SCHOOL OF
CIVIL ENGINEERING

INDIANA
DEPARTMENT OF TRANSPORTATION

JOINT HIGHWAY RESEARCH PROJECT

FHWA/IN/JHRP-96/12
Final Report

LABORATORY STUDY ON THE USE OF TIRE SHREDS AND RUBBER-SAND IN BACKFILLS AND REINFORCED SOIL APPLICATIONS

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FINAL REPORT

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Joint Highway Research Project

Project No.: HPR-2087
File No.: 6-19-16

Prepared in cooperation with the
U.S. Department of Transportation Federal Highway Administration

Purdue University
West Lafayette, Indiana 47907
December 20, 1996

TECHNICAL REPORT STANDARD TITLE PAGE

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA/IN/JHRP-96/12		

4. Title and Subtitle	5. Report Date / 6. Performing Organization Code
Laboratory Study on the Use of Tire Shreds and Rubber-Sand in Backfills and Reinformced Soil Applications	5. Report Date: December 20, 1996 6. Performing Organization Code:
7. Author(s) Andres Bernal, C. W. Lovell, and Rodrigo Salgado	8. Performing Organization Report No. FHWA/IN/JHRP-96/12
9. Performing Organization Name and Address Joint Highway Research Project 1284 Civil Engineering Building Purdue University West Lafayette, Indiana 47907-1284	10. Work Unit No. 11. Contract or Grant No. HPR-2087
12. Sponsoring Agency Name and Address Indiana Department of Transportation State Office Building 100 North Senate Avenue Indianapolis, IN 46204	13. Type of Report and Period Covered Final Report 14. Sponsoring Agency Code

15. Supplementary Notes

Prepared in cooperation with the Indiana Department of Transportation and Federal Highway Administration.

16. Abstract

Millions of scrap tires are discarded annually in the United States, the bulk of which are currently landfilled or stockpiled. This consumes valuable landfill space, or, if improperly disposed, creates a fire hazard and provides a prolific breeding ground for rates and mosquitoes. The use of tire shreds as lighweight fill material can sharply reduce the tire disposal problem. The present study, based on laboratory testing and numerical modeling, examines the feasibility of incorporating tire shreds and rubber-sand mixtures as lightweight geomaterial in embankments and backfills.

The growing interest in utilizing waste materials in civil engineering applications has opened the possibility of using reinforced soil structures with non-conventional backfills. The laboratory testing program of the present study includes the determination of volumetric behavior of rubber-sand mixtures during triaxial testing, lateral pressure coefficients for rubber-sand backfills, and interaction properties of tire shreds and rubber-sand mixtures with geogrids and geotextiles through pull-out and direct shear tests. The test results have been used to perform numerical modeling of tire shred and rubber-sand backfills in walls.

It has been found that the use of tire shreds and rubber-sand (with a tire shred to mix ratio of about 40%) in highway construction offers technical, economic, and environmental benefits. The salient benefits of using tire shreds and rubber -sand include reduced weight of fill, adequate stability, low settlements, good drainage (avoiding the development of pore water pressure during loading), separation of underlying weak or problem soils from subbase or base materials, conservation of energy and natural resources, and usage of large quantities of local waste tires, which would have a positive impact on the environment.

17. Key Words	18. Distribution Statement
highway construction materials, waste utilization, scrap tire disposal options, backfill, reinforced soil applications, lighweight material.	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22161

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	164	

Form DOT F 1700.7 (8-69)

ACKNOWLEDGEMENTS

The authors wish to acknowledge the assistance provide by Mr. Robert Swan, GeoSyntec Consultants for allowing the use of their facilities and Huesker, Inc. for providing the geosynthetics used in this research. Special thanks are extended to Professor Dana Humphrey, University of Maine for his advice and help during the course of this research.

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IMPLEMENTATION REPORT

The study is directed towards presenting a partial solution to the waste tire problem in the United States. The use of scrap shredded tires as a lightweight fill in embankments and wall backfills has the potential of disposing of large quantities in beneficial ways.

The environmental effects can be diminished by providing proper encapsulation of the tire shred fill and preventing the presence of water in the fill. It may also be desirable to reduce the risk of an exothermic reaction in situ by controlling shred size and steel content. The largely positive results observed in tire shred and rubber-soil embankments in different areas of the country support the feasibility of this application in Indiana.

The following recommendations are made to INDOT to start implementation of the present research.

1. Conduct laboratory research on the effect of the shred properties on the generation of exothermic reactions.

2. Demonstration projects involving the use of tire shreds and rubber-sand as lightweight fill should be identified.

3. The proposed Special Provision for Embankments Constructed with Shredded Tires should be implemented with some minor changes.

4. A simple monitoring program such as the use of groundwater monitoring wells parallel to the embankment or in the vicinity of the fill should be designed.

CHAPTER 1
INTRODUCTION

Lightweight fill materials can be used to solve bearing capacity and settlement problems of walls and embankments on soft compressible soils. Some common lightweight materials that have been used in the past include sawdust and bark from the lumber industry, slags and ashes from the power generation industry and engineered materials such as expanded shales and Elastizell. These materials have intrinsic disadvantages such as long term performance, settlement, environmental effects, and economic aspects that lessen their appeal as lightweight fill materials.

Field and laboratory studies indicate that the use of tire shreds and rubber-sand meets the requirements of durability, low unit weight, availability and relative cost required for lightweight fill material applications.

Millions of scrap tires are discarded annually and an even larger number is currently stockpiled throughout the country consuming valuable landfill space, or, if improperly disposed, providing a breeding ground for mosquitoes and rodents. The use of tire shreds as lightweight fill can reduce the tire disposal problem in an economically and environmentally beneficial way.

Various highway agencies in the United States (e.g. Colorado, Maine, Minnesota, Oregon, Vermont, Washington and Wisconsin) have studied applications for tire shreds as a lightweight fill material. Their experiences indicate that the use of shredded tires in embankments and backfills is feasible and beneficial.

The INDOT has been using recycled and waste products in those applications which have been proven effective. Research has also been done on the use of a variety of waste products in highway construction to find an alternative source to offset the rising costs of quality natural aggregates, waste disposal and energy. This study is part of INDOT's commitment to promote the use of waste products in highway construction and satisfy the requirements of Senate Bill No. 209 and House Bill 1056 which deal with this objective.

The purpose of this research is to investigate, based on laboratory testing and evaluation combined with computer modeling, the feasibility of using shredded tires in embankments and wall backfills. The study focuses on the stress-strain and volumetric behavior during triaxial testing, on the compressibility and lateral pressures, on the reinforced earth applications, and addresses the environmental impact of tire shreds and rubber-sand mixtures. The findings of this study provide parameters for design of embankments and walls, performance prediction and evaluation, and recommendations for use of shredded tires and rubber-sand in embankments and wall backfills.

The research objectives were accomplished by synthetizing available information from a comprehensive literature review and following a detailed testing plan.

Chapter 2 summarizes the current information on recycling, reuse and disposal options for scrap tires. The current laboratory studies and applications from various highway agencies and universities are reported. The available information on compressibility, compaction and environmental effects of tire shreds and rubber-sand is also presented.

Chapter 3 presents the results of the stress-strain and volumetric change results during triaxial testing. Chapter 4 describes the setup used to determine the compressibility and earth pressure coefficients for tire shreds and rubber-sand.

Chapter 5 presents the results of direct shear and pullout tests of geosynthetics performed to determine the parameters necessary for design of reinforced soil applications of tire sheds and rubber-sand.

Chapter 6 contains the results of the numerical modeling and finite element analysis used to predict the performance of tire shred and rubber-sand embankments and walls.

Chapter 7 presents the recommended procedures and specifications for utilizing tire shreds and rubber-sand in embankments and backfills with or without reinforcement.

Chapter 8 summarizes the main conclusions of this experimental study and provides recommendations for these type of applications in Indiana.

CHAPTER 2

TIRE SHREDS AS LIGHTWEIGHT GEOMATERIAL

2.1 Introduction

It is estimated that 0.8 to 2 billion scrap tires have been disposed of in huge piles across the United States. Furthermore, an additional 250 million tires are discarded every year. Almost 30% of these scrap tires wind up in overcrowded landfills and thousands more are left in empty lots and illegal tire dumps. These piles are a serious fire hazard, an ideal breeding ground for rodents and mosquitoes, and an ugly sight in the landscape. Since rubber tires do not easily decompose, economically feasible alternatives for scrap tire disposal must be found. Some of the current uses for recycled tires include tire derived fuel for energy generation in cement kilns and paper mills, tire retreading applications, highway crash barriers, breakwaters, reefs, and crumb rubber asphalt pavement.

Although these recycling, reuse and recovery efforts consume about 70% of the tires discarded every year, they have not significantly reduced the amount of tires in landfills and illegal dumps. A need still exists for the development of additional and practical uses for scrap tires (EPA, 1991; Scrap Tire Management Council reported by Hilts, 1996).

