Large-Scale Model Experiment (LSME): This test is recommended for fatigue model validation because it realistically simulates field conditions in terms of size.

Table B-6. Evaluation of Erodibility Tests

	Rotational Shear Device (RSD)	Manual Wet-Dry Brushing	Wheel Tracking	Jetting	Vibration Table	Cyclic Impact Erosion (CIE)
Performance Predictability	**Good** (simulates erosion under slab (Ras and Visser 2004) and field conditions (Van Wijk 1985)	**Good** (correlates with the critical shear stress of materials, but not with erosion test results) (Gass et al. 1993)	**Good** (simulates field conditions) (Scullion 2005)	**Good** (simulates field conditions)	**Average** (does not simulate field conditions)	**Good** (simulates field conditions, considers dynamic traffic effect) (Sha and Lu 2002)
Precision	**Unknown**	**Poor** (poor repeatability and reproducibility due to brushing technique) (De Beer 1994)	**Poor** (high variation) (De Beer 1994)	**Unknown**	**Unknown**	**Good**
Accuracy	**Good** (applied shear stress and critical shear stress of materials can be accurately determined) (De Beer 1994)	**Average** (highly dependent on brushing; potential error in the mass loss results owing to local cemented surfaces produced during compaction)	**Good** (direct measurement)	**Poor** (shear stress cannot be accurately determined, as the stress distribution is not uniform)	**Unknown**	**Good** (easy to adjust the load level) (Sha and Lu 2002)
Practicality	**Average** (index is based on mass loss, which could lead to incorrect interpretation of erodibility potential) (De Beer 1994)	**Good** (simple, but difficult to control the force) (De Beer 1994)	**Poor** (cumbersome to make and handle specimens) (De Beer 1994)	**Good** (simple to run) (Ras and Visser 2004)	**Good** (easy to operate)	**Good** (easy to operate)
Machine Costs	**Low**	**Low**	**High** (Scullion 2005)	**Low** (Ras and Visser 2004)	**Low**	**Low**
Time	**Short** (10 minutes) (Van Wijk 1985)	**Long** (W-D 1 month)	**Long**	**Short**	**Short**	**Short** (test is run until modulus value drops to 50% of its initial value)
Recommendation	**Recommended** for model development	**Not Recommended** because accurate shear stress cannot be measured for modeling	**Not recommended** due to machine costs	**Not recommended** due to inaccurate shear stress measurements	**Not recommended** due to inaccurate stress measurements	**Recommended** for model development

Note: Cost Ratings: High: >$50,000; Medium: $20,000 to $50,000; Low: <$20,000; Very Low: <$5,000

Table B-7. Evaluation of Fatigue Tests

	Flexural Fatigue	IDT Fatigue	Large-Scale Model Experiment
Performance Predictability	**Good** (simulates field bending conditions) (Yeo et al. 2002)	**Good** (simulates tensile behavior)	**Good** (large-scale pavement model)
Precision	**Average**	**Average**	**Unknown**
Accuracy	**Good** (direct measurement)	**Good** (indirect measurement of tensile fatigue)	**Accurate** (direct measurement)
Practicality	**Average** (hard to make and handle specimens; average repeatability;) (Yeo et al. 2002)	**Good** (relatively practical and user friendly; stress/strain control is difficult with brittle stabilized materials; suitable for lightly stabilized materials) (Yeo et al. 2002)	**Poor** (large-scale layer preparation is laborious. Boundary conditions and applied cyclic loading must be carefully chosen to replicate field conditions; requires large-scale mixing operations and curing conditions similar to those encountered in the field construction and therefore incorporates this important factor for CSL; provides realistic simulation)
Machine Costs	**Medium**	**Medium**	**High**
Recommendation	**Recommended** for bottom-up fatigue model development	**Recommended** for fatigue model	**Recommended** for model validation only

Note: Cost Ratings: High: >$50,000; Medium: $20,000 to $50,000; Low: <$20,000; Very Low: <$5,000

Shrinkage Tests

The shrinkage of CSM is evaluated for shrinkage cracking development. Table B-8 presents the appraisal and recommendations for the tests procedures.

Free Drying Shrinkage Test: This test is recommended for the free drying shrinkage strain prediction model development for the shrinkage cracking prediction. There is no standard for the free drying shrinkage of CSM; however, AASHTO T160, *Length Change of Hardened Hydraulic Cement*, can be modified for CSM. Long-term drying also is needed to measure the ultimate drying shrinkage strain and develop the ultimate shrinkage strain prediction model.

Free Autogenous Shrinkage Test: This test is recommended for measuring the autogenous shrinkage strain. During the test, the specimen should be sealed well to prevent any loss of moisture so that the shrinkage strain is due only to the hydration of the cementitious binder. There is no standard for the autogenous drying shrinkage of CSM. This test is developed and evaluated by the team.

Gradient Drying Shrinkage Test: This test is recommended for the drying shrinkage strain gradient model development for the shrinkage cracking prediction. This test can simulate field conditions, as the four sides of the surface of the beam specimen are sealed and only the top surface is subjected to drying. The shrinkage strain at different depths is measured to represent the shrinkage gradient in the field. There is no test procedure for the gradient drying shrinkage of CSM. This test is developed and evaluated by the team.

Restrained Drying Shrinkage Test: This test is recommended to induce shrinkage cracking for model development. The increase in friction between the CSL and underlying material will reduce the shrinkage crack spacing and crack width. By artificially creating a high level of bond, shrinkage cracking can be generated in a lab specimen for model development. This method is evaluated in terms of size, base restraint, relative humidity (RH) and temperature so that cracking occurs in laboratory-scale beams. There is no test standard for restrained drying shrinkage cracking testing. However, concrete researchers have conducted restrained slab tests at the base layer (Weiss et al. 1998). The slab is fixed to the base and subjected to drying to generate shrinkage cracking in a lab-scale specimen. These test procedures developed by Weiss et al. (1998) are modified for laboratory evaluation in this study.

Thermal Shrinkage/Expansion Test: The thermal shrinkage/expansion property is recommended as Level 1 input to determine thermal strain, which is a key parameter in the shrinkage cracking model. AASHTO TP 60, *Standard Method of Test for Coefficient of Thermal Expansion of Hydraulic Cement Concrete*, is designed to determine the coefficient of thermal expansion

(COTE) of concrete only. This test is developed and evaluated by the team. The method developed by Cusson and Hoogeveen (2006) is evaluated for CSM.

Friction/Bond

Even though the interface friction/bond is not a property of CSM, it significantly affects shrinkage cracking and thus is an important parameter in the shrinkage cracking model. Typical crack spacing in the field is between 3 ft and 60 ft, which is difficult to achieve in the lab. Therefore, the shrinkage cracking model is based on the large-scale restrained drying shrinkage test conducted in the lab. Epoxy and cement slurry are proposed to create a strong bond between the CSL and mold base so that shrinkage cracking can be generated within the lab specimen. The bond strength is characterized in accordance with the Iowa shear test (Iowa DOT 2000), which separates the CSL and base along the interface.

After the placement of the CSL in the field, the bond between the CSL and underlying material is typically strong, such that bond failure often occurs in the weak material between the CSL and underlying materials (Romanoschi and Metcalf 2001). The friction/bond input can be based on the shear strength of these weak materials, such as sand and gravel, which are readily available for use in the shrinkage cracking model.

Table B-8. Appraisal of Test Methods for Shrinkage and Swelling

	Free Drying Shrinkage	**Autogenous Drying Shrinkage**	**Gradient Drying Shrinkage**	**Restrained Drying Shrinkage**	**Thermal Shrinkage/Expansion**
Performance Predictability	**Good** (key parameter in shrinkage cracking model)	**Good** (key parameter in shrinkage cracking model)	**Good** (key parameter in shrinkage cracking model)	**Good** (models behavior of restrained drying shrinkage cracking)	**Good** (key parameter for thermal shrinkage prediction)
Precision	**Unknown**	**Unknown**	**Unknown**	**Unknown**	**Unknown**
Accuracy	**Good** (direct measurement)	**Good** (direct measurement)	**Good** (direct measurement)	**Good** (direct measurement; fixed base creates shrinkage cracking in a lab-scale slab)	**Good** (direct measurement)
Practicality	**Unknown**	**Unknown**	**Unknown**	**Average** (application of bond and specimen is time-consuming)	**Good**
Machine Costs	**Very low**	**Very low**	**Very low**	**Low**	**Low**
Time	**Long**	**Long**	**Long**	**Long**	**Average**
Recommendation	**Recommended** for unrestrained shrinkage strain prediction model development	**Recommended** for unrestrained shrinkage strain prediction model development	**Recommended** for unrestrained shrinkage strain prediction model development	**Recommended** for shrinkage cracking prediction model development	**Recommended** for thermal cracking strain prediction

Note: Cost Ratings: High: >$50,000; Medium: $20,000 to $50,000; Low: <$20,000; Very Low: <$5,000

CHAPTER B-3. LABORATORY TEST PROCEDURE EVALUATION

The team has identified the test procedures shown in Table B-9 that are evaluated for their applicability to the material characterization or model development. Proper selection of test procedures to characterize the material properties is important and needs to consider the application of a specific material in a pavement structure. One heavily stabilized material, i.e. cement-stabilized gravel, and one lightly stabilized material, i.e. lime-stabilized clay, are selected to evaluate the test procedures. Both materials are widely used by state highway agencies, based on the survey results.

Table B-9. Test Procedures Selected for Evaluation

Test Procedures for Evaluation	Evaluation Variables	Number of Tests
IDT modulus	Stress levels: two levels	1 mixtures×2 tests×3 replicates=6
Flexural modulus	Stress levels: two levels	2×2×3=12
Beam/IDT fatigue	Three stress levels: 65%-85% of strength	2×3×3=18
Seismic modulus/ultrasonic pulse velocity test	Precision and reliability: 10 measurements	2×10×3=60
Wet-dry	Curing time for soil-lime: 7 days accelerated curing	2×3×3=18
Freeze-thaw	Curing time for soil-lime: 7 days accelerated curing	2×3×3=18
Vacuum saturation	Curing: soil-lime: 7 days accelerated curing soil-cement: 7 days curing at 68°F	2×2×3=12
Coefficient of thermal expansion	Applicability	2×1×3=6
Bond strength	Bond conditions: epoxy and cement slurry	2×2×3=12
Free autogenous shrinkage	Free autogenous shrinkage at 68°F	2×1×3=6
Free drying shrinkage	Free drying shrinkage at relative humidity: 65% at 68°F and ultimate shrinkage after long-term curing	2×2×3=6
Gradient drying shrinkage	Gradient drying shrinkage at relative humidity: 40% and 65% at 68°F	2×1×1=2
Restrained drying shrinkage	Slab length: 48 in. and two bond conditions: epoxy and cement slurry	2×2×1=4
Rotational shear device (RSD)	Rotation Speed: 500, 1500, 2500 RPM	2×3×3=18
Cyclic impact erosion (CIE)	Displacement levels: two levels	2×2×3=12
Total		**210**

Flexural/IDT Modulus Test: Flexural modulus test is conducted for the cement-gravel, and IDT modulus test is conducted for the clay-lime. According to the Australian test standard, the maximum stress levels should not be more than 50% and 40% of the strength for the IDT and flexural tests, respectively. Therefore, the two stress levels for the IDT modulus are 20% and 30% of the IDT strength for the IDT modulus tests and 20% and 40% of the MOR for the flexural modulus tests.

Beam/IDT Fatigue Test: The fatigue tests are evaluated in terms of the stress levels to be applied. Australian researchers recommend that the stress levels should be less than 60%~90% and 80% of the flexural and IDT strength values, respectively, to avoid immediate damage to the specimen upon application of the load. Therefore, 60%, 70%, and 80% of flexural and IDT strength values are evaluated.

Seismic/Acoustic Modulus Test: Seismic modulus tests are conducted on two materials with 10 measurements for each of three replicates to determine repeatability.

Wet-Dry/Freeze-Thaw Durability Tests: Due to the low strength of lightly stabilized materials, the curing of lightly stabilized materials needs to be extended. Seven days of accelerated curing at 104°F is adopted.

Coefficient of Thermal Expansion (COTE) Test: The applicability of AASHTO TP60 to CSM is evaluated. AASHTO TP60 specifies the use of a water bath for temperatures between 50°F and 122°F. Unlike concrete, CSM are sensitive to water, especially for lightly stabilized materials. A temperature cycling method is evaluated.

Bond Strength Test: Bond strength test is conducted in accordance with the Iowa shear test (Iowa DOT 2000). Two CSL are bonded to the metal base to be used for the restrained drying shrinkage test, using epoxy and cement slurry. The bond strength and displacement values that correspond to the peak load are recorded.

Free Autogenous Shrinkage Test: This test is evaluated at room temperature (68°F) for both heavily and lightly stabilized materials. The specimens are sealed with wax to prevent any evaporation.

Free Drying Shrinkage Test: This test is evaluated in terms of relative humidity (RH) for curing. AASHTO T160 specifies 50% RH during evaluation. For this study, a different curing RH is evaluated: 65% at 68°F. The 65% RH is equivalent to field bituminous curing.

Gradient Drying Shrinkage Test: This test is evaluated in terms of RH for curing. For this study, two RH values, 40% and 65%, are evaluated.

Restrained Drying Shrinkage Test: This method is evaluated in terms of beam length, the bonding condition between the beam and underlying material, and RH. Typical shrinkage crack spacing for CSL is between 3 ft and 60 ft. If the bond strength is increased, the shrinkage crack spacing can be reduced to that for a lab-scale beam. Two bonding agents are used, epoxy and cement slurry, which create a strong bond (Atkinson 1990). The specimen length greatly affects

shrinkage cracking. The restrained slab at the base is recommended to be 4 in. × 6 in. × 48 in. to create shrinkage cracking. The restrained slab is subjected to 65% RH drying at room temperature.

Rotational Shear Device Test: The RSD test is evaluated in terms of speed and consistency. Three speeds have been used previously by other researchers: 500, 1500, and 2500 revolutions per minute (RPM). The applicability of these speeds is evaluated in terms of erosion depth and mass loss.

Cyclic Impact Erosion (CIE) Test: The CIE test is evaluated in terms of displacement level. Two displacements are evaluated in terms of erosion depth and modulus reduction.

Indirect Tensile (IDT) Test

This investigation was conducted in order to (i) determine the indirect tensile strength, (ii) determine an appropriate stress level for the IDT modulus test, and (iii) characterize fatigue behavior under different stress levels for IDT specimens using a lightly stabilized mixture (clay-lime). The dimensions of the cylindrical specimens used for this study are 6 inches in diameter and 3.1 inches in length. The specimens were kept in an environment of 68°F and 65% RH for 14 days prior to testing.

(a) Indirect Tensile Strength (IDT) Test

The Australian method (Midgley and Yeo 2008) for determining the IDT strength of stabilized materials was used with modifications. The IDT strength tests of moisture-cured specimens were performed within 30 minutes after removing the specimens from the curing room. The IDT strength test involves loading a cylindrical specimen with compressive loads distributed along two axial lines that are diametrically opposite, as shown in Figure B-6. Two loading strips that are slightly longer than the specimen were provided for each specimen. The width of the loading strips is 0.75 inch. This condition results in a relatively uniform tensile stress perpendicular to and along the loading diametric plane. Failure occurs due to splitting along this loaded plane.

Figure B-6. Indirect Tensile (IDT) Strength Test Setup

The loading rate for the IDT strength test is 15 psi/min. The IDT strength value is computed using Eq. (B-1).

$$S_{IDT} = \frac{2P}{\pi l d} \qquad \text{Eq. (B-1)}$$

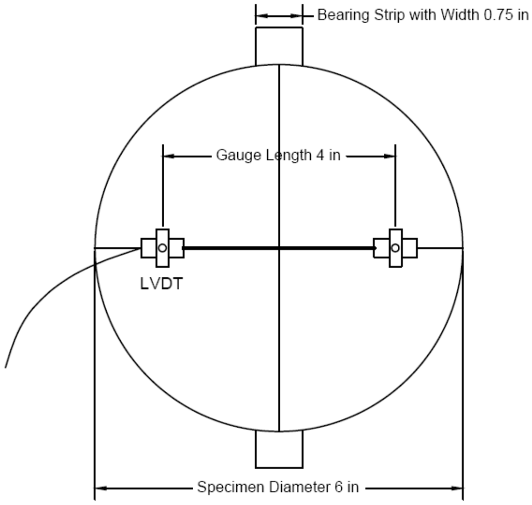
where

S_{IDT} = IDT strength, psi
P = maximum applied load, lb
l = height of the specimen, in.
d = diameter of the specimen, in.

(b) IDT Modulus and Fatigue Test

The same setup as used for the IDT strength test was followed for IDT modulus testing. Internal linear variable differential transformers (LVDTs) were fixed on both sides of the cylinder to measure the horizontal displacement, as shown in Figure B-7. For fine materials, LVDTs were mounted on the specimen using screws, whereas epoxy was used to mount the LVDTs for the granular materials. The gauge length is 4 inches in the horizontal direction.

(a) IDT Modulus Test Setup at WSU



(b) Diagrammatic View for IDT Modulus Test

Figure B-7. IDT Modulus Test Setup

The IDT modulus test was conducted at a frequency of 1 Hz. A cyclic haversine load pulse of 100 ms duration followed by a 900 ms rest period was applied for each cycle. A contact load of approximately 10 lbs was applied to the specimen. Cyclic haversine loading was applied for 100 load pulses. Both the maximum force applied to the specimen and the peak displacement for each pulse cycle were recorded. The first 90 cycles were considered preconditioning. The data from the last 10 consecutive cycles were used to calculate the IDT modulus value of the specimen based on Eqs. (B-2) through (B-4):

$$\sigma = \frac{2P}{\pi l d} \qquad \text{Eq. (B-2)}$$

$$\varepsilon = U\frac{\gamma_1+\gamma_2 \upsilon}{\gamma_3+\gamma_4 \upsilon} \qquad \text{Eq. (B-3)}$$

$$E_t = \frac{\sigma}{\varepsilon} \qquad \text{Eq. (B-4)}$$

where

σ = tensile stress, psi
P = maximum applied load indicated by the testing machine, lbs
l = height of the specimen, in.
d = diameter of the specimen, in.
U = horizontal displacement, in.
ε = cyclic horizontal tensile strain
v = Poisson's ratio, assuming 0.2 for CSM
E_t = indirect tensile modulus, psi
γ_1, γ_2, γ_3, γ_4 = constants.

Table B-10 shows the values of γ_1, γ_2, γ_3, γ_4 when the load strip width is 0.75 inch. For different diameters, gauge lengths, and load strip widths, γ can be found in Wen and Kim (2002).

Table B-10. Parameters for IDT Modulus Test

γ_1	γ_2	γ_3	γ_4
12.27	37.34	0.7781	2.7269

According to the Austroads fatigue test protocol (Midgley and Yeo 2008), the fatigue life is defined as the number of cycles applied to the specimen to reduce the modulus value to half of the initial modulus value. IDT fatigue testing was conducted at 75% of the IDT strength.

Tables B-11, B-12, and B-13 summarize the results of the IDT tests for IDT strength, modulus, and fatigue, respectively. The IDT strength of the clay-lime specimens is 13.9 psi with good repeatability. The averaged IDT modulus values were obtained as 110,611 psi and 102,000 psi for the stress levels of 20% and 30% of IDT strength, respectively. The 20% and 30% stress levels resulted in comparable modulus values. However, during the test, it was noted that the 20% results were affected by the specimen setup and signal noise from the machine. Thus, 30% IDT strength is recommended for IDT modulus testing. Table B-13 presents the IDT fatigue test results. There are significant variations in terms of fatigue life and modulus reduction. The LVDT mounts may have come loose during the fatigue tests, leading to significant errors. Thus, the IDT strength and modulus values are recommended to determine the tensile strength and stiffness of the CSM. However, the IDT fatigue test is not suitable for CSM.

Table B-11. Summary of IDT Strength Test Results

Test Item	Specimen No.	IDT Strength (psi)		Coefficient of Variation (COV)
IDT Strength	S_{IDT}-1	14.5	13.9	3.67%
	S_{IDT}-2	13.8		
	S_{IDT}-3	13.5		

Table B-12. Summary of IDT Modulus Test Results

Specimen No.	Stress Level by Percentage of IDT Modulus, %	IDT Modulus (psi)		COV
S_{IDT}-20-1	20	117014	110611	9.09%
S_{IDT}-20-2	20	115795		
S_{IDT}-20-3	20	99025		
S_{IDT}-30-1	30	107394	102000	7.99%
S_{IDT}-30-2	30	105974		
S_{IDT}-30-3	30	92631		

Table B-13. Summary of IDT Fatigue Test Results

Test Item	Specimen No.	Stress Level (%)	Initial IDT Modulus (ksi)	IDT Modulus at End of Fatigue Test (ksi)	Percent IDT Modulus at End of Fatigue Test (%)	Number of Cycles at End of Fatigue Test
IDT Fatigue	FT-75-1	75	161	19	12	12
	FT-75-2	75	498	637	128	5916
	FT-75-3	75	213	190	89	5921

Flexural Beam Tests

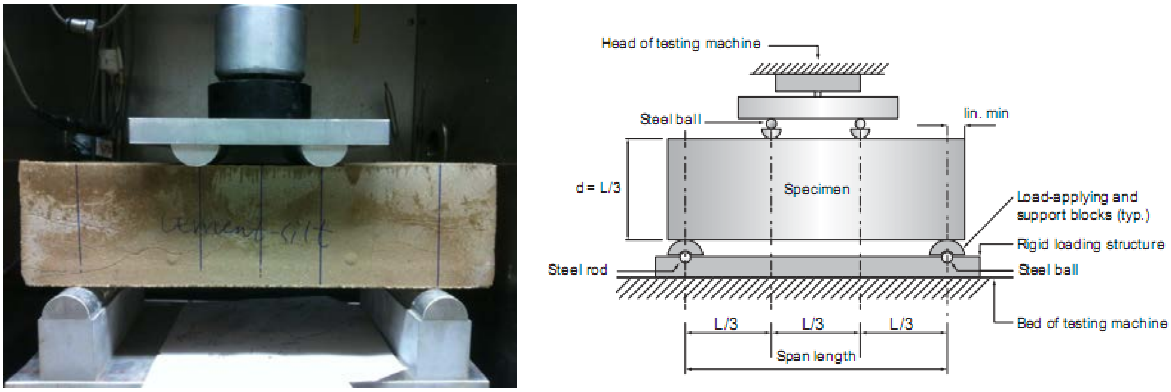
Flexural beam tests were evaluated for their ability to (i) determine MOR, (ii) determine an appropriate stress level for flexural modulus testing, and (iii) characterize fatigue behavior under different stress levels for the beam specimens produced using stabilized mixtures (gravel-cement and clay-lime).

Prismatic molds with dimensions of 4 in. × 4 in. × 15.75 in. were used to fabricate beam specimens composed of gravel-cement. The gravel-cement materials were compacted using a modified compaction method in accordance with AASHTO T180, *Standard Method of Test for Moisture-Density Relations of Soils Using a 4.54-kg (10-lb) Rammer and a 457-mm (18-in.) Drop*. The specimens were covered and allowed to cure in the molds for 48 hours at 70°F. The specimens were then removed from the molds, wrapped in plastic sheets, and kept in a curing room with 100% RH at 70°F for 28 days.

The clay-lime beam specimens were fabricated with 5% lime. The mixed material was compacted using the standard compaction method found in AASHTO T99, *Standard Method of Test for Moisture-Density Relations of Soils Using a 2.5-kg (5.5-lb) Rammer and a 305-mm (12-in.) Drop*. The specimens were allowed to cure in the molds for 3 days at 70°F and were covered to prevent moisture evaporation. Then, the specimens were subjected to 4-day accelerated curing at 158°F, because lightly stabilized materials are weak and the beam can be broken during handling without accelerated curing.

(a) Modulus of Rupture (MOR) Test

In the MOR test, a beam specimen is subjected to a bending load at a constant stress rate until failure, as shown in Figure B-8. MOR tests of the specimens were performed within 30 minutes of removal from the moisture room. The span between steel supports (L) is 12 inches. Then, the average width and depth of the specimens were measured prior to testing.



(a) MOR Test Setup at WSU

(b) Diagrammatic View of MOR (Mehta and Monteiro 2006)

Figure B-8. Modulus of Rupture Test Setup

The Australian Austroad test procedure for MOR testing (Midgley and Yeo 2008) was followed for this study. Beam specimens were tested using a servo-hydraulic machine. A constant loading rate was applied such that the extreme fiber stress at the bottom of the beam was within the limits of 100 ± 5 psi/min. The MOR is calculated by Eq. (B-5).

$$MOR = \frac{PL}{bd^2} \qquad \text{Eq. (B-5)}$$

where

MOR = modulus of rupture, psi
P = maximum applied load, lbs
L = span length, in.
b = average width of specimen, in.
d = average depth of specimen, in.

(b) Flexural Modulus and Fatigue Test

The flexural modulus test simulates the bending conditions which is a dynamic and small-strain modulus which represent the field performance of CSL. The use of flexural modulus tests is in line with the MOR tests recommended previously for strength.

Figure B-9 presents the flexural modulus test setup. The same setup as the MOR is used to conduct the flexural modulus and fatigue tests. A pair of LVDTs was used to measure the vertical displacement at the midpoint of the beam.

Figure B-9. Flexural Modulus Test Setup

The flexural modulus test was conducted at a frequency of 1 Hz. A cyclic haversine load pulse of 250 ms duration followed by a 750 ms rest period were applied for each cycle. A contact load less than 10 lbs was applied to the specimen. Cyclic haversine loading was applied for 100 load pulses. The maximum force and the peak displacement for each pulse cycle were recorded. The first 50 cycles were considered preconditioning. The data from the second 50 consecutive cycles were used to calculate the flexural modulus of the specimen using Eq. (B-6).

$$E_f = \frac{23 \cdot P \cdot L^3}{108 \cdot b \cdot d^3 \cdot \delta h} \times 1000 \qquad \text{Eq. (B-6)}$$

where

E_f = flexural modulus, MPa, 1 MPa = 145 psi
P = peak force, kN, 1kN = 225 lbs
L = beam span, mm, 1 mm = 0.03937 in.
b = specimen width, mm
d = specimen height, mm
δ_h = peak mid-span displacement, mm

The average of the modulus values during the second 50 cycles was considered as the flexural modulus value of the specimen. Flexural fatigue tests were then carried out at stress levels of 70%, 75%, and 80% of the MOR.

Tables B-14, B-15, and B-16 provide summaries of the results of the MOR, flexural modulus, and flexural fatigue tests, respectively. The MOR of the gravel-cement specimens was 113 psi with a relatively low coefficient of variation (COV). The average flexural modulus values of the gravel-cement were obtained as 84,100 psi and 131, 259 psi for the stress levels of 20% and 40% MOR, respectively. Further work used a gravel-cement mixture with 3% cement

content. In order to observe the dependence of the modulus on the stress level, measurements were taken at stress levels of 20%, 30%, and 40 percent. The specimens were tested at the curing ages of 14 days and 28 days, respectively. Table B-17 presents the flexural test results. Similar to the earlier results, 20% stress level results in a lower flexural modulus when compared to those at 30% and 40% stress levels. The 30% and 40% stress levels yield comparable results. The 20% stress level is not recommended because the load level is low and the accuracy of the response is affected by the signal noise of the machine. Because the 40% stress level increases the potential for damage to the specimen, it is recommended that the flexural modulus test should be conducted at 30% stress level.

Figure B-10 shows a typical degradation of the flexural modulus during a beam fatigue test of the gravel-cement material. Based on the stress level, the flexural fatigue test was designed originally to observe failure near 10,000 cycles or three hours, based on the Austroads test protocol (Midgley and Yeo 2008). However, fatigue tests of the specimens performed at 70% stress continued for more than 20,000 cycles without failure and then were terminated. The results show that flexural fatigue behavior is highly sensitive to the level of stress. Even 5% change in stress level can significantly affect the number of cycles to failure. In addition, the fatigue test results are highly variable at each stress level, which has been reported by previous researchers (Midgley and Yeo 2008).

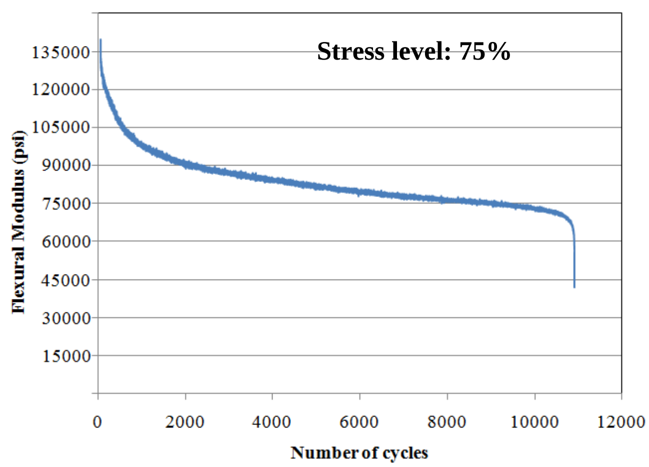


Figure B-10. Typical Degradation of Flexural Modulus in Fatigue Test of Gravel-Cement

Table B-14. Summary of MOR Test Results (Gravel-Cement 4%)

Test Item	Specimen No.	MOR (psi)		Coefficient of Variation (COV)
Modulus of rupture	GC-FS-1	123	113	7.59%
	GC-FS-2	112		
	GC-FS-3	106		

Table B-15. Summary of Flexural Modulus Test Results (Gravel-Cement 4%)

Specimen No.	Stress level by percentage of MOR (%)	Flexural modulus (psi)		COV
GC-FM-20-1	20	76,705	84,100	10.84%
GC-FM-20-2	20	81,200		
GC-FM-20-3	20	94,250		
GC-FM-40-1	40	121,365	131,225	8.81%
GC-FM-40-2	40	128,470		
GC-FM-40-3	40	143,985		

Table B-16. Summary of Flexural Fatigue Test Results (Gravel-Cement 4%)

Test Item	Specimen No.	Stress Level (%)	Initial Flexural Modulus (psi)	Flexural Modulus at End of Fatigue Test (psi)	Percent Flexural Modulus at End of Fatigue Test (%)	Number of Cycles at End of Fatigue Test	COV of Fatigue Life (%)	Note
Flexural Fatigue	GC-FT-70-1	70	147503	79336	48	28707	23.0%	Not yet failed
	GC-FT-70-2	70	157511	85137	54	20681		Not yet failed
	GC-FT-75-1	75	130389	62656	48	389	103.2%	
	GC-FT-75-2	75	125748	44091	35	12895		
	GC-FT-75-3	75	146053	68603	47	1052		
	GC-FT-75-4	75	135030	40466	30	10898		
	GC-FT-80-1	80	177091	62801	35	3139	160.8%	
	GC-FT-80-2	80	143007	94275	66	58		
	GC-FT-80-3	80	173465	107618	62	99		

Table B-17. Summary of Flexural Test Results (Gravel-Cement 3%)

Specimen No.	Stress Level (%)	Mixture: Gravel-Cement Binder Content: 3% Curing: 14-day Standard		Mixture: Gravel-Cement Binder Content: 3% Curing: 28-day Standard	
		Modulus of rupture: 86 psi		Modulus of rupture: 106 psi	
		Flexural Modulus (psi)	Average (psi)	Flexural Modulus (psi)	Average (psi)
GC-**FT-**20-1	20	112,259	102,252	152,870	133,000
GC-**FT-**20-2		92,244		112,984	
GC-**FT-**30-1	30	162,007	174,335	201,892	210,740
GC-**FT-**30-2		186,518		219,442	
GC-**FT-**40-1	40	164,328	166,358	224,808	222,633
GC-**FT-**40-2		168,389		220,312	

Tables B-18, B-19 and B-20 provide summaries of the flexural test results for the clay-lime materials for the MOR, flexural modulus and flexural fatigue tests, respectively. The average MOR is 44 psi. The flexural modulus values are 65,847, 64,252, and 69,328 psi at stress levels of 20%, 30% and 40%, respectively. The clay-lime mixture shows more consistent flexural modulus results at the different stress levels than the gravel-cement mixture shows. In order to be consistent with the stress level for the flexural modulus of heavily stabilized materials, a stress level of 30% is recommended. For the fatigue tests, the stress levels range between 65% and 85%, as shown in Table B-20. The specimens tested at the stress level of 65% were still intact at the end of 100,000 cycles. Increasing the stress level significantly decreases the number of cycles to failure. The results indicate that the flexural modulus/fatigue test is a viable method for lightly stabilized materials as well, but has a relatively high variability as well. The flexural modulus and fatigue tests are recommended because they are the only tests available to simulate the bending of CSL in the field.

Table B-18. Summary of MOR Test Results (Clay-Lime 5%)

Test Item	Specimen No.	MOR (psi)	
MOR	CL-FS-1	46	44
	CL-FS-2	41	

Table B-19. Summary of Flexural Modulus Test Results (Clay-Lime 5%)

Test Item	Specimen No.	Stress Level (%)	Flexural Modulus (psi)	
Flexural Modulus	CL-FM-20-1	20	49603	65847
	CL-FM-20-2	20	82091	
	CL-FM-30-1	30	58595	64179
	CL-FM-30-1	30	69763	
	CL-FM-40-1	40	65992	69328
	CL-FM-40-2	40	72664	

Table B-20. Summary of Flexural Fatigue Test Results (Clay-Lime 5%)

Test Item	Specimen No.	Stress Level (%)	Initial Flexural Modulus (psi)	Flexural Modulus at End of Fatigue Test (psi)	Percent Flexural Modulus at End of Fatigue Test (%)	Number of Cycles at End of Fatigue Test	Note
Flexural Fatigue	**CL-FT-65-1**	65	75130	-	-	>100000	Not yet failed
	CL-FT-65-2	65	43656	-	-	>100000	Not yet failed
	CL-FT-75-1	75	71359	37275	52	1442	
	CL-FT-75-2	75	51343	26977	53	2726	
	CL-FT-85-1	85	-	-	-	3	
	CL-FT-85-2	85	62366	44382	71	621	

Stress-Wave Base Modulus Test

The aim of the stress-wave base modulus test is to evaluate the effectiveness and reproducibility of the seismic modulus and ultrasonic pulse velocity tests as surrogate modulus tests. The evaluation of the seismic modulus test and ultrasonic pulse velocity test was performed on a gravel-cement mixture and a clay-lime mixture representing heavily stabilized and lightly stabilized soil systems, respectively. Prismatic specimens with dimensions of 4 in. × 4 in. × 4.5 in. were used for the gravel-cement mixture. The clay-lime mixture was molded to produce 4 in. (diameter) × 4.6 in. (height) cylindrical specimens. The testing age of the specimens was 28 days. Figure B-11 presents views of the test specimens.

