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16. Abstract

Researchers designed a laboratory test sequence to identify the optimum amount of portland type I cement for stabilizing two aggregates, limestone and recycled concrete, typically used in the Houston District. Smectitic compositions identified through mineralogical investigations corresponded with the poor performance of the untreated aggregates in preliminary testing and substantiated the need for stabilization. Samples were subsequently treated with 1.5, 3.0, and 4.5 percent cement and tested for strength, shrinkage, durability, and moisture susceptibility in the laboratory. Strength was determined with the Soil Cement Compressive Strength Test (TxDOT Test Method Tex-120-E), and a linear shrinkage test was developed to assess shrinkage characteristics. Durability was evaluated with the South African Wheel Tracker Erosion Test (SAWTET), and moisture susceptibility was assessed with the Tube Suction Test (TST). The limestone aggregate was also subjected to modulus testing. Based on these parameters, stabilized samples exhibited markedly improved performance with minimum additions of cement. Based on the results of the laboratory testing, the recommendation of this report is 3.0 percent cement for the limestone and 1.5 percent cement for the recycled concrete.

For future testing of aggregate base materials to determine optimum cement contents, the joint utilization of the strength test and the TST is recommended. Sufficient quantities of cement should be added to tested samples to obtain minimum unconfined compressive strengths of 300 psi in the former and maximum average surface dielectric values of 10 in the latter. The minimum amount of cement necessary to satisfy both criteria should be recommended for pavement construction. In addition to these tentative specifications, a provisional pre-cracking procedure is also suggested in this report for further evaluation.

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SELECTING OPTIMUM CEMENT CONTENTS FOR STABILIZING AGGREGATE BASE MATERIALS

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CHAPTER 1. INTRODUCTION

GENERAL

Cement has been applied in situ to modify and stabilize soils and aggregate layers for highway construction since 1935 (*1*). Recently, however, some traditionally strong markets for cement usage have reduced their use of cement treatments due to several factors, such as shrinkage cracking and faulting, that cause accelerated materials degradation and premature pavement failure. While the current procedures for determining design levels of cement treatments are based mainly on compressive strength, additional tests have become available for assessing other properties of cement-stabilized aggregates that could be used to more accurately predict performance in the field and more appropriately select optimum cement contents.

In 1998, the Houston District of the Texas Department of Transportation (TxDOT) initiated a research project with the Texas Transportation Institute (TTI) to use some of these tests to determine the impact of varying cement levels on performance-related engineering properties of aggregate base materials. Two aggregates typically used in the Houston area, limestone and recycled concrete, were selected for the testing.

The laboratory test program was designed to evaluate compressive strength, shrinkage, durability, and moisture susceptibility of the aggregates at three levels of portland type I cement treatment, including 1.5, 3.0, and 4.5 percent. The test procedures chosen to evaluate each engineering property were the Soil Cement Compressive Strength Test (TxDOT Test Method Tex-120-E), a linear shrinkage test, the South African Wheel Tracker Erosion Test (SAWTET), and the Tube Suction Test (TST), respectively. Researchers also performed modulus measurements on cement-treated samples of the limestone aggregate. The research objective was to identify an optimum cement content for each aggregate that would meet strength requirements, minimize shrinkage, improve durability, and reduce moisture susceptibility. The following chapters provide background information, detailed laboratory procedures, test results, and conclusions.

CHAPTER 2. BACKGROUND

DESIGN CONSIDERATIONS

Most problems with cement-stabilized base layers in pavements stem from the fact that current design practices are based only on strength, without consideration of long-term durability or performance. For example, many state departments of transportation require sufficient cement to achieve a minimum unconfined compressive strength as high as 750 psi after seven days (*1*). While this level of cement results in a very stiff aggregate layer characterized by a high resilient modulus, it does not necessarily guarantee acceptable long-term pavement performance.

In many roadways, for instance, shrinkage cracks within heavily cement-stabilized base layers reflect into the surface treatments and appear as transverse cracks with a spacing of between 3 ft and 60 ft (*1*). Although the cracks themselves may not present a structural problem, they often accelerate degradation of the pavement by allowing water to enter lower pavement layers. Several documented cases demonstrate the ability of moisture to disintegrate underlying base materials, causing a reduction in pavement support and a corresponding increase in pavement roughness that often leads to unacceptable riding quality (*2*).

Thus, the focus of the research conducted in this project was to evaluate supplements to strength-based design procedures (*3*). To this end, following the completion of mineralogical characterizations, researchers used a series of laboratory test procedures to analyze strength, shrinkage, durability, and moisture susceptibility characteristics at three levels of cement treatment for each of the aggregates included in this project. Modulus testing was also performed on the limestone aggregate. A discussion of each topic follows.

MINERALOGY

Aggregates are predominantly comprised of small mineral crystals that give rise to most of their physical and chemical properties (*4*). The primary purpose of mineralogical testing is to identify and quantify the individual mineral constituents. The use of x-ray diffraction (XRD) for the qualitative identification of minerals is the backbone of these laboratory investigations. Chemical tests for total potassium determination and cation exchange capacity (CEC) are useful supplements for estimating the quantity of each identified mineral. Imaging microscropy provides information about the morphology of individual particles, including shape, angularity, texture, and agglomeration (*5*).

These physical and chemical characteristics of soil minerals are important factors in considering the engineering performance of aggregate base materials. Understanding the mineralogical composition of aggregates aids in determining such physical parameters as surface area, affinity for water, and volume stability. The chemistry of reactions can be investigated through identification of individual mineral constituents that play a significant role in the strength and durability of materials. In this project, the performance of the untreated materials in the preliminary characterization was related to the mineral composition of the aggregates, with emphasis placed on the amount of smectite because of its high surface area and reactivity (*6*).

STRENGTH

Although the Portland Cement Association and the United States Army Corps of Engineers developed their individual design criteria for cement-stabilized base materials in accordance with both strength and durability requirements, most state departments of transportation have historically focused on compressive strength alone (*7, 8*). In the 1960s, for example, the California Division of Highways proposed a minimum strength criterion of 850 psi at seven days for cement-stabilized base materials, which was thereafter reduced to 750 psi following several instances of severe shrinkage cracking. During the same period, minimum strength criteria of 700 psi, 250 to 400 psi, 300 psi, and 450 psi were issued by the Texas Department of Transportation, the Road Research Laboratory in the United Kingdom, the United States Air Force, and the Iowa Department of Transportation, respectively (*1*).

The Texas Department of Transportation constructed thousands of highway miles with cement-stabilized base layers designed to meet the 700 psi requirement and experienced, like in California, unsatisfactory performance in many instances due to shrinkage cracking (*1*). This led several of the Texas districts to abandon the use of cement-stabilized bases in preference to lime or fly ash combinations. The use of cement has only made a resurgence in recent years through significant changes in design criteria, where the target seven-day unconfined compressive strength has been reduced to lower values. Although lower cement contents have been shown to improve the long-term performance of stabilized layers, there is still little agreement in the highway community on the selection of a minimum strength requirement. Unconfined compressive strength remains the most widely referenced property for the design of cement-stabilized aggregate base materials, however, and is therefore included in this laboratory test program.

SHRINKAGE

The majority of performance problems occurring in cement-stabilized aggregate base materials are related to shrinkage cracking. While fine, distantly spaced shrinkage cracks do not generally create a structural deficiency in the pavement system, poor performance can result from wide, closely spaced cracks that cause poor load transfer and unacceptable riding quality.

The shrinkage of cement-treated materials results from the loss of water by drying and from self-desiccation during the hydration of cement (*9*, *10*). Several factors influence the magnitude and rate of shrinkage, including mixture proportions and the material properties of individual constituents. Generally, materials containing higher fines contents exhibit greater shrinkage potential than coarser materials, which, depending on subbase friction and the tensile strength of the base layer, can lead to the formation of wider cracks (*9*). The type of clay mineral present also influences the drying shrinkage potential, with the smectite group exerting the most pronounced influence (*11*).

Efforts to minimize shrinkage cracking have centered on material selection, mixture design, curing quality, and specific construction techniques. Based on research in Queensland, for example, the Australian code of practice for cement applications has recently changed to control gradations of raw materials as well as to introduce laboratory-measured linear shrinkage as an indicator of shrinkage potential in the field (*11*). Furthermore, the use of low-shrinkage cement blended with fly ash was recommended to reduce shrinkage strains in treated aggregates. Limited laboratory testing in Georgia showed that the use of expansive cement also provided a marked reduction in shrinkage cracking compared to portland type I cement (*12*).

Effective retention of moisture in cement-treated materials is especially important for promoting cement hydration and corresponding strength gain while reducing shrinkage. The impacts of drying shrinkage can be combated by application of a curing emulsion immediately after construction of the cement-treated base layer (*13*). Some researchers suggest that delaying placement of the surface by as long as practical afterwards should then reduce reflective cracking through the surface layer simply because less shrinkage of the base occurs after surface application (*14*).