2.2 Background

Although automobile and truck tires manufactured today are primarily steel-belted radial ply type, other types of tires are available. Some tires are made with fiberglass, aramid, and/or rayon. Most modern tires have a complex composition of natural and synthetic rubbers, chemicals, minerals, and metals. Steel-belted radial ply tires may also contain polyester, steel, or nylon cords. Some radial tires have a fine carcass wire, whereas bias ply tires do not. Both radial and bias ply tires contain bead wires, which consist of numerous strands of high tensile strength steel. A typical tire casing is composed of 83 percent carbon, 7 percent hydrogen, 1 percent sulfur and 6 percent ash. The primary constituents include polymers, carbon black and softeners. The softeners are mostly composed of hydrocarbon oils which in combination with the polymers give the tire a very high heating value, hence the combustible nature of tires. When tires burn in uncontrolled environments, the thick black smoke that escapes contains fine particles of unburned hydrocarbons (Blumenthal, 1993).

Rubber tires are designed to withstand the rigors of the environment so that they will have a reasonable useful life on vehicles. Therefore, it is not surprising that discarded tires persist for long periods. Indeed, it has been estimated that a whole tire requires at least a hundred years to decompose fully (Hofmann, 1974; reported by Cadle and Williams, 1980). The average scrap automobile tire weighs approximately 20 pounds and makes up to 85 percent of all scrap tires. Heavy truck and industrial tires, which can weigh anywhere from 35 to several hundred pounds, constitute 14 percent of all scrap tires and are more difficult to process for further use. The remaining 1 percent are specialty tires, ranging from aircraft to heavy equipment tires (Blumenthal, 1993).

Currently three major options exist for the disposal of scrap tires: (1) landfilling, (2) recycling and reuse, and (3) incineration. The most common method currently used in the United States is incineration of tire shreds as Tire Derived Fuel (TDF). The radial tire design and low import prices have forced retreading and other forms of tire reuse out of the market place (Chafee, 1993). Nearly 30 percent of all scrap tires are disposed of in landfills. In Indiana, the cost of landfilling tires ranges from one to eight dollars for automobile tires and increases substantiously for heavy truck tires. Whole tires are no longer allowed in landfills and should be cut to quarters of the original size to be accepted. This measure increases the compactability and prevents water and air from being trapped in the tire and causing it to float to the surface of the landfill.

The Intermodal Surface Transportation Efficiency Act (ISTEA) approved by the Congress in 1991 requires that by 1997 one fifth of all road projects include 20 lb of recycled tire rubber per ton of hot mix or 300 pounds per ton of spray applied binder (Huckaba et al., 1993). The implementation of ISTEA has been postponed because of the strong opposition from state transportation departments and trade groups such as the National Paving Association and Associated General Contractors. The cost of adding scrap tire rubber to asphalt pavement is very high. The price increases between 20% to 100% above the cost of conventional asphalt pavements and the process consumes a small number of tires. Twenty pounds of recycled tire rubber is equivalent to about 1.4 tires per ton of hot mix. A mile of an interstate highway in Maine overlaid with crumb rubber modified asphalt used only 6,000 tires at a cost of $22 per tire. If ISTEA was fully implemented, it would only consume less than 20% of the yearly amount of scrap tires in Maine (Humphrey and Nickels, 1994).

In Indiana, HB1391 was signed into law on March, 1990 and establishes regulations on the disposal of lead acid batteries and scrap tires. The law creates a scrap tire management fund to pay for cleaning up tire dumps when the responsible party is unknown or cannot afford the cleanup. Permit requirements were instituted for established scrap tire storage facilities which should show proof of financial responsibility, source and quantity of tires handled, quality of material (shredded, cut or whole) shipped from the site, and documentation showing its final destination, site closure plan and a contingency plan for protecting public health and the environment. The law requires the Department of Environmental Management to establish a Waste Tire Task Force to develop market plans for waste tires and further guidelines for storage and includes a 10% price preference for state purchase of supplies that meet recycled content requirements. As of July 12, 1994, the Indiana Department of Environmental Management Waste Tire Registration program listed 60 scrap tire transporter, processing or storage companies who were registered or pending for the first six months of the state's registration program (RRI, 1995).

The "1994 Scrap Tire Use/Disposal Study" published by the Scrap Tire Management Council found that 55.4% of the new scrap tires generated in 1994 in the US had markets in 1994; this amount has increased to 70% in 1995 (see Figure 2.1). Overall markets for scrap tires have risen since 1990, when only 11% of the annually generated scrap tires were consumed. Tire Derived Fuel (TDF) continues to be the largest single market that consumed 100 million tires in 29 cement kilns (11 were testing), 15 paper mills (8 were testing), 22 industrial plants including utilities and 2 whole-tire-to-energy plants in 1994 and consumed 130 million tires in 1995.

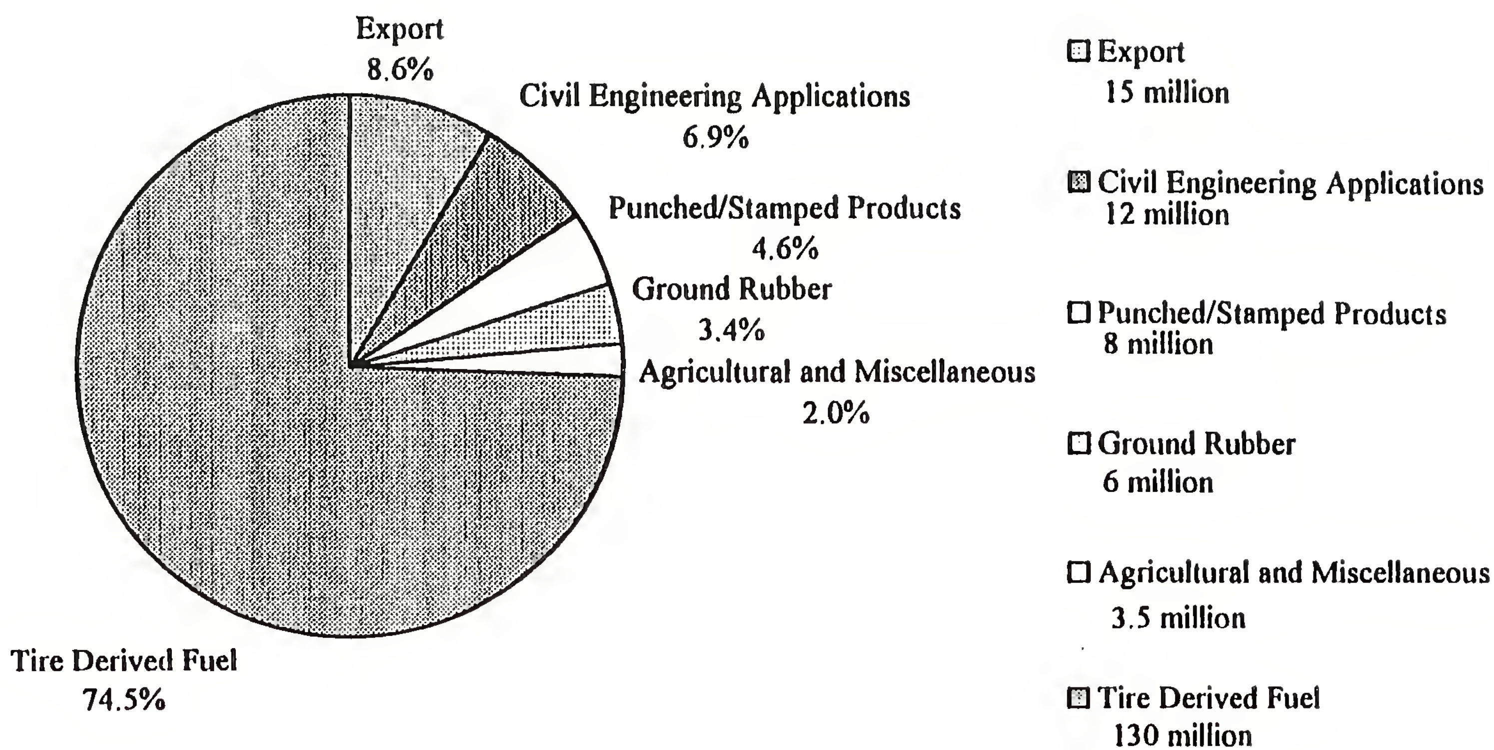


Figure 2.1 Markets for U.S. Scrap Tires - 174.5 million scrap tires consumed in 1995
(69% of the total scrap tires)

Civil engineering applications nearly doubled over the last two years utilizing 9 million tires in 1994 and 12 million tires in 1995. The total amount of crumb rubber derived from tires and used in the manufacturing of various rubberized goods including molded/extruded goods, bound rubber, new tire fill, friction materials, asphalt rubber, rubber/plastic compounds, surfacing materials (athletic, recreational, construction) increased to 6 million tires in 1995. The survey results suggest that markets for scrap tires will consume about 87% of the annually generated scrap tires in the US by the end of 1996. The 1994 and 1995 studies indicate that the inventory of above ground stockpiles in the US is estimated to be 800 million scrap tires. Earlier studies including the 1991 EPA market study estimated that two to three billion scrap tires were stored in above ground stockpiles. The RRI report is based on a survey of all 50 states and is the first attempt to clearly identify total US stockpile size (RRI, 1995; Hilts, 1996).