(a) Clay-Lime (b) Gravel-Cement

Figure B-11. View of Stress-Wave Base Modulus Test Specimens

(a) Impact Resonance Test Method

The impact resonance test method is a simple and nondestructive test for determining the modulus of pavement materials. The procedure allows the velocity of the elastic wave that propagates through a specimen to be measured. Depending on the dimensions and the stiffness of the specimen, the energy associated with one or more frequencies is trapped and magnified (resonated) as it propagates within the specimen. In the impact resonance method, a supported specimen is struck with a small impact device, and the specimen response is measured by a lightweight accelerometer on the specimen. Figure B-12 shows the free-free resonant column (FFRC) test setup used for this test method. Given the specimen's mass density (ρ), travel time (t) and length (L), the seismic modulus (E) can be determined based on the p-wave velocity (V_p), as shown in Eq. (B-7).

$$E = \rho(L/t)^2 = \rho(V_p)^2 \quad \text{Eq. (B-7)}$$

where V_p is the wave velocity.

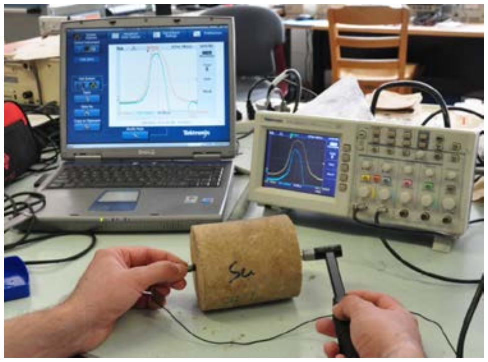


Figure B-12. Seismic Modulus Test Setup

(b) Ultrasonic Test Setup

An ultrasonic test instrument (Pundit Plus) that has a function to measure the elastic modulus was evaluated. In this method, pulses of longitudinal stress waves are generated by an electro-acoustical transducer that is held in contact to one end surface of the specimen. After traversing through the specimen, the pulses are received and converted into electrical energy by the second transducer that is in contact with the opposite face of the specimen, as shown in Figure B-13. The ultrasonic tester readily determines the modulus of the specimen based on travel time, path length travelled, and density of the CSM.

Figure B-13. View of the Ultrasonic Test Setup

Ten measurements were taken for each of the seismic tests and ultrasonic tests. This study reveals statistically significant differences and poor reproducibility in the results obtained using the FFRC test setup. The significance of difference in the results is obviously higher for the gravel-cement specimens, as shown in Figure B-14. The seismic modulus values obtained from the FFRC test range between 1,000 ksi and 7,000 ksi for one material, whereas the constrained modulus values determined by the ultrasonic test setup range between 3,000 ksi and 3,800 ksi for gravel-cement. As shown in Figure B-15, the seismic modulus values obtained from the FFRC test range between 900 ksi and 1,600 ksi for the clay-lime specimens, whereas the constrained modulus values determined by the ultrasonic test are again quite consistent, ranging between 500 ksi and 600 ksi. Thus, the ultrasonic pulse velocity test can provide more repeatable results than the FFRC test.

However, the moisture content of CSM can affect the results of the ultrasonic test method, which is also reported by Fratta et al. (2005) and shown in Figure B-16. The p-wave velocity decreases with increasing saturation and becomes nearly constant for a range of degrees of saturation. When the saturation level is close to 100% (i.e. after prolonged soaking), the constrained modulus value becomes very high, which indicates that the p-wave traveling through the water phase. After increasing the moisture content by long-time soaking, the measured

constrained modulus increases for all four types of mixtures, as shown in Figure B-17. Therefore, this study concludes that the FFRC and ultrasonic tests are not suitable for determining the modulus of CSM.

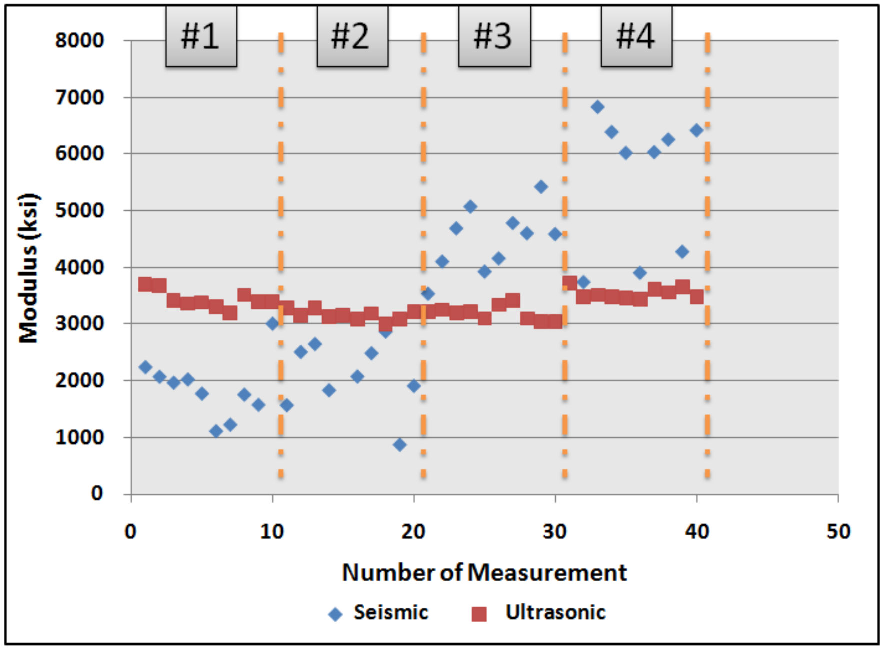


Figure B-14. Variation in Modulus Results Obtained from Seismic and Ultrasonic Tests for Gravel-Cement Specimens

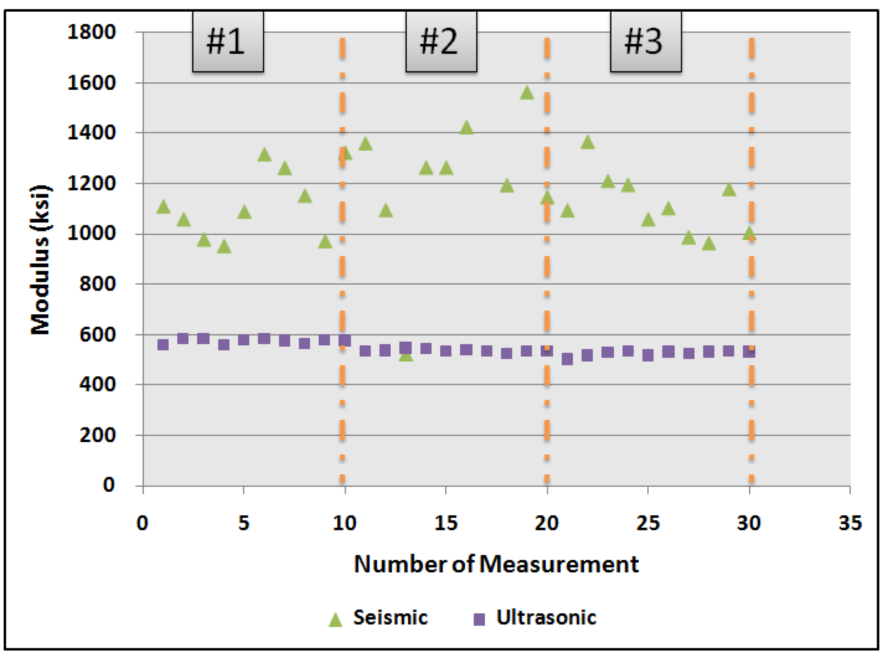


Figure B-15. Variation in Modulus Results Obtained from Seismic and Ultrasonic Tests for Clay-Lime Specimens

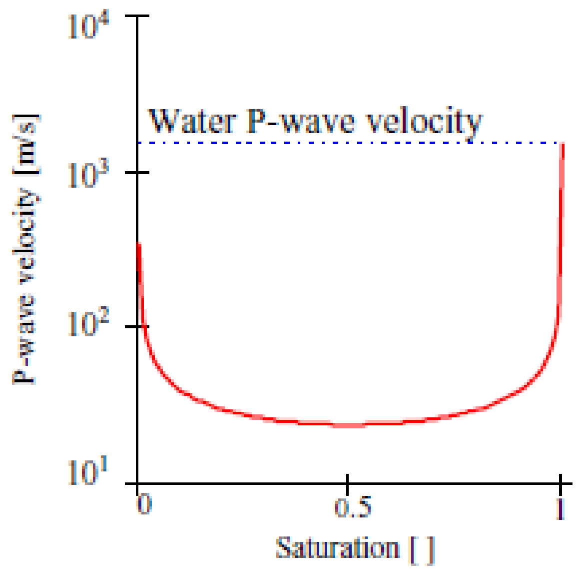
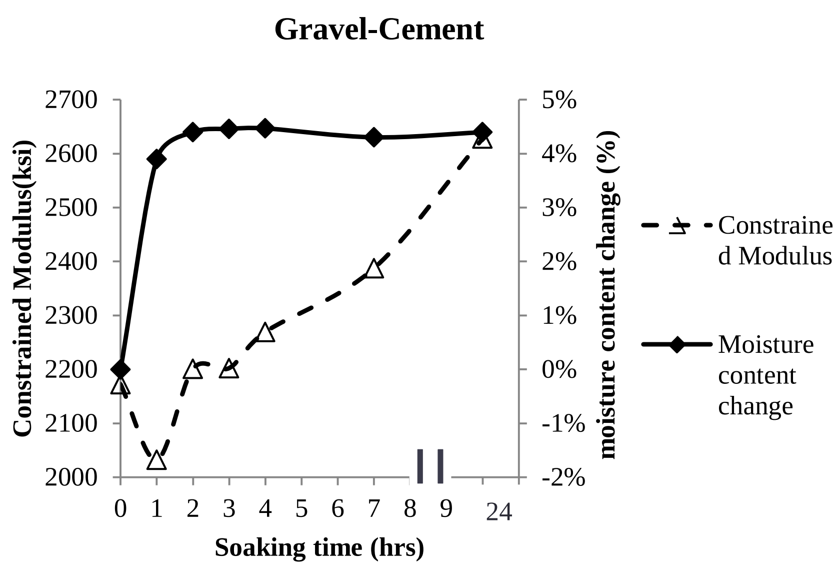
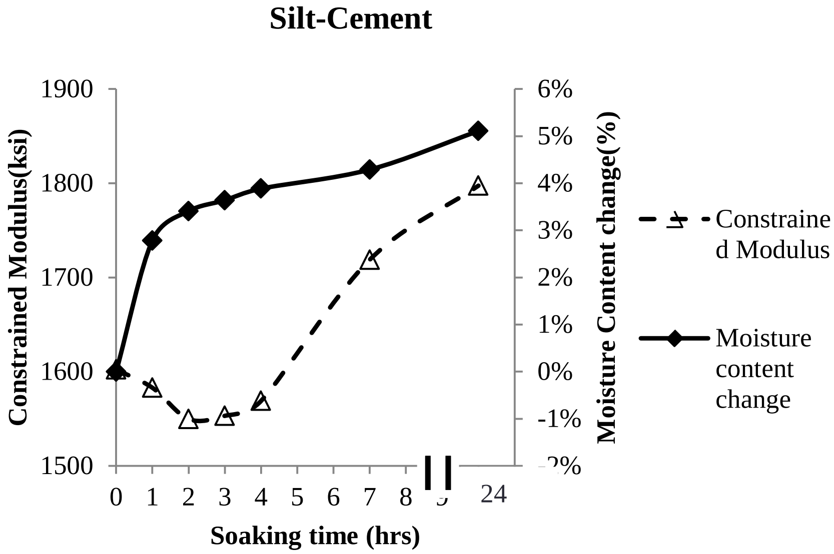


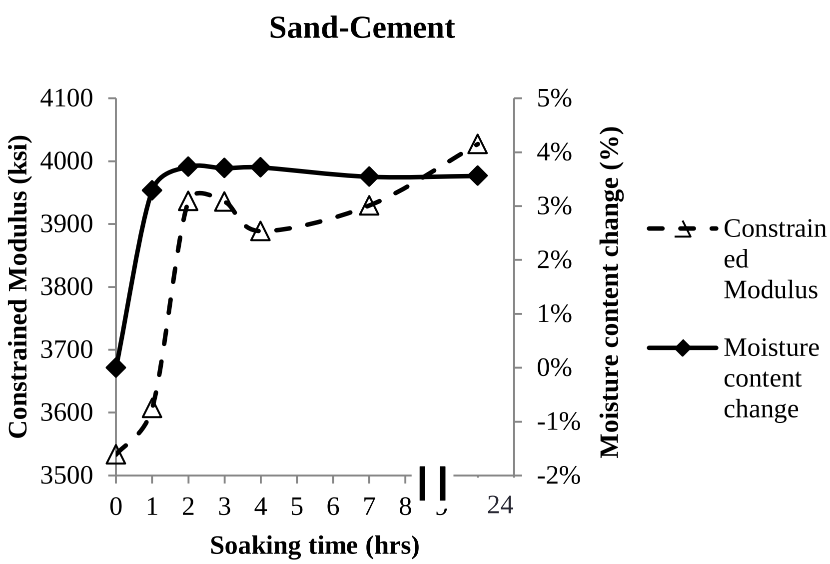
Figure B-16. Effect of Saturation on P-wave Velocity (Fratta et al. 2005)



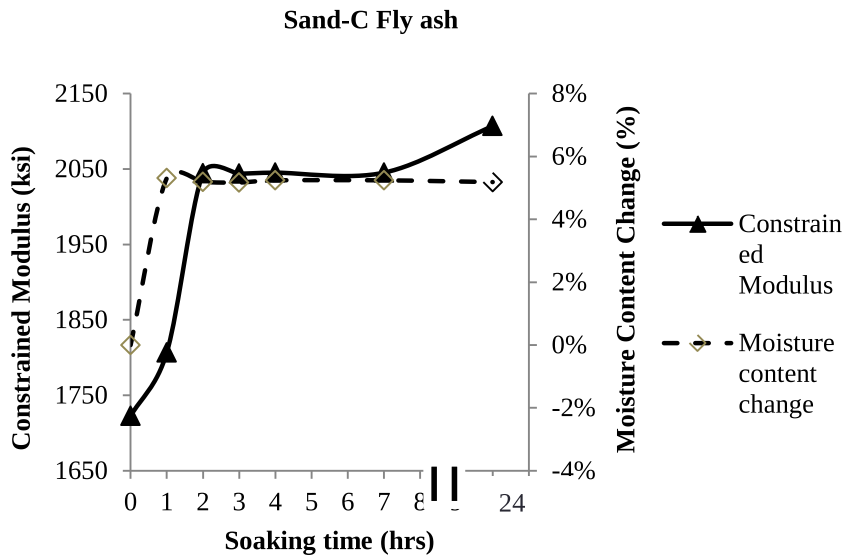
(a) Gravel-Cement



(b) Silt-Cement



(c) Sand-Cement



(d) Sand-C Fly Ash

Figure B-17. Effect of Moisture Content on Constrained Modulus

Durability Tests

The objective of the durability test evaluation is to investigate whether durability tests are applicable to CSM, especially for lightly stabilized materials.

Wet-Dry Durability Test

For the wet-dry tests, C fly ash-gravel and lime-clay were selected to represent materials with strength values from high to low. After curing at 104°F and 100% RH for 5 days, the specimens in a test group were subjected to wet-dry cycles: i.e., they were soaked in water for 4 hours at 68°F and dried in an oven at 158°F for 42 hours, in accordance with ASTM D559-03. Specimens in the control group were wrapped with a moist towel and subjected to the same temperature cycles (4 hours at 68°F and 42 hours at 158°F), which served to eliminate the effect of temperature on the CSM, leaving moisture as the only effective factor. The specimens were tested for UCS after different wet-dry cycles. Each material had 3 replicates.

Table B-21 presents the UCS test results where it is seen that after 13 wet-dry cycles, the UCS of the C fly ash-gravel dropped from 1223 psi to 1024 psi, whereas the UCS of the lime-clay specimens decreased from 373 psi to 241 psi after 9 wet-dry cycles, which is only 65% of their initial strength values. The Class C fly ash-gravel retained 84% strength of the control group. This wet-dry durability test is recommended for both lightly stabilized materials and heavily stabilized materials.

Table B-21. Wet-Dry Test Results

Specimen	No. of Cycles	UCS Values for Wet-Dry Cycles (psi)					UCS of Control Group (psi)	Residual Rate (%)
		Specimen 1	Specimen 2	Specimen 3	Average	COV		
C Fly Ash-Gravel	13	984	1014	1074	1024	4%	1223	84
Lime-Clay	9	256	226	242	241	6%	373	65

Freeze-Thaw Durability Test

Freeze-thaw tests were conducted to develop freeze-thaw durability models. These tests used two mixtures in gravel-cement and clay-lime specimens. The specimens were molded to produce 4 in. (diameter) by 4.6 in. (height) cylindrical specimens. The gravel-cement specimens were cured in a moist room at 100% RH and 70°F for 28 days in accordance with ASTM D558. The clay-lime specimens were sealed with plastic wrap and cured for 7 days in an oven at 104°F. After curing, the specimens were placed on top of a water-absorbing pad in the freezing apparatus with temperatures below -10°F for 24 hours following ASTM D560. After the freezing stage, the specimens were placed in a moist room at 70°F and 100% RH for 23 hours. The UCS values of the specimens in the control group were measured after curing. The UCS values of the specimens in the freeze-thaw group were measured after the freeze-thaw cycles. Table B-22 presents the UCS results for the freeze-thaw tests where it is seen that after two freeze-thaw cycles, the UCS of the cement-gravel dropped from 638 psi to 444 psi, whereas the UCS of the lime-clay specimens decreased from 149 psi to 57 psi. Based on the residual rate, the residual strength of the lime-clay was only 38% of its initial strength, whereas the cement-gravel had a residual rate of about 70 percent. Therefore, the lime-clay material experienced more damage during the freeze-thaw cycles than the cement-gravel material. The test results indicate that the freeze-thaw durability test is applicable to both lightly stabilized materials and heavily stabilized materials.

Table B-22. Freeze-Thaw Test Results

Specimen	UCS Values for 2 Freeze-Thaw Cycles (psi)					UCS of Control Group (psi)	Residual Rate (%)
	Specimen 1	Specimen 2	Specimen 3	Average	COV		
Cement-Gravel	535	419	377	444	18%	638	70
Lime-Clay	62	57	52	57	9%	149	38

Vacuum Saturation Test

The effects of vacuum saturation on the UCS of gravel-cement and clay-lime specimens also were studied. Six gravel-cement and six clay-lime specimens were fabricated and cured in a moist room (100% RH). The gravel-cement specimens were cured at 68°F, and the clay-lime specimens were cured at 104°F. After curing for 7 days, 3 specimens were tested for UCS, and the remaining 3 specimens were vacuum-saturated and then tested for UCS.

The vacuum saturation strength tests were carried out according to ASTM C593, *Standard Specification for Fly Ash and Other Pozzolans for Use with Lime for Soil Stabilization*, except the vacuum process was modified. The specimens were sealed and vacuumed to a pressure of 29 in-Hg (14.24 psi) using a CoreLok device and submerged in water. The vacuum-sealed bags were cut open in the water and the specimens were removed and left in the water for half an hour. Figure B-18 shows a vacuum-sealed specimen.

Figure B-18. Vacuum Test Setup

Table B-23 shows the UCS results of unsaturated and vacuum-saturated gravel-cement specimens, which are also shown in Figure B-19. The average UCS value for the control group is 1,074 psi, whereas the average UCS value after vacuum saturation is 957 psi. Vacuum saturation slightly reduces the UCS of gravel-cement.

Table B-23. Test Results of Vacuum Saturation for Gravel-Cement Specimens

UCS, Unsaturated			**UCS, Vacuum Saturated**		
Specimen No.	psi	COV	Specimen No.	psi	COV
1	1,041	4.39%	4	992	13.53%
2	1,053		5	1,066	
3	1,128		6	814	
Average	1,074		**Average**	957	

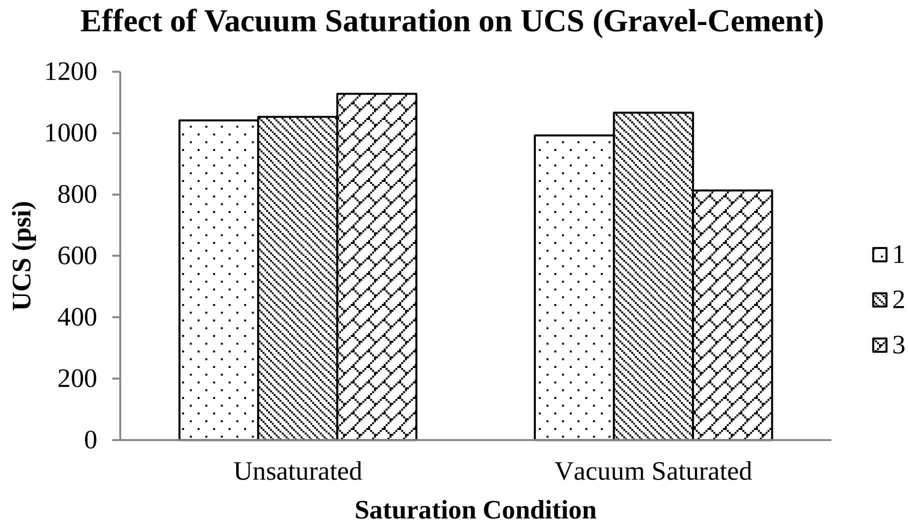


Figure B-19. UCS with and without Vacuum Saturation for Gravel-Cement

For the clay-lime specimens, vacuum saturation decreases the UCS by 55%, from 115 psi to 52 psi, as shown in Table B-24 and Figure B-20. Based on the test results, it appears that the vacuum saturation test method is applicable to both lightly and heavily stabilized materials to reduce their strength.

However, one significant shortcoming of vacuum saturation is that it provides only a qualitative evaluation of the durability of CSM. The effect of the number of freeze-thaw cycles on CSM cannot be quantified. Therefore, vacuum saturation is not recommended for model development.

Table B-24. Test Results of Vacuum Saturation for Clay-Lime Specimens

UCS, Unsaturated			**UCS, Vacuum Saturated**		
Specimen No.	psi	COV	Specimen No.	psi	COV
1	105	7.72%	4	54	12.72%
2	122		5	45	
3	118		6	58	
Average	**115**		**Average**	**52**	

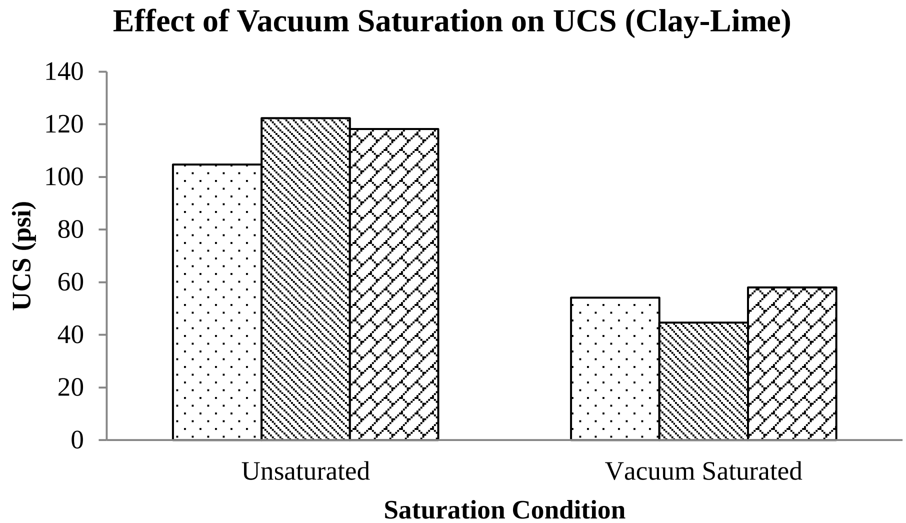


Figure B-20. UCS of Unsaturated and Saturated Specimens

Large-Scale Model Experiment (LSME)

The objective of this phase of the investigation is to identify a procedure for determining the number of load cycles to fatigue crack initiation for a cement-stabilized base course and subgrade materials using the *large-scale model experiment* (LSME). This procedure is intended to determine whether the fatigue model that is developed based on small-scale laboratory beam fatigue tests can be calibrated.

Cement-stabilized gravel, recycled pavement materials (as part of procedure development for another study), and silt were used to evaluate the applicability of the LSME. Mixing the materials was accomplished using a Gilson portable concrete mixer in 220-lb batches. The appropriate mass of the cement was added to the soil under air-dry conditions and allowed to mix for at least two minutes. Water was added slowly until the batch was evenly coated and brought to nearly optimal water content for the mixture. After mixing for approximately five minutes, the mixture was shoveled into the test pit. Additional batches were created in the same manner until enough material that could be compacted in a single lift was placed. A jumping-jack style compactor and hand tamper were used to compact the stabilized layer to the desired thickness at the maximum dry unit weight. Specimens were covered with plastic and allowed to cure under ambient laboratory conditions.

The LSME was conducted in a 10 ft × 10 ft × 10 ft test pit. The bottom 8.2 ft of the test pit was filled with dense, uniformly graded subgrade sand. The test materials were placed on top of the subgrade sand to the desired thickness. Loads were generated using a MTS 280 L/m hydraulic actuator with a 20-kip load capacity and 6.6 in. stroke supported by a steel load frame. Applied loads were transferred to the pavement profile using a 1-inch thick steel plate with a radius of 4.9 inches. The MTS equipment is capable of applying a haversine-shaped load pulse over a range of load durations, load levels and rest periods. Vertical deflections of the pavement

profile were measured using LVDTs with a precision of +/- 0.0002 inch. Surface deflections were measured using an LVDT mounted on the surface of the loading plate. The steel plate was assumed to be rigid and, thus, any movement of the plate was translated to the pavement section. Subgrade deflections were measured by attaching two small plates at either end of a thin rod. One plate was placed flush at the subgrade surface beneath the load plate. The thin rod was passed through a tube extending through the stabilized layer, allowing the rod to move freely with the subgrade. The second plate was positioned above the stabilized layer surface where another LVDT was mounted. Deflections and applied load data were collected using LabView 7.1 software. Figure B-21 shows a schematic view of the LSME. While the tests were running, the specimen surface was visually inspected for the presence of cracks.

Previous LSMEs have used the entire 10 ft × 10 ft section to evaluate pavement profiles (Tanyu et al. 2003, Kootstra 2009), whereas other experiments have used a smaller 3.3 ft × 3.3 ft section (Schaertl 2010). Using the entire test pit section would require a significantly large amount of material. Thus, test trials have been initiated using specimens that utilize the same setup as found in Schaertl (2010) with dimensions approximately 3.3 ft × 3.3 ft and 8 in. thick. With the absence of an asphalt layer, a force of 1,500 lbs was applied using a 4.9-inch steel load plate to simulate truck traffic (Kootstra 2009, Schaertl 2010). Based on the results of the MICHPAVE program, the applied force was determined to be equivalent to a 15.7 kip axle load with 101.5 psi tire pressure when an asphalt layer is present.

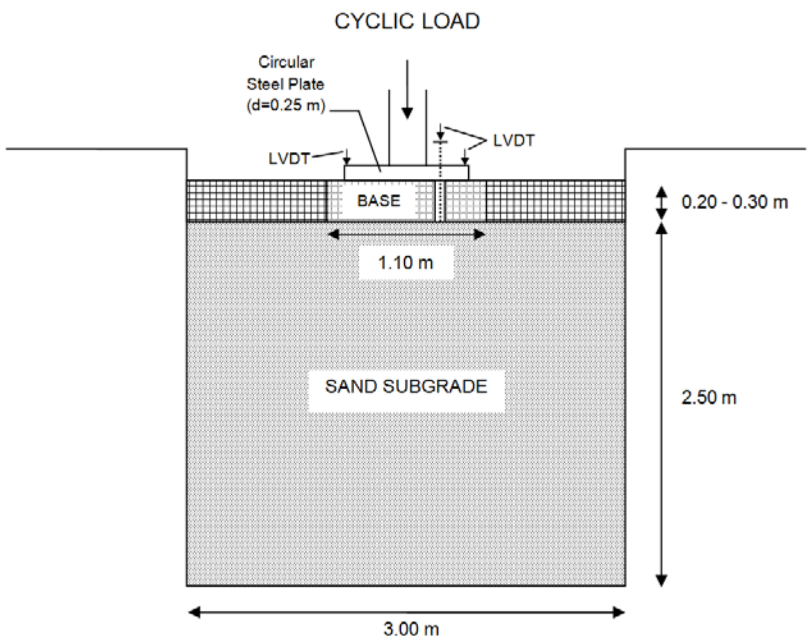


Figure B-21. Schematic of LSME Setup, 1 m = 3.3 ft (Schaertl 2010)

The initial LSME tests employed a silt-cement (4% by weight) slab approximately 3.3 ft × 3.3 ft and 8 in. thick. Surface and subgrade deflections were measured, and the specimens were monitored for fatigue cracking. The tests were conducted during the day in increments of 20,000 to 40,000 cycles. Cracking was not observed after 325,000 cycles at loads ranging from 2,500 to

7,000 lbs. A second silt-cement specimen of the same geometry was tested next. In order to increase deflections during loading and thus increase the stress/strain in the layer, a 3.3 ft × 3.3 ft piece of 2-inch thick extruded polystyrene (XPS) was placed beneath the specimen. The presence of the foam layer doubled the elastic deflections compared to the first test. No cracking was observed after 140,000 cycles at 4,000 lbs and 260,000 cycles at 5,000 lbs.

Observations of the specimen movement indicate that the silt-cement was likely compressed with little flexural response, which appears to be due to the relatively small size of the specimen compared to the steel load plate. Therefore, the specimen size was increased to a 3.3 ft × 6.6 ft test section. Additionally, the CSL slab and foam layer thicknesses were reduced to 4 in. and 1 in., respectively. The foam layer was centered beneath the specimen and remained at 3.3 ft × 3.3 ft., effectively leaving 1.6 ft at each end of the specimen supported on the subgrade sand. The intent was to simulate a simply supported lab scale beam specimen in the LSME test pit and to induce flexure.

Recycled pavement material was stabilized with 3% cement by weight and set up using the new configuration. The applied load ranged from 3,500 to 6,000 lbs. Loading commenced at 3,500 lbs for 100,000 cycles. With no observed cracking, the load was subsequently increased to 4,000, 5,000, and 5,500 lbs each for 200,000 cycles. Still no cracking was observed. After about 179,000 cycles (879,000 cycles in total) at 6,000 lbs, a fatigue crack was observed on the recycled pavement material surface approximately 14 in. to the left of the steel load plate. The crack ran perpendicular to the length of the specimen and made nearly imperceptible movements during each load application. However, deflection data gave no clear indication of the exact time of the crack initiation (Figure B-22). The elastic deflection around this time was about 0.0358 inch. Just prior to the observation of cracking, a slight increase in the surface elastic measurement was observed, up from 0.0354 in. and, afterward, deflections grew to exceed 0.0362 inch. This change is extremely small.

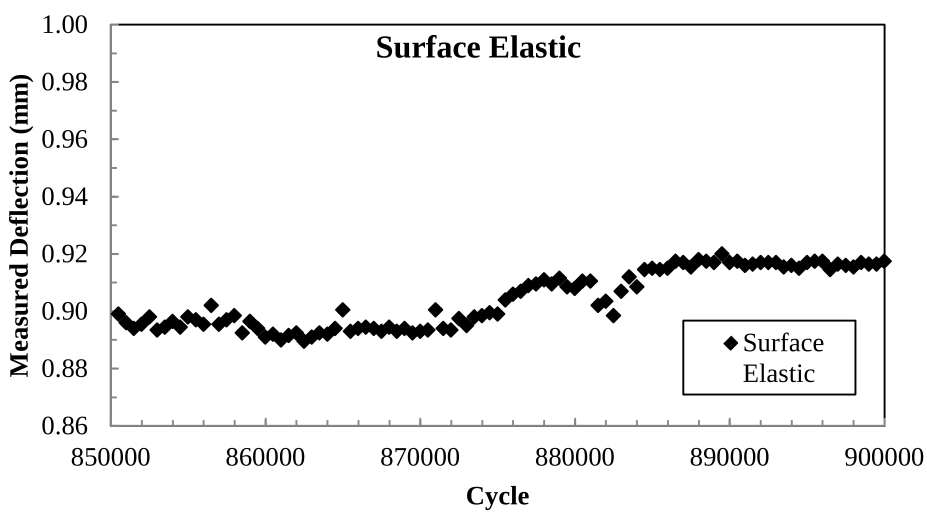


Figure B-22. Elastic Deflections of Recycled Pavement Material before and after Crack Observation (25.4 mm = 1 in.)