From the construction viewpoint, efforts to further delay the appearance of shrinkage cracks at the surface are manifest in design of the "upside-down" pavement section (*15*). An untreated granular layer between the cement-treated base and the surface layer is added to disrupt crack growth into the surface. Saw-cutting has also been used to control the spacing of cracks, where the pre-determined spacing is a function of the material strength, shrinkage stress development, and crack potential (*16*).

In the field, pre-cracking is a viable method for reducing reflection cracking through surface layers placed over cement-treated bases. Pre-cracking should occur within one to three days after placement, where heavy traffic or vibratory rollers can be used to create networks of microcracks within the base layer that eliminate the development of larger shrinkage cracks (*17*). A recommended provisional specification for pre-cracking procedures was developed at TTI and is given in the appendix of this report (*18*). In consideration of the important role shrinkage cracking plays in the performance of cement-treated materials, this research project included a measure of linear shrinkage among the laboratory tests.

DURABILITY

Traditional durability testing is generally concerned with abrasion resistance, where the Wet-Dry Test (ASTM D 559) and the Freeze-Thaw Test (ASTM D 560) are most common. With these tests, the weight loss of a cement-treated material under wire brushing is determined through 12 cycles of wetting and drying or freezing and thawing, where the maximum allowable weight loss is a function of the aggregate classification. In research performed by the Portland Cement Association, about 20 percent of the samples with a seven-day compressive strength of 300 psi would pass the freezing and thawing test, while about 70 percent would pass with a compressive strength of 500 psi (*7*). This correlation between compressive strength and resistance to freezing and thawing damage was documented for guidance in mixture design, where higher cement levels generally decrease the average pore size and reduce the permeability of treated materials, making a sample more difficult to critically saturate (*10*).

For this project, however, researchers utilized a wheel tracking test developed in South Africa for assessing the durability of cement-treated materials because it better models the in situ distress mechanisms experienced by aggregates. The SAWTET specifically simulates the stress conditions that are induced by heavy traffic loading within a base material constructed under a thin surfacing *(19)*. In the test, three rectangular specimens are submersed in water and covered with a rubber membrane. The bottom side of the membrane has a rough grit surfacing that erodes each sample under the back-and-forth motion of three overhead wheels, each 39 lb. in weight. After 5000 passes, the depth of erosion is measured at 15 points on each sample surface and averaged to evaluate durability *(19)*.

The SAWTET has been used in Texas to evaluate the durability of cement-treated materials, where the same samples were used to assess both shrinkage and erosion properties *(20)*. Researchers included this test in this project to evaluate the consequences of reduced cement contents on abrasion resistance.

MOISTURE SUSCEPTIBILITY

The permanency of stabilization is a major concern with all stabilizing materials. Many state departments of transportation have experienced problems with stabilizers "disappearing" after a few years of service *(21)*. While this predicament is more common in layers stabilized with lime and fly ash, cement-treated materials have also been found to be susceptible to chemical reversals of the stabilization process. Generally, such reversals are associated with moisture intrusion and movement within the stabilized layer. Calcium hydroxide, for example, which is one of the principal constituents in materials treated with either lime or cement, has a very high solubility in water and may be leached rather rapidly *(4)*. Combined with traffic loading, the leaching out of cementitious components from an aggregate matrix by moisture and the subsequent formation of less stable phases can accelerate deterioration of base materials and cause early failure due to loss of strength.

During the past few years, the TST has been developed at TTI for investigating the moisture susceptibility of aggregate base materials *(22)*. The moisture susceptibility ranking is based on the surface dielectric value of a compacted specimen after a 10-day capillary soak in the laboratory and depends upon the suction and permeability of the aggregate layer and the state of bonding of water that accumulates within the aggregate matrix.

Research studies in Texas and Finland have demonstrated that moisture susceptibility is related to the suction properties of soils and aggregates *(22, 23)*. Soil suction characterizations are derived from the study of moisture flow through soil media and are composed of both osmotic and matric suction components. Osmotic suction is the suction potential resulting from salts present in the water portion of a soil system. Matric suction is the suction potential due to the matrix arrangement of the soil particles themselves. Tightly compacted particles form capillaries through which water may flow *(23)*.

Materials exhibiting high suction potential are strongly hydrophilic, and, when water is available, moisture ingress can rapidly deteriorate their engineering properties. Higher permeability results from more interconnected void space within the pore structure that allows water to more easily flow into and through the aggregate matrix. Permeability is an especially important issue in moisture damage mechanisms, such as frost heave, where water must be able to rapidly respond to changes in suction within the pavement structure *(24)*.

The state of bonding of water describes the structuring of the water molecules within the soil or aggregate matrix. Adsorbed water molecules are arranged in layers around aggregate particles as displayed in Figure 1. The first layer is a tightly bound, monomolecular layer, but as the distance from the aggregate surface increases, the water layers become more and more loosely bound. So-called viscous, or capillary, water molecules beyond the zone of electrical capture are considered unbound and, depending on permeability and time, can migrate within the pavement structure *(26)*.

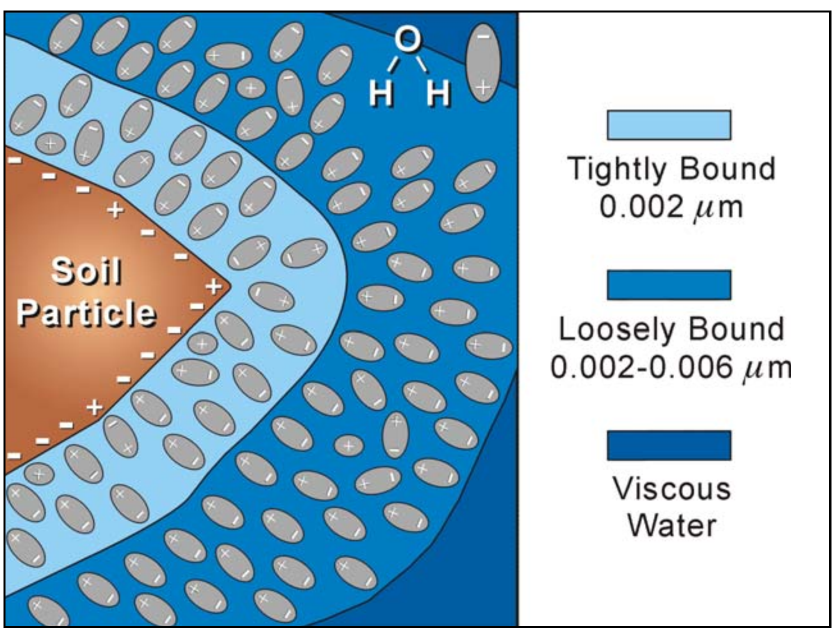

Figure 1. Structuring of Water Molecules *(25)*.

The interpretation of TST results is based on an empirical relationship between the final dielectric value and the expected performance of aggregate base materials, where the dielectric value of an aggregate in the TST is most sensitive to the amount of unbound water in the sample (*27*). Because the TST can be used to readily measure the affinity of an aggregate for moisture, researchers also included it in the laboratory test program developed for this project.

MODULUS

The modulus of a base material is often utilized in design procedures to determine the necessary thickness of the base layer, where strength and thickness are presumed interchangeable (*28*). The 2002 AASHTO Design Guide will incorporate a mechanistic-empirical design approach requiring a modulus input for each layer for relating the stress and strain distributions through the pavement profile.

In this project, the Standard Method of Test for Resilient Modulus of Subgrade Soils and Untreated Base/Subbase Materials (AASHTO T 292-91) was used despite its specific applicability to unbound base materials. Its selection for use in this project evaluating lightly stabilized materials resulted from personnel preference over the diametral test arrangement described in the Indirect Tension Test for Resilient Modulus of Bituminous Mixtures (ASTM D4123-82). A free-free resonant column method developed at the University of Texas at El Paso was also utilized (*29*).

In the former test, linear variable differential transformers (LVDTs) are utilized to measure deflections within a specified gauge length of the sample under a repeated load, and the recoverable strain after a 200-cycle conditioning period is used to calculate the resilient modulus. The latter technique is based on elastic wave propagation, where the velocity of seismic waves within a sample can be used to calculate Young's modulus, the shear modulus of elasticity, and Poisson's ratio. At one end of a sample, the energy source is given as a light tap from a hammer equipped with a load cell that measures the energy input and triggers a timing circuit. An accelerometer mounted at the other end of the sample reports the time of longitudinal and transverse wave arrivals necessary to complete the computer-automated calculations.

In longitudinal waves (P-waves), the particles of the medium move in the direction of wave travel, causing alternating expansions and contractions of the medium. In transverse waves (S-waves), the motion of the particles is perpendicular to the direction of wave travel. The equations of motion for dilatation and shear disturbances propagating through a material can be derived in terms of dilatational and rotation strains. The physical implication of these equations is that the velocities of longitudinal and transverse waves are related to the elastic properties of the material (*30*).