2.3 Alternative Uses for Scrap Tires

The uses for scrap tires can be grouped into two general categories: whole or processed tires. Uses for whole tires include retreading, artificial reefs, soil reinforcement, while uses for processed tires include rubber products, fuel shreds and other civil engineering applications. Some of the common uses are outlined in the following sections.

2.3.1 Uses for Whole Scrap Tires

Some of the common applications for whole scrap tires are listed below.

- Retreading. Retreading is the process by which the worn tread portion of the tire is replaced with a new tread. Two basic systems are used: mold cure and precure. After the tire casing is tested to see if it is suitable for retreading, the remaining tread is buffed off and the casing is shaped to accept the new tread. After the casing has cured, the retreaded tire is inspected and marked. At present some 38 million tires are retreaded annually, the majority of which are truck tires. This is mostly due to the economic advantage over new tires because three to five retreads are possible on truck tires, and the finished product is regarded as safe.

-Artificial reefs and breakwaters. Artificial reefs are built by bundling scrap tires together and then anchoring them in coastal waters. These tires soon become a permanent habitat for marine life. Breakwaters can be constructed to help protect harbors and boat marinas from the harmful effects of waves and tides. Other potential uses include dune stabilization mats and river bank erosion mats.

- Civil engineering applications. Techniques for the utilization of rubber tires in civil engineering applications have been developed over the past several years. Whole tires have been used as fill reinforcement in retaining structures and as a traveling surface over soft soils by binding them together. Scrap tires are currently being used for landfill applications either as a daily cover or as part of leachate collection systems in various areas of the United States.

2.3.2 Uses of Processed Scrap Tires

The first step in processing a recycled tire is to reduce its size by either chopping, shredding, or grinding. Most processors use fairly small mobile shredding equipment with engines ranging from 20 to 75 kW. Present shredders use a shearing process that produces more uniform sizes, cleaner cuts and minimizes partial pulling of tire belts in contrast to the old tearing process used in older shredders. The production rate ranges from 100 to 400 tires per hour and the costs range from $30 to $65 per ton corresponding to approximately 100 automobile tires (Edil and Bosscher, 1994).

After shredding, a whole tire is reduced to strips. Coarse tire shreds of about 8 in. size are produced after a single pass of the tire through the shredder. Finer shreds are produced after two or three passes through the shredder, and measure about 2 in. The bulk tire volume is reduced by up to 75 percent after this process. Processed scrap tires have been used in the following applications.

- Fuel. Scrap tires provide an excellent source of energy. Combustion facilities can be constructed or modified to burn whole tires or tire shreds exclusively or in combination with other fuel sources. Tires consist of 83 percent carbon and possess a comparatively high heating value of about 15000 BTU's/lb with respect to other fuel sources. Tires can be burned in cement kilns, pulp and paper mills, utility boilers and scrap tire incineration facilities. Since a continuous supply of scrap tires is required, the most appropriate location for these facilities would be next to a tire disposal center. Most mills require 95% removal of steel belts and this is achieved by shredding the tires to 1 inch maximum size and using magnetic separators. The cost is $35 to $90 per ton which makes it an economically competitive boiler fuel.

- Pyrolysis. Pyrolysis is the process of breaking organic bonds by heating. This process involves combustion of whole tires in the absence or under controlled concentrations of oxygen. Tires are broken down into several by-products, including carbon black, gas, oil, and steel. The gas generated is typically used to provide heat for the reactor. Possible uses for the carbon black produced in asphalt pavement and other areas are currently being researched (Park, 1995; Zeng and Lovell, 1995).
- Asphalt paving. Scrap tire rubber can be used in asphalt paving in two different processes: with ground rubber in the aggregate binding material or in the seal coat (loosely known as asphalt rubber) or with tire shreds as aggregate (rubber modified asphalt concrete). Both technologies have been proven commercially in small scale applications. However, there are some contradictions in the data available on the use and performance. Some disadvantages of tire use in asphalt paving include high initial costs, lack of ASTM or other standard specifications and questionable recyclability of this material.
- Concrete aggregate.
- Drainage pipes made with tire beads
- Engineered fill in embankments and walls. A previous report by Ahmed (1993) provided information on this and other uses for tire shreds. The information has been summarized and expanded with current information on research and applications in the following section.

2.4 Engineered fill in embankments and walls

The main objective of the present study is the use of shredded tires in embankments and fills. Some of the latest applications since the report by Ahmed, 1993 are presented here. Shredded tires have been used separately and in combination with soil in the projects listed below. A summary of the research in laboratory and field projects developed at the University of Maine can be found in Section 2.5 Laboratory Studies.

2.4.1 Vermont

A half mile section of the Oakland Station Road (Town Highway 4) in Georgia, Vermont was almost impassable during the spring mud season due to a high water table (VAT, 1991). The road section consists of two feet gravel on a silty sand subgrade (24% to 43% passing the number 200 sieve). The pumping action of the traffic, in the range of several hundred cars per day, had caused contamination of the gravel road with fine materials. Beginning in 1990, road crews placed 2 inch square tire shreds under a 330 ft long section. Two hundred cubic yards of tire shreds were used (25% large shreds and 75% small shreds). The base material and six inches of subgrade were removed with a backhoe. The tire shreds were placed with dump trucks and leveled in a 9 inch to 12 inch course with the backhoe prior to replacing the gravel. It was estimated that the compacted shred layer was 6 to 8 inches thick. One year after construction, the section was dry and free of ruts although some minor cracks were observed. The surfaces just north and south of the project area were wet, rutted and marked with cracks and boils. It was concluded that the tire shreds prevented the capillary rise of groundwater and improved the drainage conditions of the gravel layer. The success of this small section led the town to rebuild the remaining rutted section over the next three years using a total of 65,000 tires. The town paid $1.00 per cu·yd for tire shreds while local gravel costs were $3.85 per cu·yd.

The Vermont Agency of Transportation tested the use of tire shreds in September, 1990 to flatten a sloping embankment from 1:1.5 to 1:3 in Middlesex, Vermont. An estimated 2738 cu·yd (104,600 shredded tires) were placed as side slope at a height of 18 feet and eliminated the need for a guardrail. Another project to stabilize a severely eroding bank along River Road in Arlington, Vermont used 185,000 tires to construct a 400 ft long retaining wall. Construction of the wall was scheduled for August and September, 1994. The project was expected to cost $210,000 (Grodinsky, 1994).

2.4.2 Field Performance of Tire Shreds in Minnesota

The Minnesota Pollution Control Agency (MPCA) has documented over 33 sites (1986 through 1994) throughout the state which have used over 160,000 cu·yd of shredded tires (about 4.5 million tires). Over half of these projects are privately owned driveways and roads, eight are city and township roads, five are county roads, and two are Division of Natural Resources forest roads. A few of the projects used shredded tires for purposes other than in road fills. One project in downtown Minneapolis used the lightweight tire shreds as a fill material to support a park and landscaping above an underground parking lot. Another project used tires for erosion control in a fly ash pond. At another site, tire shreds were used as lightweight fill over an existing water main (MPCA, 1994, MnDOT, 1994).

2.4.3 Use of Tire Shreds to Cross Boggy Area

The Southeast Chester Refuse Authority in Pennsylvania was confronted with a problem of road construction over soft soil for movement of equipment from the landfill to the storage sheds (Biocycle, 1989). They placed an 18 inch layer of tire shreds (2x2 inch) along a 525 feet section of roadway passing over a boggy area, without compaction or any other treatment. It has been reported that the section containing tire shreds drains well and provides a good riding surface.