Another specimen composed of gravel and cement (3% by weight) was set up in the same configuration as the recycled pavement material -cement specimens. The applied load was set to 6,000 lbs at the beginning of the test. Two fatigue cracks, each 14 in. to the right and left of the steel load plate, were observed on the gravel surface in less than 5,000 cycles (Figure B-23). Like the recycled pavement material test results, the deflection data for the gravel-cement specimens appear to be unchanged by the presence of these cracks, as shown in Figure B-24. Most of the plastic strain accumulation occurred during the first 1000 cycles, after which the strain rate became relatively constant. The elastic deflections tended to decrease slowly during the test as the material became more densely compacted under loading.

Figure B-23. Observed Crack on Gravel-Cement Surface

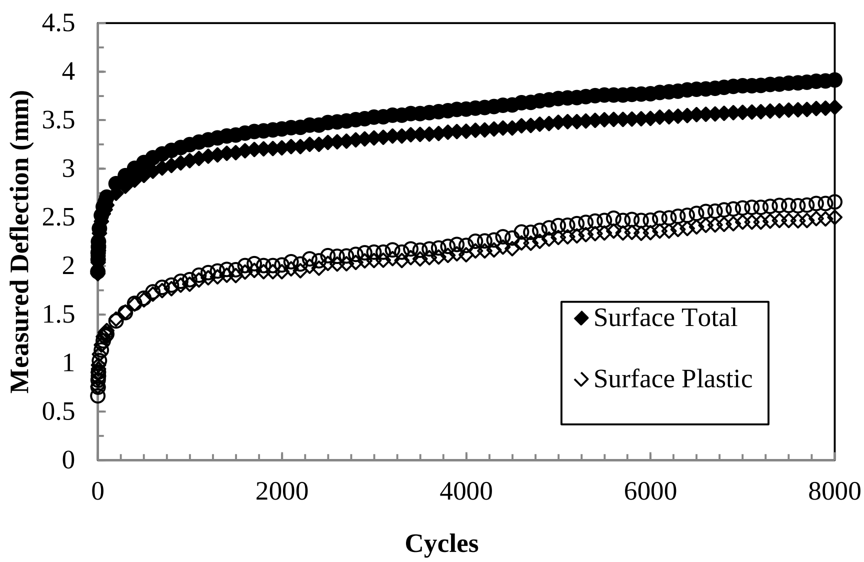


Figure B-24. Measured Deflections for Gravel-Cement with Foam at 6000 lbs

The soil was changed to silt for the next tests because the gravel-cement specimens proved difficult for observing fatigue cracks due to the rough surface. Also, the deflection data yielded no clear signs of cracking. Three silt-cement slabs, each 3.3 ft × 6.6 ft and 4 in. thick, were placed on an expanded polystyrene (EPS) support. The EPS support was manufactured by Cellofoam and typically has a density of 1 pund per cubic foot (pcf). Tanyu et al. (2003) note that the stress-strain behavior of a low-density (1.1 pcf) EPS support is comparable to Antigo silt loam, a typical soft subgrade soil found in Wisconsin. The first silt-cement slab cracked in less than 200 cycles at 1,500 lbs. This load level seems to be too extreme due to the soft EPS support. For the second specimen, the applied load was reduced to 900 lbs, and a crack was observed after about 3,000 cycles (Figure B-25). For both tests, the cracks were located near the center of the CSL. The third specimen was subjected to a load of 750 lbs. Small cracks (1 in. long) were observed after 1,500 cycles. The cracks continued to grow and connect to additional observed cracks. After 16,000 cycles, the cracks appeared to be forming a ring around the steel load plate. As with all the previous LSME tests where cracking was observed, the deflection data provided no indications of the exact time of crack initiation.

Figure B-25. Observed Crack in Silt-Cement Surface

The resilient deformation of CSL was not affected by the occurrence of fatigue cracking in the LSME. Changes in the deflection characteristics were expected to manifest during or after cracking of the stabilized materials. However, for all tests where cracking was observed, the presence of cracking did not appear to impact the measured deflections. Numerical modeling indicates that the deflection change due to cracking is very small, supporting the observations. The location of the cracks varied for each LSME test, which suggests that the fatigue behavior of the CSL is not controlled by the stress condition alone. By nature, CSL are not homogeneous and are susceptible to material defects. Therefore, cracking may not occur at the point of maximum stress within the CSL but at local weak points. The results from the LSME study indicate that

this LSME fatigue test is able to characterize the fatigue behavior of CSM. For instance, increasing the load reduces the fatigue life (based on observations of the surface, not the reduction of the modulus), and an increase in the thickness of the slab prolongs the fatigue life of the CSM. However, the resilient strain and causal modulus of the CSL remain unaffected by the occurrence of cracks, which contradicts to the field observation. Therefore, the LSME is not recommended for fatigue model calibration.

Shrinkage and Cracking

The objective of the shrinkage test evaluation is to investigate the test methods that are input factors for the shrinkage strain and shrinkage cracking models.

Coefficient of Thermal Expansion (COTE) Test

The coefficient of thermal expansion (COTE) is a key parameter in the shrinkage cracking model. The originally proposed test method for the COTE of concrete, based on AASHTO TP60, *Standard Method of Test for Coefficient of Thermal Expansion of Hydraulic Cement Concrete*, is not suitable for measuring the COTE of lightly stabilized materials or the early-age (e.g., 2 days) COTE of heavily stabilized materials which is critical for early-age shrinkage cracking. The saturation method recommended in AASHTO TP60 would severely damage the lightly stabilized materials or the early-age specimens of heavily stabilized materials. In addition, thermal expansion is coupled with autogeneous and drying shrinkage for early-age stabilized materials. A test method for COTE, recommended by Cusson and Hoogeveen (2006), which was specifically designed for early-age concrete, is evaluated in this study based on temperature cycling. The shrinkage of a CSL specimen includes autogenous, drying, and thermal shrinkage. The use of temperature cycling can separate the thermal shrinkage/expansion from autogenous and drying shrinkage.

Prism specimens with dimensions of 4 in. × 4 in. × 11.25 in. were placed in an environmental chamber at an initial ambient temperature of 77°F and 98% RH. The chamber cycled the temperature between 77°F and 86°F using a saw-tooth pattern while the RH was kept constant. After the target temperature in the chamber was reached, which takes about 20 minutes, the temperature was kept constant for 4 hours, resulting in three full cycles (or six steps) per day. The constant temperature period is long enough to ensure a stable and uniformly distributed temperature in the prism sample. The amplitude of the temperature cycle (9°F) is selected to be small enough to maximize the number of cycles per day and obtain more COTE values at early ages. With such small temperature changes, the temperature effect on the change in COTE over time can be considered small (Cusson and Hoogeveen 2006).

The displacements were measured by LVDTs. The temperatures inside the specimens and environmental chamber were monitored by thermal couples.

Figures B-26 and B-27 show the test results and COTE of the clay-lime and gravel-cement specimens, respectively. The COTE of the clay-lime specimens increased within the first two days and then stabilized at about 4.7×10^{-5}/°F. However, the COTE of the gravel-cement samples was relatively stable at about 1.4×10^{-5}/°F, which is about one-third of that of the clay-lime specimens.

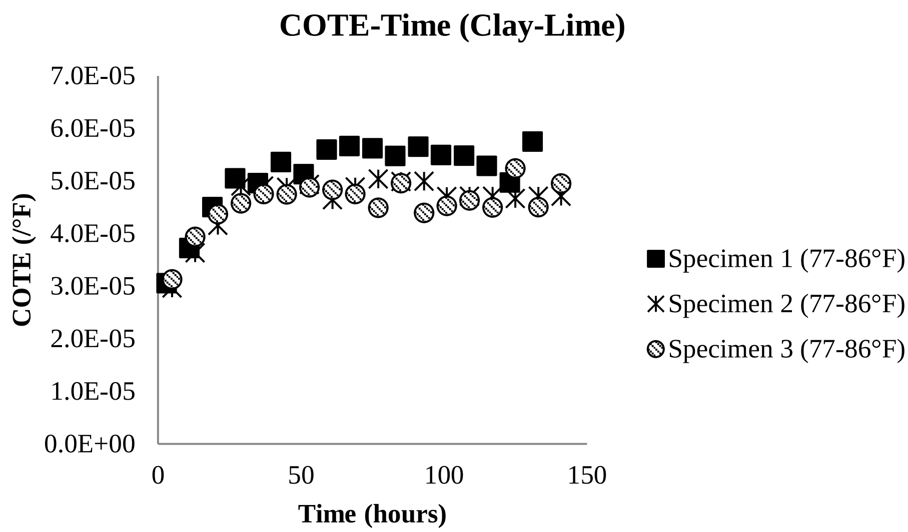


Figure B-26. COTE Development of Clay-Lime Specimens

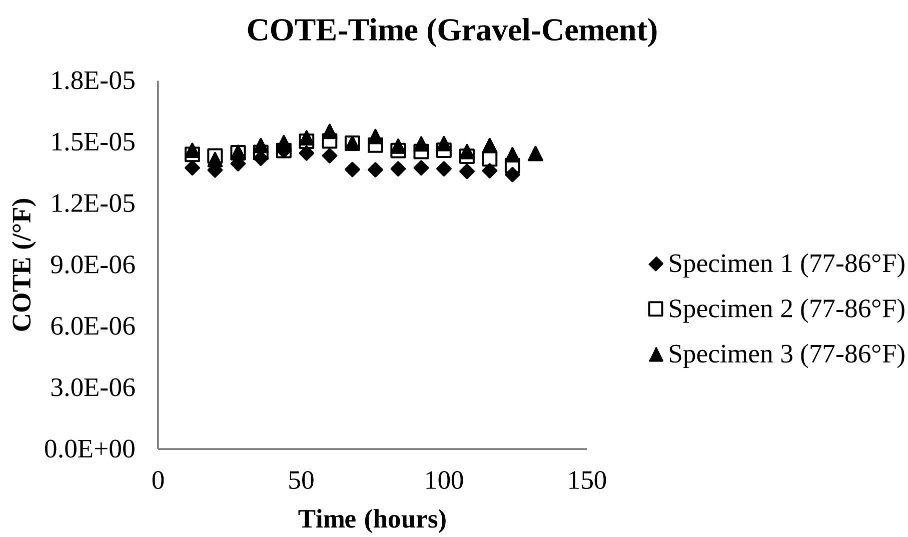


Figure B-27. COTE Development of Gravel-Cement Specimens

Overall, the COTE test, which is based on temperature cycling, is capable of measuring the COTE of both lightly and heavily stabilized materials and is recommended for model development.

Bond Strength Test

Bond strength is an input to the restrained shrinkage cracking model. Because the beam specimen for restrained shrinkage cracking is glued onto a steel substrate with epoxy or cement mortar, the bond strength between the CSM and steel substrate is measured when epoxy or cement mortar are used as bonding agents.

The bond strength tests were conducted in accordance with the Iowa shear test (Iowa DOT 2000), which separates the CSL and base along the interface. Two materials, i.e., clay-lime and gravel-cement, were used and compacted at the optimum moisture content. The mixed materials were compacted by standard Proctor compaction in a 4-inch diameter mold. The specimens were cured at 100% RH at room temperature (68°F) for 7 days and were glued to a steel substrate with epoxy or cement mortar. The cement mortar was prepared with a sand:cement:water ratio of 6:2:2.

The bond strength tests were conducted at a constant loading rate (71.6 psi/min) or constant deformation rate (0.065 in./min). Figures B-28 and B-29 show the test results for the constant loading rate and constant deformation rate for the cement-gravel specimens, respectively. Due to abrupt failure during the constant loading rate tests, a constant deformation rate is recommended for the purpose of safety. However, the repeatability of the deformation control (0.065 in./min) is poor. Thus, the load speed was increased to 0.65 in./min, which improved the repeatability.

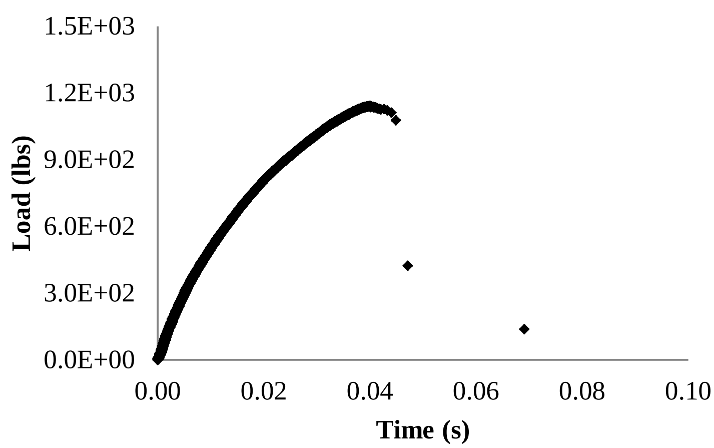


Figure B-28. Bond Strength under Constant Loading Rate for Cement-Gravel (71.6 psi/min)

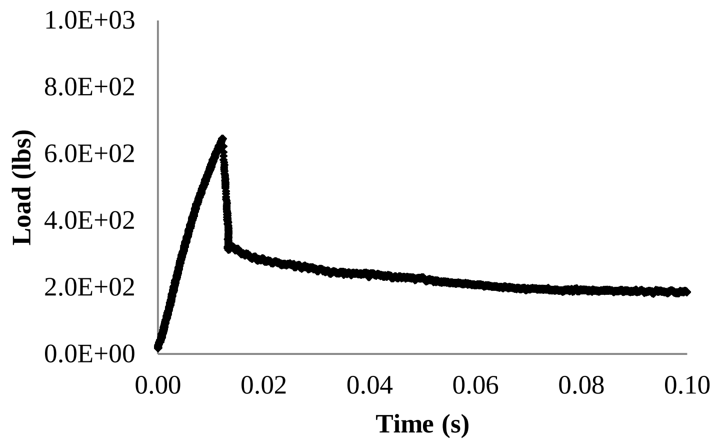


Figure B-29. Bond Strength under Constant Deformation Rate for Cement-Gravel (0.065 in./min)

Table B-25 and Figures B-30 and B-31 present the test results for three clay-lime and three gravel-cement specimens that use epoxy or cement mortar as bonding agents, respectively. Some variability is seen in the bond strength measured. As shown in Figure B-30, when cement mortar is used as the bonding agent, the bond strength values for the clay-lime and gravel-cement specimens are very close to each other, around 3 psi, because the failure occurred within the mortar. That is, the bond strength was controlled by the strength of the cement mortar. Therefore, the clay-lime and gravel-cement specimens have similar bond strength values when cement mortar is used as the bonding agent.

When epoxy is used as the bonding agent, the bond strength values of the clay-lime specimens are significantly lower than those of the gravel-cement specimens, as shown in Figure B-31. Shear failure occurred inside the clay-lime specimen, as shown in Figure B-32 (a). Therefore, the bond strength that was measured is actually the shear strength of the clay-lime. The failure plane for the gravel-cement specimen is located at the interface between the epoxy and soil specimen in Figure B-32 (b). Therefore, this measured strength is the bond strength at the interface.

Table B-25. Results of Bond Strength Tests

Binder	Materials	Shear Strength (psi)	COV
Mortar	Clay-Lime	2.53	26.2%
		2.43	
		3.80	
	Gravel-Cement	2.68	27.9%
		2.57	
		4.14	
Epoxy	Clay-Lime	19.1	32.1%
		14.0	
		26.8	
	Gravel-Cement	206.8	10.5%
		224.5	
		181.8	

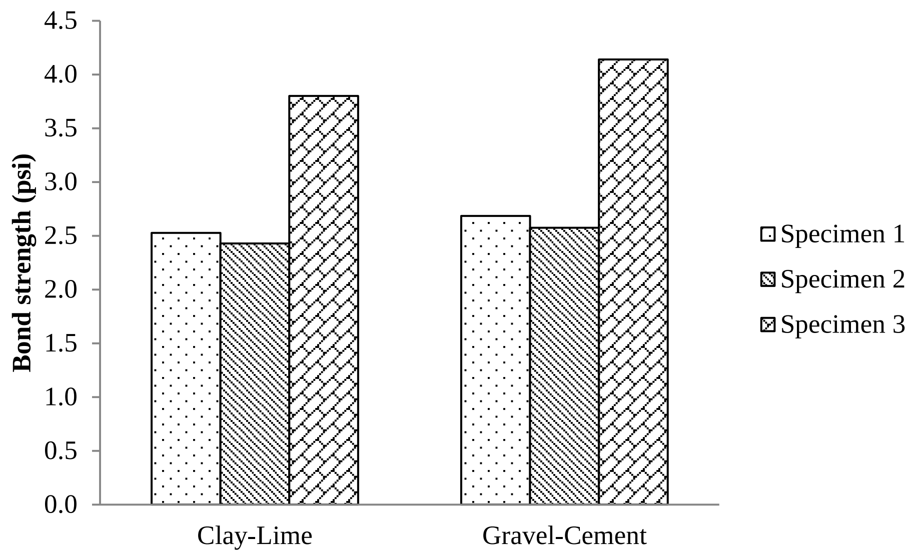


Figure B-30. Bond Strength Test when Cement Mortar is used as Bonding Agent

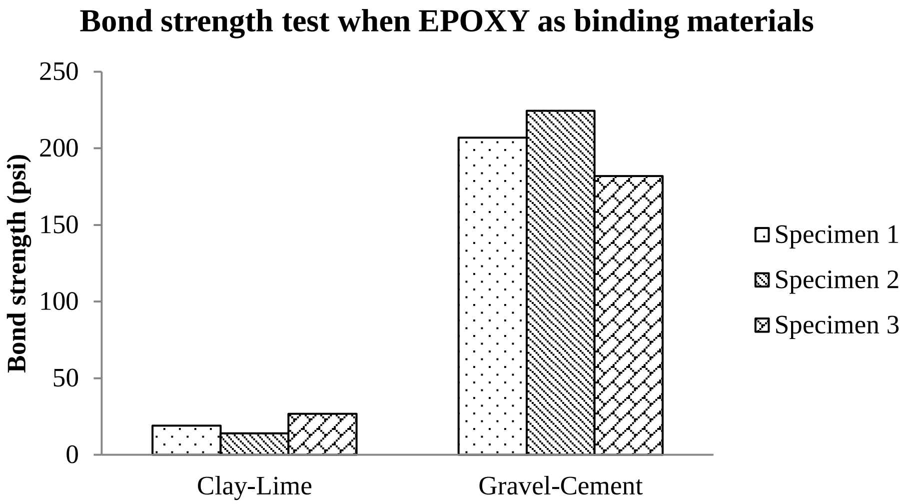


Figure B-31. Bond Strength Test when Epoxy is used as Bonding Agent

(a) Clay-Lime Specimen after Test *(b) Gravel-Cement Specimen after Test*

Figure B-32. Failure Surfaces after Bond Strength Tests

Even though some variability was found between the bond strength of the replicates, the Iowa shear test is capable of measuring the bond strength for modeling shrinkage cracking and is recommended for model development.

Free Autogenous Shrinkage

Autogenous shrinkage is the shrinkage due to hydration of the binder in CSL. Autogenous shrinkage can be measured when specimens are wrapped with plastic to prevent loss of moisture. However, it is cumbersome to wrap specimens with plastic and difficult to prevent

moisture loss and drying shrinkage. Instead, wax was used in this study to seal the specimens. Specimens of clay-lime and gravel-cement were fabricated and placed in plaster molds, as shown in Figure B-33. Hot liquid wax was poured into each plaster mold. After the excess liquid wax was drained, the specimen was covered with a thin layer of wax. The plaster mold was removed and the specimens and wax were cooled to room temperature. A plastic pad was glued onto the sides of the specimen in order to prevent permanent deformation caused by the tips of the dial gauges. The shrinkage was then measured, as shown in Figure B-34.

Figure B-33. Plaster Mold

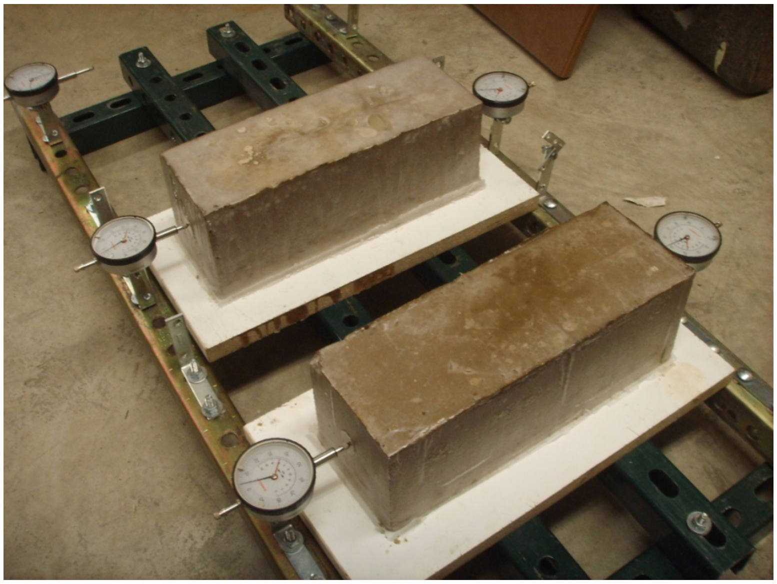


Figure B-34. Autogenous Shrinkage Test Setup

Figure B-35 shows the free autogenous shrinkage test results. The clay-lime specimens show large autogenous shrinkage, which continued to increase even after 8 days; whereas the

autogenous shrinkage of gravel-cement specimens was small and stabilized after about 3 days. This phenomenon might be due to the fact that the reaction time for the clay-lime is longer than for the gravel-cement mixture.

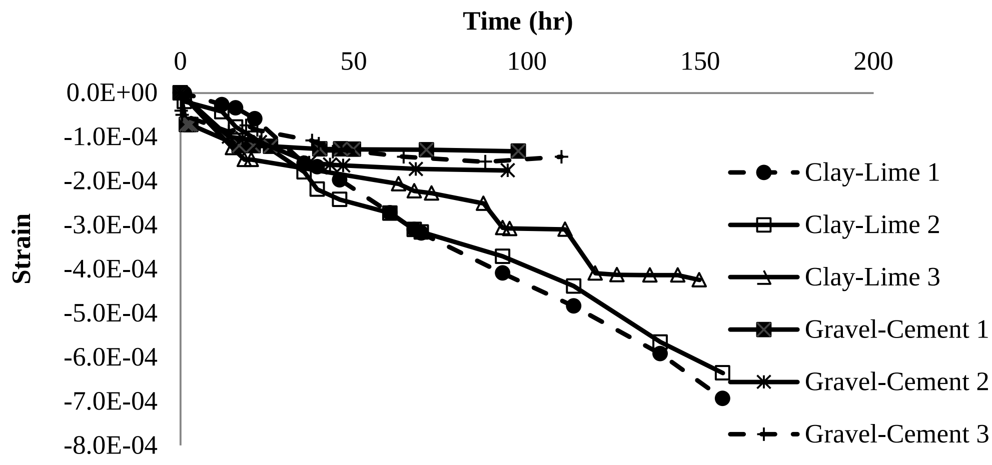


Figure B-35. Autogenous Shrinkage Results of Clay-Lime and Gravel-Cement Specimens

Based on the test results, it seems that the autogenous shrinkage of CSL can be measured using this test.

Free Drying Shrinkage

This test aims to measure free drying shrinkage due to moisture loss. Free drying shrinkage tests were conducted on both lightly stabilized materials (clay-lime) and heavily stabilized materials (gravel-cement). For measuring the drying shrinkage, clay-lime and gravel-cement specimens were fabricated with dimensions of 4 in. × 4 in. × 11.25 in., as shown in Figure B-36. The specimens were air-dried and the displacements were measured from both ends. Two dial gauges were placed on the ends to measure the displacements. The total shrinkage strain is the sum of the two measured strains. The drying shrinkage strain is equal to the total shrinkage strain minus the autogenous shrinkage strain, because the specimen is simultaneously subjected to autogenous shrinkage as well.

Each specimen was placed on a solid plate and lubricant was applied between the specimen and solid plate to allow free movement of the specimen. In addition, in order to allow the specimen to dry in 3 dimensions (6 surfaces), another specimen was placed on a metal plate with numerous small holes for moisture to evaporate from the bottom as well. Lubricant was applied between the specimen and metal plate to allow free movement of specimen. The third method that was evaluated in this study was to reduce the height of specimen (for fine-grained materials only) to mitigate the effect of the moisture gradient inside the specimen, without the need for the metal plate with holes. A 2-inch high specimen was used. The shrinkage of the three

specimens was measured side by side, as shown in Figure B-37. Figure B-38 shows the test results and indicates that the shrinkage of these three specimens is very similar to each other. Therefore, the effect of specimen dimension and evaporation from the bottom of the specimen is negligible. It is recommended that the drying shrinkage test should be conducted on specimens with dimensions of 4 in. × 4 in. × 11.25 in. on a solid plate and that lubricant should be applied between the specimen and plate.

Figure B-36. Compaction Mold for Free Shrinkage Test

Figure B-37. Test Setup to Evaluate the Effects of Specimen Dimension and Bottom Surface
(left: 4 in. high on porous plate; middle: 4 in. high on solid plate; right: 2 in. high on solid plate)

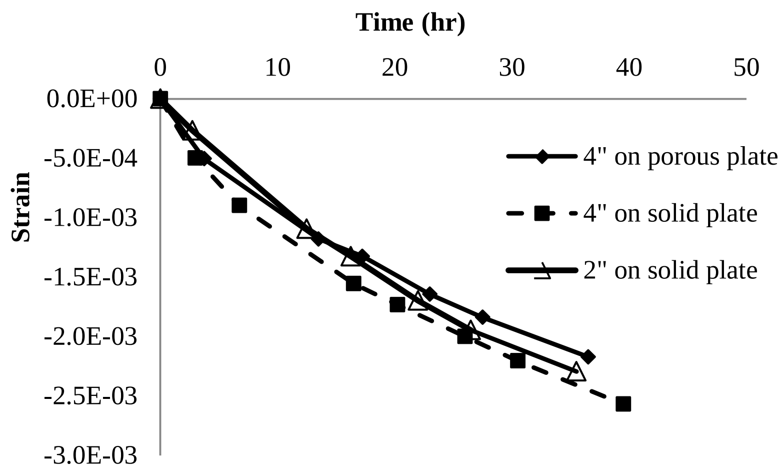
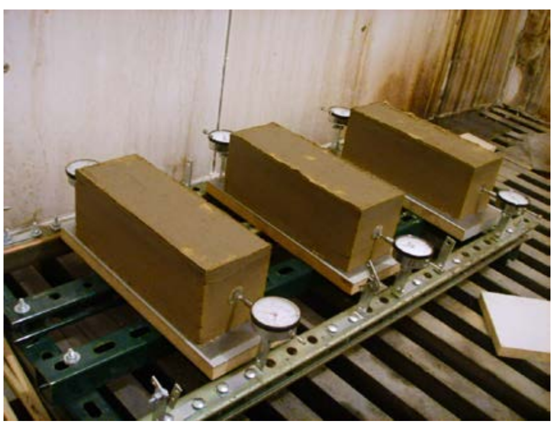
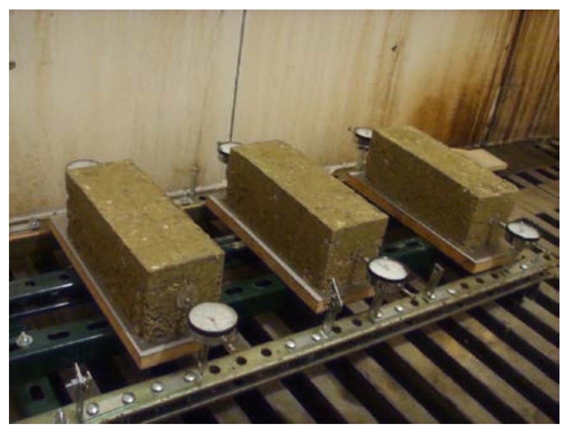


Figure B-38. Effects of Specimen Dimension and Bottom Surface

Lime-stabilized clay and cement-stabilized gravel specimens were fabricated, as shown in Figure B-39. Figure B-40 presents the test results and indicates that the results for lime-stabilized clay are very close and repeatable compared to those of cement-stabilized gravel. The shrinkage of lime-stabilized clay continued to increase, whereas the shrinkage of the cement-stabilized gravel stabilized after three days.



(a) Clay-Lime



(b) Gravel-Cement

Figure B-39. Drying Shrinkage Test

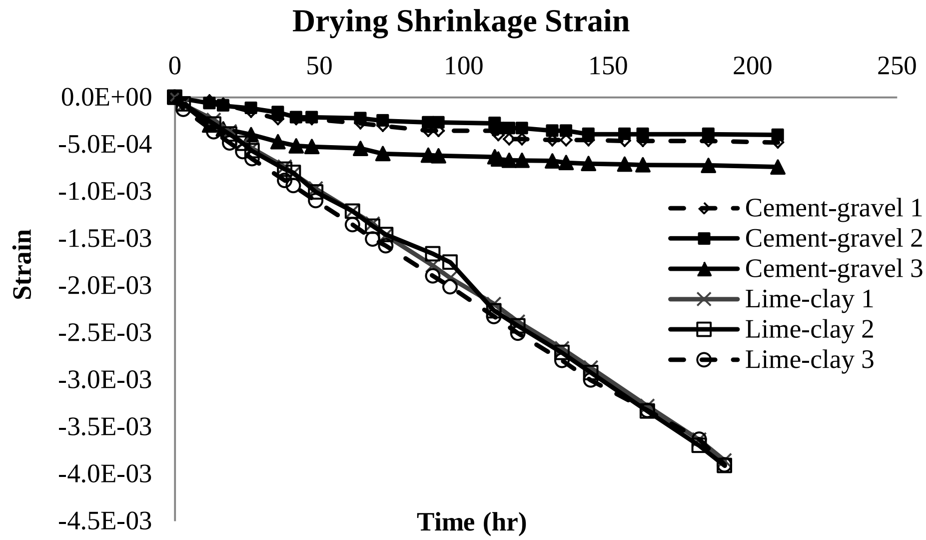


Figure B-40. Test Results for Drying Shrinkage Evaluation

The drying shrinkage and autogenous shrinkage results are plotted together in Figures B-41 and B-42 for the clay-lime and gravel-cement specimens, respectively. Free drying shrinkage is significantly larger than the autogenous shrinkage for both materials. Based on the test results, the free drying shrinkage test used in this study can be used to measure the drying shrinkage of both lightly and heavily stabilized materials and is recommended for model development.

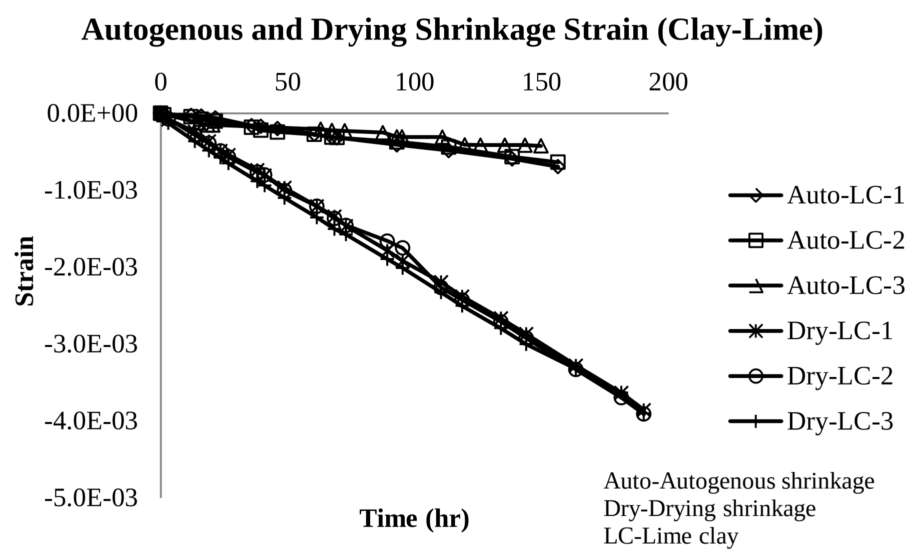


Figure B-41. Free Drying and Autogenous Shrinkage Results for Clay-Lime Specimens

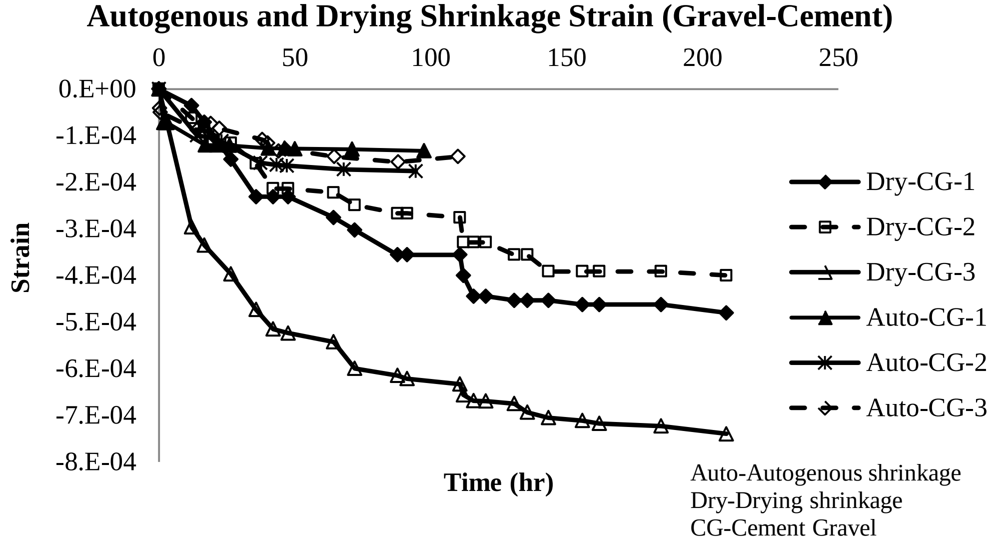


Figure B-42. Free Drying and Autogenous Shrinkage Results for Gravel-Cement Specimens

Restrained Drying Shrinkage Cracking

Tensile stress is produced when moisture or temperature changes cause the CSL base or subbase to contract or shrink with restraint. Shrinkage cracking occurs when the tensile stress exceeds the tensile strength of the cement-treated pavement layer (Cauley and Kennedy 1972). Because shrinkage cracking occurs when the CSL are restrained, the shrinkage cracking potential of each CSL material can be examined using the restrained shrinkage cracking test. The increase in friction between the CSL and underlying material reduces the shrinkage crack spacing. By artificially creating a high level of bond, shrinkage cracking can be generated in a laboratory-scale specimen for model development. The CSL slab is bonded to a steel tube to prevent curling.