CHAPTER 3. TEST PROCEDURES

AGGREGATE CHARACTERIZATION TESTS

The purpose of this project was to utilize a set of laboratory tests based on both strength and long-term durability to determine the optimum content of portland type I cement for stabilizing limestone and recycled concrete aggregates typically used in the Houston District. A mechanical sieve analysis and a determination of optimum moisture content (OMC) and maximum dry density preceded all other testing. To demonstrate the effects of cement stabilization on each aggregate, researchers then performed several tests on the untreated materials. Both aggregates were submitted in their raw condition to the TST to evaluate moisture susceptibility prior to the addition of cement. The typical testing arrangement for untreated granular materials is shown in Figure 2. Immediately following the TST, the unconfined compressive strength of each sample was measured, and the mineralogy of both aggregates was also determined.

Following this preliminary characterization testing, the aggregates were treated with 1.5, 3.0, and 4.5 percent portland type I cement and tested for strength, shrinkage, durability, and moisture susceptibility in a series of four tests. These tests included the Soil Cement Compressive Strength Test (TxDOT Test Method Tex-120-E), a linear shrinkage test, the

Figure 2. TST Arrangement for Unstabilized Aggregates.

SAWTET, and the TST. Performance criteria proposed for each test are included in the following sections describing each test procedure. Modulus measurements were also performed on stabilized limestone samples. In addition to the Standard Method of Test for Resilient Modulus of Subgrade Soils and Untreated Base/Subbase Materials (AASHTO T 292-91), a free-free resonant column test developed at the University of Texas at El Paso was also included in the laboratory test program *(29)*.

MINERALOGY TESTS

The procedures for mineralogical determinations included pretreatments, dispersion and fractionation of the bulk sample, XRD, total potassium determination, CEC determination, and scanning electron microscopy (SEM) of the clay fractions. While the limestone sample was subjected to each step of the evaluation, the SEM evaluation was not performed on the recycled concrete. An overview of these procedures follows *(5)*.

Pretreatments

A bulk limestone sample representative of the complete gradation was oven dried and sufficiently crushed to pass through a 0.0787-in. (No. 10) sieve prior to pretreatments. The bulk recycled concrete sample was scalped on the No. 10 sieve directly, which, while eliminating the need for crushing, precluded any evaluation of the mineralogical components of the larger size fractions. The pretreatments included NaOAc and H_2O_2 for removal of carbonates and organic material, respectively, which act as binding agents in soils. The sample was then split on a 0.0029-in. (No. 200) sieve, and pH 4 1 N NaOAc was used to treat the coarse fraction for removal of carbonates. The fine fraction was treated with pH 5 1 N NaOAc for the same purpose. Organic matter, sulfides, and manganese oxides were removed with H_2O_2, and residues were washed out of the system with pH 5 1 N NaOAc and 1 N NaCl.

Dispersion and Fractionation

Following removal of the above-mentioned binding agents, the sample was dispersed and fractionated. Because some minerals tend to occur in certain size fractions, the size fractionation was important for concentrating mineral phases and improving layer silicate preferred orientation. After dispersion with pH 10 Na_2CO_3, the sample was scalped on a 0.0017-in. (No.

325) sieve to separate the sand fraction. Stoke's Law was then applied to centrifugation for separation of the silt, coarse clay, and fine clay fractions. The silt and coarse clay were separated at 0.0787 mils, and the coarse clay and fine clay were separated at 0.0079 mils.

X-Ray Diffraction

For XRD, dried samples were examined on glass slides and x-rayed over a range of angles from 2° to 65° 2θ using CuK radiation. A bulk sample was first x-rayed, followed by samples of various size fractions. The sand fraction was ground sufficiently to pass through a 0.0035-in. (No. 180) sieve, and subsamples of the sand and silt fractions were front loaded into an aluminum powder mount and x-rayed. The coarse clay was washed with distilled water and dried, and the fine clay was flocculated with NaCl and subjected to dialysis and freeze-drying before XRD was performed. One subsample of each of the coarse and fine clay fractions was saturated with Mg and prepared for XRD on glass slides, while another subsample of each of the coarse and fine clay fractions was saturated with K and prepared on Vicor slides. Treatment of the Mg slides with glycerol aided in the detection of smectite, and XRD of the K slides at 77 °F, 572 °F, and 1022 °F afforded distinction of chlorite, hydroxy-interlayered phyllosilicates, mica, and kaolinite.

Total Potassium Determination

The determination of total K was necessary for estimating the amount of mica in the sample, given that 10 percent K_2O equals 100 percent mica in the absence of potassium feldspars. In this step, two samples of dried coarse clay and fine clay were each submitted to acid dissolution and fusion. Atomic absorption was then used to determine the K concentration in each of the final solutions.

Cation Exchange Capacity Determination

Because many common minerals have unique values of CEC, this testing can aid in quantifying the amounts of each mineral present in a given aggregate. CEC denotes the tendency for cations in solution to replace cations adsorbed on the mineral exchange surface. Higher CEC values indicate a greater number of cation exchange sites on the mineral surface. Exchanges generally follow the concentration-valency rule so that the apparent preference of the mineral surface for an ion of higher charge increases with dilution of the solution (*4*).

Studies in surface chemistry indicate that the thickness of the adsorbed water layer can be reduced by increasing the valence and concentration of the cations within the layer (*4*). This is a fundamental mechanism of lime stabilization, for instance, that brings about almost immediate textural changes in clayey soils. Decreasing the water layer thickness promotes flocculation of clay particles and increases the shear strength of the soil matrix (*31*).

As in the determination of total K, two samples each of dried coarse clay and fine clay were used in this procedure. The samples were saturated with Ca and then washed with 1 N $MgCl_2$. During the washing, adsorbed Ca ions were replaced by Mg ions at cation exchange sites on the mineral surface, which allowed freed Ca ions to be released into the supernatant. The supernatant was sampled, and the concentration of Ca cations was determined with atomic absorption spectroscopy. Values of replicate samples were averaged for CEC calculations, where higher substitutions of Mg for Ca yield higher CEC results. Typical values are given in Chapter 4.

Scanning Electron Microscopy

Researchers used SEM to examine the morphology and elemental composition of the bulk sample and the silt fraction of the crushed limestone material. Interpretations of SEM data were aided by prior knowledge of the mineralogy of the sample as determined from XRD.

Data Synthesis

Final descriptions of the mineralogy of each aggregate were prepared from integrated analyses of the collected test data. XRD was successful in identifying constituent minerals, and the relative sizes of some peaks, such as those for quartz and kaolinite, were used to compute the relative percentages of those minerals comprising the bulk sample. The total potassium measurement provided a direct estimate of the mica content, and the total CEC was used to calculate the percentages of minerals whose individual CEC values were independently known from the literature (*5*).

The total CEC was decomposed into a summation of products of individual CEC values and their respective compositional percentages. A system of two equations was constructed for each clay fraction of each aggregate sample, with one equation derived from CEC data and the other from mass balance rules, where the sum of all mass fractions was required to equal 100 percent. The solution for the system of equations then yielded the quantitative estimates of each mineral present in each clay fraction. For the limestone aggregate, imaging microscopy provided additional details about the physical structure of the minerals and served to confirm the XRD results. The mineral composition of both aggregates was then related to the performance of the untreated materials in preliminary testing.

STRENGTH TEST

The measurement of unconfined compressive strength in this project followed the procedures of the Soil Cement Compressive Strength Test (TxDOT Test Method Tex-120-E), except that the aggregates were scalped on the 1-in. sieve prior to sample construction, and distilled water was specified for mixing. The distilled water was used to avoid contamination of the samples by any ions present in regular tap water, which might have altered the performance of samples in the TST by increasing osmotic suction. Compacted samples were capped with gypsum prior to the testing, which was accomplished at a constant loading rate of 0.135 in./min under a floating head until failure.

For each cement level, this test was performed at three combinations of moisture and time of curing. Samples cured at 100 percent relative humidity and 77 °F were tested at seven days and at 28 days. Also, samples that had been subjected to the TST and then placed underwater until after reaching constant weight were tested in their soaked condition. Regular tap water was used for soaking purposes. For the seven-day curing, a strength of 300 psi was set as the minimum threshold, while the minimum strength after soaking was specified to be greater than 80 percent of the 28-day strength.

SHRINKAGE TEST

Researchers prepared rectangular beam samples for the linear shrinkage test. After the aggregate was scalped on the 0.75-in. sieve, mixing was performed according to TxDOT Test Method Tex-120-E. Again, distilled water was used for mixing. The beams were then constructed in three lifts inside a metal form, with each lift compacted by 56 blows of a 10-lb. hammer dropped from a height of 18 in. A leveling load of 18,000 lb. was applied across the top of the sample to complete the compaction process. As shown in Figure 3, each finished beam was 18 in. long with a square cross-section, where the length of each side was approximately 3 in.