2.4.4 Test Embankment Containing Shredded Tires

The University of Wisconsin-Madison, in cooperation with the Wisconsin DOT, has conducted a limited field experiment to determine the feasibility of incorporating shredded tires in highway embankment (Edil et al., 1990 and Bosscher, et al., 1992). They constructed a 16 feet wide and 6 feet high test embankment consisting of ten different sections, each 20 feet long, using locally available soil and shredded tires in a number of different ways, including pure tire shreds, tire shreds mixed with soil, and tire shreds layered with soil. They also varied the embankment configuration for different sections of embankment to determine the optimum slope. A geotextile was placed around the tire shreds to serve as a separator between the embankment body and the surrounding materials. The embankment was constructed parallel to the access road of a sanitary landfill and exposed to heavy incoming truck traffic. Compaction was done using a sheepsfoot roller with vibratory capability. Field observations during construction indicated that handling and placement of tire shreds was not a problem and a back hoe was appropriate for spreading the material because tracked equipment could easily maneuver on tire shreds. Neither vibratory nor static compaction significantly improved compaction of tire shreds, however, non-vibratory compaction was found more appropriate and compacted field unit weight varied from 20 to 35 pcf, depending upon shred type and size. Based on construction and initial post construction evaluation, Edil, et al. (1990) reported that construction of embankments using tire shreds did not present any unusual problems. Leachate characteristics indicated little or no likelihood that shredded tires would affect groundwater. The main problem is reportedly related to control of compressibility. A two-year monitoring and evaluation program of the test embankment supports the use of properly confined tire shreds as a lightweight fill in highways. After an initial adjustment period, the overall road performance was similar to most gravel roads. It was observed that embankment sections having 3 ft of soil cap performed better and had less plastic deformation than those having 1 ft of soil cap. The study concluded that soil and tire shred mixtures have performance similar to pure shreds sections with thicker soil caps.

2.4.5 Use of Tire Shreds on Interstate 76 in Colorado

The Colorado Department of Transportation has experimented with the use of shredded tires as lightweight fill material (Lamb, 1992). Shredded tires have been used on a 200 ft portion of Colorado's new Interstate 76, a four-lane highway that connects west Denver to Nebraska. More than 400,000 shredded tires with four-inch nominal size have been used in a 5 ft fill. The tire embankment has been instrumented to monitor the long term performance of the fill.

2.4.6 Wyoming Slide Area

The first local use shredded tires for lightweight fill was a $1.8 million road construction project scheduled to start in August, 1994. This project was expected to consume more than

500,000 shredded tires for lightweight fill in the Double Nickel Slide Repair site on WYO 28 between Lander and South Pass, which constitutes about half of what is currently stockpiled in the state. During the past two years tire recyclers have processed more than 1 million tires throughout Wyoming and Kansas with mobile tire shredders. The majority of the tires have been stockpiled at landfills awaiting a recycling or beneficial use. Each participating landfill will receive half of the $4.00 per cu·yd being paid for the product. Twelve thousand cubic yards of landfill space estimated to be worth $38 per cu·yd will be saved. The lightweight fill embankment will be constructed in a 0.2 mile portion of the roadbed where it passes the slide with tire shreds as primary fill overlaid with soil and ash layers (STN, 1994). Sliding problems have been caused by the unstable subsurface geology and the presence of a high water table and a spring that contribute to the problem. The use of the lightweight fill should relieve some of the downward pressure on the unstable subsurface and reduce the potential for continuing sliding. The replacement of 150,000 cu·yd of unstable earth and rock that will be excavated away from the slide area with tire shreds will generate a 70% weight reduction (about 15,000 tons) in the area. The tire shreds will form a four to five feet deep supporting layer under the road's subbase. The subbase will consist of two feet of conventional pit-run gravel, overlaid by a foot of crushed base and six inches of asphalt pavement. The bottom or "toe" of the slide will be reinforced with heavy iron ore from an old open pit mine. The spring will be channeled from underneath the slide area by a culvert. Underdrains will also be used to reduce subsurface moisture (WYO, 1994).

2.4.7 Route 199 in Virginia

Virginia disposed of nearly 7% of its stockpile of 25 million tires by shredding 1.7 million for use as a highway embankment fill near Williamsburg, making it the largest reported use of waste tires in a structural fill in the U.S. The project consisted of two ramp embankments (525 ft long and 263 ft long, respectively) with heights up to 20 ft for a future interchange on Virginia Route 199.

The largest tire shredding operation in the state is located only 31 miles from the site. The 100,000 cu·yd embankment was constructed during the summer of 1993 with equal proportions by volume of shredded tires and silty sand borrow material. The rubber soil was prepared by spreading alternate 6 in. layers of shredded tires and soil, mixing with a grader and then compacting with three passes of a sheepsfoot roller. An average compacted unit weight of 71.3 pcf was achieved. The resulting mix had a substantially interlayered structure. A thorough mix of tire shreds and soil appeared difficult to achieve under field conditions. The maximum shred size allowed in the fill was 10 in. long and the maximum shred area was 40 sq·in. It is estimated that 90 percent of the tire shreds met the material specifications. Construction advanced at a normal rate comparable to conventional materials. A 5 ft soil cap and a 5 ft uncompacted soil surcharge were placed on the embankment to induce complete settlement over time. Thick layers of soil cover the sides of the embankment. The project had an increased cost over traditional materials but the waste tire program agreed to pay the difference from the savings incurred by avoiding the landfill disposal fee.

The embankment was monitored periodically for settlement, vertical stress and temperature during and after construction. The measured vertical stresses at the bottom of the embankment are lower than expected and a proposed explanation involves arching taking place within the embankment. Groundwater is also being sampled to monitor for contaminants that might leak from the tire core.

2.5 Laboratory Studies

A number of lightweight tire shred fill projects include the analysis of the potential environmental hazard posed by the leachates of tire shreds on groundwater. Eight laboratory studies were identified in the literature:

1) a limited laboratory study conducted by the University of Wisconsin-Madison to determine the mechanical properties of rubber and rubber-till mix, and leachate analysis of specimens collected from a tire shred test embankment (Edil, et al., 1990 and Bosscher, et al., 1992),
2) the Minnesota laboratory study on leachates from tire and asphalt materials (MPCA, 1990),

3) the Radian Corporation "Report on the RMA TCLP Assessment Project" (Radian, 1989) prepared for the Rubber Manufacturers Association (RMA),

4) the Envirologic, Inc. "Report on the Use of Shredded Scrap Tires in On-site Sewage Disposal Systems" prepared for the Vermont Department of Environmental Conservation (Envirologic, 1990),

5) the Professional Services Industries, Inc. (PSI, 1994) "Report of Shredded Tire Testing" prepared for Maryland Environmental Services,

6) an ongoing laboratory and field study by University of Maine to determine the properties of tire shreds for lightweight fill (Humphrey, et al. 1992, 1993, 1994, Nickels, 1995),

7) a laboratory study on the properties of tire shreds and rubber-sand mixtures conducted by Purdue University (Ahmed, 1993) and,

8) a laboratory study on the engineering properties of tire shreds and soil mixtures conducted at the University of Wisconsin-Madison (Edil and Bosscher, 1994).

2.5.1 Wisconsin Study

A testing program was carried out at the University of Wisconsin-Madison to analyze the compaction and compression behavior of tire shreds, and the leachates from a test embankment made of rubber-soil (Edil, et al., 1990). Rubber shreds of different sizes alone and mixed with sand were placed in a 6 in. Proctor mold and then loaded using a disk placed on the tire shreds. The load-deformation response of tire shreds indicated that the major part of compression is irrecoverable; but there is some rebound upon unloading. The rebound is nearly the same from one cycle to another. The slope of the recompression/rebound curve is markedly lower beyond a vertical load of about 40 psi. Their test results, on specimens of sand/shred ratios varying from 100% sand to 100% shreds, indicated that compression increases significantly when tire shreds content went beyond 30% by weight of sand. Since the tests were carried out in a 6 in. compression mold with tire shred sizes of 1.5 in. and larger, it is likely that side friction was induced.

Edil et al. (1990) have also reported duplicate EP toxicity and AFS leaching tests performed on tire shred samples by the Wisconsin State Laboratory of Hygiene. The test results indicate that the shredded automobile tire samples show no likelihood of being a hazardous waste. The shredded tires appear to release no base-neutral regulated organics. The tire samples showed detectable, but very low release patterns for all substances and declining concentrations with continued leaching for most substances. By comparison with other wastes for which leach tests and environmental monitoring data are available, tire leachate data indicate little or no likelihood of shredded tires affecting groundwater. Bosscher, et al. (1992) have reported that an overall review of the available leach data and results of the recent leachate tests on samples collected from two lysimeters, installed during construction of the test embankment in December 1989, support their initial conclusions.

