BENEFICIAL USE OF SCRAP TIRE RUBBER IN LOW-VOLUME ROAD AND BRIDGE CONSTRUCTION WITH EXPANSIVE SOILS

(USDOT-MPC/274 INTERIM REPORT)

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Abstract

(In Preparation) 200 words

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Executive Summary

(In Preparation)

1. INTRODUCTION

1.1 Background

Billions of dollars in damages are attributed to expansive soils in the United States (Jones and Jones 1987) and in many other countries each year. Geotechnical design and analyses in/on/with expansive soils may involve additional complications that otherwise would not have to be dealt with if expansive soils were not present. The term expansive soil can be used to define soils that undergo a significant amount of volume change as a result of changes in the water content of the soil (Nelson and Miller 1992). While deposits of expansive soils exist throughout the United States, in areas where expansive soils predominate, mitigation efforts usually focus on reducing the swell potential of the soil to eliminate the detrimental effects due to excessive heave and swell pressures on typical engineering structures such as road pavement layers, building foundations, and retaining structures. Detailed descriptions of expansive soil stabilization techniques are discussed in the literature (Jones and Jones 1987; Nelson and Miller 1992; Petry and Little 2002; Katti et al. 2002). In general, the proposed stabilization techniques fall into one of the following categories: (1) chemical stabilization, or (2) physical stabilization.

Traditional methods for chemical stabilization of expansive soils include the addition of lime, class-C or class-F fly ash, Portland cement, or other industrial byproducts such as cement kiln dust, steel or copper slag (Basma et. al. 1998; Cokca 2001; Al-Rawas et. al. 2002). An important issue associated with the addition of lime, class-C fly ash, Portland cement, or other chemical stabilizers containing high concentrations of calcium occurs when the additives are mixed with soils that are rich in soluble sulfates. When high-calcium content additives are mixed with sulfate-rich soils, a compound known as ettringite is usually formed (Puppala et al. 2005). Formation of ettringite results in a soil-stabilizer mixture that may be even more expansive than the natural expansive soil in its original state, before addition of the chemical stabilizers (Hunter 1988). Hence, in soils containing soluble sulfates, physical stabilization techniques may be more appropriate than chemical stabilization methods.

Physical stabilization techniques aim at reducing the potential swell pressure and swell percent of the expansive soil without altering the soil chemistry. Typical physical stabilization techniques described in the literature include: (1) pre-wetting of the soil to achieve maximum swell prior to construction, (2) reduction of the compaction effort and compacted density of the soil, or (3) removal of the natural expansive soil followed by full or partial replacement with nonexpansive material (Mowafy et. al. 1985; Jones and Jones 1987; Basma et. al. 1998; Petry and Little 2002; Katti et al. 2002; Hudyma et. al. 2006). In this study, a new environmentally-friendly technique for physical stabilization of expansive soils is proposed that focuses on reducing both the swell percent and the swell pressure of an expansive soil by mixing it with scrap tire rubber (STR). The purpose of this study is to evaluate the effectiveness of adding STR on the mitigation of the swell potential of an expansive soil from Colorado in low-volume road and bridge construction applications.

1.2 Project Objectives

The objectives of this project are to (1) evaluate the geotechnical properties of expansive soil-rubber (ESR) mixtures required for the mechanistic design of low-volume road embankments and bridge abutments, (2) design a typical low-volume road embankment cross section using a computer model, (3) construct a pilot road section and monitor its performance to demonstrate the feasibility of the proposed technology, and (4) disseminate the project findings through the publication of technical papers and by holding a one-day workshop involving state, county, and city engineers as well as students and professionals interested in expansive soil stabilization.

2. LITERATURE REVIEW

2.1 Mechanical Stabilization of Soils by Admixing of Nonexpansive Geomaterials

2.1.1 Swell Potential Mitigation by Sand Admixing

Few articles have been published in the literature, to date, that address the swell potential mitigation and the reduction of the swell percent of expansive soils through sand addition. Mowafy et al. (1985) evaluated the impact of (i) adding various amounts of sand to the soil, (ii) varying the soil water content, and (iii) altering the soluble salt concentrations on the swell percent and swell pressure of an expansive soil collected near Cairo, Egypt. They found that increasing the sand content of the expansive clay-sand mixture reduced the swell percent and the swell pressure of the mixture. Addition of 20 to 25% of sand (either by mass or volume – since the specific gravity of the clay and sand tested are 2.70 and 2.71, respectively) reduced the swell percent of the expansive clay-sand mixture by 50%. Addition of 40% of sand reduced the swell pressure from 5.2 MPa to 1.4 MPa. Basma et al. (1998) studied the effect of using sand, lime, Portland cement, and pore water salts on the stabilization of an expansive soil from Al-Khod, Oman. Results from their study showed that increasing the sand content of the expansive soil-sand mixture to 52% reduced both the swell percent and the swell pressure of the mixture from 8.4% and 182 kPa to 4% and 110 kPa, respectively. Hudyma and Avar (2006) showed that the addition of 20 to 30% (the authors did not specify whether sand addition was based on a mass or volume percentage) of sand to two clays from southern Nevada reduced the swell percent of the expansive clay-sand mixture by 50%. However, addition of 20 to 30% of sand reduced the swell pressure of the mixture by 13%. Each of these studies demonstrates that addition of a nonexpansive geomaterial to expansive soils may be an effective way to mitigate the swell potential of the natural expansive soil. However, the overall costs associated with admixing nonexpansive geomaterials (such as sands or gravels) to naturally-occurring expansive soils may not be an economical alternative in some cases, particularly in areas where natural, nonexpanisve geomaterials are scarce and transportation costs are high. Perhaps as a result of these limitations, no field tests or implementation studies that look at evaluating the effectiveness of physical dilution of expansive soils with nonexpansive geomaterials have been found in the literature.

2.1.2 Soil Stabilization by Scrap Tire Rubber Admixing

A viable and sustainable alternative to the admixing of expansive soils with traditional nonexpansive geomaterials such as clean sands and gravels is evaluated in this study which addresses the beneficial use of STR to mitigate the swell potential of expansive soils. The addition of STR to expansive soils may not only provide a low cost alternative to the addition of conventional nonexpansive geomaterials but also encompasses an environmentally-friendly solution to these widely-generated waste materials.