Clay-lime and gravel-cement beams with dimensions of 48 in. × 6 in. × 4 in. were glued onto a steel tube using either epoxy or cement mortar. The beams were air-dried to generate drying shrinkage as well as autogenous shrinkage. The clay-lime beam that was glued to the steel tube with epoxy generated one shrinkage crack (Figure B-43). Another clay-lime beam was bonded to the base using cement mortar and generated two shrinkage cracks one day after casting (Figure B-44). Figures B-45 and B-46 present the growth of the cracks (in terms of width) as measured at the top and mid-span of the specimens over time.

Figure B-43. Restrained Shrinkage Test for Clay-Lime when Epoxy is used as Bonding Agent (48 in. Long)

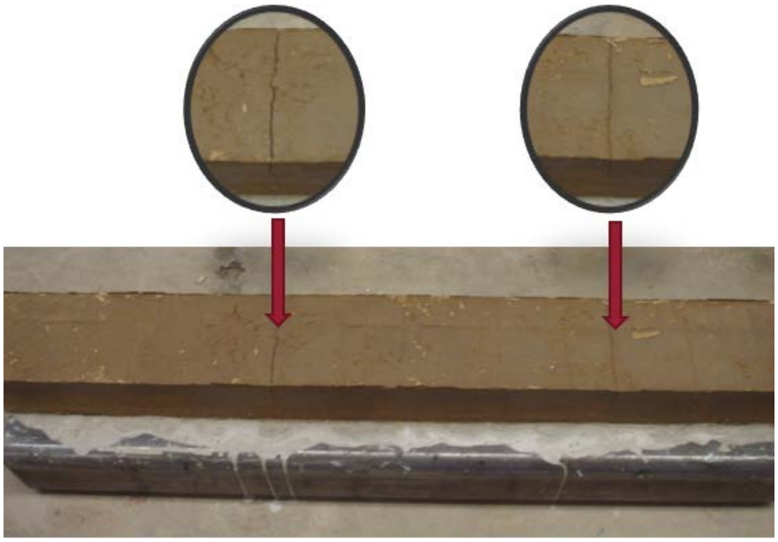


Figure B-44. Restrained Shrinkage Test for Clay-Lime when Cement Mortar is used as Bonding Agent (48 in. Long)

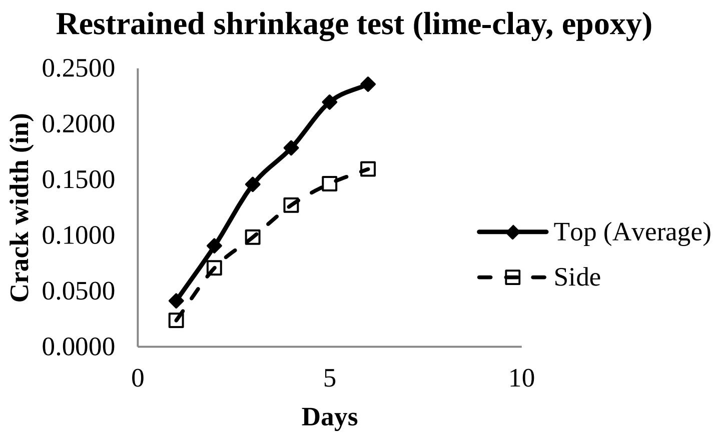


Figure B-45. Crack Width Increase over Time for Clay-Lime Specimen when Epoxy is used as Bonding Agent

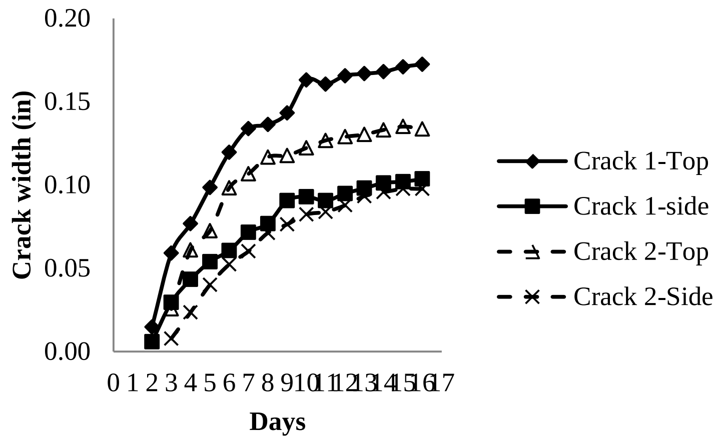


Figure B-46. Crack Width Increase over Time for Clay-Lime Specimen when Cement Mortar is used as Bonding Agent

The gravel-cement specimens, on the other hand, did not generate any shrinkage cracking when either epoxy or mortar was used as the bonding agent. This outcome is likely due to the low molding moisture content and low drying and autogenous shrinkage of the gravel-cement mixture. Three longer gravel-cement (3%, 5%, and 7% cement, respectively) beams with dimensions of 92 in. × 6 in. × 4 in. were fabricated and glued onto a concrete floor using epoxy (Figure B-47). However, still no shrinkage cracking was observed.

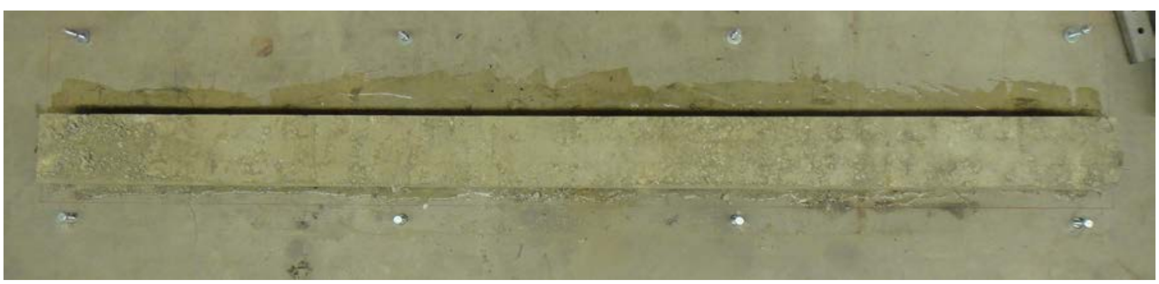


Figure B-47. Restrained Shrinkage for Gravel-Cement when Epoxy is used as Bonding Agent (92 in. Long)

The clay-lime beams experienced shrinkage cracking. However, the gravel-cement used in this study was found not to be prone to shrinkage cracking. To simulate field conditions, the side surfaces of the beam specimens were sealed in a later study and used for the cracking model development. The restrained shrinkage test is recommended for evaluating the potential of CSM for shrinkage cracking and for developing a shrinkage cracking model.

Erodibility

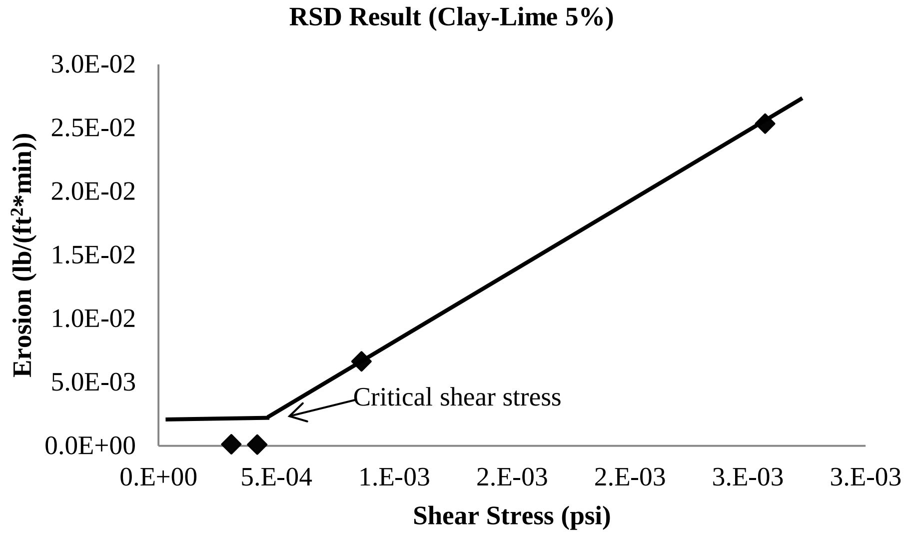
Erodibility refers to the capability of a material to resist the detachment of fines under external loading or weathering. An RSD was used to evaluate the susceptibility of the CSL to erosion. The RSD consists of a specimen placed inside a cylinder that is surrounded by water in the annular space, as shown in Figure B-48. The rotation of the outside cylinder, which is surrounded by water, applies shear force on the specimen. The shear force is measured by a load cell. An RSD was manufactured by WSU for this study with a rotational speed of up to 1,300 RPM.

Figure B-48. Rotational Shear Device

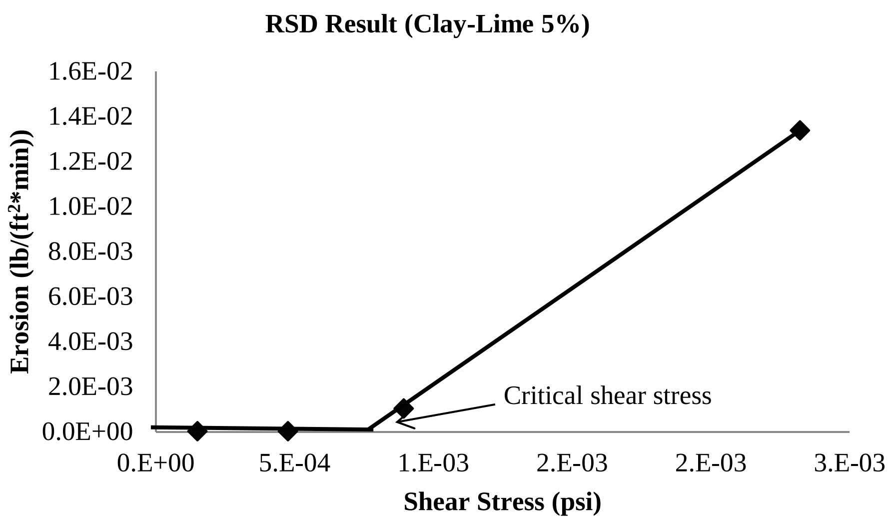
Three clay-lime (5% lime) specimens were tested using the RSD. The shear stress on the specimens was measured. Four rotational speeds were used: 200, 400, 600, and 800 RPM. After 10-minute rotations at each speed, the water was drained and passed through a No. 200 sieve. The mass retained on the sieve was dried and weighed and considered to be the mass that eroded. The erosion rate was calculated in accordance with Eq. (B-8).

$$Erosion\ [lb/(ft^2 \times min)] = \frac{Mass\ of\ Eroded\ Soil\ (lb)}{Lateral\ Area\ (ft^2) \times Time\ (min)} \qquad \text{Eq. (B-8)}$$

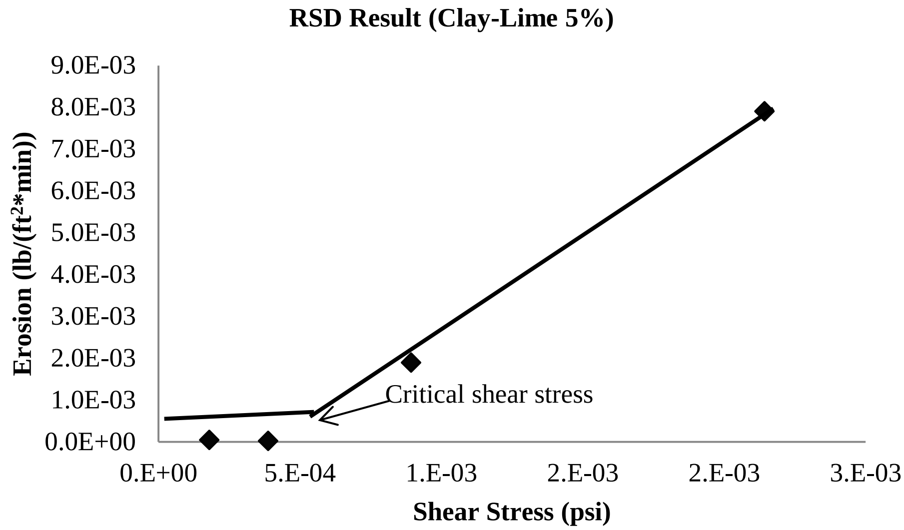
Figure B-49 presents the test results for the three specimens. The stress level at which erosion occurs is the critical shear stress beyond which the erosion increases with an increase in shear stress. In this case, the average critical shear stress for the clay-lime specimens is about 6×10^{-4} psi.



(a) Specimen 1



(b) Specimen 2



(c) Specimen 3

Figure B-49. RSD Results for Clay-Lime Specimens

The erosion potential of the gravel-cement (1% cement) samples also was evaluated. However, negligible erosion occurred. In the field, heavily stabilized materials also can erode (De Beer and Visser 1989). Therefore, strips were added to the cylinder in order to increase the shear stress on the specimen (Figure B-50).

Figure B-50. Strips inside the Cylinder

The effect of the strips on the shear stress was evaluated at various rotational speeds (Figure B-51). At the highest speed of 1300 RPM only, the shear stress increased from 3.78×10^{-3} psi to 4.37×10^{-3} psi, which produced a considerable amount of eroded material (0.31 lb) for the

lime-clay mixtures. However, the use of strips did not lead to erosion of the gravel-cement, and thus RSD is not suitable for cement-gravel material.

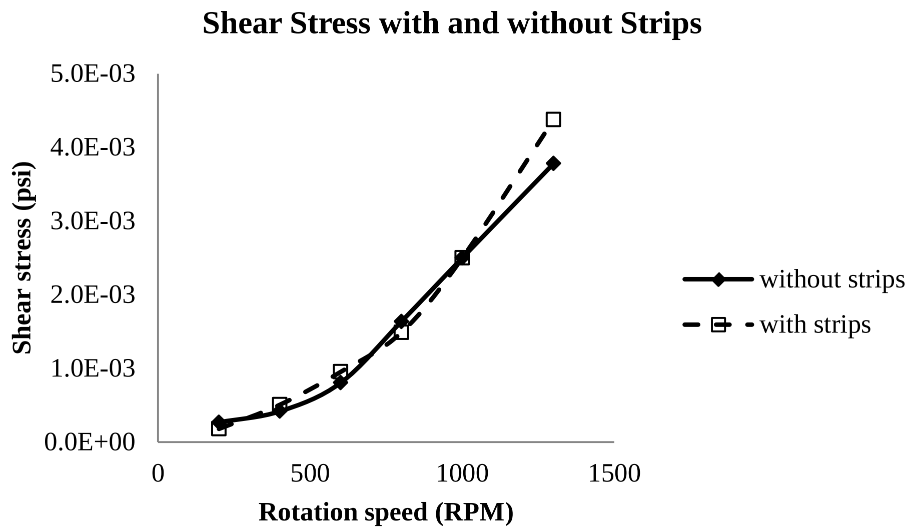


Figure B-51. Effects of Strips on Shear Stress

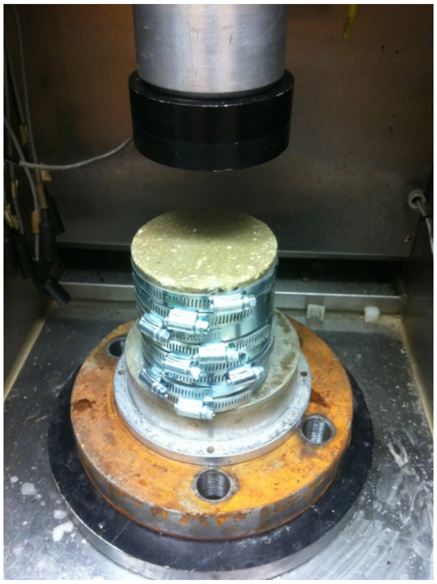
A steel brush was mounted inside the RSD to scratch the cement-gravel cylinder specimen and provide friction. The load was monitored using the load cell of the RSD. Results show that the steel brush produces 10~20 lbs force on the specimen surface, whereas the water-based RSD produces only a 9 lb maximum load. However, the steel brush could not erode the gravel-cement.

A *hole erosion test* (HET), as described by Wahl et al. (2008), was conducted using a cylindrical specimen 6 inches in diameter and 8 inches tall. Holes 1 inch and 0.5 inch in diameter were drilled through the center of the gravel-cement (3%) and clay-lime (5%) specimens, respectively. High-pressure water was used to generate erosion. The diameter change and weight of the eroded materials were monitored. The high-pressure water ran for 15 minutes. Figure B-52 shows the specimens after the HET tests. The clay-lime specimen generated a significant amount of erosion, 0.4 lb, and the diameter of the hole increased from 0.53 in. to 1.58 inches. However, the gravel-cement specimen was strong enough to withstand the high-pressure water and did not erode.

(a) Gravel-Cement *(b) Clay-Lime*

Figure B-52. HET Setup with High-Pressure Water

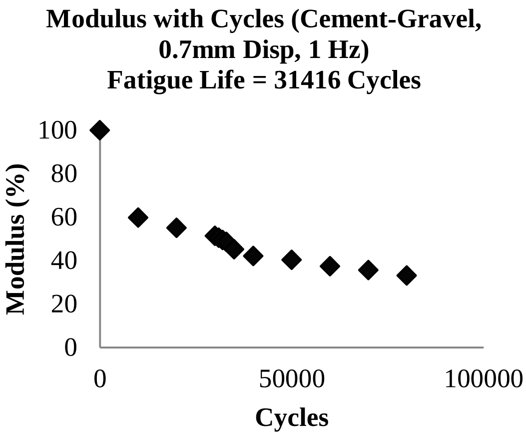
The cyclic impact erosion (CIE) test, developed by Sha and Hu (2002), also is evaluated in this study. A cyclic load with constant displacement was applied to a cylindrical specimen (4 in. diameter and 4.6 in. tall), which was submerged in water, as shown in Figure B-53 (a). The specimen was confined by clamps, as shown in Figure B-53 (b). The confinement from the clamps simulates confinement in the field and may prevent damage from impact loading. When a load is applied to the specimen, water at the interface between the loading plate and specimen gets in and out during the unloading and loading. It was found that the CIE test generates a large amount of eroded material for both the gravel-cement and clay-lime specimens. Constant displacement and frequency were applied to the specimens. When the particles detach from the specimen and erode, the corresponding load decreases for a fixed displacement, which causes the calculated modulus to decrease. This test simulates traffic loading and the interaction between the surface layer and CSL in the presence of water. The loading frequency corresponds with the traffic speed. Erosion is caused by fatigue damage to the CSM and the transportation of fine particles through the shear stress by the water. Figure B-54 shows typical modulus reduction with the increase of load cycles.



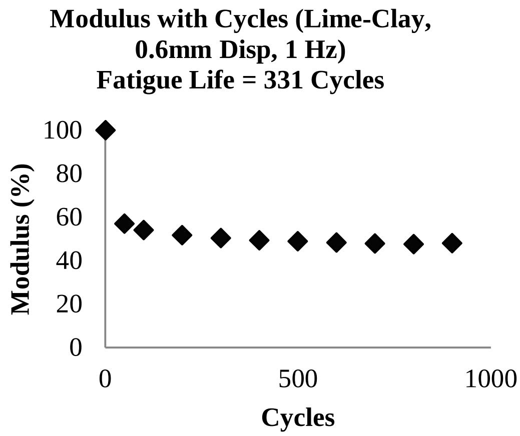
(a) Clamped Specimen

(b) Specimen under Water

Figure B-53. CIE Test Setup



(a) Cement-Gravel



(b) Lime-Clay

Figure B-54. Modulus Reduction with Number of Load Cycles

Based on the water-based RSD, steel brush-based RSD, HET, and CIE tests, it is found that erosion is a fatigue phenomenon. Erosion starts with damage to the CSM and the breakage of the bond between the fine particles and coarse particles, followed by the loss of fines caused by the movement of water. For lightly stabilized materials, fatigue occurs easily and erosion is generated only by the shear stress caused by the movement of water. However, for heavily stabilized materials, the traffic load first causes the fatigue damage. The presence of water saturates and weakens the materials under high traffic loading, and the movement of the water facilitates erosion. Therefore, the CIE test is recommended for top-down compressive-fatigue/erosion testing of CSM.

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Appendix C. Development of Experiments and Findings for Distress Models

CONTENTS

LIST OF FIGURES

LIST OF TABLES

CHAPTER C-1. LABORATORY EXPERIMENTS

Experiments were conducted to develop models for strength/modulus growth, durability, fatigue, erosion and shrinkage cracking.

Strength/Modulus Growth Rate

The strength/modulus growth rate of CSL was studied. If cement is used as a binder, a relative high modulus can form fairly quickly. For lime and/or fly ash, the early-age modulus value is relatively low when compared to that of cement. However, the modulus can grow for years due to additional pozzolanic reactions. Different curing periods were used to evaluate the mechanisms of strength gain: 3, 7, 28, 90, 180, and 360 days. Table C-1 shows the test matrix for the strength growth model development, which includes unconfined compressive strength (UCS), indirect tensile (IDT) strength, and modulus of rupture (MOR). All the specimens were cured at 68°F and 100% relative humidity (RH), except for IDT strength for which different combination of temperature and RH were used.

Table C-1. Strength of Mixtures

<table>
<tr><th colspan="5">Material Characterization</th></tr>
<tr><td colspan="2"></td><td colspan="2">Purpose</td><td>Symbol</td></tr>
<tr><td rowspan="3">Properties</td><td>Unconfined Compressive Strength</td><td colspan="2">Level 1 for top-down compressive-fatigue model and Level 2 for correlation with other strength/moduli values, and catalogue of Level 3 default values</td><td>UCS</td></tr>
<tr><td>IDT Strength</td><td colspan="2">Level 1 for cracking model and catalogue of Level 3 default values</td><td>S_{IDT}</td></tr>
<tr><td>Modulus of Rupture</td><td colspan="2">Level 1 for pavement response model and catalogue of Level 3 default values</td><td>MOR</td></tr>
<tr><td>Factors</td><td>Curing time</td><td colspan="2">Long-term behavior: 3, 7, 28, 90, 180, and 360 days.</td><td>C</td></tr>
<tr><td></td><td>Clay</td><td>Silt</td><td>Sand</td><td>Gravel</td></tr>
<tr><td>Cement</td><td>UCS, S_{IDT}, MOR</td><td>UCS, S_{IDT}, MOR</td><td>UCS, S_{IDT}, MOR</td><td>UCS, S_{IDT}, MOR</td></tr>
<tr><td>Lime</td><td>UCS, S_{IDT}, MOR</td><td>UCS, S_{IDT}, MOR</td><td>(N/A)</td><td>(N/A)</td></tr>
<tr><td>Class C fly ash</td><td>(N/A)</td><td>UCS, S_{IDT}, MOR</td><td>UCS, S_{IDT}, MOR</td><td>UCS, S_{IDT}, MOR</td></tr>
<tr><td>Replicates</td><td colspan="4">1</td></tr>
<tr><td>Strength</td><td>6 curing times × (2UCS+2 S_{IDT}+2 MOR) × 1 replicate = 36</td><td>6 × (3+3+3) × 1 = 54</td><td>6 × (2+2+2) × 1 = 36</td><td>6 × (2+2+2) × 1 = 36</td></tr>
<tr><td>Total</td><td colspan="4">54 UCS + 54 S_{IDT} + 54 MOR tests</td></tr>
</table>

Unconfined Compressive Strength (UCS)

The UCS test is the most common test performed on cementitiously-stabilized materials (CSM) for mix designs for use in pavement bases and subbases. The test methods for the different mixtures follow the recommendations in the Mechanistic-Empirical Pavement Design Guide (MEPDG) (ARA 2004). The specimens for all nine types of mixtures were compacted using standard Proctor compaction in a mold 4 inches in diameter and 4.6 inches in height. At the end of the moist-cure periods, all of the specimens except the lime-clay specimens were soaked in water for 4 hours prior to testing.

For the cement-stabilized fine-grained materials (clay, silt and sand), the ASTM D1633 test method was followed. The compression tests were conducted as soon as possible after removing the specimens from the water. The tests were conducted using a servo-hydraulic machine with the loading rate of 20 psi/sec. The AASHTO T22 test method was followed to test the cement-gravel specimens with the loading rate of 35 psi/sec. For the fly ash-stabilized soils, the ASTM C593 method was followed, and the loading rate was 35 psi/sec.

The lime-clay specimens were capillary-soaked for 24 hours prior to testing. The capillary soaking process is conducted by wrapping the specimens with wet absorptive fabric and placing them on a porous stone. The water level reaches the top of the stone and comes in contact with the fabric wrap throughout the capillary soaking process. However, the specimen is not in direct contact with the water, following the National Lime Association (NLA) procedure (NLA 2006). During the test, the load is applied with an axial deformation rate of approximately 2.0%/min, in accordance with the ASTM D5102 test method.

Figures C-1, C-2, and C-3 present the UCS, MOR, and IDT strength results for all nine mixtures, respectively.

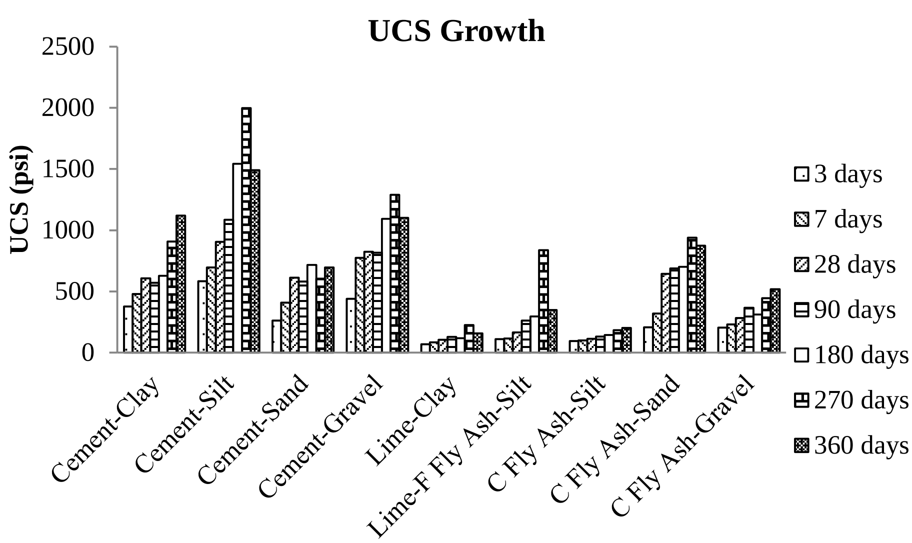

Figure C-1. UCS Growth with Curing Age

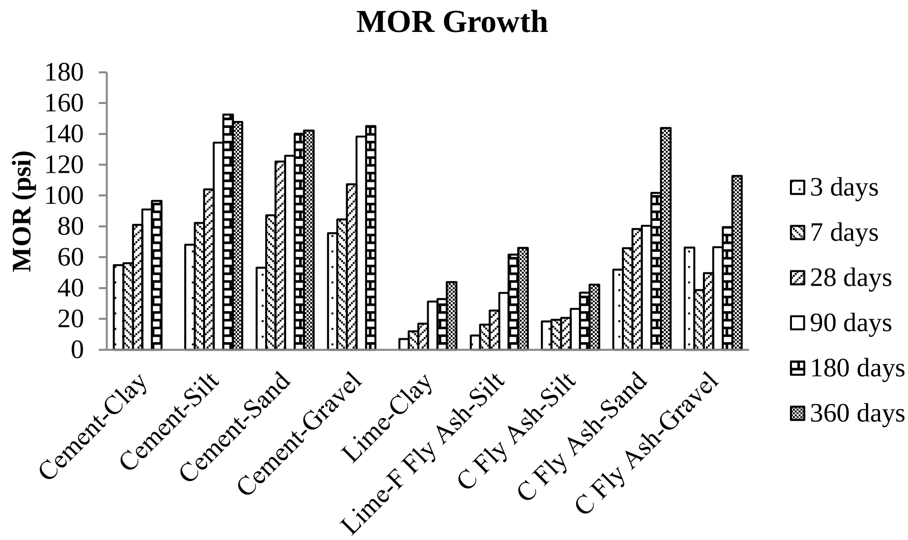

Figure C-2. MOR Growth with Curing Age

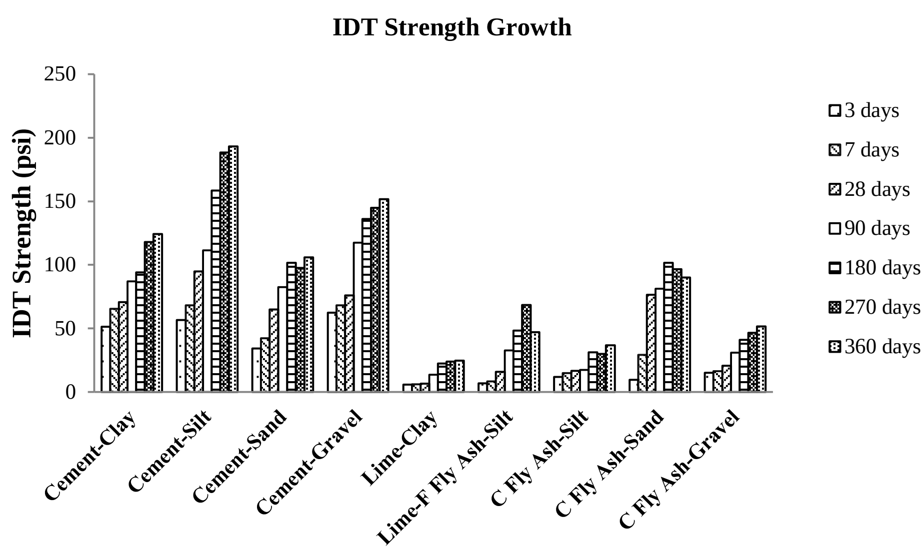

Figure C-3. IDT Strength Growth with Curing Age

In order to establish the shrinkage cracking model, the IDT strength/modulus specimens were prepared for 3, 7, 14, 28, and 56 days curing at various RH and temperatures, because early-stage shrinkage cracking and the IDT strength/moduli are sensitive to environmental conditions. Five mixtures were tested with three replicates, including cement-clay, cement-silt, cement-gravel, lime-clay, and C fly ash-silt, as shown in Table C-2. Figures C-4 and C-5 present the IDT strength and IDT modulus growth test results, respectively.