2.5.2 Minnesota Study on Tire Leachates

The Minnesota Pollution Control Agency (MPCA) sponsored a study on the feasibility of using waste tires in road subgrades (MPCA, 1990). The laboratory study was performed by the Twin City Testing Corporation (TCTC) of St. Paul, Minnesota, to evaluate the compounds produced by exposure of tires to different leachate environments. They subjected samples of old tires, new tires, and asphalt to laboratory leachate procedures at different conditions, i.e., at pH 3.5, pH 5.0, approximately neutral pH (with 0.9% sodium chloride solution), and pH 8.0. They also conducted field sampling. As a result of elaborate testing and analysis, TCTC reached the following conclusions (MPCA, 1990):

- metals are leached from tire materials and the constituents of concern are barium, cadmium, chromium, lead, selenium, and zinc;
- Polynuclear Aromatic Hydrocarbons (PAHs) and Total Petroleum Hydrocarbons are leached from tire materials in highest concentrations under basic conditions;
- asphalt may leach higher concentrations of contaminants of concern than tire materials under the same conditions;
- drinking water Recommended Allowable Limits (RALs) may be exceeded under "worst-case" conditions for certain parameters;

- co-disposal limits, EP Toxicity limits, and TCLP criteria are generally not exceeded;
- potential environmental impacts from the use of waste tires can be minimized by placement of tire materials only in the unsaturated zone of the subgrade.

2.5.3 Radian Corporation

The Rubber Manufacturers Association (RMA) authorized Radian Corporation to assess what levels of chemicals, if any, are leached from representative RMA products using EPA's Toxicity Characterization Leaching Procedure (TCLP) and the EP Toxicity Test. Tests were performed in seven products from tire manufacturers (1 sample of truck tires, 2 of light truck tires and 4 of passenger auto tires) and nine products from other industries affiliated to RMA. The tire samples were ground to appropriate size (<0.4 in.) and tested. The purpose of the TCLP as well as the EP toxicity protocol is to determine the whether a waste has the potential to pose a significant hazard to human health or to the environment due to its propensity to leach toxic compounds into the groundwater. The TCLP listed chemicals and their regulatory limits are listed in Table 2.1.

A known amount of the material to be tested is place in a containment jar which is filled with a leaching solution at a pH of 4.9 for alkaline wastes and pH of 2.9 for acid wastes. The sealed jar is attached to a rotary tumbler spinning at a rate of 30 rpm for 16 hours for the TCLP or 24 hours for the EP Toxicity Test. After rotation the leachate is forced through a filter, effectively separating the sample for the leaching medium without exposure to air (Radian, 1989). The results of the TCLP study indicate that none of the products tested, cured or uncured, exceeded TCLP regulatory levels. Most compounds were found at trace levels (near method detection limits) from ten to one hundred times less than regulatory levels and similar results were found for ground and unground samples. The EP toxicity results were compared with the TCLP results and it was found that both leachate methods give comparable results.

2.5.4 Envirologic, Inc.

The Vermont Department of Environmental Conservation commissioned Envirologic, Inc. to investigate the use of scrap shredded tires to replace crushed stone in on-site sewage disposal systems. The review of leachate studies (including Radian, 1989) indicates that leachate from tire shreds would not be a significant source of groundwater pollution and that the physical characteristics of tire shreds would allow them to serve as in on-site disposal systems.

Table 2.1 TCLP listed chemicals and regulatory levels (Radian, 1989)

Contaminant	Regulatory Level (mg/L)	Contaminant	Regulatory Level (mg/L)
Volatile organics		Semivolatile Organics	
Acrilonitrile	5.0	o,m,p-cresols (ea)	10.0
Benzene	0.07	Hexachlorobenzene	0.13
Bis(2-chloroethyl)ether	0.05	Hexachloroethane	4.3
Carbon Disulfide	14.4	Nitrobenzene	0.13
Carbon thetrachoride	0.07	Pentachlorophenol	3.6
Chlorobenzene	1.4	Phenol	14.4
Chloroform	0.07	Pyridine	5.0
1,2-Dichlorobenzene	4.3	2,3,4,6-Tetrachlorophenol	1.5
1,4-Dichlorobenzene	10.8	2,4,5-Trichlorophenol	5.8
1,2-Dichloroethane	0.40	2,4,6-Trichlorophenol	0.30
1,1-Dichloroethylene	0.10		
2,4-Dinitrotoluene	0.13		
Hexachlorobutadiene	0.72		
Isobutanol	36.0		
Methylene Choride	8.6		
Methyl Ethyl Ketone	7.2	Metals	
1,1,1,2-Tetrachloroethane	10.0	Arsenic	5.0
1,1,2,2-Tetrachloroethane	1.3	Barium	100
Tetrachloroethylene	0.1	Cadmium	1.0
Toluene	14.4	Chromium	5.0
1,1,1,-Trichloroethane	30.0	Lead	5.0
1,1,2,-Trichloroethane	1.2	Mercury	0.20
Trichloroethylene	0.07	Selenium	1.0
Vinyl Chloride	0.05	Silver	5.0

Envirologic also analyzed the results of samples taken from a tire pond for the storage of whole tires in Hamden, Connecticut. Most of the compounds tested were below detection limits. Iron was the only compound tested which was occasionally above groundwater standards (Envirologic, 1990). The presence of iron could be attributed to rusting steel tire belts.

2.5.5 Professional Service Industries, Inc.

Professional Service Industries, Inc. ran a series of tests on shredded tires for Maryland Environmental Services which provided the testing material. Due to the large particle diameter of the shredded tire, the samples were manually reduced to pass a ¾ in. sieve, but be retained on the No. 4 sieve. The percent loss after the Modified Leachate Compatibility Test was 0.7% (see Table 2.2). Permeability was 1.2 cm/sec (0.5 in./sec) at a 30.5 pcf compacted density. The softening point (there is no uniform melting point) was recorded at 243°F and the spontaneous combustion point (flashpoint) was recorded at a temperature higher than 610°F even though volume loss was observed at 554°F and 610°F due to the heterogeneous composition of the material.

Table 2.2 PSI Test Results

Contaminant	Concentration Level (mg/L)	Contaminant	Concentration Level (mg/L)
Benzene	BRL	Pyridine	BRL
Carbon thetrachoride	BRL	Tetrachloroethylene	BRL
Chlorobenzene	BRL	o,m,p-cresols (ea)	BRL
Chloroform	BRL		
1,4-Dichlorobenzene	BRL		
1,2-Dichloroethane	BRL	Metals	
1,1-Dichloroethylene	BRL	Arsenic	<0.10
2,4-Dinitrotoluene	BRL	Barium	0.20
Hexachlorobenzene	BRL	Cadmium	<0.10
Hexachloro-1,3-butadiene	BRL	Chromium	<0.10
Hexachloroethane	BRL	Lead	<0.10
Methyl Ethyl Ketone	BRL	Mercury	<0.001
Nitrobenzene	BRL	Selenium	<0.10
Pentachlorophenol	BRL	Silver	<0.10

The results show that the concentrations were Below Reporting Limits (BRL) for all organics and the concentration for metals were BRL except for barium. The concentration levels are below regulatory levels for all compounds analyzed.

2.5.6 University of Maine

Currently the largest research on tire shreds properties and applications is being conducted at the University of Maine. Large direct shear tests have been run on tire shreds of different sizes. Earth pressure coefficients have been calculated by filling a 12 in. PVC pipe with tire shreds and applying a vertical load; the deformation of the tube has been measured with strain gages and the horizontal pressure of the tire shreds on the tube has been calculated from the strains. Environmental effects are being assessed by performing TCLP tests on steel-belted and fiberglass belted tire shreds and by studying the leachate from the tire shred fills with leachate collection systems and monitoring wells. The thermal properties of tire shreds are also being measured in the laboratory.

Field tests include the use of tire shreds as an insulation layer to limit freezing and subsequent thawing of soils underlying the pavement structure, tire shreds as lightweight embankment fill, tire shreds as wall backfill and evaluation of leachates from tire shreds fill under groundwater level in three soils (clay, till and peat).

2.5.6.1 Water Quality Testing

The Dingley Road Tire Shred Test Project is located in the town of Richmond, Maine. The road follows the northeast shoulder of a broad, flat ridge that trends northwest-southeast. During summer and fall, no standing water or wet areas are evident near the test site. However during the spring melt the generally flat topography leads to poor drainage and areas of standing water (Humphrey and Katz, 1995). The native soils range from gray silty clay to gray-brown silty gravelly sand. Glacial till or bedrock can be found at depths ranging form 9 to 18 ft. The water table during the summer and fall is 3 to 10 ft below the ground surface.

The test site is 950 ft long and was divided into five tire shred sections and one control section. Two different layer of tire shreds (0.5 ft and 1 ft thick) were used to investigate the thickness required to provide adequate insulation. Three different layers of granular soil cover (12 in., 18 in. and 24 in. thick) were placed over the 2 in. nominal size tire shreds to determine the thickness required to provide a stable riding surface (see Figure 2.2). Approximately 20,000 tires were used in his project.