Samples were removed from the form after several hours of curing in an environmental chamber maintained at 100 percent relative humidity and 77 °F. Afterwards, metal gauge studs were glued onto the ends of the sample with epoxy to facilitate shrinkage measurements over the following 21 days. Although both aggregates were tested at approximately 77 °F, the limestone was tested in an environment of less than 50 percent relative humidity, while the recycled concrete remained in the curing chamber for the duration of the testing to accommodate use of the same beams in durability testing afterwards. (Only a limited quantity of recycled concrete was retained for testing in this project.) In all cases, samples were tested on a smooth plastic surface.

The device used for shrinkage measurements, shown in Figure 4, was equipped with a dial gauge capable of measuring deflection to the nearest 0.0004 in. The Australian specification stating that the shrinkage strain should not exceed 0.000250 in./in. after 21 days was proposed for judging the performance of both sets of samples (*11*). This specification was developed from extensive laboratory testing of beam samples of similar size and has been implemented in quarry and plan mixing processes in Queensland.

Figure 3. Beam Sample.

Figure 4. Shrinkage Measurement Device.

DURABILITY TEST

The same size samples used for shrinkage measurements were prepared for evaluating durability in the SAWTET. As mentioned earlier, the very same shrinkage samples could be used in the case of the recycled concrete. However, for the limestone, sufficient material was available for a new set of beams to be constructed, and these beams were allowed to cure at 100 percent relative humidity for a 28-day period before testing. After curing, each sample was trimmed with a tile saw to a length of approximately 12 in. and cast in plaster of paris in a rectangular mold so that only one rectangular face was exposed for wheel tracking. A typical sample is shown in Figure 5. The cast sample was then placed underwater until reaching constant weight.

Figure 6 depicts the wheel tracking machine, which allowed testing in groups of three. In this case, every cement level could be represented in each batch. Before and after testing, depth measurements were made in a jig over an array of 15 points across the sample surface to facilitate calculation of the average depth of erosion. Figure 7 is a picture of the jig and the digital caliper used for measuring depth. South African specifications for the test state that the average depth of erosion should be less than 0.04 in. after 5000 wheel passes (*19*).

Figure 5. Cast Beam Sample.

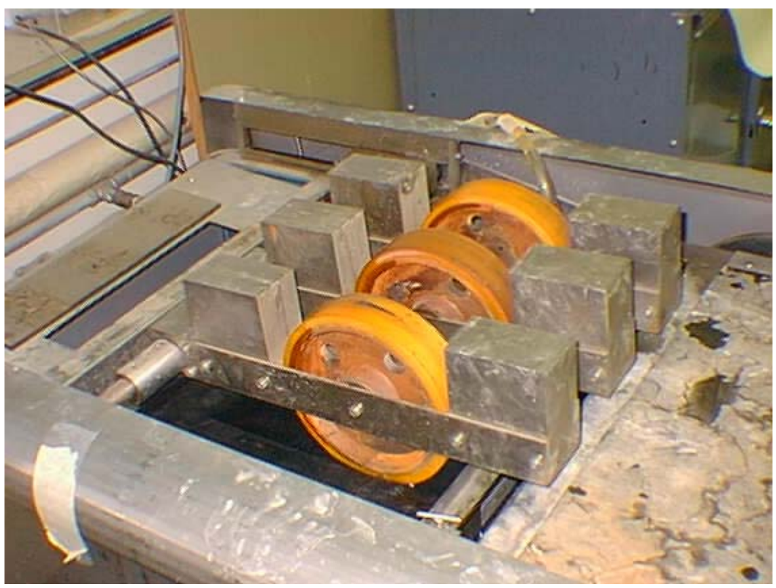

Figure 6. Wheel Tracking Machine.

Figure 7. Measurement Jig.

MOISTURE SUSCEPTIBILITY TEST

To investigate moisture susceptibility, the TST used samples of 6-in. diameter and 8-in. height that were prepared in the same manner as for the unconfined compressive strength test. After being extruded from the metal compaction mold, the limestone and recycled concrete specimens were placed for 28 days and seven days, respectively, in an environmental chamber maintained at 100 percent relative humidity and 77 °F for curing. (Research has shown that TST results for samples cured for seven days are not significantly different than results for samples cured for 28 days (*1*).)

Following the curing period, samples were moved for four days to another environmental chamber maintained at 104 °F and 50 percent relative humidity for drying. Samples were then placed in a 0.50-in.-deep bath of distilled water for capillary rise over a 10-day period. During this time, the surface dielectric value of each sample was monitored daily with an Adek Percometer™ probe, as shown in Figure 8. At each measurement time, five readings were taken around the perimeter of each specimen and a sixth in the center. The highest and lowest readings were discarded as a means of reducing variability, and the remaining four were averaged.

Aggregates whose final average dielectric values in the TST are less than 10 are expected to provide superior performance as base materials, while those with dielectric values above 16 are expected to provide poor performance. Aggregates having final dielectric values between 10 and 16 are expected to be marginally moisture susceptible (*22*).

Figure 8. Using the Adek Percometer™.

MODULUS TEST

Modulus testing was performed on only the limestone aggregate evaluated in this project, with two methods being used. In the Standard Method of Test for Resilient Modulus of Subgrade Soils and Untreated Base/Subbase Materials (AASHTO T 292-91), a repeated axial deviator stress of fixed magnitude, 0.1-s load duration, and 1.0-s cycle duration was applied to an unconfined cylindrical specimen. The total resilient, or recoverable, axial deformation response of the specimen was measured with LVDTs and used to calculate the resilient modulus. In addition to measuring deformation over the full height of a sample with one set of LVDTs, a second set was placed to measure deflections across a 6-in. gauge length to mitigate end effects, which are discussed in Chapter 4. As shown in Figure 9, mounts were glued to the sample 1 in. from each end. One soaked sample treated at each cement level was subjected to this test.

For the free-free resonant column method developed at the University of Texas at El Paso, cylindrical specimens were placed on their sides on a sheet of styrofoam insulation in the laboratory, and an accelerometer was affixed to one end of the sample. As shown in Figure 10, a hammer instrumented with a load cell was used to lightly tap the other end. A computer display of the measured wave response shapes was used to determine the quality of the test run, and the average of three measurements was used in a software package to calculate Young's modulus, the shear modulus of elasticity, and Poisson's ratio based on the sample mass and dimensions. In this method, four soaked samples of each cement level were tested. Test results are given in Chapter 4.

Figure 9. Resilient Modulus Testing.

Figure 10. Seismic Testing.

CHAPTER 4. TEST RESULTS

AGGREGATE CHARACTERIZATION TEST RESULTS

Preliminary testing of each aggregate in its untreated state included a mechanical sieve analysis, determination of OMC and maximum dry density, subjection to the TST, and an assessment of unconfined compressive strength immediately following the TST. For the limestone, 10 samples were submitted to the TST, and three were tested for strength. In this and most of the other testing regimes in this project, only one sample of recycled concrete was tested due to limited material availability. The results of the preliminary testing are given in Figure 11 and Tables 1 through 3.

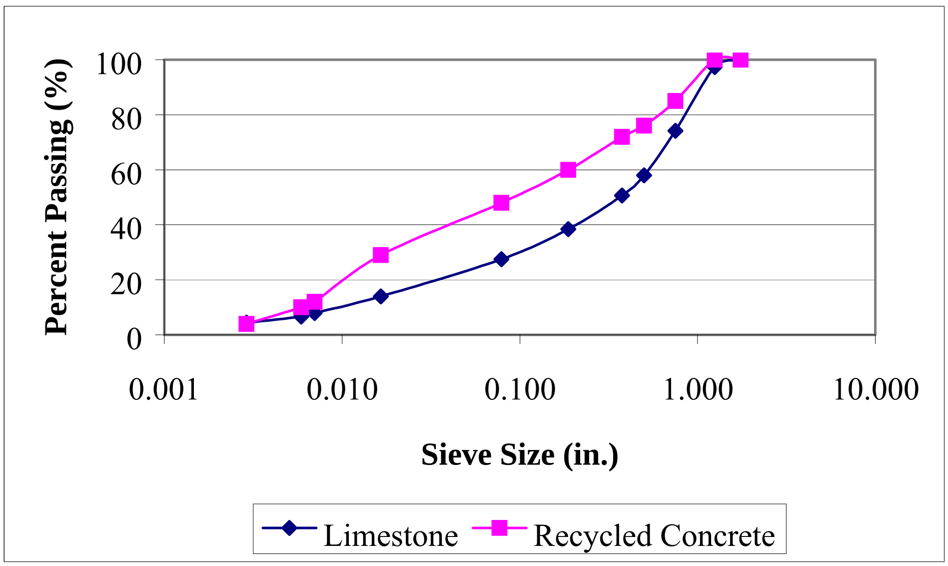

Figure 11. Sieve Analyses.

Table 1. Optimum Moisture Content and Maximum Dry Density.

Aggregate	OMC (%)	Maximum Dry Density (pcf)
Limestone	7.0	144.1
Recycled Concrete	10.5	123.5

Table 2. Moisture Susceptibility Test Results for Untreated Aggregates.