Table C-2. IDT Strength and Modulus of Mixtures

<table>
<tr><th colspan="5">Material Characterization</th></tr>
<tr><td colspan="2"></td><td colspan="2">Purpose</td><td>Symbol</td></tr>
<tr><td rowspan="2">Properties</td><td>IDT Strength</td><td colspan="2">Level 1 for cracking response model and catalogue of Level 3 default values</td><td>S_{IDT}</td></tr>
<tr><td>IDT Modulus</td><td colspan="2">Level 1 for cracking response model and catalogue of Level 3 default values</td><td>E_T</td></tr>
<tr><td>Factors</td><td>Curing time</td><td colspan="2">Long-term behavior: 3, 7, 14, 28, and 56 days at various temperatures and RH</td><td>C</td></tr>
<tr><td></td><td>Clay</td><td>Silt</td><td>Sand</td><td>Gravel</td></tr>
<tr><td>Cement</td><td>S_{IDT}, E_t</td><td>S_{IDT}, E_t</td><td>-</td><td>S_{IDT}, E_t</td></tr>
<tr><td>Lime</td><td>S_{IDT}, E_t</td><td>-</td><td>(N/A)</td><td>(N/A)</td></tr>
<tr><td>Class C fly ash</td><td>(N/A)</td><td>S_{IDT}, E_t</td><td>-</td><td>-</td></tr>
<tr><td>Replicates</td><td colspan="4">3</td></tr>
<tr><td>Total</td><td colspan="4">12 materials/environment × 5 curing time × 3 replicates =180 IDT Strength and 180 IDT Modulus Tests</td></tr>
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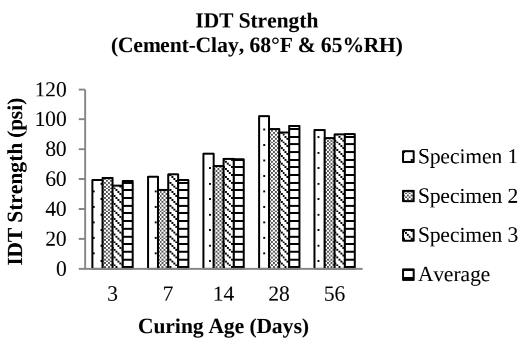
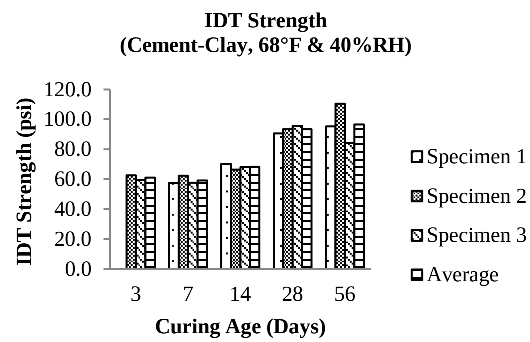
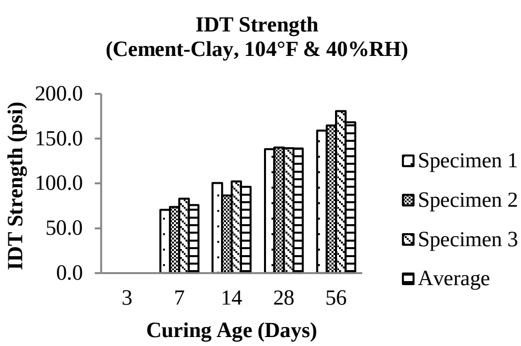


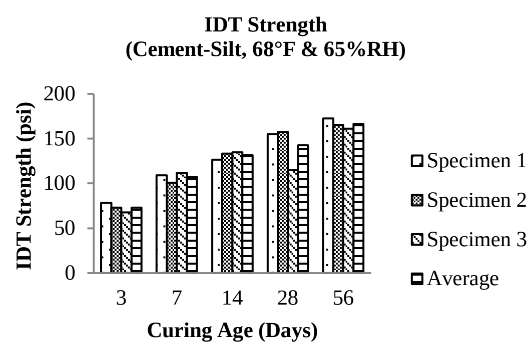
(a) Cement-Clay, 68°F & 65%RH



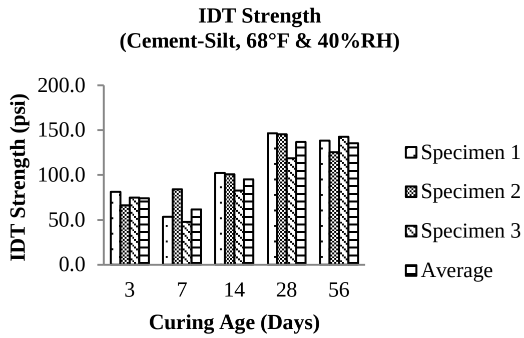
(b) Cement-Clay, 68°F & 40%RH



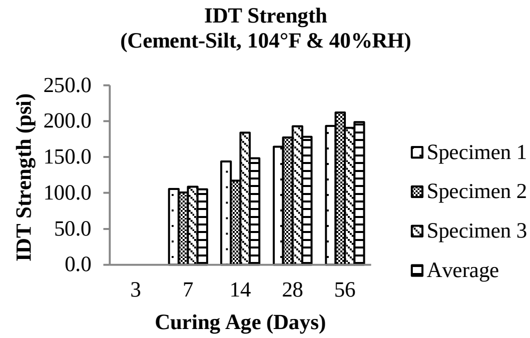
(c) Cement-Clay, 104°F & 40%RH



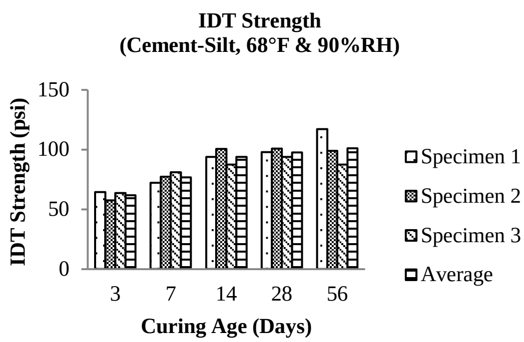
(d) Cement-Silt, 68°F & 65%RH



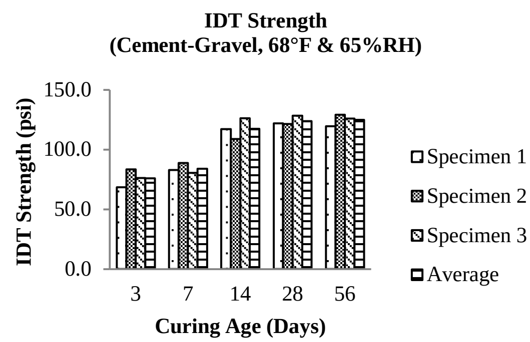
(e) Cement-Silt, 68°F & 40%RH



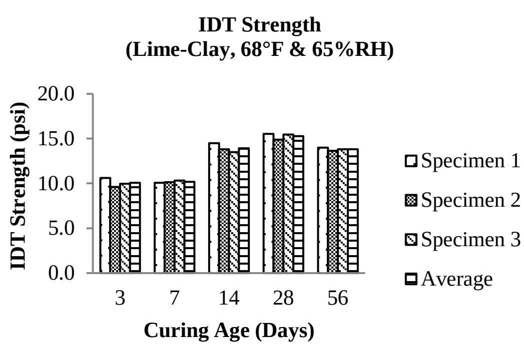
(f) Cement-Silt, 104°F & 40%RH



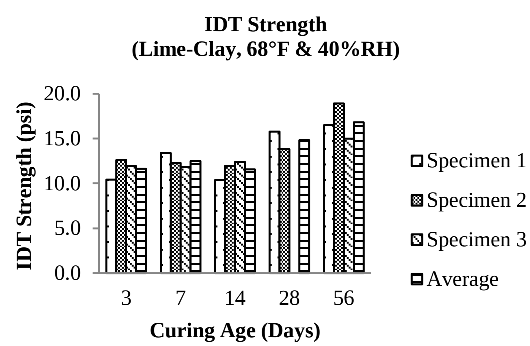
(g) Cement-Silt, 68°F & 90%RH



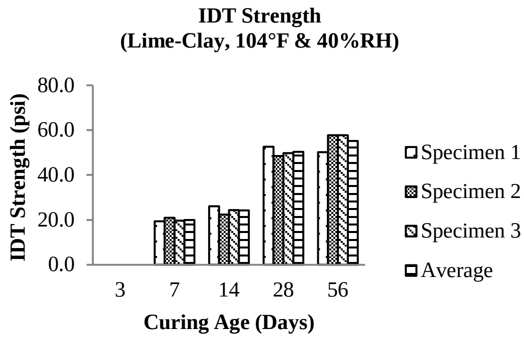
(h) Cement-Gravel, 68°F & 65%RH



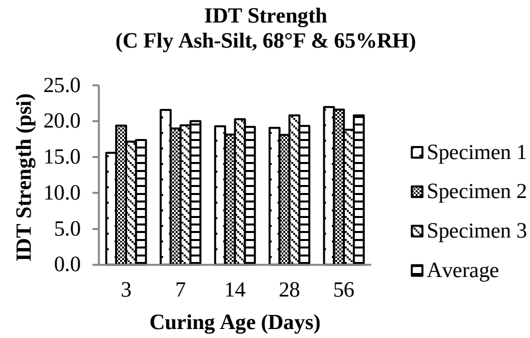
(i) Lime-Clay, 68°F & 65%RH



(j) Lime-Clay, 68°F & 40%RH

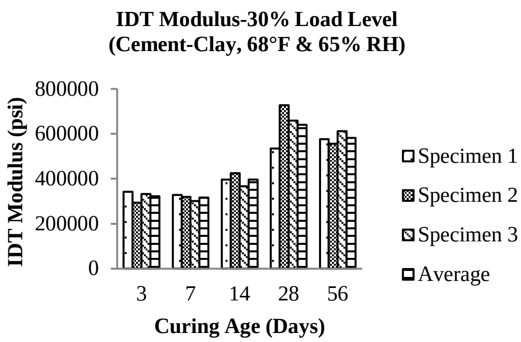


(k) Lime-Clay, 104°F & 40%RH

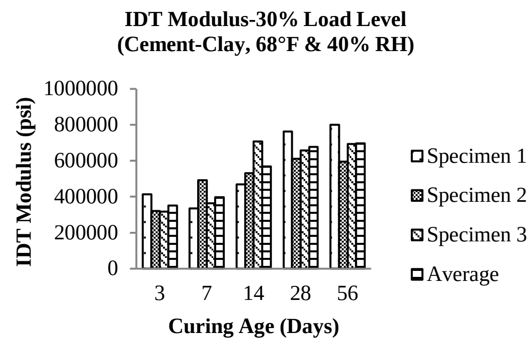


(l) C Fly Ash-Silt, 68°F & 65%RH

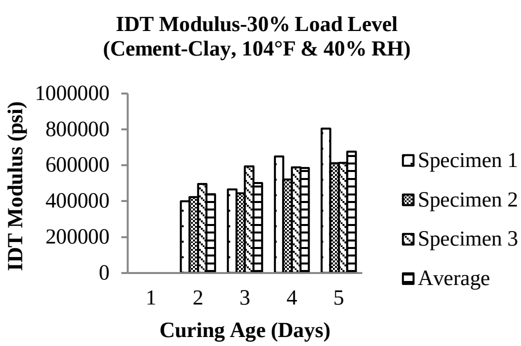
Figure C-4. IDT Strength Values at Various RH and Temperatures with Curing Age



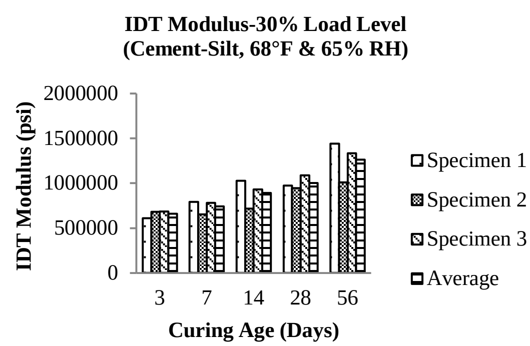
(a) Cement-Clay, 68°F & 65% RH



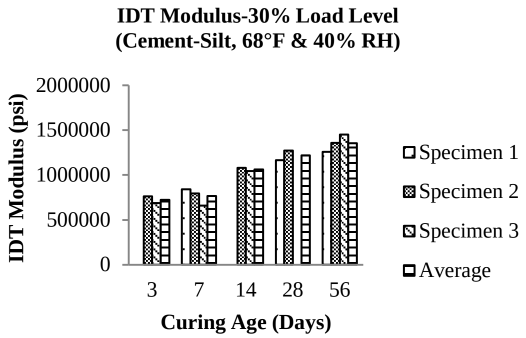
(b) Cement-Clay, 68°F & 40% RH



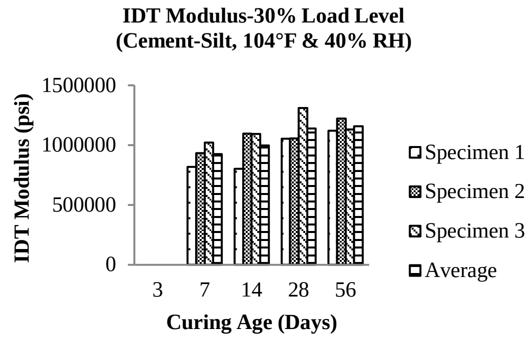
(c) Cement-Clay, 104°F & 40% RH



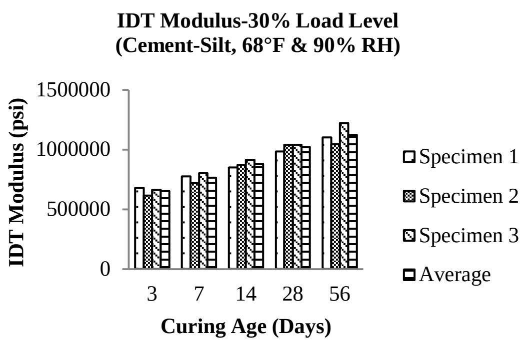
(d) Cement-Silt, 68°F & 65% RH



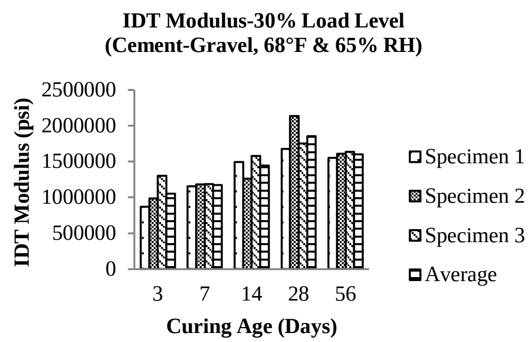
(e) Cement-Silt, 68°F & 40% RH



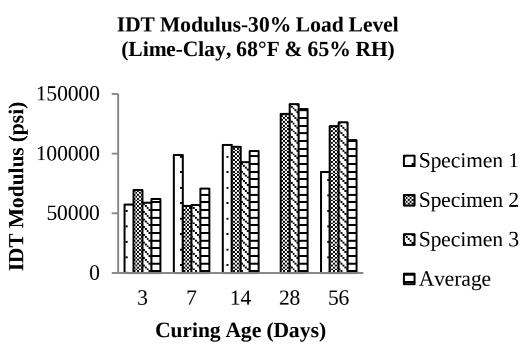
(f) Cement-Silt, 104°FS & 40% RH



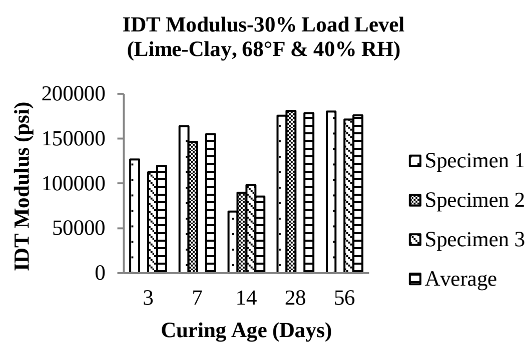
(g) Cement-Silt, 68°F & 90% RH



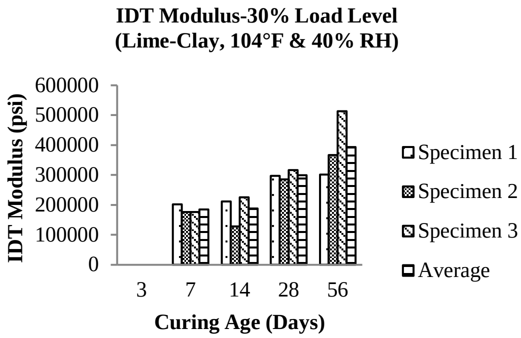
(h) Cement-Gravel, 68°F & 65% RH



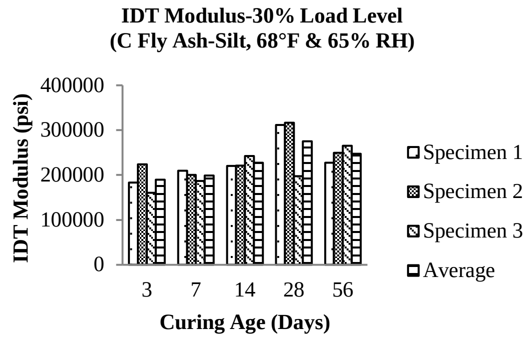
(i) Lime-Clay, 68°F & 65% RH



(j) Lime-Clay, 68°F & 40% RH



(k) Lime-Clay, 104°F & 40% RH



(l) C Fly Ash-Silt, 68°F & 65% RH

Figure C-5. IDT Modulus Values at Various RH and Temperatures with Curing Age

Flexural Modulus

The flexural modulus test was conducted for all nine original mixtures. Additional mixtures, including gravel with 5% cement, sand with 8% cement, and silt with 18% Class C fly ash, were included. Table C-3 presents the test matrix.

Table C-3. Modulus of Mixtures

Material Characterization				
		Purpose		Symbol
Properties	Flexural Modulus	Level 1 for pavement response model and catalogue of Level 3 default values		E_f - flexural modulus;
Factors	Curing time	Cement-stabilized mixtures: 100% RH, 73.4°F for 28 days Fly ash-stabilized mixtures, lime-clay and lime-F fly ash-silt: sealed with plastic wrap and cured in an oven set to 104°F for 7 days.		C
	Clay	Silt	Sand	Gravel
Cement	E_f	E_f	E_f	E_f
Lime	E_f	E_f	(N/A)	(N/A)
Class C fly ash	(N/A)	E_f	E_f	E_f
Replicates	3			
Total	**36 E_f**			

Different curing procedures were applied to different mixtures depending on the binder. The cement-stabilized mixtures (gravel, sand, silt, and clay) were cured in a moist room (100% RH, 73.4°F) for 28 days (ASTM D558). The fly ash-stabilized mixtures (sand, silt, and gravel), lime-clay and lime-F fly ash-silt were sealed with plastic wrap and cured in an oven set to 104°F (ASTM C593) for 7 days. Table C-4 presents the flexural modulus test results and corresponding UCS and MOR test results. The relationship between the flexural modulus and UCS values and

between the flexural modulus and the MOR values can be developed. The growth of the UCS and MOR was used to establish the growth of the flexural modulus, as shown in the main report.

Table C-4. Flexural Modulus Results with Corresponding UCS and MOR Results

Materials	UCS (psi)	MOR (psi)	Flexural Modulus (psi)
Clay–cement (12%)	534	94	144,893
Gravel–cement (3%)	640	79	134,740
Sand–cement (6%)	521	71	141,122
Silt–cement (8%)	653	81	131,114
Clay–lime (6%)	149	27	87,168
Gravel–class C fly ash (13%)	289	49	119,076
Sand–class C fly ash (13%)	206	51	104,427
Silt–class C fly ash (13%)	91	18	65,847
Silt–lime–class F fly ash (4%-12%)	271	48	124,297
Gravel–cement (5%)		145	193,625
Sand–cement (8%)		145	200,007
Silt–class C fly ash (18%)		28	85,137

Durability

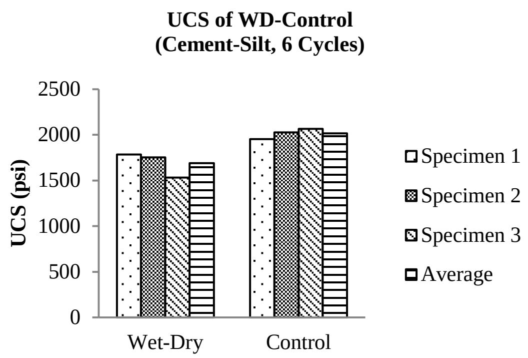
Durability tests, including wet-dry and freeze-thaw tests, were conducted to develop the durability models. Cement-silt, C fly ash-gravel, lime-clay, and C fly ash-silt materials were selected to represent materials with strength values from high to low for wet-dry testing. Table C-5 presents the test matrix.

Table C-5. Laboratory Test Plan for Durability Model

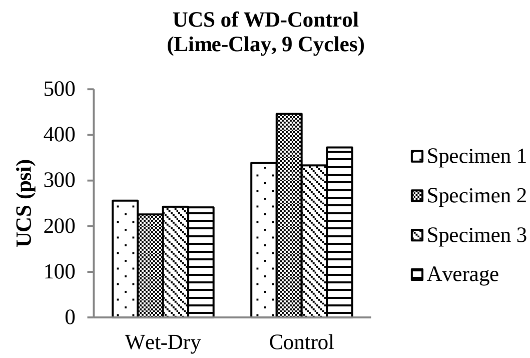
<table>
<tr><th colspan="5">Durability Model</th></tr>
<tr><td colspan="3"></td><td>Purpose</td><td>Symbol</td></tr>
<tr><td rowspan="2">Durability Tests</td><td colspan="2">Freeze-thaw</td><td>Develop freeze-thaw durability model</td><td>F-T</td></tr>
<tr><td colspan="2">Wet-dry</td><td>Develop wet-dry durability model</td><td>W-D</td></tr>
<tr><td></td><td>Clay</td><td>Silt</td><td>Sand</td><td>Gravel</td></tr>
<tr><td>Cement</td><td>F-T</td><td>F-T, W-D</td><td>F-T</td><td>F-T</td></tr>
<tr><td>Lime</td><td>F-T, W-D</td><td>F-T</td><td>(N/A)</td><td>(N/A)</td></tr>
<tr><td>Class C fly ash</td><td>(N/A)</td><td>F-T, W-D</td><td>F-T</td><td>F-T, W-D</td></tr>
<tr><td>Total</td><td colspan="4">39 F-T + 30 W-D Tests</td></tr>
</table>

Wet-Dry

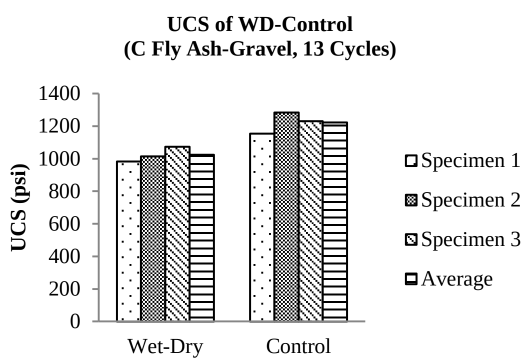
Figure C-6 presents the UCS results of the wet-dry tests. It is noted that the control group is subjected to the same temperature as the wet-dry group, except that the control group is not submerged in water, as per the procedures outlined in Appendix B. This is to single out the effect of moisture only and to eliminate the effects of high temperature on the material properties.



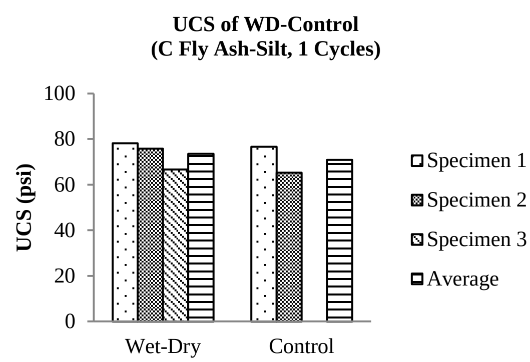
(a) Cement-Silt (6 Cycles)



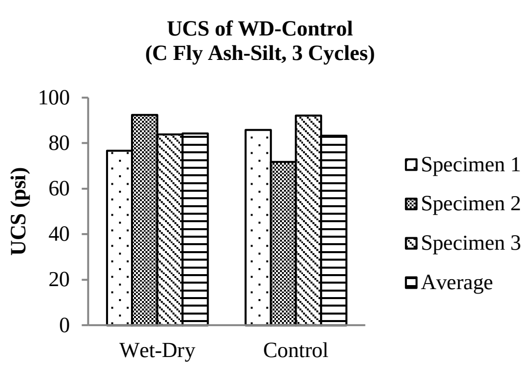
(b) Lime-Clay (9 Cycles)



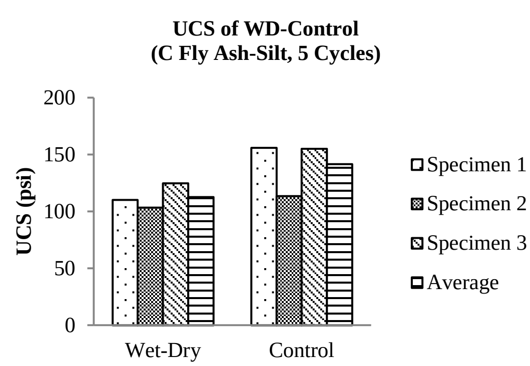
(c) C Fly Ash-Gravel (13 Cycles)



(d) C Fly Ash-Silt (1 Cycles)



(e) C Fly Ash-Silt (3 Cycles)

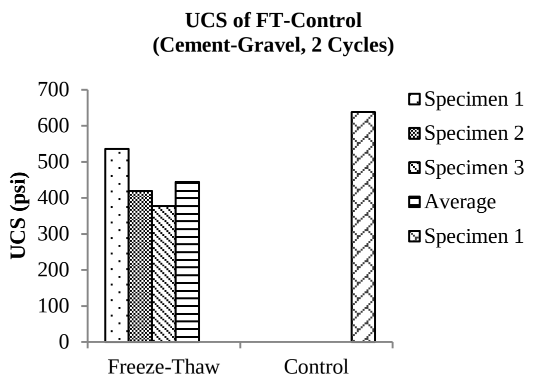


(f) C Fly Ash-Silt (5 Cycles)

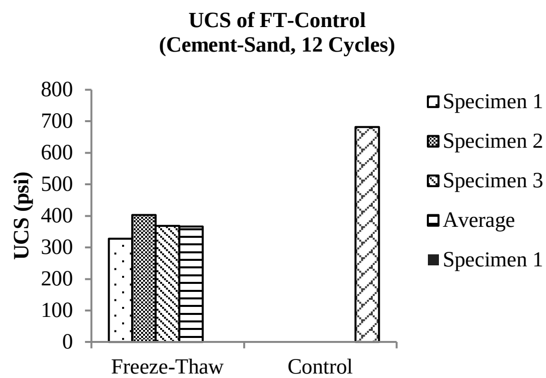
Figure C-6. UCS Test Results of Wet-Dry and Control Specimens

Freeze-Thaw

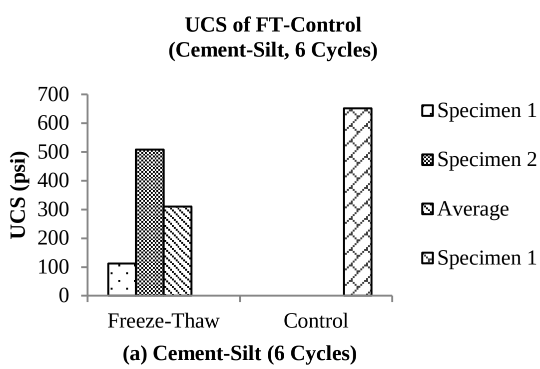
Freeze-thaw tests were conducted to develop freeze-thaw durability models. The cement-stabilized mixtures (gravel, sand, silt, and clay) were cured in a moist room at 100% RH and 70°F for 28 days in accordance with ASTM D558. Based on the test procedure evaluation results presented in Appendix B, the fly ash-stabilized mixtures (sand, silt, and gravel), clay-lime, and silt-lime-Class F fly ash were sealed with plastic wrap and cured for 7 days in an oven set to 104°F in accordance with ASTM C593. The number of freeze-thaw cycles was varied amongst the different mixtures to incorporate the effect of the number of cycles in the model. The UCS of the specimens was measured after the designated freeze-thaw cycles. Figure C-7 presents the UCS results before and after the freeze-thaw tests.



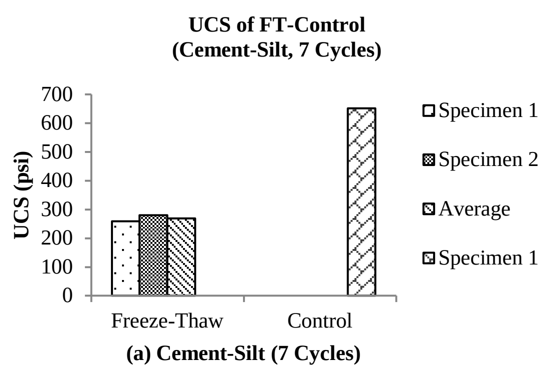
(a) Cement-Gravel (2 Cycles)



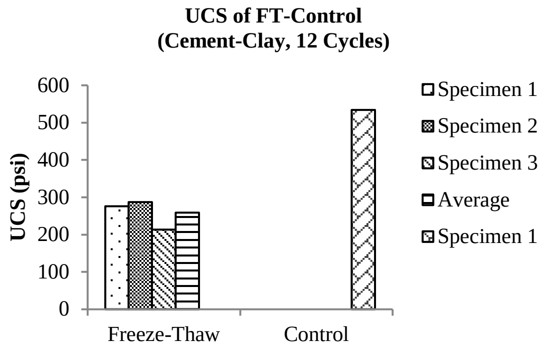
(b) Cement-Sand (12 Cycles)



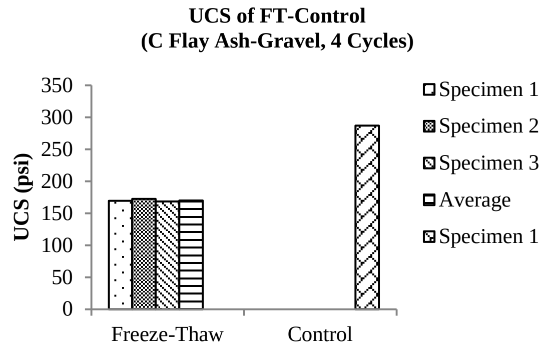
(c) Cement-Silt (6 Cycles)



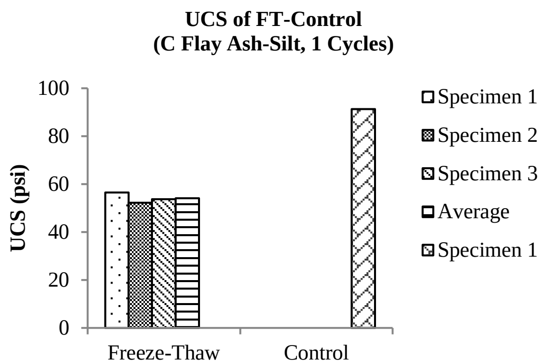
(d) Cement-Silt (7 Cycles)



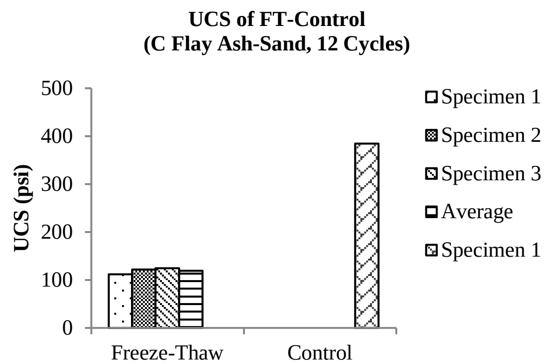
(e) Cement-Clay (12 Cycles)



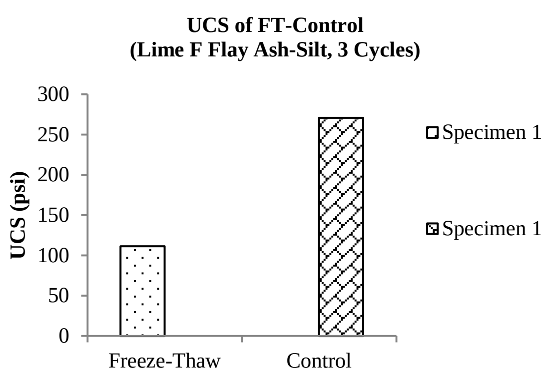
(f) C Fly Ash-Gravel (4 Cycles)



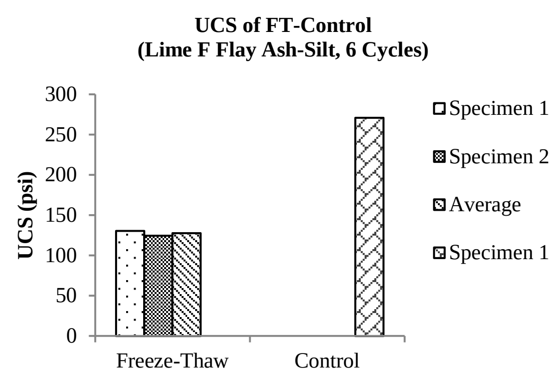
(g) C Fly Ash-Silt (1 Cycles)



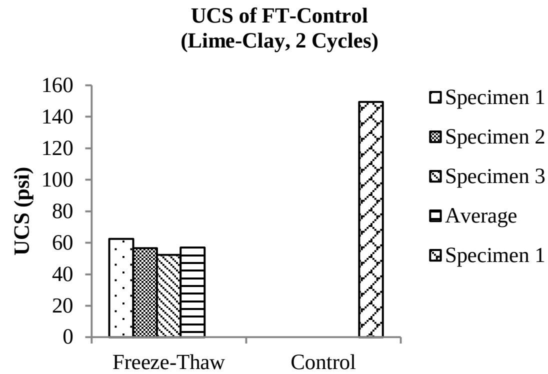
(h) C Fly Ash-Sand (12 Cycles)



(i) Lime C Fly Ash-Silt (3 Cycles)



(j) Lime C Fly Ash-Silt (6 Cycles)



(k) Lime-Clay (2 Cycles)

Figure C-7. UCS Test Results of Freeze-Thaw and Control Specimens

Fatigue Test

Bottom-Up Tensile-Fatigue Test

The bottom-up tensile-fatigue test was initiated immediately after the flexural modulus test on the same specimen. The same setup as that used in the flexural modulus test was followed for the bottom-up tensile-fatigue test. The peak magnitude of the haversine load pulses ranges from 45% to 95% of the breaking load (based on the beams that were tested for strength). The bottom-up tensile-fatigue test was conducted with a haversine pulse width of 250-ms duration with 250-ms rest between pulses for a total 500-ms pulse periods (2 Hz frequency) to expedite the tests, which is different from the test procedure evaluation presented in Appendix B. Table C-6 presents the laboratory test plan.

Top-Down Compressive-Fatigue-Erosion Test

The top-down compressive-fatigue-erosion test was developed based on the cyclic impact erosion (CIE) test. Table C-7 presents the test plan. Different displacement levels and frequencies were used. Table C-8 presents the measured fatigue life data. *Fatigue life* is defined as the number of cycles that can reduce the initial modulus value by 50 percent.