Scrap tires are encountered all over the world in increasing numbers. One of the main issues associated with the management of scrap tires has been their proper disposal. The United States Environmental Protection Agency (EPA) reports that approximately 290 million scrap tires are generated annually within the United States (EPA 2003). Around 80% of these tires are being beneficially used or recycled (RMA 2004) in the various markets currently available for scrap tires, tire shreds, tire chips, and granulated rubber. The State of Colorado has been identified as one of the states with the largest number of stockpiled scrap tires within its EPA region (EPA Region 8). Since scrap tires and STR products are not biodegradable, the required amount of landfill space along with the health and environmental threats related to scrap tire disposal provide ample motivation to find new uses for these waste materials.

Scrap tires and scrap tire products such as tire bales, tire shreds, tire chips, and granulated rubber have been used in a variety of engineering applications including highway subgrades, embankments, backfills, asphalt mixture designs, leach fields or as erosion control and sorptive media (Ahmed and Lovell 1993; Upton and Machan 1993; Newcomb and Drescher 1994; Kershaw and Pamukcu 1997; Edil 2005; Zornberg et al. 2005; Ashmanwy et al. 2006). Some of the desirable properties of STR in backfill and embankment applications are due to the light weight of rubber (Edil and Bosscher 1994). The dampening characteristics of STR also have been used to advantage in railroad track beds to minimize disturbance to nearby residents (Feng and Sutter 2000). Although many STR products are currently available for beneficial use, emphasis is placed in this study on the beneficial use of STR products that are relatively smaller in size such as granulated rubber. According to ASTM D6270 "Standard Practice for Use of Scrap Tires in Civil Engineering Applications," granulated rubber is defined as a scrap tire product that contains rubber particles smaller than 12 mm.

The Colorado Department of Local Affairs lists a number of Colorado scrap tire vendors on its website (http://www.dola.state.co.us). Vendors located along the Front Range corridor that sell STR products with nominal maximum particle size ranging from 4.8 mm to 50 mm include:

- a. Snowy River Tire Recycling, 4450 Mulligan Dr., Longmont, CO 80504
- b. Academy Sports Turf, 3740 S. Jason Street, Englewood, CO 80110
- c. Jai Tire, Inc., 5050 Colorado Blvd., Denver, CO 80216
- d. Midway Tire Disposal/Recycling, Inc., P.O. Box 352, Fountain, CO 80817
- e. North West Rubber CO, Inc., 7623 N. Lavaun Drive, Louviers, CO 80131
- f. Front Range Tire Recycle, P.O. Box 184, Sedalia, CO 80135-0184
- g. Imagination Playgrounds and Safety Surface, 3407 E. 115th Drive, Thornton, CO 80233
- h. Tire Mountain Inc., 12311 Weld County Road 41, Hudson, CO 80642

Figure 2.1 shows the locations of each of the STR suppliers listed above.



Figure 2.1 Scrap Tire Rubber Suppliers along the Front Range Corridor in Colorado (http://www.mapquest.com).

The price of STR materials varies significantly with particle size ranging from approximately \$20 per ton for 25.4-mm tire chips (Front Range Tire Recycle, Sedalia, CO) to \$450 per ton (Jai Tire, Denver, CO) for the 4.8-mm and 6.7-mm particle sizes used in the laboratory evaluation of ESR mixtures contained in this report. This price range does not include delivery.

2.1.2.1 Soil-Rubber Applications

Since earthwork construction typically requires large volumes of geomaterials, the beneficial use of STR products in these high-volume projects may significantly reduce the number of tires originally destined to be disposed of in landfills and reduce the costs of reworking soils (Tatlisoz et al. 1997). Some concerns associated with the use of STR products, which are due to the deformability of STR products as well as the potential development of spontaneous exothermic reactions, have been addressed in recent technical publications. Edil and Bosscher (1994) showed that the inclusion of as low as 30% of sand (by weight) in sand-tire chip mixtures restored the compressibility of the mixtures to a level comparable to that of pure sand. ASTM D6270 "Standard Practice for Use of Scrap Tires in Civil Engineering Applications" limits the thickness of STR layers to 3 m in road construction applications to minimize the potential for fire hazards.

The types of geomaterials most commonly used in previous studies involving soil-rubber mixtures are clean sands, silts, and low-plasticity clays. While some studies have proposed the use of STR in embankments over deposits of soft soils (Ahmed and Lovell 1993; Humphrey et al. 1993; Newcomb and Drescher 1994; RMA 2004), other studies have focused on the effect of STR on the shear strength, compressibility, and permeability of soil-rubber mixtures (Ahmed and Lovell 1993; Humphrey et al. 1993; Edil and Bosscher 1994; Newcomb and Drescher 1994; Tatlisoz et al. 1997; Lee et al. 1999; Edil 2005). Articles pertaining to the use of STR to mitigate the swell potential of expansive clays have not been found in the literature with the only exception of the preliminary work carried out by the PI's research group (Seda et al. 2007), which indicated that both the swell percent and swell pressure of an expansive soil from Berthoud, CO, could be reduced by STR admixing.

2.1.2.2 Soil-Rubber Field Construction

The literature available on field construction of mixtures of soil with tire shreds or tire chips appears to be limited to embankment fills in highway construction (Siddiki et. al. 2004, Zornberg et. al. 2004, Yoon et. al. 2005). Siddiki et. al. (2004) constructed a 3-m high highway embankment using a 50:50 sand-tire shred mixture. Special consideration was taken to minimize infiltration of water through the sand-tire shred mixture to eliminate the potential for groundwater contamination. Construction using a 50:50 sand to tire shred ratio was chosen to eliminate the potential for exothermic reaction in the tire shreds (ASTM D6270). Zornberg et. al. (2004) evaluated three embankment sections in Colorado. Their first embankment section was constructed using native soil (silty sand with SM classification and standard Proctor maximum dry unit weight and optimum water content of 18.6 kN/m³ and 12.6%, respectively), the second consisted of successive layers of soil and tire shreds, and the third section was constructed using 10% (by weight) of tire shreds mixed with the native soil. Construction of the embankment sections evaluated six methods for mixing soil and rubber, and a 6.7-ton (6078-kg) sheepsfoot roller was used to compact the embankment lifts. The 15-cm lifts used were shown to have minimal increases in dry unit weight after two passes of the sheepsfoot roller for all three embankment lift types. The five mixing procedures evaluated for the soil-rubber mixture embankment construction are summarized in Table 2.1