Aggregate	Beginning Moisture (%)		Ending Moisture (%)		Dielectric Value	
	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.
Limestone	2.0	0.4	6.1	0.2	19.9	1.3
Recycled Concrete	8.2	NA	12.6	NA	30.9	NA

Table 3. Strength Test Results for Untreated Aggregates Conditioned in TST.

Aggregate	Moisture (%)		Unconfined Strength (psi)	
	Mean	Std. Dev.	Mean	Std. Dev.
Limestone	6.1	0.8	39	3
Recycled Concrete	12.6	NA	48	NA

Although the percentage passing the 0.0029-in. (No. 200) sieve is the same for both aggregates, the recycled concrete has a finer gradation overall. The effects of this finer matrix are manifest in the higher OMC of the recycled concrete and its higher dielectric value in the TST. Because the dielectric value of both aggregates exceeded 16, however, they would both be expected to provide poor performance as base materials in their untreated condition from the perspective of moisture susceptibility.

During the drying period prior to the TST, the recycled concrete retained its moisture more strongly than the limestone samples, and while both aggregates imbibed approximately 4 percent moisture during the test, the recycled concrete aggregate ended the TST with a moisture content more than 2 percent higher than optimum. Unconfined compressive strength tests were performed on the samples immediately following the TST, with these moisture conditions being considered representative of those likely to exist in the field given the availability of moisture. As shown in Table 3, only the recycled concrete meets the strength requirement of 45 psi for a Triaxial Class 1 material, perhaps due to secondary chemical reactions causing recementation of the sample matrix. The following section presents the mineralogy of each aggregate.

MINERALOGY TEST RESULTS

Results and calculations for pretreatments, dispersion and fractionation, XRD, total potassium determination, CEC, and SEM are given in the following discussion.

Pretreatments

After dispersion and fractionation were completed, it was possible to calculate the percentage of insoluble material contained in each sample. For the limestone, the original sample was 0.4409 lb., and only 0.0210 lb. remained after pretreatment. Thus, the insoluble fraction constituted only 4.77 percent of the total sample, which is typical of Georgetown limestone according to similar studies by TxDOT (*32*). The soluble fraction was composed primarily of calcite, with trace impurities, that was readily dissolved during chemical pretreatment. For the recycled concrete, the original sample was 0.2227 lb., and 0.1784 lb., or 80.12 percent, was recovered. One of the components of the soluble fraction was asphalt coatings on some of the aggregate particles. The remainder of this mineralogical analysis focuses on the compositions of the aggregate residuals.

Dispersion and Fractionation

Table 4 shows the relative compositions of each aggregate residual obtained by fractionation. Based on the percentage of each constituent, the United States Department of Agriculture (USDA) classifications for the insoluble fractions of limestone and recycled concrete samples are clay and sandy loam, respectively.

Table 4. Fractionation Results.

Aggregate	Composition (%)			
	Sand	Silt	Coarse Clay	Fine Clay
Limestone	13.6	23.1	12.4	50.9
Recycled Concrete	76.9	14.2	5.2	3.7

X-Ray Diffraction

Tables 5 and 6 list the mineralogical components in each size fraction identified from XRD patterns. Among these, smectite is the most reactive constituent, with a specific surface area of approximately 3,906,000 ft^2/lb. Kaolinite, in comparison, has a specific surface area of only 98,600 ft^2/lb. Smectite is additionally characterized by high CEC due to its high negative surface charge and is a significant cause of suction in aggregate base materials. Some studies have linked the presence of smectite, even in small quantities, to the poor performance of these materials in the field (6). In order to determine the amount of each of these constituents in the aggregate residuals, researchers performed further testing as described below.

Table 5. XRD Results for Limestone.

Fraction	Mineral Components
Sand	Quartz, feldspar
Silt	Quartz, feldspar, kaolinite
Coarse Clay	Quartz, kaolinite, mica, smectite, goethite
Fine Clay	Kaolinite, mica, smectite, goethite

Table 6. XRD Results for Recycled Concrete.

Fraction	Mineral Components
Sand	Quartz, feldspar
Silt	Quartz, feldspar
Coarse Clay	Quartz, kaolinite, mica, smectite
Fine Clay	Kaolinite, mica, smectite

Total Potassium Determination

Two samples of coarse clay and two samples of fine clay were analyzed to determine the average percentage of mica in each fraction reported in Table 7 (33). The average K concentration provided a basis for directly estimating the mica content, given the absence of potassium feldspars in the coarse and fine clay samples.

Table 7. Results of Total Potassium Determination.

Aggregate	Mica (%)			
	Coarse Clay		Fine Clay	
	Mean	Std. Dev.	Mean	Std. Dev.
Limestone	13.86	3.29	16.91	2.35
Recycled Concrete	16.30	0.65	13.60	0.41

Cation Exchange Capacity Determination

The CEC was also determined for the coarse and fine clay fractions, as shown in Table 8 (33). The CEC values for kaolinite, mica, and smectite in later data synthesis calculations were assumed to be 0.1102 mol/lb., 0.2205 mol/lb., and 2.4251 mol/lb., respectively (4).

Table 8. Results of Cation Exchange Capacity Determination.

Aggregate	Cation Exchange Capacity (mol/lb.)			
	Coarse Clay		Fine Clay	
	Mean	Std. Dev.	Mean	Std. Dev.
Limestone	0.3874	0.0028	1.5043	0.0131
Recycled Concrete	1.2025	0.0029	1.2874	0.0037

Scanning Electron Microscopy

SEM work was completed on the bulk limestone sample as well as on the silt fraction obtained from fractionation of the limestone residual. Figure 12 shows the morphology of the calcite particles comprising about 95 percent of the bulk limestone sample. This figure illustrates the aggregated nature of the limestone and suggests an unusually high surface area compared to calcite in general *(34)*.

Figures 13 through 17 are from the silt fraction of the limestone sample. The oval-shaped particles in Figure 13 may be opal, even though the presence of opal was not observed in the XRD patterns. The aggregated particles to the upper and lower right are quartz. Figure 14 shows a distinctly hexagonal kaolinite particle adhered to a quartz particle. The conchoidal fracture at the left of the photo is typical of quartz. The main feature of Figure 15 is a kaolinite vermiform, with quartz in the background. Figure 16 displays the quartz morphology typical of this sample, and Figure 17 depicts an iron oxide, which is probably goethite. The fossil structure likely formed by small scale, progressive substitution of goethite for the parent material.

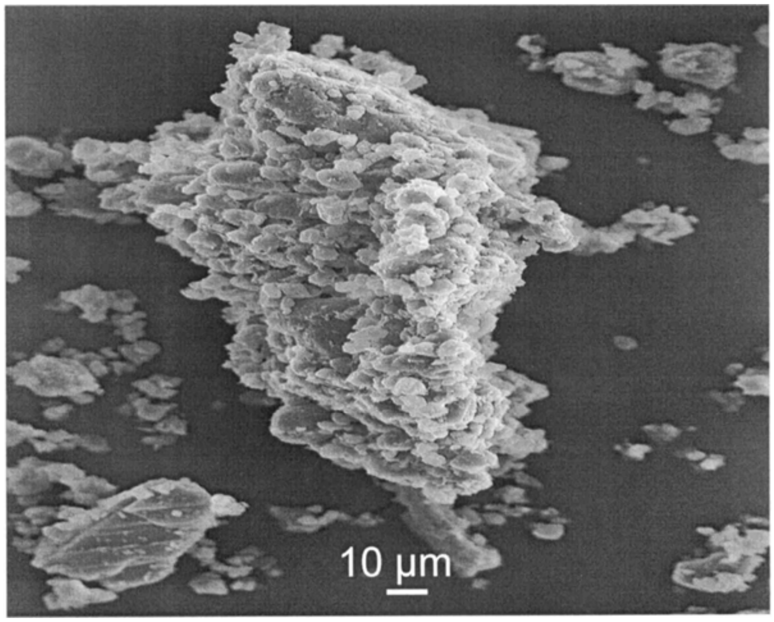

Figure 12. Aggregated Calcite Particles.

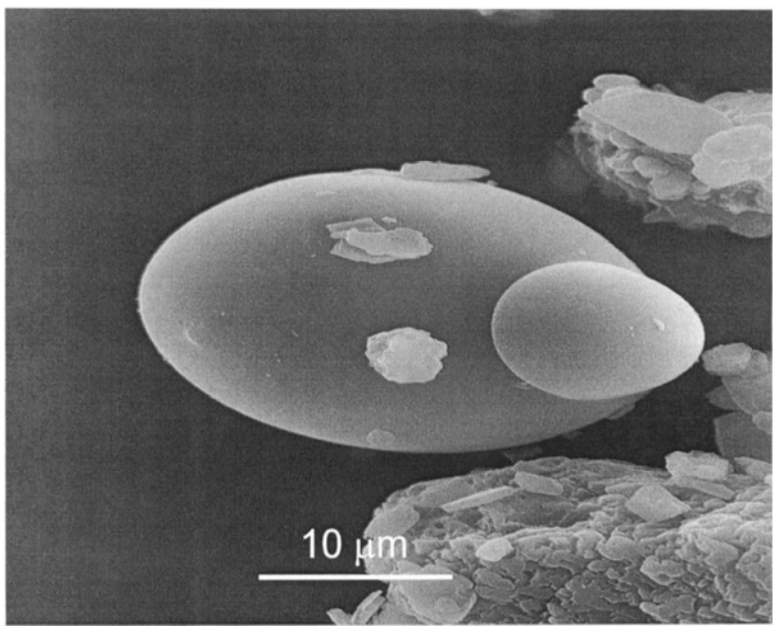

Figure 13. Opal and Quartz Particles.