Table C-6. Laboratory Test Plan for Bottom-Up Tensile-Fatigue Test

<table>
<tr><th colspan="5">Fatigue Model</th></tr>
<tr><td colspan="2"></td><td colspan="2">Purpose</td><td>Symbol</td></tr>
<tr><td rowspan="2">Material Properties</td><td>Modulus of Rupture</td><td colspan="2">Level 1 material properties in the fatigue model and catalog of Level 3 default values</td><td>MOR</td></tr>
<tr><td>Unconfined Compressive Strength</td><td colspan="2">Correlation with other strength values</td><td>UCS</td></tr>
<tr><td>Fatigue Tests</td><td>Beam Fatigue</td><td colspan="2">Fatigue model development</td><td>Beam</td></tr>
<tr><td></td><td>Clay</td><td>Silt</td><td>Sand</td><td>Gravel</td></tr>
<tr><td>Cement</td><td>MOR, UCS, Beam</td><td>MOR, UCS, Beam</td><td>MOR, UCS, Beam</td><td>MOR, UCS, Beam</td></tr>
<tr><td>Lime</td><td>MOR, UCS, Beam</td><td>MOR, UCS, Beam</td><td>(N/A)</td><td>(N/A)</td></tr>
<tr><td>Class C fly ash</td><td>(N/A)</td><td>MOR, UCS, Beam</td><td>MOR, UCS, Beam</td><td>MOR, UCS, Beam</td></tr>
<tr><td>Replicate</td><td colspan="4">1</td></tr>
<tr><td>Subtotal</td><td>2 MOR +2 UCS + 2 beam</td><td>3+3+3</td><td>2+2+2</td><td>2+2+2</td></tr>
<tr><td>Total</td><td colspan="4">12 MOR + 12 UCS + 12 beam fatigue tests</td></tr>
</table>

Table C-7 Laboratory Test Plan for Top-Down Compressive-Fatigue-Erosion Test

<table>
<tr><th colspan="5">Erosion Model</th></tr>
<tr><td colspan="2"></td><td colspan="2">Purpose</td><td>Symbol</td></tr>
<tr><td>Material Property</td><td>Unconfined Compressive Strength</td><td colspan="2">Material property in erosion model</td><td>UCS</td></tr>
<tr><td>Model Development</td><td>Cyclic Impact Erosion</td><td colspan="2">Model validation</td><td>CIE</td></tr>
<tr><td></td><td>Clay</td><td>Silt</td><td>Sand</td><td>Gravel</td></tr>
<tr><td>Cement</td><td>-</td><td>-</td><td>-</td><td>UCS, CIE</td></tr>
<tr><td>Lime</td><td>UCS, CIE</td><td>-</td><td>(N/A)</td><td>(N/A)</td></tr>
<tr><td>Class C fly ash</td><td>(N/A)</td><td>UCS, CIE</td><td>UCS, CIE</td><td>UCS, CIE</td></tr>
<tr><td>Total</td><td colspan="4">5 UCS + 21 CIE</td></tr>
</table>

Table C-8 Laboratory CIE Test Results

Materials	Frequency (Hz)	Initial Displacement (in.)	Measured Fatigue Life
Cement-Silt	1	0.031	388
Lime-Clay	4	0.031	31
C Fly Ash-Silt	10	0.016	90
C Fly Ash-Sand	10	0.015	11,416
C Fly Ash-Gravel	1	0.031	3,435
Cement-Gravel	1	0.028	31,416
	1	0.034	3,098
	1	0.041	313
	1	0.048	424
C Fly Ash-Silt	2	0.015	710
	2	0.017	1,344
	1	0.028	870
	2	0.023	41
	2	0.026	16
	2	0.031	30
Lime-Clay	1	0.015	97
	1	0.021	331
	1	0.029	80
	1	0.036	70
	1	0.044	46
	1	0.051	35

Shrinkage and Cracking Test

The shrinkage cracking models consist of models for ultimate shrinkage strain, gradient drying shrinkage strain, crack spacing and width. Table C-9 presents the laboratory test plan for the shrinkage model.

Table C-9. Laboratory Test Plan for Shrinkage Model

Shrinkage Model				
		Purpose		Symbol
Material Properties	IDT Modulus	Material property in the cracking spacing and width model		E_t
	IDT Strength	Material property in the cracking spacing and width model		S_{IDT}
Model Development Tests	Ultimate Drying Shrinkage	Input to shrinkage strain model		ε_{su}
	Gradient Drying Shrinkage Strain	Shrinkage strain model development		ε_g
	Restrained Shrinkage Cracking	Shrinkage cracking model development		R
	Interface Friction	Input to shrinkage cracking model		μ
	Thermal Shrinkage	Thermal shrinkage measurement		ε_T
	Clay	Silt	Sand	Gravel
Cement	E_t, S_{IDT}, ε_{su}, R, μ, ε_T	E_t, S_{IDT}, ε_{su}, ε_g, R, μ, ε_T	-	E_t, S_{IDT}, ε_{su}, μ, ε_T
Lime	E_t, S_{IDT}, ε_{su}, ε_g, R, μ, ε_T	-	(N/A)	(N/A)
Class C fly ash	(N/A)	E_t, S_{IDT}, ε_{su},	-	ε_{su},
Subtotal	**$90E_t+90S_{IDT}+2\varepsilon_{su}+\varepsilon_g+6R+9\mu+12\varepsilon_T$**	**$75E_t+75S_{IDT}+2\varepsilon_{su}+\varepsilon_g+R+12\mu+6\varepsilon_T$**	-	**$15E_t+15S_{IDT}+2\varepsilon_{su}+0\varepsilon_g+0R+3\mu+6\varepsilon_T$**
Total	**180 IDT modulus + 180 IDT strength +2 gradient drying shrinkage + 7 restrained drying shrinkage+24 bond strength tests + 24 thermal shrinkage/expansion**			

Ultimate drying shrinkage strain is an input for the gradient drying shrinkage, crack spacing and width models. Table C-10 presents the measured ultimate shrinkage strain results.

Table C-10. Measured Ultimate Shrinkage Strain

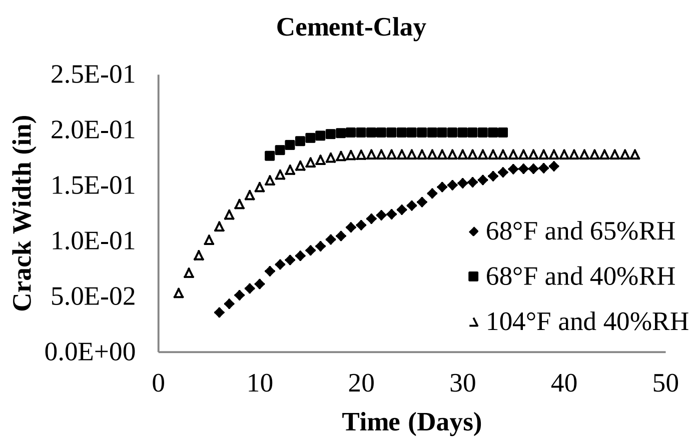
Material	**Optimum Moisture Content (%)**	**Measured Ultimate Strain ($\times 10^{-6}$)**
C Fly Ash-Silt	10.0	1313
C Fly Ash-Gravel	7.4	628
Cement-Gravel	6.2	1467
Cement-Clay	18.0	12430
Cement-Silt	11.1	3259
Lime-Clay	19.4	14438

The gradient drying shrinkage model was developed based on the measured drying shrinkage strain at four depths from the exposed surface, i.e., 0, 2.25, 4.5 and 6.75 inches, respectively. Two beam specimens were fabricated for this test at room temperature (68°F), a cement-silt specimen was monitored at 65% RH, and a lime-clay specimen was monitored at 40% RH.

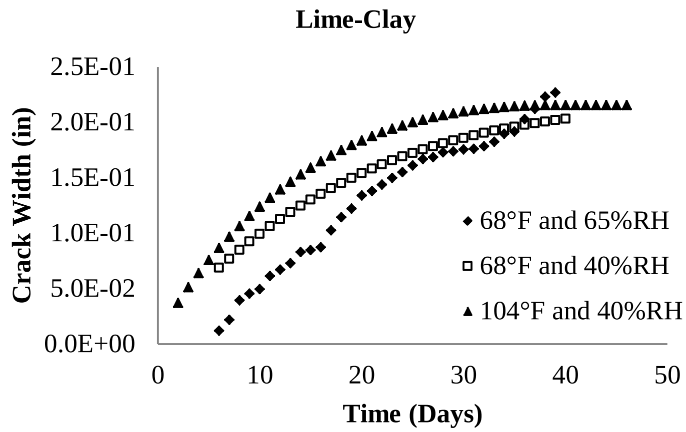
The restrained shrinkage cracking tests were conducted to create shrinkage cracking and develop the crack spacing and width models. Three environmental conditions, 68°F and 65% RH, 68°F and 40% RH, and 104°F and 40% RH, were used for the restrained shrinkage cracking tests. Epoxy was used as the bonding agent. One replicate was used for each of the combinations. Three mixtures, i.e. cement-clay, cement-silt, and lime-clay, were selected for the restrained shrinkage cracking tests. Seven specimens experienced shrinkage cracking. Table C-11 and Figure C-8 show the measured crack spacing and width results, respectively.

Table C-11. Summary of Laboratory Crack Tests

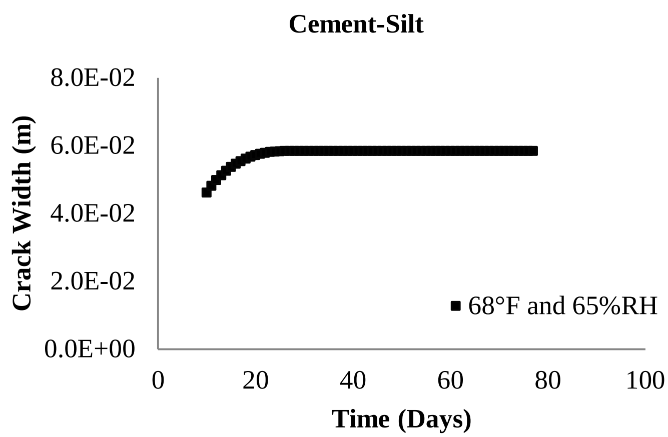
Materials	Environment	No. of Cracks	Crack Spacing (in.)	Age when Crack Occurred (days)
Cement-Clay	68°F and 65% RH	1	24	6
	68°F and 40% RH	2	16	11
	104°F and 40% RH	2	16	2
Lime-Clay	68°F and 65% RH	1	24	6
	68°F and 40% RH	1	24	4
	104°F and 40% RH	1	24	2
Cement-Silt	68°F and 65% RH	1	24	10



(a) Cement-Clay



(b) Lime-Clay



(c) Cement-Silt

Figure C-8. Measured Crack Width

CHAPTER C-2. DEVELOPMENT OF PERFORMANCE MODELS

The performance models, including fatigue, durability, erosion, and shrinkage cracking models, were developed based on the laboratory experiments.

Fatigue Models

Fatigue is one of the major distresses of cementitiously-stabilized layers (CSL) in a pavement. A fatigue model of CSL is needed to predict the fatigue performance of CSL in the field. Two types of fatigue have been identified: bottom tension and top compression. Bottom tension fatigue is due to the repeated tensile stress/strain at the bottom of the CSL due to repeated traffic loads. Top compression fatigue stems from repeated compressive stress at the top of the CSL.

Bottom-up Fatigue Models

The fatigue life (N) of CSL due to tension at the bottom of the layer usually is considered to be a function of the applied tensile stress (σ_t), the applied tensile strain (ε_t), or a ratio of these responses to the breaking stress (σ_b) or breaking strain (ε_b) of the material. The general form of the relationship typically is shown as Eq. (C-1).

$$\log N = f_n\left[\frac{\sigma_t}{\sigma_b}\ or\ \frac{\varepsilon_t}{\varepsilon_b}\right] \qquad \text{Eq. (C-1)}$$

Various forms of the fatigue models are summarized by Yeo (2008), as follows.

1. Strain-based models

<u>(a) South Africa Department of Transport model (Freeme et al. 1982)</u>

$$Log\ N = 9.1\ \left[1 - \frac{d_s\varepsilon_t}{\varepsilon_b}\right] \qquad \text{Eq. (C-2)}$$

where

d_s = factor to account for shrinkage crack-induced stress concentration, as shown in Table C-12.

The above South African model was revised in 1995 with a shift factor (*SF*) to account for crack propagation induced by load repetition (Theyse et al. 1995), as follows.

$SF = 10^{(0.00285d - 0.293)}$ if 102 < d < 419;

$SF = 1$ if $d < 102$;
$SF = 8$ if $d > 419$,

where

d = the thickness in mm, 1 inch = 25.4 mm

Table C-12. Correction Factors for Tensile Stress (Otte 1978)

Type of cracking	Unconfined compressive strength (psi)	Factor, d for total thickness of cemented material (in.)	
		<8	>8
Weakly cemented:			
Moderate cracking; crack widths less than 0.08 in. (e.g., natural materials with lime or 2% to 3% cement)	109 - 218	1.1	1.2
	218 - 435	1.15	1.3
Strongly cemented			
Extensive cracking; crack widths more than 0.08 in. (e.g., high-quality natural gravel and crushed stone with 4% to 6% cement)	435 - 1740	1.25	1.4

(b) Australian models

The relationship between the maximum tensile strain in cemented materials produced by a specific load and the allowable number of repetitions of that load is given by the following expression (National Association of Australian State Road Authorities (NAASRA) 1987).

$$Log\ N = 18\ Log\ \left[\frac{C}{\varepsilon_t}\right] \qquad \text{Eq. (C-3)}$$

where

$C = 280$; $E = 290$ psi
$C = 200$; $E = 725$ psi
$C = 150$; $E \geqslant 1450$ psi
E = modulus of the cemented material.

Litnowicz and Brandon (1994) note a disadvantage with the above fatigue relationship for cemented materials, as the implied values of the strain at the breaking point (ε_b) for the various cemented material moduli tend to exceed the upper bound of the laboratory ε_b values

reported in the literature. As such, the current Austroads fatigue relationship for cemented materials is reworked as:

$$Log\ N = RF \cdot 12 \left[\frac{\frac{11300}{E^{0.804}} + 191}{\epsilon_t} \right] \qquad \text{Eq. (C-4)}$$

where

RF = reliability factor (RF = 2 for 90% reliability, RF = 1 for 95% reliability and RF = 0.5 for 97.5% reliability), and the other terms are as described previously.

2. Stress-based Models

(a) French model

The French design for CSL is based on direct tensile strength, as follows (Corte and Goux 1996).

$$Log\ N = \frac{1}{B}\left(\frac{\sigma_t}{\sigma_b} - 1\right) \qquad \text{Eq. (C-5)}$$

where

B = regression constant.

(b) MEPDG fatigue model

The MEPDG proposes the following fatigue distress model (ARA 2004).

$$\log N = \left[\frac{k_1 \beta_{c1} - \left[\frac{\sigma_s}{MOR} \right]}{k_2 \beta_{c2}} \right] \qquad \text{Eq. (C-6)}$$

where

N = number of repetitions to fatigue cracking

σ_s = tensile stress at the bottom of CSL, psi

MOR = modulus of rupture, psi

k_1, k_2, β_{c1}, and β_{c2} = regression/calibration coefficients.

The accumulated damage is based on Miner's law in the MEPDG, as shown in Eq. (C-7) (ARA 2004).

$$D = \sum_{i=1}^{j} \frac{n_i}{N_i} \qquad \text{Eq. (C-7)}$$

where

n_i = actual number of repetitions of load group i
N_i = allowed number of repetitions to fatigue of load group i
j = total number of load groups.

According to the MEPDG, after fatigue damage is induced, the modulus of the CSL is reduced, which affects the pavement response. The modulus of the deteriorated CSL in the MEPDG is expressed as (ARA 2004):

$$E_{CSM}(t) = E_{CSM}(min) + \frac{E_{CSM}(max) - E_{CSM}(min)}{1+e^{(-4+14D)}} \qquad \text{Eq. (C-8)}$$

where
$E_{CSM}(t)$ = new CSL modulus at a damage level of D, psi.
$E_{CSM}(max)$ = maximum CSL modulus for intact layer, psi.
$E_{CSM}(min)$ = minimum CSL modulus after total layer destruction, psi.
D = CSL damage level, in decimal form.

For the minimum modulus values for CSL after damage, Table C-13 is used in the MEPDG (ARA 2004).

Table C-13. Deteriorated Modulus of CSL

Chemically Stabilized Material	Deteriorated M_f Typical, psi
Lean concrete	300,000
Cement-stabilized aggregate	100,000
Open-graded cement-stabilized	50,000
Soil-cement	25,000
Lime-cement-fly ash	40,000
Lime-stabilized soils	15,000

After the damaged modulus of the CSL is determined, the damaged modulus is then used in the MEPDG for pavement analysis within each time interval of the analysis.

The fatigue cracking of the CSL in the MEPDG is determined based on Eq. (C-9) (ARA 2004).

$$C = \frac{1000}{1+e^{(1-D)}} \qquad \text{Eq. (C-9)}$$

where
C = CSL cracking in units of feet per 500-ft long sections.
D = accumulated fatigue damage.

Top-down Compressive-Fatigue Model

The South Africa study proposes a top-down compressive-fatigue model as shown by Eq. (C-10) (De Beer 1990).

$$log\,N = k_1\left(1 - \frac{\sigma_c}{k_2\,UCS}\right) \qquad \text{Eq. (C-10)}$$

where

N = number of repetitions to fatigue cracking
σ_c = compressive stress at the top of CSL, psi
UCS = unconfined compressive strength, psi
k_1 *and* k_2 = regression/calibration coefficients, k_1 = 8.21, and k_2 = 1 in original South African model.

Evaluation of Fatigue Models

Otte (1978) developed a strain ratio-based fatigue model, which is adopted by the South Africa design guide. Yeo (2008) evaluated the Austroad fatigue model based on laboratory testing and FHWA ALF pavement performance. Yeo (2008) found that the load-strain exponent in the initial strain-based fatigue model is different for different CSM. Carteret et al. (2009) studied the strain-based fatigue model, including the South African strain ratio-based fatigue model and Austroad's initial strain-based fatigue model, and concluded that there is no relationship between the fatigue life and flexural modulus in the Austroad initial strain-based fatigue model. However, Carteret et al. (2009) found that the strain ratio-based fatigue model is consistent for different CSM and, therefore, recommend the model.

Nussbaum and Childs (1975) evaluated the fatigue of soil-cement and found that the stress ratio-based fatigue model is effective in characterizing the fatigue life of soil-cement. Dempsey et al. (1984) used the stress ratio-based fatigue model to evaluate the fatigue resistance of CSM and recommend the model for fatigue analysis. Based on the Portland Cement Association's large-scale fatigue tests, Scullion et al. (2008) calibrated the MEPDG fatigue model for soil-cement. They found good agreement based on the MEPDG fatigue model.

Due to the existence of shrinkage cracking in CSL, individual CSL cannot be considered to be a continuous layer. The occurrence of shrinkage cracking affects the stress/strain state. The effects of shrinkage cracking on the tensile stress at the bottom of CSL are accounted for by a correction factor in South Africa's current fatigue model, which is based on finite element analysis. Similarly, a correction factor similar to that of the South African model also is used in the French pavement design of CSL when determining tensile stress (σ_t)in Eq. (C-5) (Corte and Goux 1996).

Yeo (2008) found that the load-strain exponents in the Austroad fatigue model differ between the lab and field data. It is believed that poor construction quality in the field resulted in this discrepancy. Therefore, it is necessary to account for this factor in the fatigue model.

In the South African fatigue model, a shift factor is used to account for the crack propagation from the bottom to the surface of CSL, based on the thickness of the CSL. Dempsey et al. (1984) also emphasize the importance of the shift factor. Therefore, a shift factor is needed for fatigue life prediction.

After reviewing the aforementioned forms of fatigue models used worldwide, the team recommends that the current form of the fatigue model in the MEPDG (Eq. (C-6)) can be used and calibrated for bottom-up tensile-fatigue testing, because it has been proven effective in characterizing the bottom-up tensile-fatigue of CSL. The South African model (Eq. (C-10) can be used and calibrated for top-down compressive-fatigue testing. However, these models need to be improved to account for three aspects:

(a) Different CSM may follow different fatigue curves, indicating that there is no one set of model parameters for different materials. For bottom-up tensile-fatigue testing, Eq. (C-11) is recommended in this study. k_1 is used for field calibration, and k_2 and k_3 represent the material properties.

$$ln\, N_{ft} = k_1 \left(\frac{k_2 - \frac{\sigma_t}{MOR}}{k_3}\right) \qquad \text{Eq. (C-11)}$$

where

N_{ft} = bottom-up tensile-fatigue life
σ_t = tensile stress at the bottom of beam, psi
MOR = modulus of rupture, psi
k_1 = parameter used for field calibration
k_2 and k_3 = regression parameters.

Based on the laboratory experiments conducted for this study, k_2 and k_3 are determined for each type of material. Figure C-9 and Table C-14 present the model parameters for the different CSM with different binder content at different levels of maximum dry density (MDD).

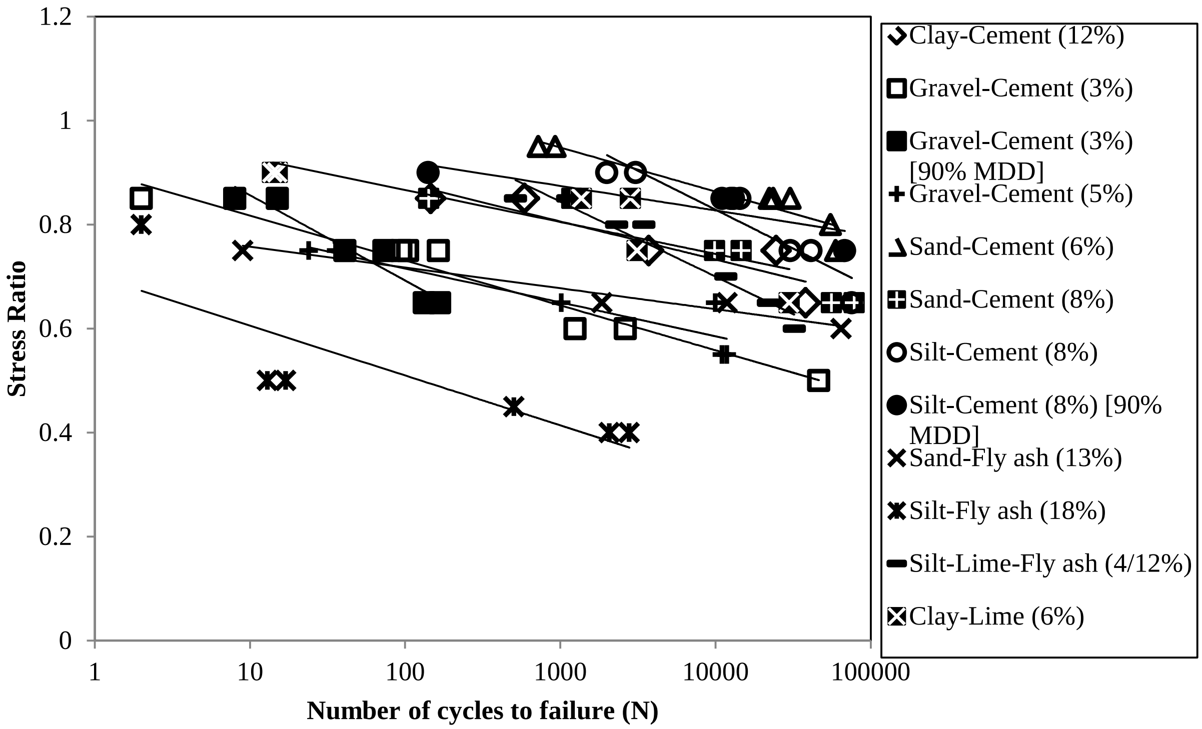


Figure C-9: Relationship between Bottom-Up Tensile-Fatigue Life and Stress Ratio

Table C-14. Regression Parameters of Bottom-Up Tensile-Fatigue Life Model for Different CSM

Specimen (Binder Content %)	Regression Parameters		R^2
	k_2	k_3	
Clay-cement (12%)	0.03	1.03	0.82
Gravel-cement (3%)	0.04	0.9	0.95
Sand-cement (6%)	0.04	1.2	0.88
Silt-cement (8%)	0.06	1.43	0.87
Sand-fly ash (13%)	0.02	0.8	0.95
Silt-lime-fly ash (4/12%)	0.06	1.28	0.94
Clay-Lime (6%)	0.03	0.99	0.72
Gravel-cement (3%) [90% MDD]	0.07	1.02	0.93
Silt-cement (8%) [90% MDD]	0.02	1.02	0.74
Gravel-cement (5%)	0.03	0.85	0.89
Sand-cement (8%)	0.03	1.06	0.89
Silt-fly ash (18%)	0.04	0.7	0.7
Average	**0.04**	**1.02**	**n/a**

(b) Another consideration for bottom-up tensile-fatigue testing is that the current MEPDG (ARA 2004) form of the fatigue model predicts the number of loads to fatigue crack

initiation. Additional load cycles are needed for the fatigue crack to propagate through the CSL. Therefore, a shift factor is needed to account for crack propagation. South African researchers have used finite element methods to obtain the correction factor for the effect of shrinkage cracking on the increase of tensile stress and conducted field studies of the shift factor for crack propagation. Therefore, in Eq. (C-11), the team recommends the incorporation of k_1 for field calibration to address the crack propagation effect.

(c) The modulus reduction due to damage must meet the boundary conditions. From the MEPDG model, Eq. (C-8), it can be seen that when D is 100%, $E_{CSM}(t)$ is equal to $E_{CSM}(min)$. However, when D = 0, $E_{CSM}(t)$ is not equal to $E_{CSM}(max)$, which is not reasonable. Therefore, the team proposes Eq. (C-12) as the modulus reduction model in order to meet the boundary conditions.

$$E(D) = E_{current}\left[\frac{m_2/ln(UCS_{28})}{1+e^{[sinh(n_2\,D)]}} + 1 - \frac{m_2/ln(UCS_{28})}{2}\right] \qquad \text{Eq. (C-12)}$$

where

$E(D)$ = modulus after accumulated damage, D
UCS_{28} = 28-day UCS, psi
$E_{current}$ = modulus before fatigue damage at age t in months, psi
m_2, n_2 = regression parameters.

For Eq. (C-12), when D is 0, $E(D)$ is equal to $E_{current}$, and when D is 1, $E(D)$ is equal to $E_{current}\left[1 - \frac{m_2/ln(UCS_{28})}{2}\right]$, which is the minimum modulus value as a function of 28-day UCS. Table C-15 shows the field-calibrated parameters for Eq. (C-12).

Table C-15. Parameters for Fatigue Modulus Reduction Model

Bottom-up tensile-fatigue modulus reduction	m_2	3.1055
	n_2	3.9919

Shrinkage Cracking Models

Shrinkage cracking is one of the most critical distresses found in CSL and must be considered in a pavement design. Shrinkage cracking often occurs at the early stage of pavement life and significantly affects the performance of the pavement. Shrinkage cracking is not considered in the MEPDG, even though it is the most recognized distress for CSL.

Shrinkage Cracking Models

A few researchers have developed shrinkage cracking models for either CSL or concrete pavement. Due to the similarities between CSL and concrete pavement, the shrinkage cracking issues in both areas are considered here.

(a) Zhang and Li model

Zhang and Li (2001) derived a closed-form solution for shrinkage stress in concrete pavements. Based on their model, crack spacing and width can be derived, respectively, as:

$$L = \frac{1}{\beta} cosh^{-1}\left(\frac{1}{\frac{S(t)}{E\epsilon}+1}\right) \quad \text{Eq. (C-13)}$$

$$W = \frac{2\epsilon}{\beta} tanh(\beta L) \quad \text{Eq. (C-14)}$$

where

L = shrinkage crack spacing

W = crack width

$\beta = \sqrt{\frac{\tau_0}{EH\delta_0}}$

$S(t)$ = tensile strength of CSL

E = modulus of CSL

ε = drying and thermal shrinkage strain

τ_0, δ_0 = maximum friction stress and displacement from slab friction test

H = CSL thickness.

(b) George model

George (1968) developed a simple model for crack spacing and width, as follows.

$$L = \frac{2\sigma_u}{\mu\gamma} \quad \text{Eq. (C-15)}$$

$$W = \varepsilon_c L - \frac{\mu\gamma L^2}{4E_t} \quad \text{Eq. (C-16)}$$

where

L = shrinkage crack spacing

W = crack width

σ_u = ultimate tensile strength of CSL

μ = coefficient of sliding coefficient

γ = unit weight of CSL

E_t = modulus of elasticity of CSL

ε_c = shrinkage strain.

Shrinkage Strain Models

Shrinkage strain is a key parameter in the models developed by both Zhang and Li (2001) and George (1968). Shrinkage strain primarily consists of drying and thermal shrinkage strain. Currently, there is no drying shrinkage strain for CSL to be developed.

(a) Drying shrinkage strain

It is very time-consuming to measure drying shrinkage strain in the laboratory, especially the ultimate drying shrinkage strain (ARA 2004). Several drying shrinkage strain prediction models are available to predict the drying shrinkage strain of concrete. Mokarem et al. (2003) evaluated five drying shrinkage strain prediction models for concrete:

- American Concrete Institute – ACI 209
- Euro-International Concrete Committee - CEB 90 Code
- Bazant B3
- Gardner/Lockman
- Sakata

It is reported that the CEB 90 code model is the best for matching measured drying shrinkage strain. The current MEPDG provides a drying shrinkage strain prediction model for concrete that is similar to the CEB 90 code model (ARA 2004). The MEPDG drying shrinkage strain model is:

$$\varepsilon_{shr}(t) = \varepsilon_{su}\left\{1 - \left(\frac{RH_c}{100}\right)^3\right\} \qquad \text{Eq. (C-17)}$$

where

ε_{shr}(t) = unrestrained drying shrinkage strain at any time t days from placement, $\times 10^{-6}$
ε_{su} = ultimate drying shrinkage strain
RH_c = relative humidity, %,
where $RH_c = RH_{ai} + (100 - RH_{ai})f(t)$
RH_{ai} = atmospheric relative humidity, %
$f(t) = 1/(1+t/b)$,
where t = time since placement in days
$b = 35d^{1.35}(w/c-0.19)/4$
d = half of depth of layer
w/c = water cement ratio.

The ultimate drying shrinkage strain of concrete (ARA 2004) is:

$$\varepsilon_{su} = C_1 C_2 \left\{ 26 w^{2.1} f_c^{\prime\,-0.28} + 270 \right\} \qquad \text{Eq. (C-18)}$$

where

ε_{su} = ultimate shrinkage strain, $\times 10^{-6}$
C_1 = cement type factor
C_2 = type of curing factor
w = water content by weight
$f_c^{\prime}$ = 28-day PCC compressive strength.

In this study, ultimate shrinkage and drying shrinkage models are modified using the lab results of CSM testing, as shown in Eq. (C-19) and (C-20).

$$\varepsilon_{su} = C_1 \cdot [w^{m_1} + m_2] \qquad \text{Eq. (C-19)}$$

where

ε_{su} = ultimate drying shrinkage strain, $\times 10^{-6}$
ω = optimum water content, lb/ft^3
C_1 = binder type factor
0.993 for cement
1.026 for lime
0.366 for C fly ash
m_1, m_2 = regression parameters, m_1 = 3.17, m_2 = 313.76.

The drying shrinkage strain of CSL with moisture gradient is modified based on the MEPDG model for concrete, as follows:

$$\varepsilon_g(t) = \varepsilon_{su} \left[1 - \left(\frac{RH_c}{100} \right)^{a_6} \right] \qquad \text{Eq. (C-20)}$$

where

$\varepsilon_g(t)$ = drying shrinkage strain with moisture gradient at any time t (in days) from placement, $\times 10^{-6}$
ε_{su} = ultimate drying shrinkage strain, $\times 10^{-6}$, can be estimated from Eq. (C-19) for Level 2 input
RH_c = relative humidity, %,
where $RH_c = RH + (100 - RH) f(t)^{a_5}$
RH = atmospheric relative humidity, %
$f(t) = 1/(1+t/b)$
where: t = time since placement in days

$b = a_1(d + a_2)^{a_3}(w/c)^{a_4}$
d = depth from evaporation surface of CSL, ft
w/c = water calcium ratio in mass
a_i = parameters, i = 1, 2, 3, 4, 5, 6, as shown in Table C-16.

Table C-16. Parameters for Gradient Drying Shrinkage Strain Model

a_1	a_2	a_3	a_4	a_5	a_6
1289202	0.085	0.94	1.24	10209.3	4.51

(b) Thermal shrinkage strain
The thermal contraction/expansion strain for concrete (ARA 2004) is:

$$\varepsilon_T = \alpha_C \Delta T \qquad \text{Eq. (C-21)}$$

where

α_C = coefficient of thermal expansion
ΔT = temperature difference.

Evaluation of Shrinkage Models

The aforementioned models have been evaluated in terms of the lab shrinkage test results and field data obtained from the literature. The common limitation of the aforementioned cracking models is that they do not consider the effect of moisture gradient and drying shrinkage gradient along the depth of CSL for shrinkage cracking, which could be one of the major causes of cracking in the field. Also, the George model (1968) does not consider the bond condition between the CSL and sublayer that is used in the Zhang and Li model (2001).

Therefore, a shrinkage cracking model is developed in this study based on the strain gradient theory developed by Westergaard (1927) for concrete. Given a cementitiously-stabilized slab with constant thickness H, as shown in Figure C-10, and given the slab is completely free of bottom restraint, then the loss of moisture from the slab's top surface will result in a moisture gradient, which in turn, causes the shrinkage gradient through the thickness/depth of the CSL. The shrinkage at any depth, $\varepsilon(y)$, can be calculated from Eqs. (C-19) and (C-20).

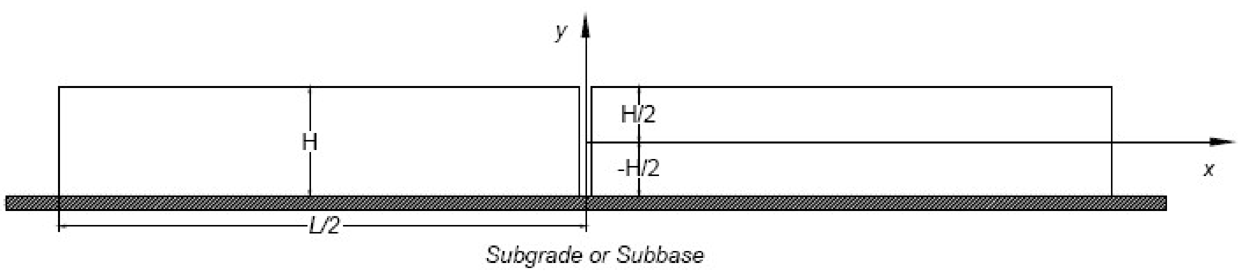


Figure C-10. Schematic Diagram of Slab Drying from Top Surface Only

Then, the tensile stress *σ (y)* can be expressed by Eq. (C-22), as shown by Westergaard (1927):

$$\sigma(y) = \frac{1}{1-\nu}\left(E_t \cdot \varepsilon(y) - \frac{E_t}{H}\int_{-\frac{H}{2}}^{\frac{H}{2}} \varepsilon(y)dy - \frac{12 \cdot E_t \cdot y}{H^3}\int_{-\frac{H}{2}}^{\frac{H}{2}} \varepsilon(y)ydy\right) \qquad \text{Eq. (C-22)}$$

where

ν = Poisson's ratio, assuming 0.2 for cementitiously-stabilized material
H = thickness of slab, in.
$\varepsilon (y)$ = strain at depth y
y = distance from the half depth of thickness, in.