Procedure	Used Equipment	Procedure	Comments
P1	One wheeled front- end loader with a bucket equipped with a straight blade edge (capacity 1.6 m ⁴).	a) The target volume of tire shreds was loaded and dumped on the ground; b) the target volume of soil was loaded and dumped over the tire shreds; c) both materials were mixed together with the loader.	Collecting the materials from the ground was difficult to control using this equipment. A significant amount of subgrade soil contaminated the backfill soil-tire shred mixture.
P2	One wheeled front- end loader, with bucket equipped with a tooth-set edge (capacity 1.6 m ³).	Same as Procedure P1.	The bucket equipped with a tooth set allowed better mixing than a bucket with straight edge. Contamination of the backfill soil-tire shred mixture was less than in Procedure P1.
P3	Two wheeled front- end loaders similar to that used in Procedure P1.	a) The target volume of tire shreds was dumped from bucket of loader 1 into bucket of loader 2; b) the target volume of soil was dumped from bucket of loader 1 into bucket of loader 2; c) the total volume was dumped into the bucket of loader 1. and then back to loader 2.	This method generated a better mixture and was faster than methods 1 and 2: however, this procedure required two pieces of equipment. Also, a considerable amount of mixture fell from the buckets during the mixing process.
P4	Two wheeled front- end loaders similar to those used in method 3 and a dump truck.	 a) The target volume of soil was loaded into the track with loader 1; b) the target volume of tire shreds was loaded into truck with loader 2; c) the loaders continued this process until all of the mixture had been placed into the dump truck; d) the mixture was dumped on the ground 	This method was faster than method 3, and also generated cleaner mixtures without losses. However, it mobilized three pieces of equipment.
P5	Wheeled front-end loaders used in Procedure P1 and a scruper.	 a) The loaders dumped a layer of tire shreds next to a layer of soil; b) the scraper ran across these layers in order to mix them. 	Materials were not mixed thoroughly using this procedure.

 Table 2.1 Mixing Procedures for Soil-Rubber Embankment Construction (Zornberg et al. 2004)

2.2 Mechanistic Pavement Design (In Preparation)

Pavement design evaluated three different potential road cross sections. Each of the cross sections was evaluated using the software PerROAD 3.2. The software was suggested by the City of Loveland, and is used frequently for the design of City of Loveland roads.

3. EXPERIMENTAL PROGRAM

3.1 Site Description

The field site from which the samples for the experimental program were collected is located along the East-West section of Byrd Drive, north of Crossroads Boulevard and west of I-25 in Loveland, CO. Figure 3.1 shows the existing site conditions prior to construction.



Figure 3.1 Field Site Prior to Construction with I-25 Traffic Shown in the Back (Photo is Facing East)

3.2 Field Sampling

Field sampling was conducted in early April 2007. Weather conditions during sampling were overcast, approximately 25° F, and snowing. Disturbed samples, two block samples, three ring samples, and water content samples were collected. The *in situ* water content ranged from 12.6% to 17.1%. The *in situ* dry unit weight, sampled from a depth of 0.6 m, was 15.8 kN/m³. The soil profile is fairly uniform down to a depth of 0.6 m, where a change in color is visible (Figure 3.2).

Two 1-m³ cubic blocks of soil were excavated using a rubber tired backhoe. Once the large blocks were excavated, the blocks were trimmed down to sampling size. The final blocks were 0.27 m by 0.3 m by 0.3 m. The trimmed block samples were then wrapped in cheese cloth and coated in paraffin wax (Figure 3.3). Three layers of cheese cloth and wax were applied. The block samples were packed into a wooden crate using sawdust packing material and transported back to the CSU geotechnical laboratory. Block sampling was done in accordance with ASTM D 70 15-04.



Figure 3.2 Test Pit Sidewall (South Wall of the Excavation)



Figure 3.3 Block Samples Prior to Removal

3.3 Materials

3.3.1 Soil

According to the USGS Geologic Map of the Boulder-Fort Collins-Greeley Area, Colorado, (Colton 2004) the soil at the Loveland Byrd Drive field site belongs to the Eolium formation which consists of light-brown to reddish-brown to olive-gray deposits of windblown clay, silt, sand and granules.



Figure 3.4 Loveland Soil (top), 19-mm (left), 6.7-mm (bottom) and 4.8-mm (right) STR

3.3.2 Scrap Tire Rubber Materials

Three sizes of STR materials were used in the experimental program: (1) the largest granulated rubber sample had a maximum particle size of 6.7-mm and specific gravity equal to 1.16; (2) the smallest granulated rubber sample had a maximum particle size of 4.8-mm and specific gravity equal to 1.17, and (3) the tire chip sample had a maximum particle size of 19-mm and specific gravity equal to 1.20. The granulated rubber samples used in the experimental program were provided by Jai Tire Industries (Denver, CO). The tire chips were provided by Front Range Tire Recycle (Sedalia, CO).

Edil and Bosscher (1994) showed that the compressibility of soil-rubber mixtures containing less than approximately 30% by volume of rubber tire chips is very similar to the compressibility of untreated soil (thus much less than the compressibility of pure tire chips). Therefore, in order to minimize potential issues related to the high compressibility of ESR mixtures (due to the addition of rubber to the expansive soil), the ESR mixtures tested in the present study consisted of and were limited to approximately 30% rubber by volume (or approximately 20% rubber by mass).

3.4 Experimental Methods

The experimental program was carried out on samples of the soil alone (used as a reference) and six ESR mixtures. The ESR mixture that contained 10% of rubber and 90% of soil (by mass) with maximum particle size of 4.8-mm was given the notation 10% 4.8-mm ESR. The remaining ESR mixtures are referred to in a similar way using the following notation: 20% 4.8-mm ESR, 10% 6.7-mm ESR, 20% 6.7-mm ESR, 10% 19-mm ESR, 20% 19-mm ESR, depending upon the STR particles size used and amount of rubber present in the mixture. The rubber content (*RC*) used to quantify the amount of rubber present in the ESR mixtures is defined as the ratio of the dry mass of rubber (*m*_R) to the total dry mass of solids in the mixture according to:

$$RC(\%) = \frac{m_R}{m_R + m_S} \times 100$$

where m_s is the dry mass of soil.

3.4.1 Index Properties

The liquid and plastic limits of the soil were determined in accordance with ASTM D 4318 using a multipoint method. The specific gravity of the soil and ESR mixtures was determined in accordance with ASTM D 854.

3.4.2 Grain Size Distribution

The grain size distribution curves for the Loveland soil, ESR mixtures and STR materials were determined in accordance with ASTM D 422. The grain size distribution curves of the ESR mixtures were determined for ESR mixtures with 4.8-mm and 6.7-mm granulated rubber only, whereas the grain size distribution curves of the STR materials were determined for all three types of STR.