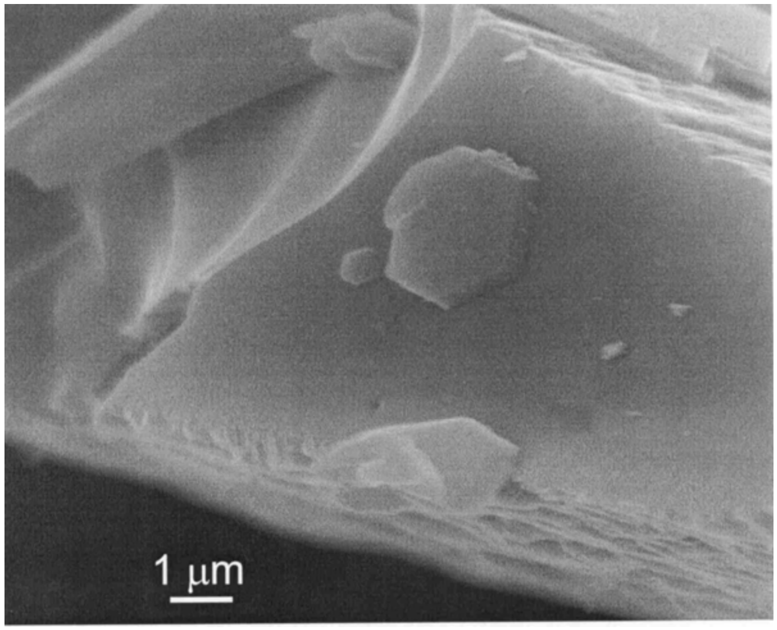

Figure 14. Conchoidal Quartz Fracture.

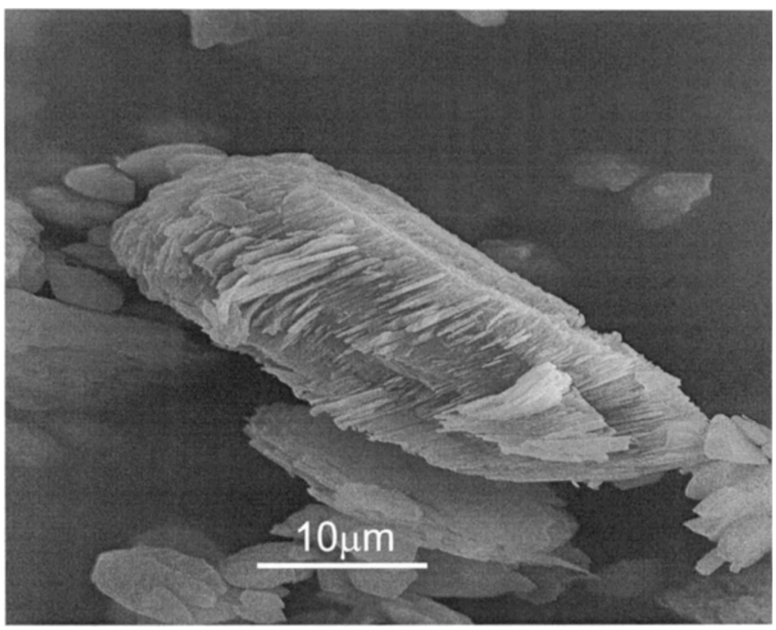

Figure 15. Kaolinite Vermiform.

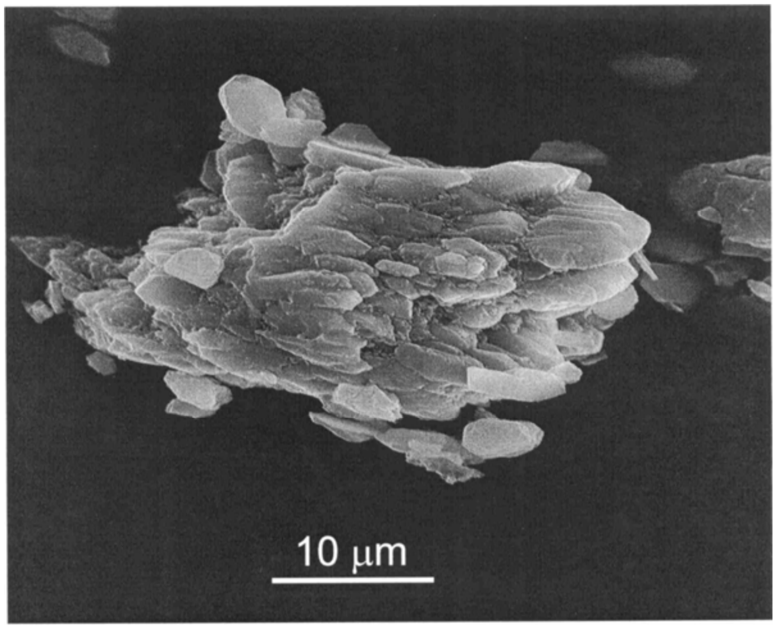

Figure 16. Quartz Morphology.

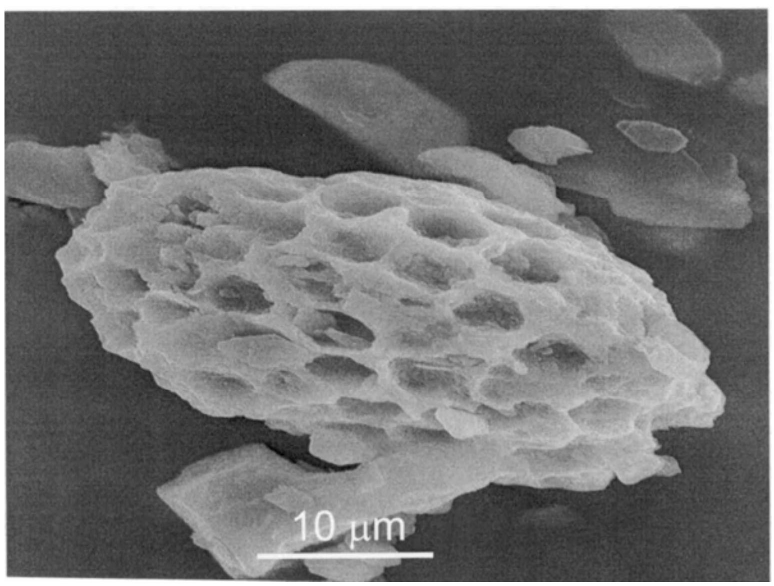

Figure 17. Iron Oxide Fossil.

Data Synthesis

The percentages of quartz, kaolinite, mica, and smectite in each aggregate residual were computed from composite CEC measurements, the percentage of mica calculated from total K determinations, and the ratio of quartz to kaolinite determined from the relative sizes of the respective peaks measured with XRD. Tables 9 and 10 present the resulting quantification of mineral constituents for the aggregate residuals *(33)*.

The smectitic composition of the clay fractions in both the limestone and recycled concrete residuals was determined to be the culprit in the poor performance of these materials in their untreated conditions. Based on these data, the bulk limestone aggregate was composed of 1.35 percent smectite by weight, while the bulk recycled concrete contained 3.35 percent smectite. The percentages of smectite for the two aggregate samples cannot be compared directly, however, because the samples were not prepared alike. As explained in Chapter 3, the recycled concrete sample was scalped on the No. 10 sieve before the mineralogical testing was performed, while the limestone sample contained portions from all particle sizes representative of the complete gradation. Thus, inferences about the total smectite content of the recycled concrete aggregate cannot be made with certainty. However, higher amounts of smectite in the recycled concrete than in the limestone would correspond well with the higher dielectric value of the recycled concrete aggregate in the TST, as reported in Table 2.

Table 9. Composition of Limestone Clay Fractions.

Clay Fraction	Composition (%)			
	Quartz	Kaolinite	Mica	Smectite
Coarse	25.3	48.4	13.8	12.5
Fine	0.0	30.6	16.9	52.5

Table 10. Composition of Recycled Concrete Clay Fractions.

Clay Fraction	Composition (%)			
	Quartz	Kaolinite	Mica	Smectite
Coarse	17.1	20.9	16.3	45.7
Fine	0.0	37.5	13.6	48.9

STRENGTH TEST RESULTS

Unconfined compressive strengths were measured for both aggregates at seven-day and 28-day cures and in a soaked condition, as shown in Tables 11 and 12. One sample of recycled concrete was tested in each condition. Two samples of limestone were tested in each condition at seven-day and 28-day cures, and four limestone samples were tested in the soaked condition.