The restraint from the underlying layer beneath the CSL hinders shrinkage. The restrained stress has a linear relationship with slab slippage, and the slope is the coefficient of friction (Zhang and Li 2001). The boundary conditions include that slippage is at its maximum at the end of the slab and is zero at the center. Thus, the shear stress τ can be expressed as Eq. (C-23).

$$\tau = \frac{1}{1-\nu} \cdot \mu \cdot \varepsilon\left(-\frac{H}{2}\right) \cdot x \qquad \text{Eq. (C-23)}$$

where

$\varepsilon\left(-\frac{H}{2}\right)$ = strain at bottom of the slab
x = distance from the center of slab in horizontal direction, in.
μ = coefficient of friction, psi/in., equals to 77.12×IDT strength, which is determined in the lab when epoxy is used as bonding agent.

From Eq. (C-23), the restrained force, *G*, can be obtained, as shown in Eq. (C-24).

$$G = \int_0^{\frac{L}{2}} \tau dx = \frac{\mu \cdot \varepsilon\left(-\frac{H}{2}\right)}{1-\nu}\frac{L^2}{8} \qquad \text{Eq. (C-24)}$$

where

G = restrained force, lbs
L = slab length, in.

From Eqs. (C-22) and (C-24), the stress distribution *σ (y)* can be obtained by the superposition of stress levels due to the shrinkage gradient and restraint, as shown in Eq. (C-25).

$$\sigma(y) = \frac{1}{1-\nu}\left(E_t \cdot \varepsilon(y) - \frac{E_t}{H}\int_{-\frac{H}{2}}^{\frac{H}{2}} \varepsilon(y)dy - \frac{12 \cdot E_t \cdot y}{H^3}\int_{-\frac{H}{2}}^{\frac{H}{2}} \varepsilon(y)ydy\right) + \frac{G}{H} \qquad \text{Eq. (C-25)}$$

Replace σ *(y)* with tensile strength S_t, and reform Eq. (C-25), and the restraint stress σ_{re} can be obtained as Eq. (C-26).

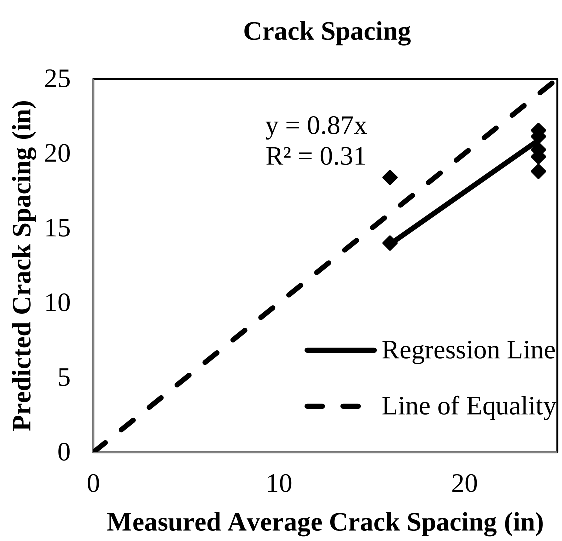
$$\sigma_{re} = \frac{G}{H} = S_t - \frac{1}{1-\nu}\left(E_t \cdot \varepsilon(y) - \frac{E_t}{H}\int_{-\frac{H}{2}}^{\frac{H}{2}} \varepsilon(y)dy - \frac{12 \cdot E_t \cdot y}{H^3}\int_{-\frac{H}{2}}^{\frac{H}{2}} \varepsilon(y)ydy\right) \qquad \text{Eq. (C-26)}$$

Therefore, the crack spacing and crack width can be obtained from Eqs. (C-27) and (C-28).

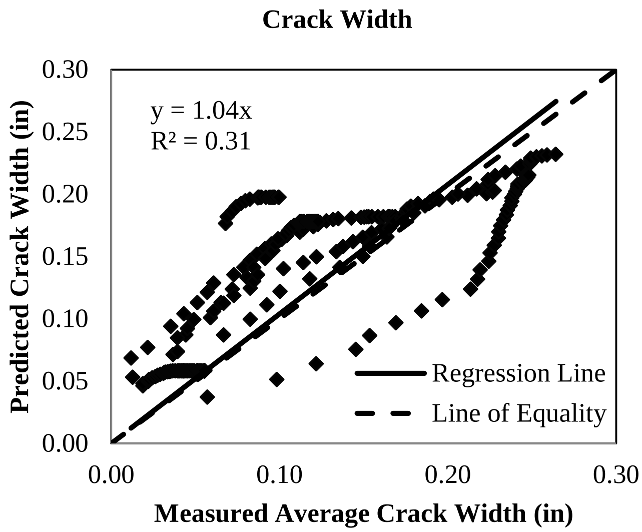
$$Crack\ Spacing = \frac{L}{2} = \frac{1}{2}\sqrt{\frac{8 \times (1-\nu) \times G}{\mu \times \varepsilon\left(-\frac{H}{2}\right)}} \qquad \text{Eq. (C-27)}$$

$$Crack\ Width = \left(\varepsilon\left(\frac{H}{2}\right) \cdot L - \frac{G \cdot (1-\nu)}{H \cdot E_t} \cdot L\right) \qquad \text{Eq. (C-28)}$$

Figure C-11 presents the comparison of measured and calculated crack spacing and width based on the restrained shrinkage cracking tests in the laboratory.



(a) Crack Spacing



(b) Crack Width

Figure C-11. Comparison of Measured and Predicted Crack Spacing and Width from Lab Tests

However, the predicted crack spacing and width obtained using the aforementioned models that are based on a purely mechanical approach do not match the field shrinkage crack data. Therefore, it was decided to develop an empirical shrinkage model based on dimensional analysis. Details about dimensional analysis are introduced by Andrew (2007). The calibration of the shrinkage cracking models is based on the restrained shrinkage cracking test results obtained from the laboratory and field pavement data. Because the stabilized fine material and stabilized coarse material show different shrinkage properties according to Kodikara and Chakrabarti (2001), the calibration was carried out for fine host material and coarse host material separately with different parameter constraints. Eqs. (C-29) and (C-30) show these crack spacing and crack width models, respectively. Figures C-12 and C-13 present the comparison of the predicted and measured shrinkage crack spacing results for the CSL for stabilized fine and coarse materials, respectively. Figures C-14 and C-15 present the comparison of the predicted and measured shrinkage crack width results for the stabilized fine and coarse materials, respectively. Tables C-17 and C-18 present the model parameters for the crack spacing and width models, respectively.

$$\log\left(\frac{L}{H}\right) = \left[\left(\frac{\mu}{\rho \cdot H \cdot t^{-2}}\right)^{l_1} \cdot (c\%)^{l_2} \cdot \left(\frac{\omega}{\rho}\right)^{l_3} \cdot (\Delta T \cdot COTE)^{l_4} \cdot \left(\frac{UCS_{28}}{\rho \cdot H^2 \cdot t^{-2}}\right)^{l_5} \cdot (RH\%)^{l_6} \cdot \left(\frac{\varepsilon_{top}}{\varepsilon_{ult}}\right)^{l_7} \cdot \left(\frac{\varepsilon_{top} \cdot E_t}{S_{IDT}}\right)^{l_8}\right] \cdot l_9 \qquad \text{Eq. (C-29)}$$

$$W = \left[\left(\frac{\mu}{\rho \cdot H \cdot t^{-2}}\right)^{w_1} \cdot (c\%)^{w_2} \cdot \left(\frac{\omega}{\rho}\right)^{w_3} \cdot (\Delta T \cdot COTE)^{w_4} \cdot \left(\frac{UCS_{28}}{\rho \cdot H^2 \cdot t^{-2}}\right)^{w_5} \cdot (RH\%)^{w_6} \cdot \left(\frac{\varepsilon_{top}}{\varepsilon_{ult}}\right)^{w_7} \cdot \left(\frac{\varepsilon_{top} \cdot E_t}{S_{IDT}}\right)^{w_8}\right] \cdot w_9 \qquad \text{Eq. (C-30)}$$

where,

L = crack spacing, in.
W = crack width, in.

H = thickness of CSL, in.
μ = coefficient of friction, psi/in.
ρ = dry density, lb/ft^3
t = age when crack survey conducted, days
$c\%$ = calcium content, %
ω = water content, lb/ft^3
ΔT = average daily maximum temperature variation, °F
$COTE$ = coefficient of thermal expansion, /°F
UCS_{28} = 28-day UCS at 68°F and 100% RH
RH = atmospheric relative humidity, %
ε_{top} = shrinkage on the top surface
ε_{ult} = ultimate drying shrinkage
S_{IDT} = IDT strength, calculated from IDT strength growth model, psi
E_t = IDT modulus, calculated from IDT modulus growth model, psi
l_i = regression parameters for crack spacing model, i = 0, 1, 2, 3, 4, 5, 6, 7
w_i = regression parameters for crack width model, i = 0, 1, 2, 3, 4, 5, 6, 7.

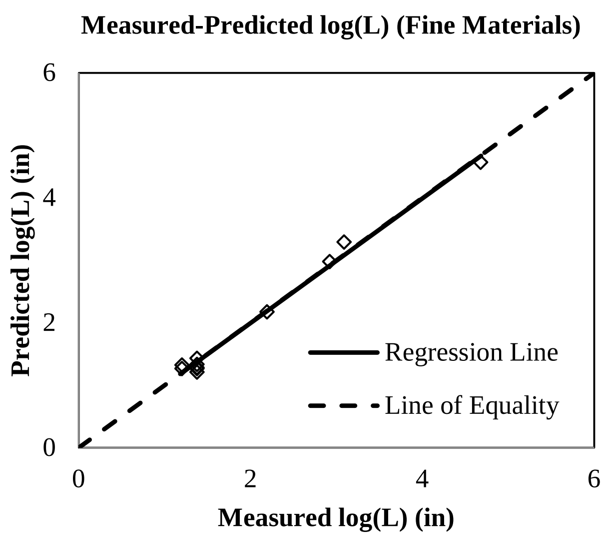


Figure C-12. Comparison of Predicted and Measured CSL Shrinkage Crack Spacing Results for Stabilized Fine Materials

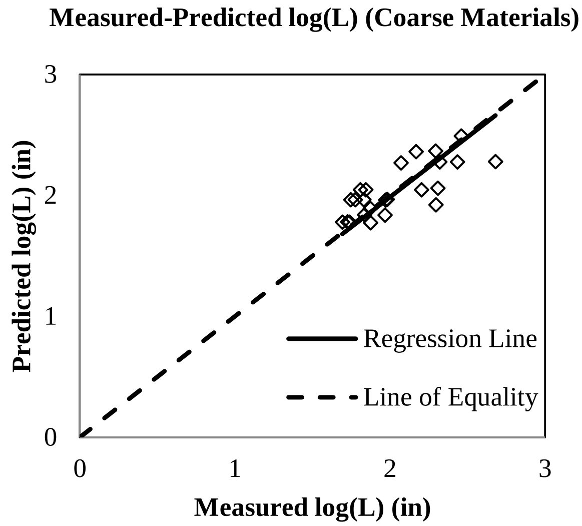


Figure C-13. Comparison of Predicted and Measured CSL Shrinkage Crack Spacing Results for Stabilized Coarse Materials

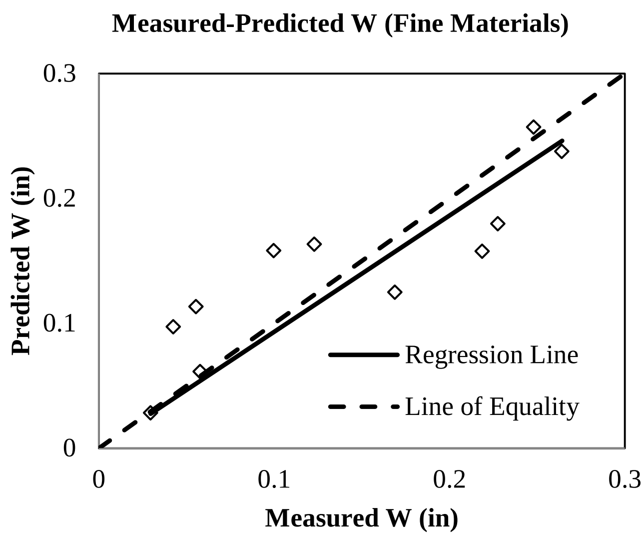


Figure C-14. Comparison of Predicted and Measured CSL Shrinkage Crack Width Results for Stabilized Fine Materials

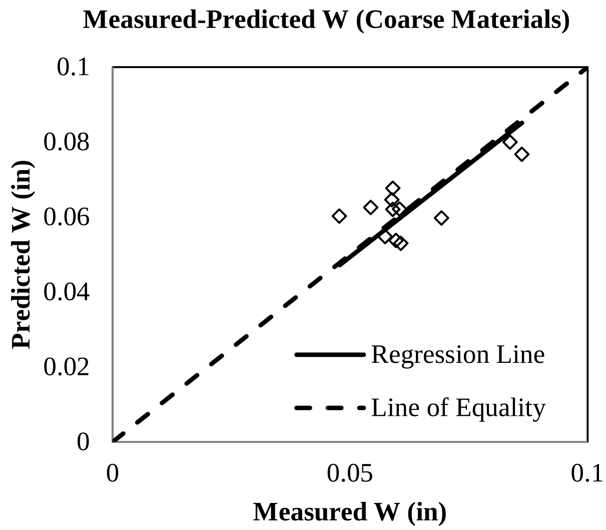


Figure C-15. Comparison of Predicted and Measured CSL Shrinkage Crack Width Results for Stabilized Coarse Materials

Table C-17. Field-Calibrated Parameters of CSL Shrinkage Crack Spacing Model for Fine and Coarse Host Materials

Parameters	Fine Materials	Parameters	Coarse Materials
l_1 *(<0)*	-1.19E-01	l_1 *(<0)*	0
l_2 *(>0)*	5.98E-01	l_2 *(<0)*	-1.39E-01
l_3 *(<0)*	-7.78E-01	l_3 *(<0)*	-1.36E-04
l_4 *(<0)*	0	l_4 *(<0)*	0
l_5 *(>0)*	0	l_5 *(<0)*	-1.46E-01
l_6 *(>0)*	0	l_6 *(>0)*	2.11E+00
l_7 *(<0)*	-2.20E-03	l_7 *(<0)*	0
l_8 *(<0)*	-2.53E-01	l_8 *(<0)*	0
l_9	8.74E+00	l_9	3.85E+00

( ) contains the parameter constraints.

Table C-18. Field-Calibrated Parameters of CSL Shrinkage Crack Width Model for Fine and Coarse Host Materials

Parameters	Fine Materials	Parameters	Coarse Materials
w_1 (>0)	7.81E-03	w_1 (>0)	0
w_2 (<0)	-1.20E+00	w_2 (>0)	0
w_3 (>0)	7.67E-01	w_3 (>0)	1.34E+00
w_4 (>0)	0	w_4 (>0)	1.76E-05
w_5 (<0)	0	w_5 (>0)	3.63E-02
w_6 (<0)	0	w_6 (<0)	0
w_7 (>0)	6.69E-01	w_7 (>0)	0
w_8 (>0)	4.71E-01	w_8 (>0)	5.36E-02
w_9	8.63E-04	w_9	1.78E-01

( ) contains the parameter constraints.

Durability Models

In the field, the CSL are subjected to weathering, such as wetting and drying and freezing and thawing, which must be characterized.

Introduction to Durability Models

Currently, there is no durability model for CSL in the MEPDG. The team has identified one durability model for concrete specimens that are subjected to freeze-thaw cycles, which has the potential to be used for CSL. The revised durability model is as follows (Wei et al. 2003).

$$E(N) = E_0 e^{\frac{k_1 T (\frac{\Delta L}{L})^2 N}{3}} \qquad \text{Eq. (C-31)}$$

where

E = modulus of CSM after N cycles of freeze-thaw
E_0 = modulus of CSM before freeze-thaw cycles
k_1 = regression constant
T = duration of one freeze-thaw cycle
ΔL = increase of specimen length after freezing
L = original length of specimen
N = number of freeze-thaw cycles.

Khoury and Zaman (2008) developed an empirical model for CSL subjected to wet-dry cycles, as follows.

$$M_r = A \times B^{WDC} \times C^{CSAFR} \times D^{DMR} \times E^{\sigma_3} \times F^{\sigma_d} \qquad \text{Eq. (C-32)}$$

where

A, B, C, D, E, F = regression coefficients
WDC = number of wet-dry cycles
$CSAFR$ = ratio of free lime to SAF (silica, alumina, and ferric oxide)
DMR = ratio of maximum dry density to optimum moisture content
σ_3 = confining pressure
σ_d = deviatoric stress.

Evaluation of Durability Models

The durability model shown in Eq. (C-31) requires the length change of the specimen, which is only applicable to laboratory experiments, not field pavements. The durability model shown in Eq. (C-32) requires the chemical composition of the soils, which may not be available. Therefore, wet-dry and freeze-thaw durability models are developed in this study, as shown in Eq. (C-33), but with different model parameters for the wet-dry and freeze-thaw models.

$$UCS(N) = UCS_{current}\left[\frac{m_1/ln(UCS_{28})}{1+e^{(n_1\,N)}} + 1 - \frac{n_1\,/ln(UCS_{28})}{2}\right] \qquad \text{Eq. (C-33)}$$

where

$UCS(N)$ = UCS after N cycles of freeze-thaw or wet-dry, psi
$UCS_{current}$ = UCS before freeze-thaw or wet-dry cycle, at age t in months (considering the strength growth from 28 days to time of freeze-thaw or wet-dry cycle), psi
UCS_{28} = 28-day UCS, psi
N = number of freeze-thaw or wet-dry cycles
m_1, n_1 = regression parameters.

Table C-19 presents the model parameters for the wet-dry and freeze-thaw models based on the laboratory test results. Figures C-16 and C-17 present the comparisons of the predicted and measured UCS values for the wet-dry and freeze-thaw models, respectively.

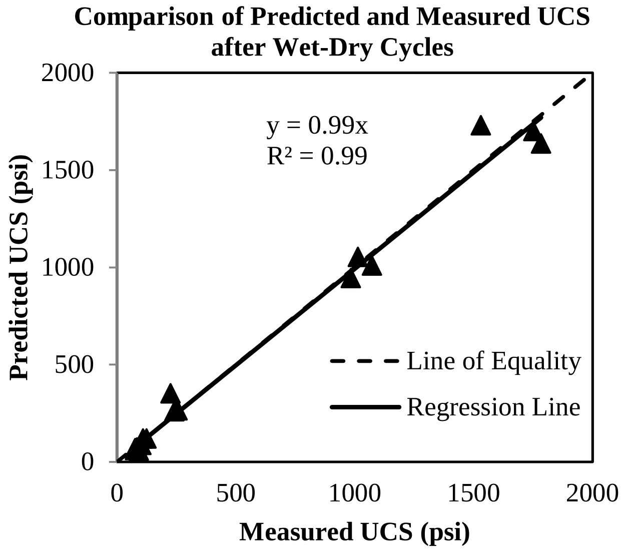


Figure C-16. Comparison of Predicted UCS and Measured UCS Values after Wet-Dry Tests

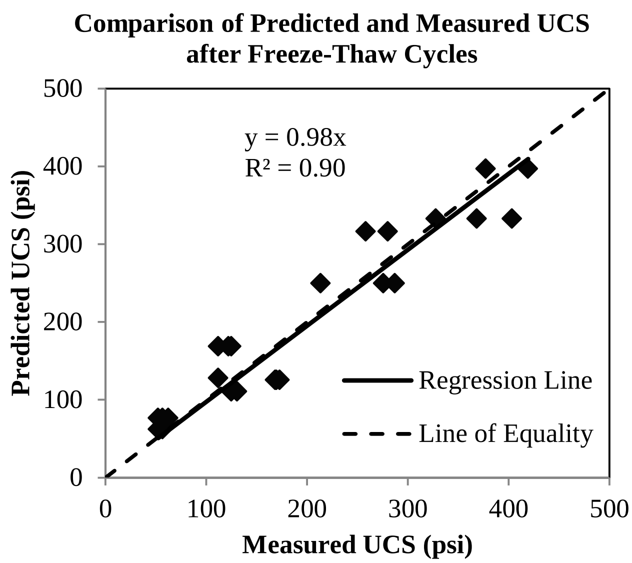


Figure C-17. Comparison of Predicted UCS and Measured UCS Values after Freeze-Thaw Tests

Table C-19. Regression Parameters of Durability Models

Wet-Dry	m_1	2.58
	n_1	0.62
Freeze-Thaw	m_1	6.68
	n_1	0.93

Erosion Models

Introduction to Current Erosion Models

A few models are available for estimating the erosion of the base/subbase in concrete pavement, as follows.

(a) Markow (1984) model

An empirical model (Markow 1984) that is based on the AASHTO road test data relates erosion to slab thickness, equivalent single-axle loads (ESALs), and subbase drainage conditions. The model is simple but does not consider many important factors. The pumping index indicates the potential of erosion, which increases with the increase in the cumulative number of ESALs and with a decrease in drainage, but decreases quickly with an increase in slab thickness. A drainage adjustment factor is considered based on subbase permeability.

$$P_i = m \sum ESAL f_d \qquad \text{Eq. (C-34)}$$

$$\log m = 1.07 - 0.34\, D \qquad \text{Eq. (C-35)}$$

where

P_i = pumping index
D = slab thickness, in.
$ESAL$ = equivalent 80 kN (18,000 lb) single-axle loads
f_d = drainage adjustment factor
= 0.2 for good drainage (k = 10,000 ft/day)
= 0.6 for fair drainage (k = 100 ft/day)
= 1.0 for poor drainage (k = 0.1 ft/day)
k = subbase permeability.

(b) Larralde (1984) model

Another empirical model (Larralde 1984) that is based on the AASHTO road test data relates erosion to the amount of deformation energy imposed by the application of the load. The deformation energy is computed using finite element modeling. A pumping index is normalized to eliminate the effects of slab length and reinforcement. The model shown in Eq. (C-36) is empirical in nature and consequently does not consider many important facts related to erosion.

$$NPI = exp\left[-2.884 + 1.652 \cdot log\left(\frac{\sum ESAL.DE}{10{,}000}\right)\right] \qquad \text{Eq. (C-36)}$$

where

NPI = normalized pumping index of volume of pumped material, $in.^3$
ESAL = equivalent 80 kN (18,000 lb) single-axle loads
DE = deformation energy per one application of ESAL
= log(*DE*) = 3.5754 – 0.3323 D
D = slab thickness, in.

(c) Rauhut (1982) model

In the Rauhut (1982) model, the level of pumping damage, which is based on nonlinear regression analysis of the Concrete Pavement Evaluation System (COPES) database, is related empirically to many comprehensive factors, such as precipitation, drainage, subbase type (stabilized or not), subgrade type (soil type), load transfer, slab thickness, freezing index, the Thornthwaite moisture index, and traffic. Eq. (C-37) is separated for jointed plain concrete pavement (JPCP) in Eqs. (C-38) and (C-39) and for jointed reinforced concrete pavement (JRCP) in Eqs. (C-41) and (C-42).

$$g = \left(\frac{ESAL}{\rho}\right)^{\beta} \qquad \text{Eq. (C-37)}$$

JPCP

$$ln\rho = 1.39 \cdot DRAIN + 4.13 \qquad \text{Eq. (C-38)}$$

$$\beta = \frac{0.772(D-2.3)^{1.61}}{PPTN} + 0.0157.JLTS.D + 0.104.STAB + 0.17.DRAIN + 0.137.SOILTYP - 0.247 \qquad \text{Eq. (C-39)}$$

JRCP

$$ln\rho = 1.028 \cdot STAB + 0.0004966 \cdot D^{3.47} - 0.01248 \cdot FRINDEX + 1.667 \cdot CBR + 5.476 \qquad \text{Eq. (C-40)}$$

$$\beta = -0.01363 \cdot DMOIST + 0.02527 \cdot D - 0.423 \qquad \text{Eq. (C-41)}$$

where

g = amount of distress as a fraction of pumping level 3 (severe)
DRAIN = 0 is no underdrainage, 1 is underdrainage
PPTN = average annual precipitation (cm)
JLTS = 0 is undoweled, 1 is doweled
STAB = 0 is unstabilized subbase, 1 is stabilized subbase
SOILTYP = 0 is granular foundation soil, 1 is coarse foundation soil
DMOIST = Thornthwaite moisture index
FRINDEX = freezing index

CBR = California bearing ratio of foundation soil
D = slab thickness.

(d) Van Wijk (1985) model

Eqs. (C-42) and (C-43) include factors derived from field data to make improvements to the Larralde model (1984) to predict the volume of eroded material as a function of the deformation energy produced by traffic. The effects of many factors, such as subbase and subgrade type, drainage, load transfer, and climate conditions, on pumping are considered in this model (Van Wijk 1985). Because this model is empirical in nature, its application is limited to the variable ranges included in the database. (Note that Eq. (C-43) is the same as Eq. (C-37) but with a modification factor F.)

$$P = 36.67 \cdot NPI \qquad \text{Eq. (C-42)}$$

$$NPI = F \cdot exp\left[-2.884 + 1.652 \cdot log\left(\frac{\sum ESAL \cdot DE}{10{,}000}\right)\right] \qquad \text{Eq. (C-43)}$$

where

P = volume of pumped material (ft^3/mile)
NPI = normalized pumping index ($in.^3$)
DE = deformation energy per application (in.-lb)
Log (DE) = 3.5754 – 0.3323D
D = slab thickness
$F = f_{JPCP}$ if nonreinforced PCC, f_{JRCP} if reinforced PCC

$$f_{JPCP} = f_{sbl} \cdot f_d \cdot f_{prec} \cdot f_{sg}$$

f_{sbl} = subbase adjustment factor = 1 if unstabilized,
= 0.65+0.18 log(ΣESAL) if stabilized
f_d = drainage adjustment factor
= 1, poor drainage
0.91 +0.12 log(ΣESAL) – 0.03 D, fair drainage
0.68 +0.15 log(ΣESAL) – 0.04 D, good drainage
0.01, excellent drainage
f_{lt} = load transfer adequacy adjustment factor
1, with dowel
1.17 - 0.68 log(ΣESAL) – 0.07 D, without dowel
f_{prec} = rainfall adjustment factor
0.89 + 0.26 log(ΣESAL) – 0.07 D, dry climates
0.96 + 0.06 log(ΣESAL) + 0.02 D, wet climates
f_{sg} = subgrade adjustment factor
1, granular subgrade

0.57 + 0.21 log(ΣESAL), coarse subgrade

$f_{JRCP} = f_{sb2}.f_e$

f_{sb2} =subbase adjustment factor

1 is unstabilized

0.91 - 0.02 log(ΣESAL), coarse subgrade

f_e = adjustment for climate

0.011 + 0.003 log(ΣESAL) – 0.001D, dry, warm climates

1.44 -0.03 log(ΣESAL) – 0.06D, wet, warm climates

1.04- 0.32 log(ΣESAL) – 0.08 D, dry, cold climates

0.54 - 0.85 log(ΣESAL) – 0.19 D, wet, cold climates.

(e) Jeong and Zollinger (2001) model

A mechanistic empirical model is developed using the water-induced shear stress model proposed by Jeong and Zollinger (2001), as shown in Eq (C-44). Key factors, such as vehicle load and speed, load transfer, number of applications, and climatic conditions, are included in the model to predict erosion. The erosion potential increases with a high initial edge gap and lift-off distance due to the effect of upward curling at the slab corners and edges, which induces shear stress on the base layer by pumping out the trapped water. The magnitude of the shear stress depends on the dynamic viscosity of the water, which is governed by the water temperature and the rate of slab deflection. Higher slab deflection velocity and lower viscosity of water result in more erosion of the base. An effective load transfer cuts down the erosion rate, as detailed in the performance equation. The accuracy of the model should be calibrated using performance data such as those available in the Long-Term Pavement Performance (LTPP) database. This model can be improved by considering abrasive erosion caused by friction between the concrete and subbase layers.

$$\upsilon = \upsilon_0 exp\left[-\left(\frac{\rho}{N_i}\right)^a\right] \qquad \text{Eq. (C-44)}$$

where

υ_0 = ultimate erosion depth (L)

N = number of axle loads per load group

ρ = calibration coefficient based on local performance

$a = a'\alpha_f$

a' = environmental calibration coefficient

α_f = inverse of the rate of void development

$$=\left[\frac{\partial \upsilon_i}{\partial t}\right]^{-1} = \left[\frac{log^{-1}(a_m\tau + b_m)}{\gamma_b}\right]^{-1} = \left[\frac{\beta}{\gamma_b}\right]^{-1}$$

τ = shear stress $= \frac{\eta B}{\delta_{void}}\left(1 - \frac{LTE}{100}\right)$

η = dynamic viscosity ($FL^{-2}T$)
$= \{2056.82 + 10.56T - 284.93\sqrt{T} - 265.02e^{-T}\}10^{-6}$
T = water temperature (°C)
$B = V_{z_i} sin\theta + 6V_{z_i}\left[\frac{sin\theta}{2} + \frac{cos^2\theta}{sin\theta}\right]$ (L/T)
δ_{void} = void space below slab for water movement
θ = slab angle = $\tan^{-1}\left[\frac{z_0}{s}\right]$
z_0 = edge gap (L) = $\frac{(1+\nu)}{H}\Delta\epsilon_{tot}l^2$
$V_{z_i} = \frac{\delta_{int}}{s/V_i}$
$\delta_{int} = \frac{P_i}{8kl}\left\{1 + \left[0.3665 log\left(\frac{a_L}{l}\right) - 0.2174\left(\frac{a_L}{l}\right)^2\right]\right\}$
a_L = loaded radius (L)
P_i = axle load (F)
s = slab lift-off distance (L) = $\sqrt{2}l(\gamma - 1)$
$\gamma = \sqrt{\frac{z_0}{w_0}}$
$w_0 = \frac{\rho H}{k}$

Evaluation of Erosion Model

The above models are either empirical or difficult to implement. Most importantly, in the field, the effects of top compression fatigue and erosion are combined. Therefore, the cyclic impact erosion test that includes both top compression fatigue and the effects of erosion is developed in this study. Details about the test method are introduced in Appendix B in the section entitled *Erodibility*. The fatigue-erosion model is modified from Eq. (C-10) and is expressed as Eq. (C-45) and called the *top-down compressive-fatigue-erosion model*.

$$log\, N_{fc} = k_4 log\left(\frac{\rho}{\omega}\right)\left(1 - \frac{\sigma_c}{k_5\, UCS}\right) \quad \text{Eq. (C-45)}$$

where

N_{fc} = top-down compressive-fatigue-erosion life, defined as the number of cycles when the modulus drops to 50% of the initial modulus
ρ = maximum dry density, lb/ft^3
ω = optimum moisture content, %
σ_c = compressive stress applied on the top of specimen, psi
UCS = current unconfined compressive strength, psi
k_4, k_5 = regression parameters.

The incorporation of maximum dry density and optimum moisture content in the model differentiates the susceptibility of the fine and coarse materials. The modulus reduction model for top-down compressive-fatigue-erosion is based on Eq. (C-12), but uses different parameters. Table C-20 presents the model parameters that are based on laboratory CIE tests. Figure C-18 presents the comparison of measured and predicted fatigue life data.

Table C-20. Parameters for Top-Down Compressive-Fatigue-Erosion Model

Model	Parameter	
Top-Down Compressive-Fatigue-Erosion Life	k_4	2.79
	k_5	3.39
Top-Down Compressive-Fatigue-Erosion Modulus Reduction	m_2	6.77
	n_2	1.99

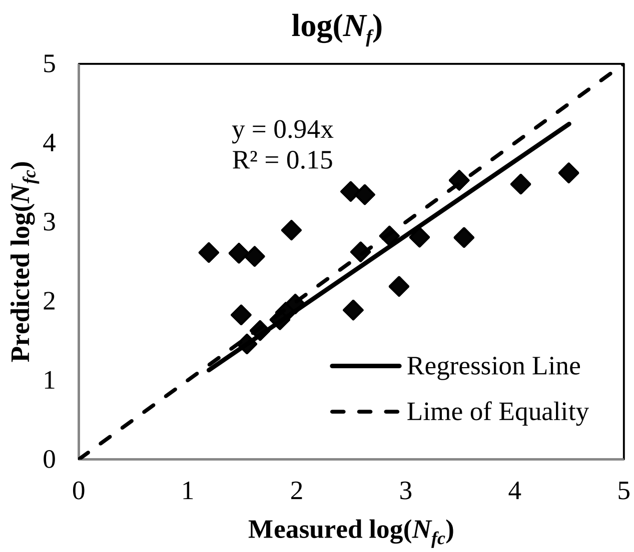


Figure C-18. Comparison of Measured and Predicted Modulus Values for Top-Down Compressive-Fatigue-Erosion Model

The performance models were developed based on laboratory experiments. However, these models need to be calibrated using field data. Typically, there is a discrepancy between laboratory results and field results.