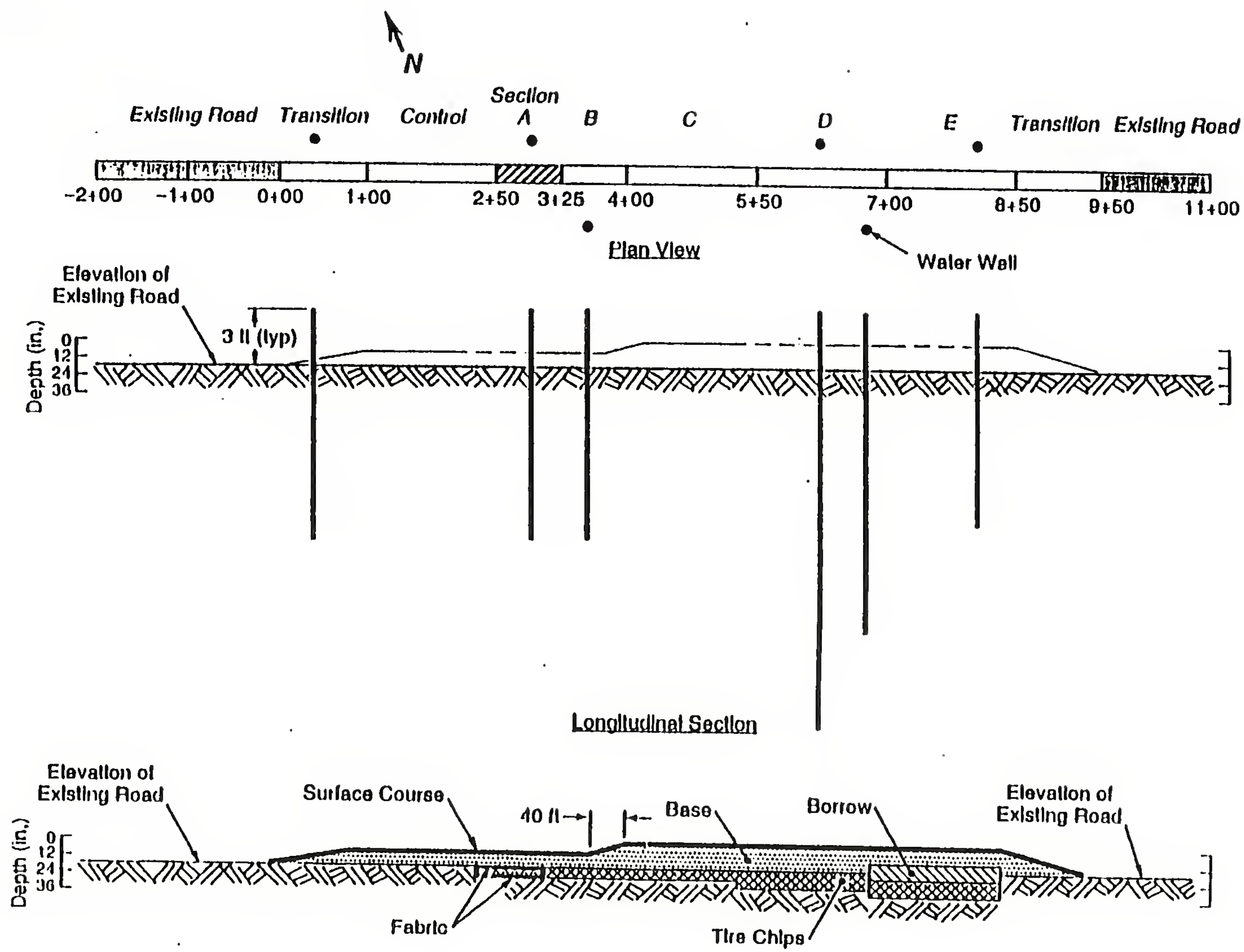


Figure 2.2 Dingley road test site (Humphrey and Katz, 1995)

Groundwater monitoring wells were installed at the six location shown in Figure 2.2. One well was adjacent of the control section (no tire shreds) and the other wells were adjacent to tire shred sections. The wells consist of a 2 in. diameter Schedule 40 PVC pipe with a cap glued to the bottom and slots cut in the bottom 1.6 to 3 ft. The pipe was placed in a 5 in. diameter hole drilled by a trailer mounted power auger. The slotted lower portion was surrounded by concrete sand. An impermeable seal was formed on top of the sand with 1 ft to 2 ft thickness of bentonite balls to prevent surface water from reaching the slotted tip. The remainder of the hole was filled with native soil. The well at station 3+42 had no bentonite seal.

The wells have been sampled three times during the two years after construction. The water samples were obtained with a one liter HDPE bailer. Prior to sampling three well volumes were bailed to minimize contamination from PVC pipe leachates. The sample was taken from the groundwater that recharged the well. Three samples were taken from each well:

1) filtered through a 0.3 micron filter and preserved with nitric acid (1.5 mL/L),
2) unfiltered and preserved with nitric acid (1.5 mL/L) and,
3) unfiltered with no acid.

The samples were stored in HDPE bottles and refrigerated to minimize degradation. Tests were performed on both acid preserved filtered and unfiltered samples, however, drinking water standards are applicable to filtered samples. Unfiltered sample provide supplementary information but should not be compared to drinking water standards.

The results indicate that barium, cadmium, chromium, copper, lead and selenium were present in trace amounts or were below detection levels and, more importantly, their concentrations were well below drinking water standards.

The test on substances with secondary drinking water standards such as aluminum, iron, manganese, zinc, chloride, sulfate and dissolved solids, which have an aesthetic concern, were below the applicable standards except for manganese. The manganese concentration was above the secondary standard for the control well and three of the wells adjacent to tire shreds. It appears that manganese is present in the natural groundwater since it was also detected in the control well.

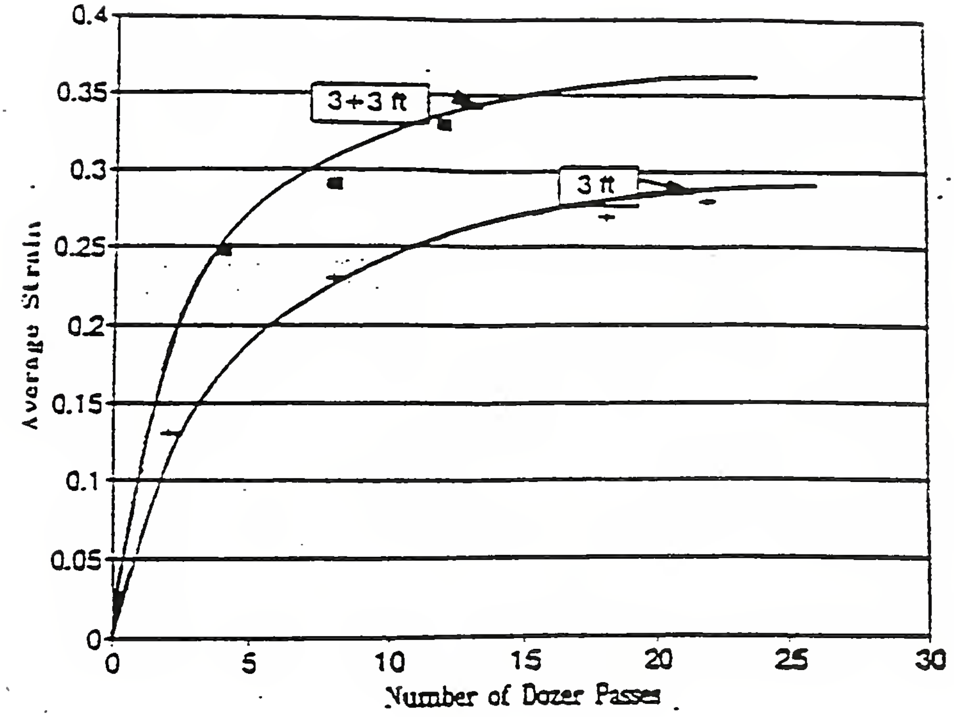


Figure 2.3 Average strain versus number of dozer passes (after Drescher and Newcomb, 1994)

The test on substances with no drinking water standards such as calcium, magnesium, sodium and the conductivity, hardness, alkalinity, pH, biological oxygen demand (BOD) and chemical oxygen demand (COD) tests showed the following results. The levels of calcium and magnesium indicate that the water is hard as was confirmed by the hardness results. The levels of calcium, sodium and chloride and conductivity are high in three wells which may be due to deicing road salt and calcium chloride used for dust control. The alkalinity results are typical for drinking water from bedrock aquifers in Maine. The pH is neutral and acceptable for drinking water. The BOD and COD are low and acceptable for drinking water.

The study shows that for the first 28 months since construction, no significant levels of inorganic contaminants have migrated from the tire shreds to the monitoring wells. However, the time since construction is too short for some substances to migrate, therefore its too early to make definite conclusions on the long term effects of tire shreds on groundwater quality . This study was limited to inorganic contaminants. Two additional ongoing studies will make it possible to reach definite conclusions (Humphrey and Katz, 1995).