3.4.3 Soil Sample Preparation

The soil samples used in the experimental program were sieved through sieve No. 4 (4.75 mm). The soil was then portioned out for specific tests accordingly. For each sample, the initial water content of the soil was determined according to ASTM D 22 16-05. The dry mass of soil was then calculated and water was added to bring the sample to its target water content. Then, the STR was admixed to the soil to achieve the target RC of the mixture.

3.4.4 Compaction

The dry unit weight-water content relationships were determined for the soil alone and all six ESR mixtures tested in the present study using both standard and modified Proctor efforts following ASTM D 698 and ASTM D 1557, respectively. The optimum water content for each material was systematically determined by (i) taking the derivative of the third-order polynomial fitted to the data points, and (ii) making the derivative equal to zero, and (iii) solving for the water content, as outlined by Howell et al. (1997). The maximum dry unit weight was then calculated by inputting the optimum water content back into the original third-order polynomial equation.

3.4.5 Swell-Consolidation Tests

Swell consolidation (SC) tests, conforming to ASTM D 4546-03, were conducted on compacted specimens. Compacted specimens consisted of soil compacted to optimum water content and at 95% and 100% relative compaction (CR). This method of sample preparation was selected to allow better control of the specimen density. It is possible to compact the soil into a proctor mold and then drive a ring into the extruded compacted specimen. Both methods, however, produce disturbed samples and the latter method does not produce consistent specimens in terms of the dry unit weight.

The initial water content of the soil was determined and subsequently used to determine the appropriate amounts of soil and rubber to be added to the mixture. The appropriate amount of water was then calculated and added to the soil or ESR samples. The sample was then left to hydrate for 24 h. Using the volume of the SC test ring and the target sample density, the required sample mass was compacted into the ring. This process was carried out using two to three lifts pressing the soil into the ring with the SC setup top cap. The ends of the specimen were then smoothed to fill in surface voids. A fixed ring setup was used to allow the soil to swell and to minimize disturbance during sample preparation. The specimens were subjected to an initial seating stress of 6.0 kPa and were inundated at the same stress level.

3.4.6 Preparation of Specimens for Strength Testing

Soil specimens for strength testing were prepared in the laboratory by a static compaction method in accordance with AASHTO T 307-99 (2003). The specimens were compacted in a series of five layers of equal mass. The apparatus consists of an aluminum split mold that is bolted together prior to compaction and a series of aluminum plugs used to compact the different layers. The central layer is the first one to be compacted. Subsequent layers are then compacted on alternating sides of the central layer. The nominal dimensions of the compacted specimens after all five layers are compacted are 71 mm in diameter and 142 mm in height. The aluminum plugs are pressed into the mold at a low strain rate using a Clockhouse 90-kN load frame. The approximate vertical stress during compaction was monitored for each layer using a 50-kN load ring and ranged from about 500 kPa to 900 kPa depending on the specimen. The mold used in the compaction of the test specimens for this project was manufactured at Colorado State University based on the AASHTO T 307-99 (2003) designation and is pictured in Figure 3.5.



Figure 3.5 Compaction Mold (as per AASHTO 307 Resilient Modulus Testing Protocol)

3.4.7 Unconfined Compression Tests

Unconfined compression (UC) tests were conducted on laboratory compacted specimens of the soil alone and the 20% 6.7-mm ESR mixture. The specimens were compacted at optimum water content and CR of about 95%. The ESR mixtures were compacted to CR of 98% within the compaction mold and expanded to approximately CR of 94% when the mold was removed. The soil mixtures were compacted to CR of 94% within the compaction mold in order to eliminate density as a variable in the unconfined strength study.

Testing of each specimen was conducted using a GeoTest S5760 10-kN load frame. The approximate vertical strain rate for each of the tests was 1% per minute. The standard for unconfined compressive strength of cohesive soils (AS TM D 2166-01) was used as a guideline for testing of all UC specimens. The load and displacement were monitored using digital force and displacement transducers connected to a GeoTac data acquisition system manufactured by Trautwein. The vertical force was measured at the top of the compacted specimen using an 80-mm diameter loading plate with a ball bearing. The UC test setup is pictured in Figure 3.6.



Figure 3.6 Unconfined Compression Test Setup

3.4.8 Unconfined Tension Tests

Unconfined tension (UT) tests were conducted on laboratory compacted specimens of the soil alone and the 20% 6.7-mm ESR mixture. The specimens were compacted at optimum water content and target C_R of about 95%. The ESR mixtures were compacted to C_R of 98% within the compaction mold and expanded to approximately C_R of 94% when the mold was removed. The soil mixtures were compacted to C_R of 94% within the compaction mold in order to eliminate density as a variable in the UT testing.

Testing of each specimen was conducted using the same load frame used in the UC testing. The approximate diametral strain rate for each of the tests was 0.5%/min. The standard for splitting tensile strength of cylindrical concrete specimens (ASTM C 496/C 496M-04) was used as a general guideline. The load and displacement were monitored using digital force and displacement transducers connected to a GeoTac data acquisition system produced by Trautwein. The horizontal line load was applied along the length of the cylinder using an 80 mm by 160 mm top plate. Two 3.2 mm by 6.4 mm wooden strips were used on top and bottom of the sample to simulate a line load. The UT test setup is pictured in Figure 3.7.



Figure 3.7 Unconfined Tension Test Setup

3.4.9 Triaxial Tests

Triaxial tests were conducted on laboratory compacted specimens of soil as well as the 20% ESR mixtures with either 4.8-mm or 6.7-mm granulated rubber. The specimens were compacted at optimum water content and target CR of 95% within the compaction mold.

Consolidated isotropically undrained (CIU) triaxial tests were carried out on each of the compacted specimens at an effective confining stress of 25 kPa which corresponds to typical effective confining stresses experienced by road subgrade materials. The samples were back pressure saturated at an effective stress of 15 kPa and then isotropically consolidated to an effective stress of 25 kPa. Due to the expansive nature of the soil and deformability of the STR material, volume changes were maintained throughout the test. The testing of each specimen was conducted using a GeoJac automated triaxial setup produced by Trautwein. The cell and back pressures were controlled using a pressure panel board also produced by Trautwein. The vertical load, vertical displacement, pore pressure and cell pressure were monitored using digital transducers connected to the GeoJac data acquisition computer. The triaxial test setup is pictured in Figure 3.8.