While the recycled concrete aggregate samples were allowed to soak underwater for 10 days before being tested, the limestone samples remained underwater for 12 weeks, due to a delay in the repair of needed testing equipment. Thus, in the case of the latter, the otherwise detrimental effects of moisture on strength were presumably overcome by additional cement hydration and corresponding strength gain.

On the other hand, the recycled concrete sample was not soaked sufficiently long for the effects of additional hydration to overcome the impact of a nearly saturated moisture condition, and, at 4.5 percent cement, the soaked sample strength was less than 50 percent of the 28-day cured sample strength. However, the recycled concrete samples treated with 1.5 and 3.0 percent cement had soaked strengths greater than 80 percent of the 28-day cured strength and so passed the specification. At 1.5 percent cement, one of the two limestone specimens had an unconfined compressive strength less than the proposed seven-day minimum of 300 psi, suggesting that this aggregate should be stabilized at a higher cement level. The strength of the recycled concrete at 1.5 percent cement was nearly twice that of the limestone and passed the specification. For both aggregates, samples treated with 3.0 and 4.5 percent cement exhibited seven-day strengths much greater than the minimum target value of 300 psi.

Table 11. Strength Test Results for Limestone.

Cement Level (%)	Unconfined Compressive Strength (psi)					
	7-day		28-day		Soaked	
	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.
1.5	303	18	393	68	529	31
3.0	523	28	768	86	1103	113
4.5	861	50	1296	273	1465	174

Table 12. Strength Test Results for Recycled Concrete.

Cement Level (%)	Unconfined Compressive Strength (psi)		
	7-day	28-day	Soaked
1.5	596	409	433
3.0	653	708	653
4.5	822	865	425

SHRINKAGE TEST RESULTS

The environmental conditions for the shrinkage tests performed in this project were different for the limestone and recycled concrete aggregates. For the limestone, samples were tested at less than 50 percent relative humidity, with total shrinkage results given in Table 13. The recycled concrete samples were monitored in an environmental chamber maintained at 100 percent relative humidity, during which time the samples were conditioned for subsequent durability testing as mentioned earlier.

While an overall retarding effect of these humid conditions on shrinkage was a possibility, an unexpected shrinkage trend presented in Table 14 resulted from testing the recycled concrete aggregate, where a minimum value was obtained at a 3.0 percent cement level. Cement levels of 1.5 and 4.5 percent incurred shrinkage in excess of the specified maximum of 0.000250 in./in. Test results for replicate samples would have provided the basis for a more reliable explanation of these observations, but instrumentation and measurement difficulties and variations in material constituents between the samples could also have adversely influenced the testing.

The limestone samples experienced shrinkage greater than the specified maximum limit in all cases, with shrinkage strain increasing with higher levels of cement treatment. This finding suggests that the Australian specification limit proposed for judging shrinkage characteristics of cement-treated materials deserves further investigation in Texas, as the importance of simulating the environment to which the aggregate base material will be exposed cannot be over-emphasized in shrinkage testing. In addition to the difficulty of determining reasonable climatic conditions for the test, the separate roles of drying and self-desiccation should also be considered in the performance of materials in shrinkage tests. Therefore, further research is needed to evaluate the sensitivity of this test to these variables.

Table 13. Shrinkage Test Results for Limestone.

Cement Level (%)	Shrinkage Strain (in./in.)
1.5	0.000267
3.0	0.000289
4.5	0.000333

Table 14. Shrinkage Test Results for Recycled Concrete.

Cement Level (%)	Shrinkage Strain (in./in.)
1.5	0.000933
3.0	0.000111
4.5	0.000556

DURABILITY TEST RESULTS

The evaluation of durability in the SAWTET yielded the results given in Tables 15 and 16 for the limestone and recycled concrete, respectively. All samples passed the specification of less than 0.04 in. of erosion, but high variability is evident from the standard deviations reported in Table 15. In fact, for the limestone stabilized at the 1.5 percent cement level, the sum of the mean and one standard deviation just exceeds the allowable maximum erosion. Thus, as also elucidated in the unconfined compressive strength testing, a higher cement level such as 3.0 percent should be utilized for stabilizing this aggregate.

While the limestone samples exhibited the expected trend of decreasing erosion with higher levels of cement treatment, the performance of the recycled concrete did not yield this trend. In fact, probably because the set of recycled concrete beams was the same for the shrinkage and durability evaluations, the results are similar for the two tests, where a minimum value of erosion was measured at the 3.0 percent level. This particular sample may have been compacted to a higher density, for example, than the other beam specimens to bring about this unexpected performance. Like the shrinkage test, the SAWTET requires further evaluation before its use is recommended.

Table 15. Durability Test Results for Limestone.

Cement Level (%)	Erosion (in.)	
	Mean	Std. Dev.
1.5	0.025	0.016
3.0	0.015	0.008
4.5	0.011	0.006

Table 16. Durability Test Results for Recycled Concrete.

Cement Level (%)	Erosion (in.)
1.5	0.020
3.0	0.017
4.5	0.029

MOISTURE SUSCEPTIBILITY TEST RESULTS

TST results in Tables 17 and 18 for the limestone and recycled concrete aggregates suggest that 1.5 percent cement is adequate to impede moisture ingress and migration. With all final dielectric values less than 10, cement was successful in raising the moisture susceptibility rating for both aggregates from "poor" to "good." In this test, four samples of limestone at each cement level and one sample of recycled concrete were evaluated at each cement level.

While the average dielectric value of the limestone samples treated with 1.5 percent cement was sufficiently reduced to achieve a "good" moisture susceptibility rating, the average water intake was still more than 4 percent, equivalent to the average amount of water imbibed during testing of the untreated limestone samples. The difference in dielectric values arose from different moisture profiles, where the treated samples exhibited a strong moisture gradient that prohibited detection of unbound moisture near the sample surface. That is, the moisture remained concentrated in the lower portions of the treated samples while, conversely, the untreated limestone samples developed a uniform moisture profile in which higher amounts of water reached the surface. With special consideration given to this issue of moisture ingress at the 1.5 percent cement level, stabilization of the limestone at a higher cement level should be considered. The water intake at 3.0 and 4.5 percent cement was only 2.8 and 2.2 percent, respectively, for the limestone. The recycled concrete wicked in approximately 2 percent moisture in all cases, about half the amount imbibed in its untreated condition.

Table 17. Moisture Susceptibility Test Results for Limestone.

Cement Level (%)	Beginning Moisture (%)		Ending Moisture (%)		Dielectric Value	
	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.
1.5	2.5	0.7	6.8	0.5	6.6	1.3
3.0	1.8	0.1	4.6	0.1	5.5	0.7
4.5	1.9	0.4	4.1	0.5	5.5	0.7

Table 18. Moisture Susceptibility Test Results for Recycled Concrete.

Cement Level (%)	Beginning Moisture (%)	Ending Moisture (%)	Dielectric Value
1.5	6.6	8.7	5.9
3.0	6.9	9.3	6.1
4.5	7.0	9.3	5.8

MODULUS TEST RESULTS

Researchers performed modulus tests on the limestone samples according to the Standard Method of Test for Resilient Modulus of Subgrade Soils and Untreated Base/Subbase Materials (AASHTO T 292-91). One sample of each cement level was tested under unconfined stress conditions, and axial deformation was measured across the full sample height and across a 6-in. gauge length centered at the sample midsection. The results are given in Tables 19 through 21. For comparisons of resilient modulus for the 6-in. and 8-in. gauge lengths at a deviator stress of 100 psi, the former is higher than the latter by factors of 6, 13, and 20 at cement levels of 1.5, 3.0, and 4.5 percent, respectively. The disparity may be attributable to end effects, suggesting that the compaction and deformation characteristics of the ends vary considerably from the matrix within the midsection of a given sample. This aspect of resilient modulus testing has also been documented in other research *(35, 36)*.

In the tables, the resilient modulus increases with increasing deviatoric stress at the 1.5 percent cement level, but decreases with increasing deviatoric stress at the 3.0 and 4.5 percent cement levels. Because only one sample was tested at each cement level, the validity of this behavior cannot be ascertained without additional testing. Thus, no explanation is attempted in this report.

Four samples were tested at each cement level with the free-free resonant column equipment, and the results are shown in Table 22. Young's modulus and the shear modulus increase proportionally, related through Poisson's ratio in the equations of elastic behavior. The implications of these parameters on materials quality and expected pavement performance are undergoing investigation at the University of Texas at El Paso.

Table 19. Resilient Modulus Test Results for 1.5 Percent Cement Level.

Deviator Stress (psi)	Resilient Modulus (psi)	
	6-in. Gauge Length	8-in. Gauge Length
30	1,278,347	138,905
40	1,339,133	165,576
50	1,374,103	184,317
80	1,432,510	222,264
100	1,454,600	243,695

Table 20. Resilient Modulus Test Results for 3.0 Percent Cement Level.