<u>2.5.6.2 Tire Shreds as Insulation Under Gravel Surfaced Roads</u>

The main objective of the Dingley Road project was to determine the performance of 1 inch tire shreds as an insulating layer beneath a gravel surfaced road. About 20,000 shredded tires were used in this project. Five test sections with two different layers of tire shreds (6 in. and 12 in. thick)) and three overlying gravel covers (12 in., 18 in. and 24 in. thick) and a control section with no tire shreds were studied. The purpose of the tire layer was to minimize penetration of freezing temperatures into the underlying frost susceptible soils. The tire shred layers were compacted with 6 passes of a vibratory roller. One section was completely enclosed in a nonwoven geotextile. An extensive monitoring program consisting of vertical strings of thermocouples (to measure temperature) installed at two locations, resistivity gages (to determine if the soil is thawed or frozen and monitor the location of the freezing front) and two frost free benchmarks were installed to evaluate the thermal behavior of the project.

After construction was completed in September, 1992, temporary distress due to rutting under repetitive passes of loaded double and triple axle dump trucks was observed in two sections where 12 in. of soil cover on 6 in. of tire shreds were used. In subsequent months these sections experienced minimal rutting under the same loading, however elastic deflections could be detected visually and thin cracks were observed in the wheel paths. It appears that these sections tended to strengthen in the first month after placement, perhaps due to consolidation of both the gravel and tire shred courses. Cracks were not evident in the remaining sections. Total depths of frost penetration up to 5 ft under the existing road and control section were measured while the sections with a 6 in. tire shred layer had frost penetration of around 26 in. and frost did not penetrate through the 12 in. tire shred layer. This indicates that the tire shred layer is effective in reducing the depth of frost penetration (Humphrey and Eaton, 1995). At the end of the study it was established that the control section had heaved 6 in., while the insulated sections had heaved 1 to 2 in. (CE, 1994).

2.5.6.3 Compressibility

Humphrey, et al. (1992 and 1993) have reported the engineering properties of 3-inch size tire shreds from three suppliers. Their tests showed that tire shreds are composed of uniformly graded gravel sized particles that absorb only a small amount of water. Their compacted density is 38.6 to 40.1 pcf. The shear strength was measured in a large scale direct shear apparatus. The reported strength angle and strength intercept ranged from 19° to 25° and 1.11 to 1.67 psi, respectively. Compressibility tests showed that tire shreds are highly compressible on initial loading but that the compressibility on subsequent loading/unloading cycles is less. The measured horizontal stress indicated that the coefficient of lateral earth pressure at rest varied from 0.26 for tire shreds with a large amount of steel belt exposed at the cut edges to 0.47 for tire shreds obtained from glass belted tires.

A major concern in using tire shreds in embankments are the large settlements (about 10 to 11 in.) observed in various field and laboratory studies (e.g., Geisler, et al., 1989; Edil, et al., 1990; Lamb, 1992; and Read, et al., 1991; Ahmed, 1993). Holtz (1989) comments that no research reported in the literature discusses limiting settlements of highway embankments. NCHRP (1971) has reported that post-construction settlements during the economic life of a roadway of as much as 1 to 2 ft are generally considered tolerable provided they: 1) are reasonably uniform; 2) do not occur adjacent to a pile-supported structure; and 3) occur slowly over a long period of time. Post-construction settlements of shredded tire embankments can be reduced by placing a thick soil cap over tire fills and increasing the confining pressure, or by using a rubber-soil mix instead of tire shreds alone. The detrimental effects of anticipated excessive settlements can be reduced by using tires under flexible pavements only and letting the tire shreds compress under traffic before placing the final surface course.

Several recent studies have included compressibility testing of tire shreds and tire shred/soil mixtures. The results have been used to determine Poisson's ratio (μ), Young's modulus (E) and resilient modulus for analytical models of pavement deflection. Manion and Humphrey (1992) tested 2 in. minus tire shreds and tire shred/soil mixtures. A 12..5 in. long schedule 40 PVC pipe with a nominal 12 in. internal diameter supported on a 0.75 in. thick steel plate was used in the experiments. The pipe was filled with the compacted testing material. Strain gages were attached horizontally to measure circumferential strain which was correlated to the circumferential stress exerted by the sample. Vertical strain gages were calibrated to measure the vertical force transmitted to the pipe through friction. The vertical load was applied through a circular plate using an Instron 4204 Universal Testing Machine at a rate of 0.5 in. per minute. Three groups of tests were done on tire shreds only, mixture of 50% (by weight) of 2 in. minus MDOT type D gravel and tire shreds, and mixtures of 25%, 50% and 75% (by weight) of ¾ in. minus gravel with tire shreds. The result of the tire shreds test indicate high compressiblity at low stresses and decreasing compressibility with increasing stresses. Their research indicate that below 23 psi the ratio between horizontal and vertical stresses ($K_0=\sigma_h/\sigma_v$) is relatively low and above this point it tends to 1. The initial compression may represent a reduction in the voids. At higher vertical stress the behavior may be controlled by the deformation response of the rubber itself. The results for all testing materials can be seen in Table 2.3.

Humphrey and Manion (1992) also studied tire shreds from three different suppliers results are summarized in Table 2.4.

Table 2.3 Tire shred compressibility (Manion and Humphrey, 1992)

% gravel	$K_{0\ average}$	$\mu_{average}$	$E_{secant\ (avg)}$	ε_{vert} (σ_v= 50 psi)
0	0.44	0.30	18.1	39%
50% 2 in.	0.34	0.25	20.9	25%
25% ¾ in.	0.32	0.24	30.2	33%
50% ¾ in.	0.33	0.25	35.6	27%
75% ¾ in.	0.54	0.35	71.7	17%

The value of E was determined from the unload-reload cycles because it represents the field deformation behavior under vehicle loading. The values of K_0 and μ decreased with increasing amounts of exposed steel belts.

Table 2.4 Summary of compressibility results (Humphrey and Manion, 1992)

Supplier	$K_{0\ average}$	$\mu_{average}$	$E_{average}$ (psi)
Pine State Recycling	0.41	0.28	165
Palmer Shredding	0.26	0.20	161
F&B Enterprises	0.47	0.32	112

Ahmed (1993) measured the compressibility of tire shreds and tire shred-soil mixtures. A 12 inch stainless steel compression mold with a 1.25 in. thick steel base was used. The mold could be split into two 6 in. height halves for smaller samples and to reduce side friction. A MTS soil testing system was used for loading and to measure the load-deformation response. Tire shreds ranging from 0.5 to 2 in. were tested. The rubber-soil mixtures were produced by combining Ottawa sand (A-3, AASHTO) or Crosby Till (A-4, AASHTO) and tire shreds. The tire shred and the Crosby-tire mixtures were compacted using the following compacting efforts: no compaction, 50% standard Proctor, standard Proctor and modified Proctor compactive energy. The rubber-sand mixtures were compacted using a vibratory table. Testing was done on 0.5, 1.0, 1.5 and 2.0 in. tire shreds and rubber-soil mixtures described above with various tire shred/soil ratios. Test data were plotted as vertical strain versus logarithm of vertical stress. The following conclusions were obtained:

1. Tire shreds are highly compressible. The air voids are reduced by increasing the overburden pressure, which in turn reduces the compressibility.
2. Modified Proctor compacted samples of various sizes show little variation in the load deformation response. Higher vertical strains were measured in 0.5 in. tire shreds than in the larger size shreds compacted at 50% standard.
3. Standard and modified Proctor compacted samples produced similar compression curves. Increased vertical strains were observed in the first loading cycle for 50% standard Proctor compacted samples. The subsequent unload-reload cycles showed little effect form the initial compactive effort.
4. Compressibility was lowered by increasing the sand to tire shreds ratio. A small vertical strain of about 3% for the third loading cycle was measured for a tire shred/soil ratio of 37%. This mixture yields a compacted unit weight of about two thirds that of soil.

Drescher and Newcomb (1994) studied the compressibility of uncompacted, 2 in. minus tire shreds. A 25.7 in. high steel cylinder with an interior diameter of 29.3 in. and a wall thickness of 0.4 in. was used. Circumferential strains (ε_θ)were measured by averaging the measurements made by four horizontal strain gages attached at a 12 inch height from the bottom of the cylinder. The resulting horizontal stress (σ_θ) was calculated as:

$$\sigma_h = \frac{t}{r}\varepsilon_\theta E \qquad (2.1)$$

where t is the wall thickness, r is the container radius and E is the modulus of elasticity of the container material. The load (P) was applied at a rate of 0.2 in./min through two 0.25 in. thick bearing plates. The vertical stress was calculated as:

$$\sigma_v = \frac{P}{A} \qquad (2.2)$$

Where A is the area of the bearing plates. The vertical strain was calculated as

$$\varepsilon_v = \frac{\Delta H}{H_0} \qquad (2.3)$$

where ΔH is the change in sample height and H_0 is the initial sample height.