Figure 3.8 GeoJac Triaxial Testing Apparatus

4. MECHANISTIC PAVEMENT DESIGN ANALYSIS

(in preparation)

- 4.1 Model Pavement Cross Sections and Properties
- 4.2 Mechanistic Pavement Design Software
- 4.3 Mechanistic Pavement Design Analysis

5. FIELD TEST SECTION

(in preparation)

5.1 Field Construction Methods

- 5.1.1 ESR Layer Mixing
- 5.1.2 ESR Layer Placement
- 5.1.3 ESR Layer Compaction

5.2 Quality Control /Quality Assurance Methods

5.3 Test Section Monitoring

6. PRELIMINARY RESULTS

6.1 ExperimentalResults

6.1.1 Index Properties

The soil at the Loveland Byrd Drive site has a liquid limit (WL) of 42%, a plastic limit (WP) of 17%, and a plasticity index (PI) of 26%. These values are the averaged results of two Atterberg limit tests. The soil is classified by the USCS classification system as CL. The ESR mixtures all have the same USCS classification of CL as the soil because all rubber would be retained on sieve No. 40 (0.425 mm). The specific gravity results for soil and ESR mixtures are presented in Table 6.1.

Material	RC (%)	Specific Gravity
Loveland Soil	0	2.64
10% 4.8-mm ESR	10	2.35
10% 6.7-mm ESR	10	2.34
20% 4.8-mm ESR	20	2.11
20% 6.7-mm ESR	20	2.10
4.8-mm Rubber	100	1.17
6.7-mm Rubber	100	1.16
19-mm Rubber	100	1.20

Table 6.1 Specific Gravity of the Materials Tested

6.1.2 Grain Size Distribution

The grain size distributions for the soil, ESR mixtures and STR materials are plotted in Figure 6.1. The STR grain size distribution curves show that most rubber particles are less than or equal to the materials nominal particle diameter. The STR samples are fairly uniform. If the USCS system were used, the granulated STR products would classify as SP and the tire chip STR products would classify as GP.



Figure 6.1 Grain Size Distributions of Soil, ESR and STR

6.1.3 Specimen Homogeneity

(In Preparation)

6.1.4 Compaction

The standard and modified Proctor compaction curves and zero air voids curves for the soil and 4.8- mm and 6.7-mm ESR mixtures are plotted in Figure 6.2. A third STR particle size, (19-mm) produced by Front Range Tire Recycle, was also included to further evaluate the effect of rubber particle size on the compaction characteristics of ESR mixtures. The solid horizontal lines shown in Figure 6.2 represent the *in situ* range of dry density and water contents of the soil. Table 6.2 presents the numerical results of optimum water content and maximum dry density for the compaction tests.

These curves depict four different observations: (1) the maximum dry unit weight systematically decreases with increasing rubber content (Ahmed and Lovell 1993), (2) the optimum water content is not significantly affected by the addition of rubber to the soil, (3) the optimum water content and maximum dry density are unaffected by STR particle size within the bounds of testing variability, and (4) the compaction effort appears to have the same effect on both native soil and ESR mixtures.



Figure 6.2a Standard Compaction Curves for the Loveland soil, 10% and 20% ESR Mixtures with 4.8-mm, 6.7-mm, or 19-mm STR



Figure 6.2b Modified Compaction Curves for the Loveland soil, 10% and 20% ESR Mixtures with 4.8-mm, 6.7-mm, or 19-mm STR

Material	Effort	Optimum Water Content (%)	Maximum Dry Density (kN/m ³)
Loveland Soil	Standard	18.6	16.3
10% 4.8-mm ESR	Standard	18.3	15.4
10% 6.7-mm ESR	Standard	17.4	15.4
10% 19-mm ESR	Standard	18.8	15.5
20% 4.8-mm ESR	Standard	18.9	14.0
20% 6.7-mm ESR	Standard	17.9	13.9
20% 19-mm ESR	Standard	18.7	14.4
Loveland Soil	Modified	14.7	18.0
10% 4.8-mm ESR	Modified	15.3	16.3
10% 6.7-mm ESR	Modified	16.1	16.7
20% 4.8-mm ESR	Modified	15.4	14.9
20% 6.7-mm ESR	Modified	14.8	15.2

Table 6.2 Compaction Test Results

6.1.5 Swell-Consolidation Tests

SC tests were carried out for six specimens compacted with the Standard effort: three for the soil and three for the ESR mixtures. The tests were performed at two target C_R levels: 95% and 100%. These values were chosen to simulate typical field compaction conditions as well as to characterize the SC response of the compacted specimens for C_R levels similar to those used in the other tests, namely the UC, TC and triaxial tests. The SC test results are summarized in Table 6.3. The three tests with target C_R of 95% were performed recently and thus the consolidation curves still have not been completely developed.

Material	Water Content (%)	Cr (%)	Cc	Cr	Swell (%)	Swell Pressure (kPa)
Loveland Soil	20.0	94.2	-	-	1.4	-
Loveland Soil	19.4	98.5	0.256	0.0475	1.6	96.0
Loveland Soil	19.7	99.7	0.27 1	0.0543	1.9	96.1
20% 4.8-mm ESR	19.0	95.0	-	-	0.1	-
20% 6.7-mm ESR	16.4	96.4	-	-	0.6	-
20% 6.7-mm ESR	18.3	99.0	0.233	0.111	0.1	25.9

 Table 6.3 Swell-Consolidation Test Results

Addition of 20% of rubber to the native soil (i) reduced the swell percent by at least 50% for both target CR levels tested, and (ii) reduced the swell pressure by 75% for the target CR of 100%. The reduction in both swell percent and swell pressure can be viewed on Figure 6.3 which shows the variation of the vertical strain as a function of the vertical stress during the SC tests. This same general trend was also observed for another 20% 6.7-mm ESR mixture compacted with another soil from Berthoud, CO (Seda et al. 2007). The ESR mixture response, shown in dashed line, shows a very small swell at inundation stress (~6 kPa), unlike the swell percent observed for the other two mixtures. Although a small swell was recorded after inundation for the ESR mixture, it was not enough to bring the specimen height back to its initial height.



Figure 6.3 Swell-Consolidation Test Response for Soil and ESR Mixture

The slope of the normal consolidation line (NCL) is essentially identical for all three specimens. This is qualitatively shown in Fig. 6.3 and quantitatively shown by the compression index (Cc) values presented in Table 6.3. The largest difference shown by these values is found in the Cr value for the expansive soil versus the ESR mixture.