Deviator Stress (psi)	Resilient Modulus (psi)	
	6-in. Gauge Length	8-in. Gauge Length
30	5,905,742	191,247
40	5,289,583	220,215
50	5,691,395	243,858
80	4,245,658	296,590
100	4,215,800	334,384
120	4,021,136	374,041

Table 21. Resilient Modulus Test Results for 4.5 Percent Cement Level.

Deviator Stress (psi)	Resilient Modulus (psi)	
	6-in. Gauge Length	8-in. Gauge Length
30	24,648,368	227,464
40	20,401,046	260,882
50	16,041,472	310,838
80	11,078,611	381,522
100	8,694,884	441,508
120	7,251,854	500,861
140	6,675,368	545,118

Table 22. Seismic Modulus Test Results.

Cement Level (%)	Young's Modulus (psi)		Shear Modulus (psi)		Poisson's Ratio	
	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.
1.5	1,071,345	33,631	207,538	21,998	0.35	0.02
3.0	1,710,546	58,324	414,739	13,956	0.34	0.00
4.5	2,176,937	42,267	526,389	10,485	0.34	0.00

CHAPTER 5. CONCLUSION

SUMMARY

The task of selecting the type and amount of stabilizer in upgrading a marginal base material requires evaluation of multiple factors. Strength-based design procedures, which require a minimum unconfined compressive strength after a specified curing time, frequently result in very stiff bases that shrink and crack. Every district within the Texas Department of Transportation can identify over-stabilized projects that have performed poorly. The trend in recent years has been to lower strength requirements, thus reducing the amount of stabilizer required. However, several case studies have shown that in many instances the stabilizer disappeared after only a few years in service, suggesting that a higher percentage should have been used. The optimum stabilizer content for an aggregate must provide sufficient strength for carrying the imposed traffic loads, as well as provide adequate durability so that its properties are not severely impacted by environmental effects.

In this project, researchers designed a laboratory test sequence to identify the optimum amounts of portland type I cement for stabilizing two aggregates, limestone and recycled concrete, typically used in the Houston District. The mineralogy of each material was investigated, and the performance of the aggregates in their untreated condition was documented to study the impacts of three levels of cement stabilization. Samples treated with 1.5, 3.0, and 4.5 percent cement were tested for strength, shrinkage, durability, and moisture susceptibility in the laboratory, and performance characteristics were compared to specifications proposed for each test.

Strength was determined with the Soil Cement Compressive Strength Test (TxDOT Test Method Tex-120-E), and a linear shrinkage test was developed to assess shrinkage characteristics. Durability was evaluated in the SAWTET, and moisture susceptibility was assessed in the TST. Modulus values for the limestone were obtained through the Standard Method of Test for Resilient Modulus of Subgrade Soils and Untreated Base/Subbase Materials (AASHTO T 292-91) and the free-free resonant column method developed at the University of Texas at El Paso.

FINDINGS

Both the limestone and recycled concrete aggregates were comprised of some percentage of smectite, which, even in small quantities, can govern the overall behavior of these materials. Preliminary moisture susceptibility testing of the aggregates in their untreated condition yielded poor performance, requiring stabilization of the aggregates to achieve acceptable properties necessary for use as base materials. Unconfined compressive strength testing following the TST showed that only the recycled concrete met the requirement of 45 psi for the Triaxial Class 1 specification in its untreated condition.

The required test specifications for strength, durability, and moisture susceptibility were satisfied with the addition of 3.0 percent cement for the limestone and 1.5 percent cement for the recycled concrete. For the limestone stabilized at 1.5 percent, samples yielded unconfined compressive strengths straddling the specified minimum value, and large variability in the SAWTET data resulted in the inclusion of the specified maximum allowable erosion value within one standard deviation above the mean. Furthermore, while the dielectric value of limestone specimens stabilized at the 1.5 percent cement level was adequately reduced, the moisture ingress was not significantly impeded compared to the moisture ingress experienced by untreated limestone samples. Specifications for maximum allowable shrinkage were exceeded by both aggregates at all cement levels, however, with one exception discussed in Chapter 4 that could have been due to unusually high sample density. Overall, instrumentation and measurement difficulties and possible variations in material constituents produced relatively high variability in results obtained in both the shrinkage and durability tests.

Modulus values obtained with the Standard Method of Test for Resilient Modulus of Subgrade Soils and Untreated Base/Subbase Materials (AASHTO T 292-91) were apparently influenced by end effects, where the modulus values computed from 6-in. gauge lengths were 6, 13, and 20 times higher than those computed from measurements over 8-in. gauge lengths at cement levels of 1.5, 3.0, and 4.5 percent, respectively. The seismic method was also used to measure Young's modulus and the shear modulus of elasticity for the limestone samples, the application of which is subject to ongoing research at the University of Texas at El Paso.

RECOMMENDATIONS

The recommendations of this report address selection of optimum cement contents for the limestone and recycled concrete evaluated in this project, laboratory procedures proposed for design of cement-treated materials, and a field measure for inhibiting the development of reflection cracking in surface layers placed over cement-treated bases.

Selection of Optimum Cement Contents

Based on the results of this project, 3.0 percent cement should be used for stabilizing the limestone aggregate, and 1.5 percent cement should be adequate for the recycled concrete. This selection satisfies the specifications required for unconfined compressive strength, durability, and moisture susceptibility testing. These recommendations are based on laboratory data presented in this report and should be valid as long as variations in mineralogy, gradations, or other important physical or chemical properties are not significant between samples tested in this project and aggregates proposed for use in actual highway construction.

Proposed Laboratory Procedures

For future testing of aggregate base materials to determine optimum cement contents, the joint utilization of the Soil Cement Compressive Strength Test (TxDOT Test Method Tex-120-E) and the TST is recommended. Sufficient quantities of cement should be added to tested samples to obtain minimum seven-day unconfined compressive strengths of 300 psi in the former and maximum average surface dielectric values of 10 in the latter. The minimum amount of cement necessary to satisfy both criteria should be recommended for pavement construction. For reasons cited earlier, this report does not at this time recommend the use of the linear shrinkage test or the SAWTET used in this project.

Proposed Field Procedure

As discussed earlier in this report, pre-cracking is a viable method for reducing reflection cracking through surface layers placed over cement-treated bases. In conjunction with proper laboratory testing conducted to determine optimum cement contents as described above, pre-cracking can be used in the field during construction to enhance the performance of cement-treated base layers. Pre-cracking of cement-treated materials should occur within one to three days after placement, where heavy traffic or vibratory rollers can be used to create networks of microcracks within the base layer that eliminate the development of large shrinkage cracks. As part of a separate project, a provisional pre-cracking specification was developed based on experimental techniques employed during new construction of a residential subdivision in College Station, Texas. Several issues potentially affecting this proposed specification, which is given in the appendix of this report, are recommended for further evaluation. Additional research is needed to determine the necessity of potential adjustments in the methodology to accommodate construction of base layers of different thicknesses and varying unconfined compressive strengths. The effects of ambient temperature on the curing of cement-treated materials and the subsequent efficacy of pre-cracking should also be investigated for different seasons.

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APPENDIX:

PROVISIONAL PRE-CRACKING SPECIFICATION

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Significance and Use

The pre-cracking methodology presented in this section resulted from new construction of a residential subdivision in College Station, Texas (*18*). This specification is based on limited data obtained during experimental construction of base layers treated with 6 to 8 percent cement. Construction occurred in the fall with an air temperature ranging between 75 °F and 80 °F. Further research is needed to evaluate possible adjustments to this specification necessitated by different layer thicknesses, lower unconfined compressive strengths, or construction during seasons substantially colder or warmer than conditions present in this project.

Procedures

After compaction, the finished cement-treated base is maintained continuously moist for between 24 and 48 hours. The stiffness of the base is then determined by the contractor with an approved device, such as a Humboldt stiffness gauge or a falling-weight deflectometer (FWD), where measurements are taken at marked 100-ft intervals, as shown in Figure 18. The finished base course is then subjected to two passes of a 12-ton steel-wheel vibratory roller, traveling at a speed of approximately 2 mph and vibrating at maximum amplitude. The section should receive 100 percent coverage exclusive of the outside 1 ft so as to induce minute cracks in the treated base layer while avoiding damage to installed gutters or unconfined shoulders. Figure 19 depicts this operation.

After these first passes, the stiffness is again measured at the same test points, and the section is inspected. Additional passes may be required to achieve the desired crack pattern or section modulus as directed by the project engineer, but rolling should be terminated when the average base stiffness has decreased by 40 percent. (For 6-in. layers designed to have seven-day unconfined compressive strengths of 500 psi according to TxDOT specification 272, an additional two passes are usually necessary.) Figure 20 shows the development of networks of microcracks in the surface of a pre-cracked layer. After completion of the pre-cracking operation, the section is moist-cured for a period of 48 hours.

Figure 18. Using the Humboldt Stiffness Gauge (*18*).

Figure 19. Pre-cracking Operations with Vibratory Roller (*18*).

Figure 20. Microcracks in Cement-Treated Base Layer (*18*).