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Appendix D. Evaluation of FWD Backcalculation

Introduction

Because cementitiously-stabilized layers (CSL) typically are used as base or subbase layers, the performance of CSL in the field cannot be measured directly or visualized. The field cores of CSL are not available for characterization in this study. Therefore, backcalculated modulus values obtained from falling weight deflectometer (FWD) tests, conducted in previous studies, are used in this study to detect the performance of CSL. The moduli of the CSL directly affect the pavement responses and performance of pavement in the field. However, it is well-known that FWD backcalculation is not a science and could cause significant error in backcalculated modulus values (Meier 1995, William 1999, Goktepe et al. 2006, Gopalakrishnan 2009). The accuracy of backcalculation of pavements that contain CSL was evaluated.

Appraisal of Backcalculation Approach

In order to analyze the error associated with FWD backcalculation, commonly used FWD backcalculation programs, such as Evercal, Modcomp 6, and MODULUS, are selected for this study. Table D-1 lists seven types of pavement structures that have different layer thicknesses and layer modulus values. For each pavement structure, pavement deflections at locations with distances of 8 in., 12 in., 18 in., 24 in., 36 in., and 60 in. from the loading center were determined using the EverStress program (developed by the Washington State Department of Transportation). In each of these pavement structures, the CSL modulus values vary from 50 ksi to 1000 ksi. These deflections are used to backcalculate the CSL modulus values, based on the backcalculation programs. The backcalculated CSL modulus values then are compared with the given CSL modulus values. The backcalculated modulus error is calculated by Eq. (D-1).

$$Backcalculated\ Modulus\ Error = \frac{(Backcalculated\ CSL\ Modulus - Given\ CSL\ Modulus)}{Given\ CSL\ Modulus}\ 100\% \qquad \text{Eq. (D-1)}$$

Table D-2 shows the backcalculated CSL modulus errors for Evercal, Modcomp 6, and MODULUS. The results indicate that all three FWD backcalculation programs result in large error for at least some of the pavement structures. Modcomp 6 shows more consistent backcalculation errors than Evercal for the structures simulated in this study. Structures E and F have relatively low backcalculation errors, probably because the CSL are thick. If a relatively thin stiff layer is sandwiched between weak layers, the backcalculation error could be large. Because the error is related to a specific pavement structure, it was decided to use the concept of modulus ratio. The CSL modulus ratio is defined by the ratio of each layer modulus to the original modulus for one pavement structure, simulating the change in CSL modulus values over time. Table D-2 presents the analysis results for the modulus ratios and associated errors. In most of the cases, the modulus ratio error is smaller than that of the backcalculated modulus error. Therefore, the modulus ratio concept is used based on the Modcomp 6 program.

Table D-1. Assumed Pavement Structures in FWD Backcalculation Error Analysis

Pavement Type	Layer 1			Layer 2			Layer 3			Layer 4			Layer 5		
	Type	Thickness (in.)	Modulus (psi)	Type	Thickness (in.)	Modulus (psi)	Type	Thickness (in.)	Modulus (psi)	Type	Thickness (in.)	Modulus (psi)	Type	Thickness (in.)	Modulus (psi)
A	HMA	4.9	500000	Granular Base	8	50000	CSL	6	variable	Granular Subbase	12	70000	Sub-grade	n/a	10000
B	HMA	9.4	800000	Granular Base	11.8	35000	CSL	6	variable	Granular Subbase	11.5	30000	Sub-grade	n/a	20000
C	HMA	15.7	500000	CSL	6	variable	Granular Subbase	9	50000	Subgrade	n/a	30000			
D	HMA	7.2	900000	Granular Base	8.5	50000	Granular Subbase	22.8	40000	CSL	12	variable	Sub-grade	n/a	30000
E	HMA	7.2	900000	Granular Base	8.5	50000	CSL	12	variable	Subgrade	n/a	5000			
F	HMA	7.2	900000	Granular Base	8.5	50000	CSL	12	variable	Subgrade	n/a	5000			
G	HMA	4.0	600000	CSL	8	variable	Subgrade	n/a	25000						

Table D-2. FWD Backcalculation Error Analysis Results

Assumed Pavement Structure	Assumed CSL E	Assumed CSL E Ratio	Backcalculated E						Backcalculated E Error			Backcalculated E Ratio Error		
			Evercal	Ratio	Modcomp6	ratio	MODULUS	Ratio	Evercal	Modcomp6	MODULUS	Evercal	Modcomp6	MODULUS
A1	1000000	0.70	842550	0.66	202000	0.91			-15.7%	-79.8%		-5.0%	29.4%	
A2	700000	0.40	560280	0.38	183000	0.77			-20.0%	-73.9%		-5.3%	93.1%	
A3	400000	0.15	319280	0.15	156000	0.63			-20.2%	-61.0%		-2.0%	322.4%	
A4	150000	0.07	123800	0.02	128000	0.43	2267700		-17.5%	-14.7%	1411.8%	-67.4%	511.7%	
A5	70000		19210		86500				-72.6%	23.6%				
B1	1000000	0.70	222020	0.78	39400	0.95			-77.8%	-96.1%		11.8%	36.3%	
B2	700000	0.40	173700	0.45	37600	0.88			-75.2%	-94.6%		11.5%	120.2%	
B3	400000	0.10	99010	0.46	34700	1.48			-75.2%	-91.3%		357.3%	1377.2%	
B4	100000	0.05	101540	0.19	58200	0.64			1.5%	-41.8%		275.9%	1189.3%	
B5	50000		41730		25400				-16.5%	-49.2%				
C1	1000000	0.80	134430	0.95	121000	5.04	173400	0.88	-86.6%	-87.9%	-82.7%	18.7%	530.2%	9.7%
C2	800000	0.50	127610	0.54	610000	0.85	152200	0.56	-84.0%	-23.8%	-81.0%	8.2%	70.2%	12.8%
C3	500000	0.20	72750	0.17	103000	0.63	97800	0.30	-85.5%	-79.4%	-80.4%	-16.3%	216.1%	49.4%
C4	200000	0.07	22510	0.52	76500	0.44	51800	0.18	-88.7%	-61.8%	-74.1%	639.3%	531.6%	157.0%
C5	70000		69570		53500		31200		-0.6%	-23.6%	-55.4%			
D1	1000000	0.70	3000000	1.00	305000	0.80			200.0%	-69.5%		42.9%	14.3%	
D2	700000	0.40	3000000	1.00	244000	0.54			328.6%	-65.1%		150.0%	35.2%	
D3	400000	0.15	3000000	1.00	165000	0.29			650.0%	-58.8%		566.7%	95.4%	
D4	150000	0.07	3000000	0.06	89400	0.17	115500		1900.0%	-40.4%	-23.0%	-11.4%	138.9%	
D5	70000		185990		51000				165.7%	-27.1%				
E1	1000000	0.70	968740	0.72	524000	0.68	212600		-3.1%	-47.6%	-78.7%	2.4%	-3.5%	
E2	700000	0.40	694700	0.43	354000	0.42			-0.8%	-49.4%		7.6%	4.0%	
E3	400000	0.10	416880	0.11	218000	0.13			4.2%	-45.5%		8.7%	31.7%	
E4	100000	0.05	105260	0.06	69000	0.09			5.3%	-31.0%		12.9%	77.1%	
E5	50000		54680		46400				9.4%	-7.2%				
F1	1000000	0.70	968740	0.72	536000	0.69	212600	0.77	-3.1%	-46.4%	-78.7%	2.4%	-1.4%	10.3%
F2	700000	0.40	694700	0.43	370000	0.39	164100	0.32	-0.8%	-47.1%	-76.6%	7.6%	-2.1%	-19.0%
F3	400000	0.20	416880	0.22	210000	0.21	68900	0.28	4.2%	-47.5%	-82.8%	7.5%	6.3%	39.9%
F4	200000	0.10	208290	0.11	114000	0.11	59500	0.06	4.1%	-43.0%	-70.3%	8.7%	9.1%	-41.8%
F5	100000		105260		58500		12374		5.3%	-41.5%	-87.6%			
G1	1000000	0.50	174360	0.69	933000	0.51	983300	0.49	-82.6%	-6.7%	-1.7%	38.6%	2.3%	-2.5%
G2	500000	0.20	120850	0.42	477000	0.21	479600	0.21	-75.8%	-4.6%	-4.1%	111.9%	3.4%	3.3%
G3	200000	0.10	73910	0.28	193000	0.10	203200	0.10	-63.0%	-3.5%	1.6%	176.3%	1.7%	1.3%
G4	100000	0.05	48170	0.17	94900	0.06	99600	0.05	-51.8%	-5.1%	-0.4%	247.1%	11.0%	4.5%
G5	50000		30260		51800		51400		-39.5%	3.6%	2.8%			

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Appendix E. Appraisal of Performance Models

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CHAPTER E-1. DURABILITY MODEL APPRAISAL

The models developed and calibrated in this study were appraised in terms of their reasonableness and behaviors of different CSM.

The wet-dry and freeze-thaw durability models developed in this study share the same form of Eq. (E-1), but with different parameters.

$$UCS(N) = UCS_{current}\left[\frac{m_1/ln(UCS_{28})}{1+e^{(n_1\,N)}} + 1 - \frac{n_1\,/ln(UCS_{28})}{2}\right] \qquad \text{Eq. (E-1)}$$

where

UCS(N) = UCS after *N* cycles of freeze-thaw or wet-dry, psi
$UCS_{current}$ = UCS before freeze-thaw or wet-dry cycle, at age *t* in months (considering the strength growth from 28 days to the time of freeze-thaw or wet-dry occurrence), psi
UCS_{28} = 28-day UCS, psi
N = number of freeze-thaw or wet-dry cycles
m_1 and n_1 = regression parameters.

Table E-1 presents the model parameters, based on laboratory experimental results.

Table E-1. Regression Parameters of Durability Models

Wet-Dry	m_1	2.58
	n_1	0.62
Freeze-Thaw	m_1	6.68
	n_1	0.93

There are two boundary conditions of Eq. (E-1): a) when N is 0, $UCS(N)$is equal to $UCS_{current}$; b); if the number of W-D and F-T cycles is large, the CSM become granular material and the UCS will approach the minimum residual value. The residual value depends on the strength of the original CSM, e.g., the 28-day UCS, as shown in Equation (E-2) and Figure E-1.

$$UCS(residual) = UCS_{current}\left[1 - \frac{m_1\,/ln(UCS_{28})}{2}\right] \qquad \text{Eq. (E-2)}$$

where

UCS(residual) = residual UCS after a large number of freeze-thaw or wet-dry cycles
m_1 = regression parameter.

Figure E-1 shows that the weak CSM are susceptible to wet-dry and freeze-thaw cycles. The freeze-thaw cycles cause more damage than the wet-dry cycles. Also, when the strength of CSM increases, the damage of wet-dry and/or freeze-thaw cycling on CSM decreases.

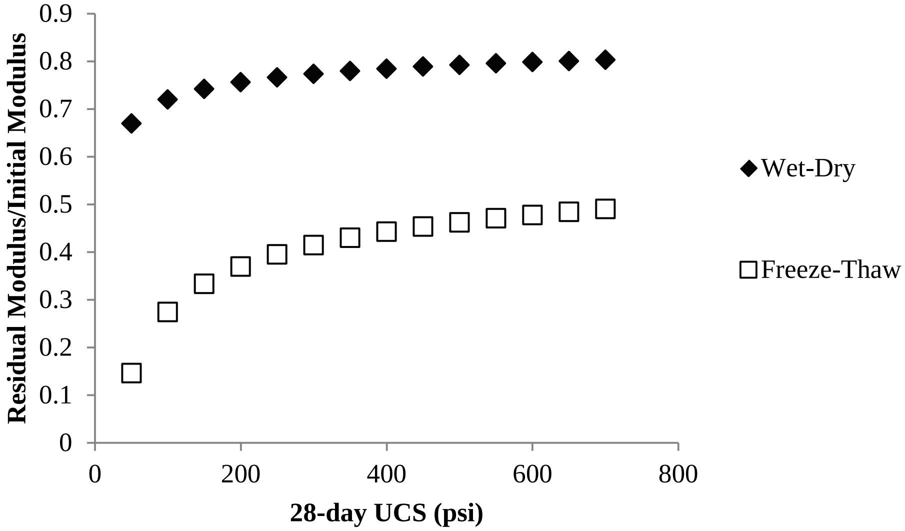

Figure E-1. Residual Modulus/Initial Modulus for Different 28-day UCS Values

CHAPTER E-2. FATIGUE MODEL APPRAISAL

The fatigue model includes the bottom-up tensile-fatigue life model, top-down compressive-fatigue-erosion life model, accumulated damage model, and modulus reduction models, as shown by Eq. (E-3) through Eq. (E-7), respectively.

The bottom-up tensile-fatigue life model is:

$$ln\, N_{ft} = k_1 \left(\frac{k_2 - \frac{\sigma_t}{M_{rup}}}{k_3} \right) \qquad \text{Eq. (E-3)}$$

The top-down compressive-fatigue-erosion life model is:

$$log\, N_{fc} = k_4 log\left(\frac{\rho}{\omega}\right)\left(1 - \frac{\sigma_c}{k_5\, UCS}\right) \qquad \text{Eq. (E-4)}$$

The accumulated damage model is:

$$D = \sum_{i=1}^{j} \frac{n_i}{N_{fci}} \qquad \text{Eq. (E-5)}$$

The bottom-up tensile-fatigue modulus reduction model is:

$$E(D) = E_{current}\left[\frac{m_2/ln(UCS_{28})}{1+e^{[sinh(n_2\, D)]}} + 1 - \frac{m_2/ln(UCS_{28})}{2}\right] \qquad \text{Eq. (E-6)}$$

The top-down compressive-fatigue-erosion modulus reduction model is:

$$E(D) = E_{current}\left[\frac{m_3/ln(UCS_{28})}{1+e^{[sinh(n_3\, D)]}} + 1 - \frac{m_3/ln(UCS_{28})}{2}\right] \qquad \text{Eq. (E-7)}$$

where

N_{ft} = bottom-up tensile-fatigue life
σ_t = tensile stress at the bottom of beam, psi
M_{rup} = modulus of rupture, psi
N_{fc} = top-down compressive-fatigue-erosion life, defined as the number of cycles when the modulus drops to 50% of the initial modulus value
ρ = maximum dry density, lb/ft^3
ω = optimum moisture content, %
σ_c = compressive stress applied on top of specimen, psi
UCS = current unconfined compressive strength, psi
UCS_{28} = 28-day unconfined compressive strength, psi
D = accumulated damage
j = total number of load groups
n_i = actual number of repetitions of load group i

N_i = allowed number of repetitions to fatigue of load group i
$E(D)$ = modulus after accumulated damage D
$E_{current}$ = modulus before fatigue damage, at age t in months (considering the strength growth from 28 days to the time of top-down compressive-fatigue-erosion), psi
k_2 and k_3 = lab-fitted regression parameters
k_1 , k_4, k_5 = field-calibrated regression parameters
m_2, n_2 m_3, n_3 = regression parameters.

Tables E-2 and E-3 show the model parameters.

Table E-2. Model Parameters of Bottom-Up Tensile-Fatigue Model for Different Materials

Specimen (Binder Content %)	Regression Parameters		R^2
	k_2	k_3	
Clay-cement (12%)	0.03	1.03	0.82
Gravel-cement (3%)	0.04	0.9	0.95
Sand-cement (6%)	0.04	1.2	0.88
Silt-cement (8%)	0.06	1.43	0.87
Sand-fly ash (13%)	0.02	0.8	0.95
Silt-lime-fly ash (4/12%)	0.06	1.28	0.94
Clay-lime (6%)	0.03	0.99	0.72
Gravel-cement (3%) [90% MDD]	0.07	1.02	0.93
Silt-cement (8%) [90% MDD]	0.02	1.02	0.74
Gravel-cement (5%)	0.03	0.85	0.89
Sand-cement (8%)	0.03	1.06	0.89
Silt-fly ash (18%)	0.04	0.7	0.7
Average	**0.04**	**1.02**	**n/a**

Table E-3. Fatigue Model Parameters after Field Calibration

Model	Equation	Parameter	Value
Bottom-Up Tensile-Fatigue Life	(E-3)	k_1	1.07
		k_2	Table E-2
		k_3	Table E-2
Top-Down Compressive-Fatigue-Erosion Life	(E-4)	k_4	10.85
		k_5	1.47
Bottom-Up Tensile-Fatigue Modulus Reduction	(E-6)	m_2	3.10
		n_2	3.99
Top-Down Compressive-Fatigue-Erosion Modulus Reduction	(E-7)	m_3	5.08
		n_3	2.01

These fatigue models are appraised by varying the stress ratios and CSM. Materials with 28-day UCS values of 300 psi and 700 psi are assumed. Table E-4 lists the analysis results. Figures E-2 through E-4 show the fatigue life data, reduction of CSL modulus value due to

fatigue damage, and residual CSL modulus value after 100% damage, respectively. Figure E-3 indicates that, at the same damage level, top-down compressive-fatigue-erosion has more damaging effect on modulus reduction than bottom-up fatigue for materials with the same 28-day UCS. Figure E-3 also indicates that for the same level of damage, materials with higher 28-day UCS values have smaller deterioration of modulus than the weak CSM. Figure E-4 shows that the CSM with high strength values have high residual modulus values after 100% damage.

Table E-4. Reasonableness Analysis Results of Field-Calibrated Fatigue Models

Fatigue Life			Modulus Reduction Ratio						Residual Modulus		
Assumed Value	Predicted Value		Assumed Value	Predicted Value (assumes 28-day UCS = 300 psi)		Assumed Value	Predicted Value (assumes 28-day UCS = 700 psi)		Assumed Value	Predicted Value	
Stress Ratio	Bottom-Up Fatigue Life (N)	Top-Down Fatigue Life (N)	Damage (D)	Bottom-Up Modulus Reduction Ratio	Top-Down Modulus Reduction Ratio	Damage (D)	Bottom-Up Modulus Reduction Ratio	Top-Down Modulus Reduction Ratio	28-Day UCS (psi)	Bottom-Up Residual Modulus Ratio (D = 1)	Top-Down Residual Modulus Ratio (D = 1)
0	3.5E+11	1.6E+08	0	1.00	1.00	0	1.00	1.00	50	0.62	0.34
0.1	2.6E+10	3.3E+07	0.1	0.97	0.93	0.1	0.97	0.94	100	0.68	0.44
0.2	1.9E+09	6.9E+06	0.2	0.94	0.86	0.2	0.94	0.88	150	0.70	0.48
0.3	1.4E+08	1.4E+06	0.3	0.90	0.79	0.3	0.92	0.81	200	0.72	0.51
0.4	1.1E+07	3.0E+05	0.4	0.87	0.71	0.4	0.89	0.75	250	0.73	0.53
0.5	8.0E+05	6.1E+04	0.5	0.83	0.64	0.5	0.86	0.69	300	0.74	0.54
0.6	6.0E+04	1.3E+04	0.6	0.80	0.59	0.6	0.83	0.65	350	0.75	0.56
0.7	4.4E+03	2.6E+03	0.7	0.77	0.56	0.7	0.80	0.62	400	0.75	0.57
0.8	3.3E+02	5.4E+02	0.8	0.76	0.55	0.8	0.79	0.61	500	0.76	0.58
0.9	2.5E+01	1.1E+02	0.9	0.74	0.54	0.9	0.78	0.60	600	0.77	0.59
1	1.8E+00	2.3E+01	1	0.74	0.54	1	0.77	0.60	700	0.77	0.60

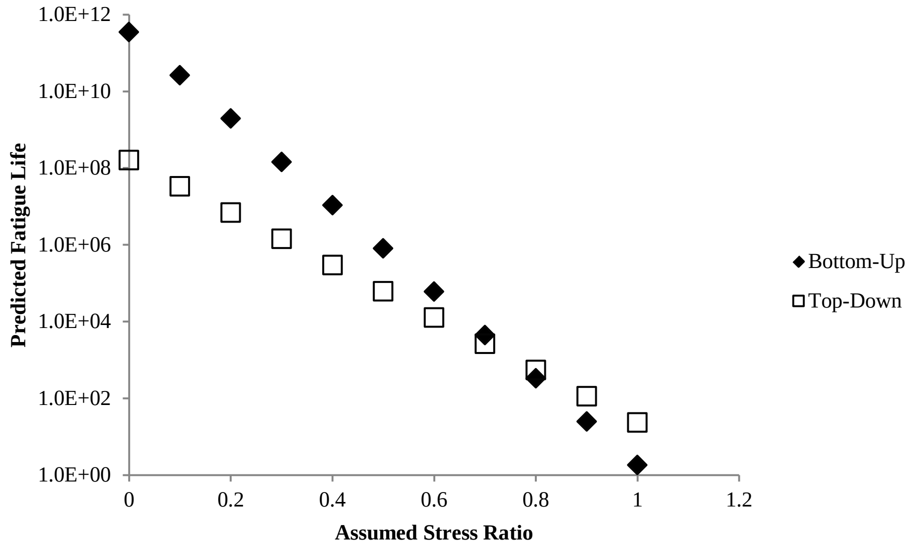

Figure E-2. Predicted Fatigue Life in Terms of Stress Ratio

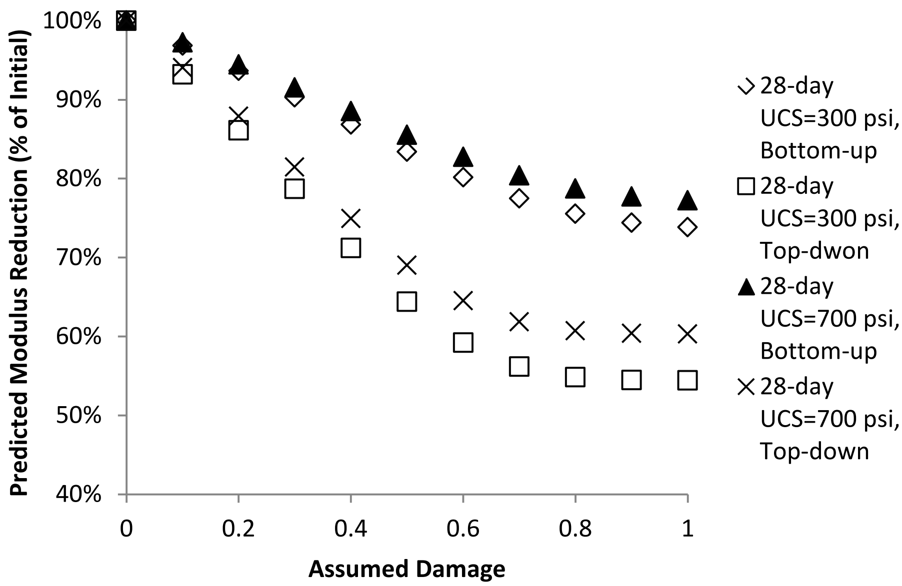

Figure E-3. Predicted Modulus Reduction in Terms of Damage (Assumes 28-day UCS = 300 psi and 700 psi)

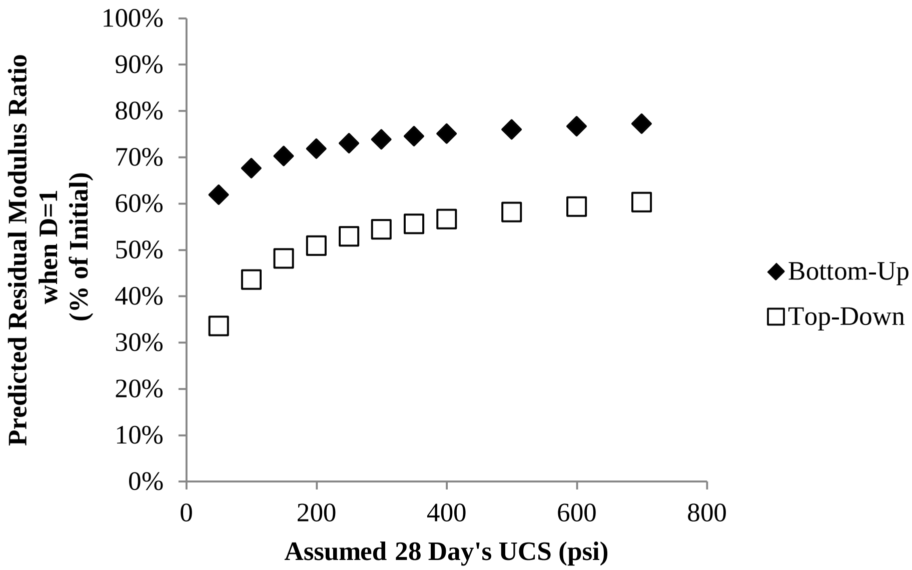

Figure E-4. Predicted Residual Modulus Ratio for Different 28-day UCS Values (when D = 1)

CHAPTER E-3. SHRINKAGE CRACKING MODEL

Eqs. (E-8) and (E-9) present the shrinkage cracking models for crack spacing and crack width, respectively. Tables E-5 and E-6 present the model parameters, respectively.

$$log(\frac{L}{H}) = \left[\left(\frac{\mu}{\rho \cdot H \cdot t^{-2}}\right)^{l_1} \cdot (c\%)^{l_2} \cdot \left(\frac{\omega}{\rho}\right)^{l_3} \cdot (\Delta T \cdot COTE)^{l_4} \cdot \left(\frac{UCS_{28}}{\rho \cdot H^2 \cdot t^{-2}}\right)^{l_5} \cdot (RH\%)^{l_6} \cdot \left(\frac{\varepsilon_{top}}{\varepsilon_{ult}}\right)^{l_7} \cdot \left(\frac{\varepsilon_{top} \cdot E_t}{S_{IDT}}\right)^{l_8}\right] \cdot l_9 \qquad \text{Eq. (E-8)}$$

$$W = \left[\left(\frac{\mu}{\rho \cdot H \cdot t^{-2}}\right)^{w_1} \cdot (c\%)^{w_2} \cdot \left(\frac{\omega}{\rho}\right)^{w_3} \cdot (\Delta T \cdot COTE)^{w_4} \cdot \left(\frac{UCS_{28}}{\rho \cdot H^2 \cdot t^{-2}}\right)^{w_5} \cdot (RH\%)^{w_6} \cdot \left(\frac{\varepsilon_{top}}{\varepsilon_{ult}}\right)^{w_7} \cdot \left(\frac{\varepsilon_{top} \cdot E_t}{S_{IDT}}\right)^{w_8}\right] \cdot w_9 \qquad \text{Eq. (E-9)}$$

where

L = crack spacing, in.
H = thickness of CSL, in.
μ = coefficient of friction, psi/in.
ρ = dry density, lb/ft^3
t = age at time of crack survey, days
$c\%$ = calcium content, %
ω = water content, lb/ft^3
ΔT = average daily maximum temperature variation, °F
$COTE$ = coefficient of thermal expansion, /°F
UCS_{28} = 28-day UCS at 68°F and 100% RH
RH = atmospheric relative humidity, %
ε_{top} = shrinkage on top surface under daily environmental conditions
ε_{ult} = ultimate drying shrinkage
S_{IDT} = IDT strength, calculated from IDT strength growth model, psi
E_t = IDT modulus, calculated from IDT modulus growth model, psi
l_i = regression parameters for crack spacing model, i = 0, 1, 2, 3, 4, 5, 6, 7
w_i = regression parameters for crack width model, i = 0, 1, 2, 3, 4, 5, 6, 7.

Table E-5. Field-Calibrated Parameters of CSL Shrinkage Crack Spacing Model for Fine and Coarse Host Materials

Parameters	Fine Materials	Parameters	Coarse Materials
l_1 (<0)	-1.19E-01	l_1 (<0)	0
l_2 (>0)	5.98E-01	l_2 (<0)	-1.39E-01
l_3 (<0)	-7.78E-01	l_3 (<0)	-1.36E-04
l_4 (<0)	0	l_4 (<0)	0
l_5 (>0)	0	l_5 (<0)	-1.46E-01
l_6 (>0)	0	l_6 (>0)	2.11E+00
l_7 (<0)	-2.20E-03	l_7 (<0)	0
l_8 (<0)	-2.53E-01	l_8 (<0)	0
l_9	8.74E+00	l_9	3.85E+00

( ) contains the parameter constraints.

Table E-6. Field-Calibrated Parameters of CSL Shrinkage Crack Width Model for Fine and Coarse Host Materials

Parameters	Fine Materials	Parameters	Coarse Materials
w_1 (>0)	7.81E-03	w_1 (>0)	0
w_2 (<0)	-1.20E+00	w_2 (>0)	0
w_3 (>0)	7.67E-01	w_3 (>0)	1.34E+00
w_4 (>0)	0	w_4 (>0)	1.76E-05
w_5 (<0)	0	w_5 (>0)	3.63E-02
w_6 (<0)	0	w_6 (<0)	0
w_7 (>0)	6.69E-01	w_7 (>0)	0
w_8 (>0)	4.71E-01	w_8 (>0)	5.36E-02
w_9	8.63E-04	w_9	1.78E-01

( ) contains the parameter constraints.

The shrinkage cracking models are appraised for three different materials (cement-clay, cement-silt and cement-gravel) and constant environmental conditions (68°F and 90% RH). The underlying layer under the CSL is clay with a coefficient of friction of 21.8 psi/in. Table E-7 presents the material properties.

Table E-7. Input for Simulation of Shrinkage Cracking Model

	Cement-Clay	Cement-Silt	Cement-Gravel
μ (psi/in.)	21.8	21.8	21.8
cement content (%)	12.0	8.0	3.0
c%	7.6	5.0	1.9
ω (lb/ft3)	18.6	13.3	8.7
γ (pcf)	103.2	119.9	140.1
H (in.)	4.0	4.0	4.0
UCS_{28}	607.3	905.4	823.9
Ave Temp (°F)	68.0	68.0	68.0
ΔT (°F)	1.0	1.0	1.0
RH (%)	90.0	90.0	90.0
COTE ($\times 10^{-6}$/°F)	12.6	9.3	8.5

Figures E-5 and E-6 present the appraisal results. The crack spacing and crack width are predicted for five years. The figures show that both crack spacing and crack width stabilize after two years. It is noted that in the field, reflective cracking could continue to develop in the asphalt layer after two years. The cement-silt has relatively large crack spacing compared to that of cement-clay and cement-gravel. Figure E-6 shows that the crack widths of the stabilized fine material are larger than those of the stabilized coarse material. Overall, the characteristics of the host materials directly affect the shrinkage behavior of the CSL, and the fine material exhibits more severe shrinkage crack distress than the coarse material.

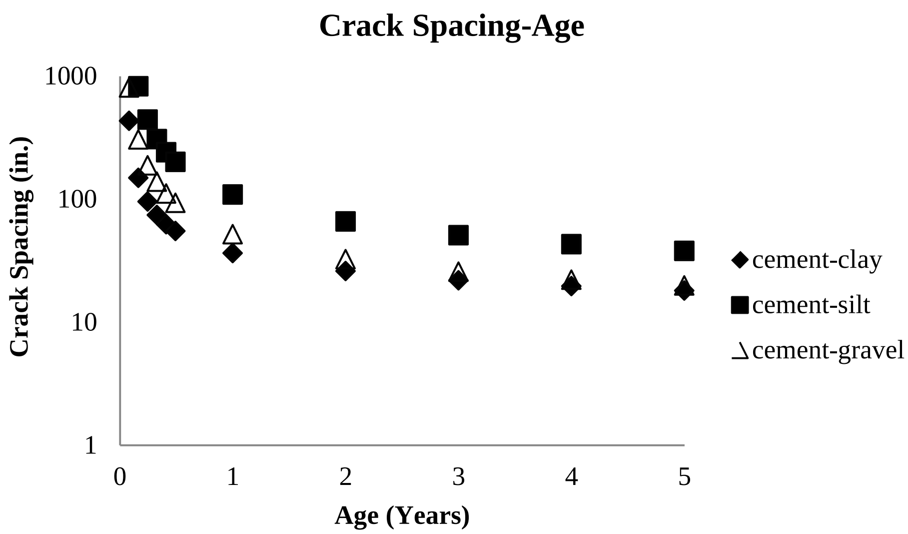

Figure E-5. Predicted Crack Spacing of Three Materials

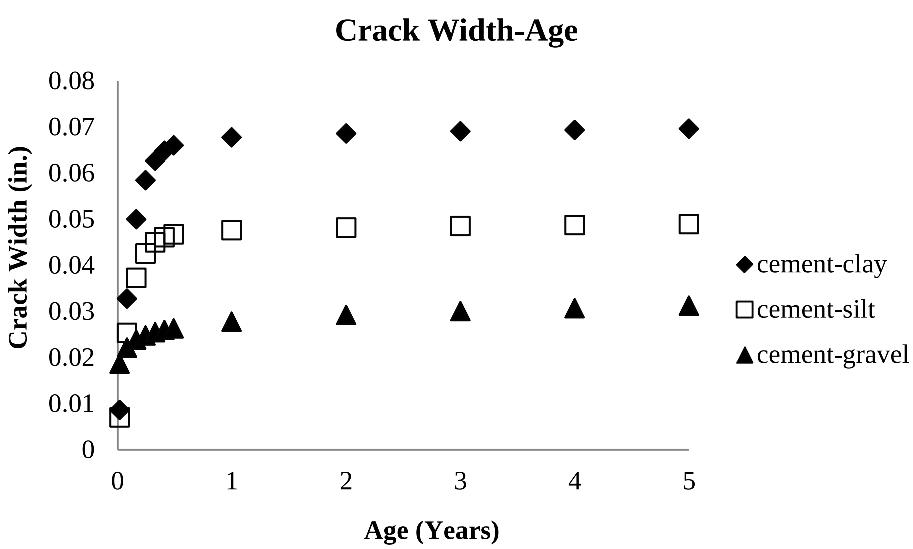

Figure E-6. Predicted Crack Widths of Three Materials

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