Some variation may be observed in the testing results due to the minor variation of the compaction parameters of the specimens. For example, the test carried out at target $C_R = 95\%$ (actual $C_R = 96.4\%$) should have a lower swell percent relative to its counterpart compacted at target C_R of 100% (actual $C_R = 99.0\%$). From Table 6.3, it may be noted that this characteristic is not met as the 6.7 mm ESR specimen at lower relative compaction swelled 0.43 percentage points more than its denser counterpart. However, this may be explained by the water content variation as the denser specimen was in fact compacted at a water content above optimum as opposed to its looser counterpart, which was compacted at a water content below optimum. A similar argument can be used to explain the small difference in swell percent observed for the untreated native soil.

6.1.6 Unconfined Compression Tests

UC tests were carried out on two laboratory compacted specimens for each mixture in accordance with ASTM D 2166-00. The required allowable range of compressive strength and allowable range of strain at peak strength between two test results was satisfied. The stress-strain response for the UC tests is plotted in Figure 6.4.



Figure 6.4 Stress-Strain Responses for Unconfined Compression Tests

The UC stress-strain response for the 20% 6.7-mm ESR mixture is very different from that of the soil alone. The ESR specimens experience a gradual build up in stiffness and stress as the ESR material deforms. The gradual strain hardening response observed in these test results suggest that, if the ESR material were subjected to an initial surcharge or confining stress, the initial part of the stress-strain plots would be eliminated. The ratio of the average peak unconfined compressive strength for the ESR mixture to that of the soil alone is approximately 1:1.9 (or 53%). The ratio the of average strain at peak for the ESR mixture to that of the soil alone is approximately 5.8:1 (or 580%). Pictures of the UC tests are included in the appendix.

Table 6.4 presents the compaction conditions of the specimens as well as the peak strength values as required by ASTM D 2166-00. The water content achieved was repeatable and within 0.3% of the optimum water content from the standard Proctor tests. The C_R values obtained once the confinement of the mold was released were less than 95% and can be explained by the rebound of the compressed rubber particles. In order to have comparable soil states, the C_R of the soil was targeted at values similar to achieve for the ESR after the mold was removed.

Material	Water Content (%)	Cr (%)	σz _{peak} (kPa)	ε z peak (%)
Loveland Soil	18.3	93.8	206	2.3
Loveland Soil	18.3	93.8	210	2.7
20% 6.7-mm ESR	17.5	93.4	115	14.1
20% 6.7-mm ESR	17.7	93.3	106	14.9

 Table 6.4 Unconfined Com^Pression Test Results

6.1.7 Unconfined Tension Tests

UT tests were carried out on two laboratory compacted specimens for each mixture using ASTM C 496/C 496M-04 as a general guideline. The repeatability of the tests was examined using the criterion from the UC test standard and the range for two test results was satisfied. The stress-diametral strain response for the UT tests is plotted in Figure 6.5.



Figure 6.5 Stress-Diametral Strain Responses for Unconfined Tension Tests

The unconfined stress-diametral strain response for the 20% 6.7-mm ESR mixture is significantly different from that of the soil alone. The ESR mixture experiences a more gradual strain hardening response than the untreated soil. The ratio of the average peak UT strength of the ESR mixture to that of the soil alone is approximately 1:2.2 (or 45%). The ratio of the average strain at peak for the ESR mixture to that of the soil alone is approximately 1.5:1 (or 154%). Pictures of the UT tests are included in the appendix.

Table 6.5 presents the compaction conditions of the specimens as well as the peak tensile strength values and the corresponding diametral strains. The water content achieved was repeatable and with 0.3% of the optimum water content from the standard Proctor tests. The C_R for the tests was similar and comparable to that obtained for the UC tests.

Material	Water Content (%)	Cr (%)	σt _{peak} (kPa)	ε d peak (%)
Loveland Soil	18.7	93.1	18.0	1.3
Loveland Soil	18.6	93.4	17.7	1.2
20% 6.7-mm ESR	17.9	93.9	8.4	1.9
20% 6.7-mm ESR	17.9	94.1	7.7	2.1

 Table 6.5
 Unconfined Tension Test Results

The UT strength of the soil is equal to 8.5% of its unconfined compressive strength while the unconfined tensile strength of the ESR mixture is equal to 7.2% of its unconfined compressive strength. Both the soil alone and ESR mixture have σ_Z/σ_T ratios similar to those that would be expected for typical geomaterials.

6.1.8 Triaxial Tests

Three consolidated isotropically undrained (CIU) triaxial tests were carried out at an effective stress of 25 kPa at the end of the consolidation stage. The CIU specimens were sheared to approximately 25% axial strain. All of the specimens failed with a barreling mechanism. The triaxial response was examined by looking at the deviator stress and excess pore pressure versus axial strain in addition to the stress paths and stiffness degradation curves. The deviator stress (q) and excess pore pressure (Δu) in axisymmetric compression are defined as:

$$q = \sigma_1 - \sigma_3$$
$$\Delta u = u u_c$$

where: σ_1 and σ_3 are the total major and minor principal stresses, respectively, and *u* and *u*_c are the current pore pressure and the pore pressure at the end of the isotropic consolidation stage, respectively. The deviator stress and excess pore pressure plots are presented in Figures 6.6 and 6.7 respectively.



Figure 6.6 Deviator Stress versus Axial Strain for Triaxial Tests



Figure 6.7 Excess Pore Pressure versus Axial Strain for Triaxial Tests

The triaxial response observed in Figures 6.6 and 6.7 suggests that the effect of STR particle size (where all other factors are the same) is minimal. The response of the soil is significantly different from that of the ESR mixtures. The stress-strain curve in Figure 6.6 illustrates that the soil has a larger strength and stiffness then the ESR mixtures, particularly at small strains. The ESR mixtures generate positive excess pore pressure throughout the tests while the soil generates initial positive excess pore pressure which then decreases and eventually becomes negative.

The peak and critical state friction angles of the three specimens were calculated using the effective stress ratio σ'_1/σ'_3 , where σ'_1 and σ'_3 are the vertical and horizontal effective stresses, respectively, defined at specific points during the shearing stage. The peak strength was defined as the point with the largest effective stress ratio and the critical state strength was defined as the point of largest strain. The peak and critical state friction angles for the three tests are given in Table 6.6.

Material	Water Content (%)	CR (%)	Peak	Critical State
		(End of Consolidation)	Friction Angle	Friction Angle
Loveland Soil	18.8	98.9	46.8°	33.4°
20% 4.8-mm ESR	18.5	99.1	40.8°	38.8°
20% 6.7-mm ESR	17.7	100.2	42.6°	40.3°

Table 6.6 Triaxial Test Results

The compaction water content for each specimen was similar to the optimum water content of their corresponding mixtures from the standard Proctor curves. The maximum variation in compaction water content from the optimum values was 0.4%. The specimens were all compacted to C_R of 95% within the mold and they all ended up having similar C_R values at the end of the saturation and consolidation stages.

Both the peak and critical state friction angle for the two ESR mixtures slightly increase with increasing STR particle size. This may suggest that admixing of larger STR particles results in slightly increased strength properties particularly at small strains where the peak effective stress ratio is typically observed. The peak friction angle of the soil is significantly larger than the peak friction angle of the ESR mixtures, while the critical state friction angle of the soil is significantly less than the critical state friction angle of the soil is significantly less than the critical state friction angle of the soil structure and packing is substantially reduced, the STR particles (which are larger than the soil particles) might help increase the frictional component of the shear strength of the ESR mixtures. At smaller strains, the presence of STR particles might help create a soil-STR particle structure with higher contractive tendency than its soil counterpart, possibly due to the higher deformability of the STR particles at relatively lower mean effective stress levels.

The *p*' versus *q* stress paths for the triaxial specimens were determined where *p*' is defined as the average of the three effective principal stresses (σ'_1 , σ'_2 and σ'_3) and *q* is the deviator stress. The effective stress paths along with the peak and critical state failure surfaces are presented in Figure 6.8.



Figure 6.8 Effective Stress Paths for Triaxial Tests

The ESR mixtures have very similar stress paths with only slightly different failure surfaces. The soil stress path is significantly different than that of the ESR mixtures and experiences a large reduction in strength from peak to critical state conditions.

The stiffness degradation curves for the triaxial specimens were determined by taking the secant slope of the deviator stress-strain curve over the range of strain applied during the shearing stage. The stiffness degradation curves were also determined for the UC tests by taking the secant slope of the stress strain curve over the range of strain applied during the UC test. The stiffness degradation curves for all specimens are plotted in Figure 6.9 and 6.10.



Figure 6.9 Stiffness Degradation Curves for Triaxial and Unconfined Compression Tests



Figure 6.10 Stiffness Degradation Curves for Triaxial and Unconfined Compression Tests

The curves in Figures 6.9 and 6.10 allow for the evaluation of the effect of both saturation and effective confinement on the stiffness degradation response of the soil and ESR specimens when subjected to axisymmetric compression. These curves must be interpreted taking into consideration the changes in both C_R and soil suction of the specimens. The UC tests were carried out at optimum water content conditions while the triaxial tests were carried out under saturated conditions.

The stiffness degradation curves for the UC tests show that the addition of rubber to the native soil significantly reduces the material stiffness for specimens tested at similar states (constant C_R and no confinement). This loss in stiffness, which is particularly significant at small strains, appears to be due to the fact that the rubber particles themselves deform while taking minimal load. The ESR specimens became stiffer once the rubber particles were compressed. If the samples were confined, much of the rubber deformation would likely have already taken place resulting in increased material stiffness. Figure 6.10 looks closely at the portion of the stiffness degradation curves where the ESR mixtures reach a peak. The stiffness response of the ESR mixtures in this region further suggests that with confinement (starting with deformed rubber particles) the ESR would have a considerably larger stiffness at small strains.

The stiffness degradation curves for the triaxial tests show that the addition of rubber to native soil significantly reduces the stiffness at small strain and maintains approximately the same stiffness at large strains. The results of triaxial tests on ESR mixtures with other expansive soils suggest that the difference between the soil and ESR stiffness decreases as the confinement is increased. The size of the rubber in the ESR mixture appears to have minimal impact on the material stiffness.

6.2 Mechanistic Pavement Design Results

(in preparation)

REFERENCES

Ahmed, I. and Lovell, C.W. (1993). "Rubber Soils as Lightweight Geomaterials." *Lightweight Artificial and Waste Materials for Embankments over Soft Soils, Transportation Research Record,* (1422), National Academy Press, Washington, DC, 6 1-70.

Al-Rawas, A., R. Taha, J. Nelson, T. Al-Shab, H. Al-S iyabi, (2002) "A comparative evaluation of various additives used in the stabilization of expansive soils," *Geotechnical Testing Journal*, GTJODJ, 25 (2), 199-209.

Ashmanwy, A., McDonald, R., Carreon, D. and Atalay, F. (2006). "Stabilization of Marginal Soils Using Recycled Materials." *Florida Department of Transportation*, (BD-544-4)

Basma, A.A., Al-Rawas, A.A., Al-Saadi, S.N., Al-Zadjali, T.F., (1998). "Stabilization of expansive clays in Oman," *Environmental and Engineering Geoscience*, 4 (4), 503-510.

Colton, R.B. (2004). Geologic Map of the Boulder-Fort Collins-Greeley Area, Colorado. U.S. Geologic Survey.

Cokca, E. (2001) "Use of class C fly ashes for the stabilization of an expansive soil," *Journal of Geotechnical and Geoenvironmental Engineering*, 127 (7), 568-573

Edil, T.B. (2005). "A Review of Mechanical and Chemical Properties of Shredded Tires and Soil Mixtures." *Recycled Materials in Geotechnics: Proc. of Sessions of the ASCE Civil Engineering Conference and Exposition*, ASCE, GSP No. 127, 1-21.

Edil, T.B. and Bosscher, P.J. (1994). "Engineering Properties of Tire Chips and Soil Mixtures." *Geotechnical Testing J.*, GTJODJ, 17 (4), 453-464. Environmental Protection Agency. (2003). "2003 EPA Report on the Environment." <u>http://www.epa.gov/indicators/index.htm</u>

Feng, Z. and Sutter, K.G. (2000). "Dynamic Properties of Granulated Rubber/Sand Mixtures." *Geotechnical Testing J.*, GTJODJ, 23 (3), 338-344. Hudyma, N., Burcin Avar, B., (2006). "Changes in swell behavior of expansive clay soils from dilution with sand," *Environmental and Engineering Geoscience*, 12 (2), 137-145.

Howell, J.L., Shackelford, C.D., Amer, N.H. and Stern, R.T. (1997). "Compaction of Sand-Processed Clay Soil Mixtures." *Geotechnical Testing Journal*, 20(4), 443-458.

Humphrey, D.N., Sandford, T.C., Cribbs, M.M. and Manion, W.P. (1993). "Shear Strength and Compressibility of Tire Chips for use as Retaining Wall Backfill." *Lightweight Artificial and Waste Materials for Embankments over Soft Soils, Transportation Research Record*, (1442), National Academy Press, Washington, DC, 29-35.

Hunter, D. (1988). "Lime-induced heave in sulfate-bearing clay soils." *Journal of Geotechnical Engineering*, 114 (2), 150-167.

Jones, D. E., and Jones, K. A., (1987). "Treating expansive soils." Civil Engineering, 57 (8), 62-65.

Katti, R.K., Katti, D.R. and Katti, A.R. (2002). *Behavior of Saturated Expansive Soil and Control Measures*, A.A. Balkema, India.

Kershaw, D.S., and Pamukcu, S. (1997). "Use of Ground Tire Rubber in Reactive Permeable Barriers to Mitigate BTEX Compounds." *Testing Soil Mixed with Waste or Recycled Materials, ASTM STP 1275, M.A. Wasemillier, K.B. Hoddinott, Eds., ASTM, 3* 14-329.

Lee, J.H., Salgado, R., Bernal, A. and Lovell, C.W. (1999). "Shredded tires and rubber-sand as lightweight backfill." *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 125 (2), 132-141.

Nelson, J. D., Miller, D. J., (1992). "Expansive Soils: Problems and practice in foundation and pavement engineering," John Wiley and Sons, New York.

Newcomb, D.E. and Drescher, A. (1994). "Engineering Properties of Shredded Tires in Lightweight Fill Applications." *Transportation Research Record*, (1437), National Academy Press, Washington, DC, 1-7.

Petry, T. M., and Little, D. N., (2002). "Review of stabilization of clays and expansive soils in pavements and lightly loaded structures—History, Practice, and Future." *Journal of Materials in Civil Engineering*, 14 (6), 447-460.

Puppala, A.J., Intharasombat N. and Vempati R.K. (2005). "Experimental Studies on Ettringite-Induced Heaving in Soils." *Journal of Geotechnical and Geoenvironmental Engineering, ASCE*, 131 (3), 325-337.

Rubber Manufacturers Association. (2004). "U.S Scrap Tire Markets 2003 Edition." http://www.rma.org/scrap tires/

Siddiki, N.Z, Kim, D. and Salgado, R. (2004). "Use of recycled and waste materials in Indiana." Transportation Research Record, No 1874, 78-8 5.

Tatlisoz, N., Benson, C.H., and Edil, T.B. (1997). "Effect of Fines on Mechanical Properties of Soil-Tire Chip Mixtures." *Testing Soil Mixed with Waste or Recycled Materials, ASTM STP 1275, M.A. Wasemillier, K.B. Hoddinott, Eds., ASTM,* 93-108.

Upton, R.J. and Machan, G. (1993). "Use of Shredded Tires for Lightweight Fill." *Lightweight Artificial and Waste Materials for Embankments over Soft Soils, Transportation Research Record,* (1442), National Academy Press, Washington, DC, 36-45.

Yoon, S., Prezzi, M. Siddiki, N.Z. and Kim, B. (2005). "Construction of a test embankment using a sandtire shred mixture as fill material." *Waste Management*, No 26, 1033-1044.

Zornberg, J.G., Costa, Y.D. and Vollenweider, B. (2004). "Performance of prototype embankment built with tire shreds and nongranular soil." *Transportation Research Board*, 1874, 70-77.

Zornberg, J.G., Christopher, B.R. and Oosterbaan, M.D. (2005). "Tire Bails in Highway Applications: Feasibility and Properties Evaluation." *Colorado Department of Transportation Research Branch*, (CDOT-DTD-R-2005-2)

APPENDIX A: LABORATORY EXPERIMENTAL RESULTS

A.1 Index Properties

Project Name Loveland

Date <u>4/26/2007</u>

Sample Location Loveland Temporary Road at Crossroads Blvd. and Byrd

<u>Dr.</u>

Sample ID Sample

1

Sample Descrpt.

Liquid Limit						
# of drops	# of drops 17 26 31					
tare id	L10	L1	L7			
tare mass	21.20	20.95	20.87			
tare + wet mass	33.32	31.38	31.42			
tare + dry mass	29.57	28.25	28.29			
wet mass	12.12	10.43	10.55			
dry mass	8.37	7.30	7.42			
Liquid Limit	44.8	42.9	42.2			

Plastic Limit		
tare id	L4	A6
tare mass	21.07	21.07
tare + wet mass	32.82	34.73
tare + dry mass	31.12	32.72
wet mass	11.75	13.66
dry mass	10.05	11.65
Plastic Limit	16.9	17.3



Summary	of Results
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Plastic Limit	17
Liquid Limit	43

Plasticity	Index	26
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Project Name Loveland

_____Date <u>4/26/2007</u>

Sample Location Loveland Temporary Road at Crossroads Blvd. and Byrd

<u>Dr.</u>

Sample ID Sample 2

Sample 2 Sample Descrpt.

Liquid Limit			
# of drops	15	25	39
tare id	L6	ON2	L14
tare mass	21.11	21.22	21.31
tare + wet mass	33.39	37.86	35.60
tare + dry mass	29.74	32.94	31.51
wet mass	12.28	16.64	14.29
dry mass	8.63	11.72	10.20
Liquid Limit	42.3	42.0	40.1

Plastic Limit			
tare id	L13	C14	
tare mass	21.16	21.1	
tare + wet mass	31.88	32.66	
tare + dry mass	30.34	31.07	
wet mass	10.72	11.56	
dry mass	9.18	9.97	
Plastic Limit	16.8	15.9	



Summary of Results		
Plastic Limit	16	
Liquid Limit	42	

Plasticity Index	25

Summaries

	Sample 1	Sample 2	Average
Plastic Limit	17.1	16.4	17
Liquid Limit	43.2	41.6	42
Plasticity Index	26	25	26
Classification		СН	

A.2 Specimen Homogeneity

A.3 Compaction



ESR Sample Compacted in Proctor Mold



ESR Sample Extruded from Proctor Mold

A.4 Swell-Consolidation Tests



Swell-Consolidation Testing Setup

A.5 Unconfined Compression Tests

Loveland Soil

20% 6.7-mm ESR

A.6 Unconfined Tension Tests

Loveland Soil

20% 6.7-mm ESR

A.7 Triaxial Tests



Loveland Soil Triaxial at Start and End of Shearing

20% 4.8-mm ESR Triaxial at Start and End of Shearing



20% 6.7-mm ESR Triaxial at Start and End of Shearing