Test results indicated that the compressibility of tire shreds is higher at low vertical stress levels and decreases significantly with increasing vertical stress. The coefficient of lateral earth pressure (K_0) was calculated to determine Young's modulus and Poisson's ratio by plotting horizontal versus vertical stress and calculating the slope of the curve. A bilinear relationship was observed: for vertical stresses below 25 psi an average K_0 of 0.4 was computed and for vertical stresses above 25 psi an average K_0 of 0.96 was obtained.

Young's modulus (E) and Poisson's ratio (μ) were calculated assuming that the tire shred fill was isotropic and as a transversely isotropic material. In the isotropic analysis E=113 psi and μ=0.45, in the transverse isotropic analysis E=236 psi and μ=0.43. The compressibility index (C_c=0.5) and swelling index (C_s=0.27) were also calculated.

Excessive deformation of underlying soils is an important factor in the deflection and long term performance of pavement structures. Ahmed (1993) determined the resilient modulus (M_R) for a coarse grained and a fine grained soil at different tire/shred/soil ratios. The resilient modulus is a measure of the elastic properties of soils under repeated loading.

The tests were performed in accordance with AASHTO T472-82 (1986), "Standard Method of Test for Resilient Modulus of Subgrade Soils". Four inch diameter samples of 0.5 in. and 0.75 in. tire shreds mixed with Ottawa sand and Crosby till , respectively were tested with the MTS Soil Testing System. The tire shed/soil ratios varied form 0 to 100%. The results are summarized in Table 2.5. The resilient modulus is related to the stress level by the following equation:

$$M_R = A\theta^B \qquad (2.4)$$

where θ is the bulk stress ($\theta=\sigma_1+\sigma_2+\sigma_3$),

σ_1,σ_2 and σ_3 are the three principal stresses

A and B are regression constants.

The major conclusions of this study were:

1. The resilient modulus decreases up to 80% or more with increasing tire shred to soil ratios. Rubber-Crosby mixes showed greater reductions than rubber-sand mixes.

2. The resilient modulus for rubber-soil mixes was not significantly affected by tire shred size.

Table 2.5 Resilient modulus test results (Ahmed, 1993)

Test	Shred Size (in.)	Compaction Method	% Tire shreds	Soil type	A	B	r^2
AH01	None	Vibratory	None	Sand	1071.5	0.84	0.95
AH02	0.5	Vibratory	15	Sand	524.8	0.83	0.95
AH03	0.5	Vibratory	30	Sand	269.2	0.90	0.67
AH04	0.5	Vibratory	38	Sand	42.7	1.15	0.89
AH05	0.5	Vibratory	50	Sand	38.9	0.83	0.84
AH06	0.5	Vibratory	100	Sand	36.3	0.55	0.74
AH07	0.75	Vibratory	38	Sand	34.7	1.21	0.92
AH08	None	Standard	None	Crosby Till	3162.3	0.49	0.83
AH09	0.5	Standard	15	Crosby Till	53.7	1.15	0.91
AH010	0.5	Standard	29	Crosby Till	61.7	0.91	0.94
AH011	0.5	Standard	38	Crosby Till	55.0	0.67	0.95

2.5.6.4 Compactability

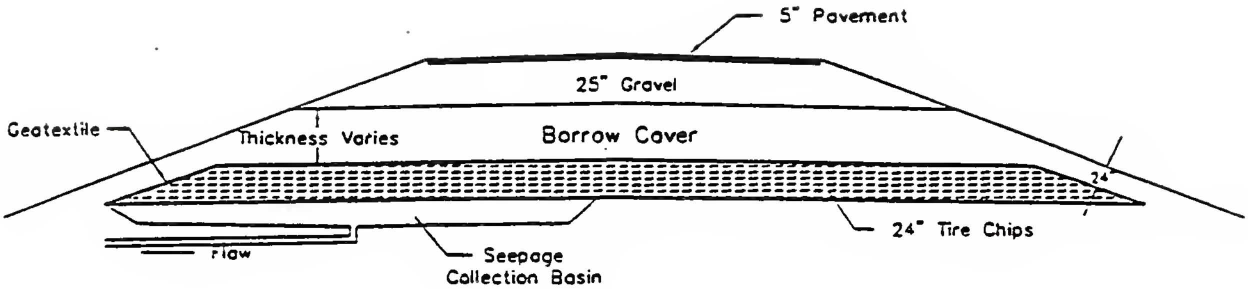
Field studies on the compaction of tire shreds are limited. Upton and Machan (1993) reported on the use of a D-8 dozer for compacting 3 ft lifts of a tire shred embankment in Oregon. A maximum size tire shred of 24 in., 80% passing the 8 in. sieve and 50% greater than 4 in. were the specifications used in this project and taken from work done in Minnesota. The dozer moved back and forth longitudinally until the whole tire shred section had been compacted by one track width of the dozer. Transverse back and forth cycles were done subsequently using the same criteria. Full longitudinal and transverse coverage was considered one pass. Each 3 ft lift received at least 3 compaction passes. A compaction pass was made with a lighter D-6 dozer on a 3 ft test lift. It was visually determined that the tire shred fill was looser after compaction with the D-6 dozer and the D-8 was used to finish the compaction of the layer.

A project in Eden Prairie, Minnesota also used a D-8 dozer for compaction purposes but the compaction pattern and number of passes required was not reported (Engstrom and Lamb, 1994).

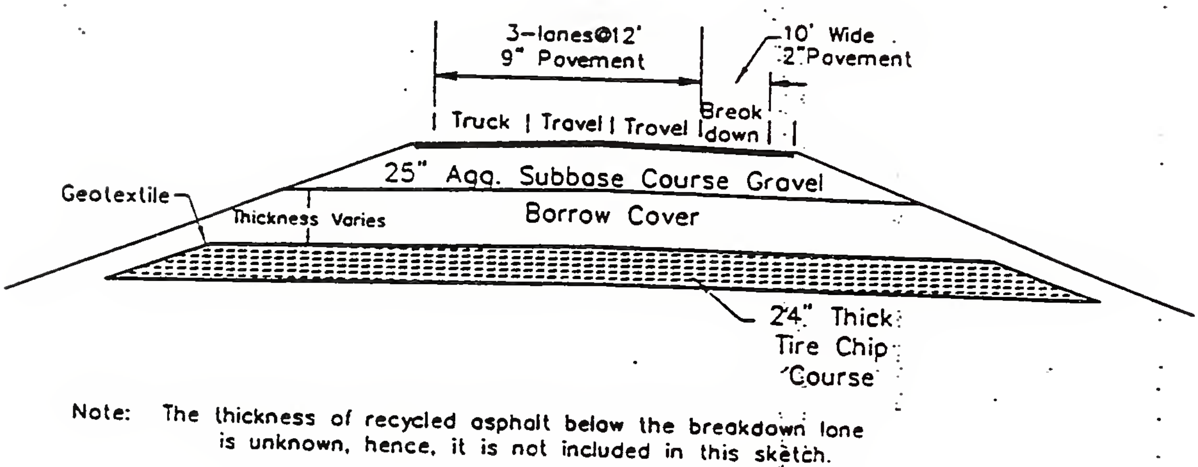
A section of a road near Mora, Minnesota was constructed with two 3 ft lifts of tire shreds (size not reported) as subgrade. A 27 ton D7F Caterpillar dozer rolling at 5 mph was used to compact the two tire shred layers. The dozer made three longitudinal back and forth trips to cover the whole section with one track of the dozer. Three trips were equivalent to one pass. The first 3 ft lift received a total of 22 passes and the second 3 ft lift received 12 passes. The average settlement of the entire section after a certain number of passes was divided by the original layer thickness to compute the average vertical strain which was plotted against the number of dozer passes (see Figure 2.3). Drescher and Newcombe (1994) concluded that maximum compacted unit weight could be achieved after 15 passes on both lifts. Settlements of approximately 30% and 37% were recorded for the first and second 3 ft layers, respectively.

Two field projects were implemented in Maine to study the effects of tire shreds as subgrade fill in paved roads (Nickels, 1995). The North Yarmouth project is located in Route 231 in North Yarmouth, Maine. The TWP31-MD project is located on a section of Route 9 near Calais, Maine. The North Yarmouth project had a seepage collection system under the tire shred course to study the leachate of the tire shreds but the results are not available yet. Cross sections of both projects can be seen in Figure 2.4.

The North Yarmouth project measures 400 ft and was divided into four sections with 2 ft thickness of tire shreds under the soil cover. The tire shred course was completely enclosed in a non-woven geotextile which was used as a separator from the underlying and overlying soils and did not provide reinforcement. Approximately 100,000 waste tires were used. Section 1 contained 12 in. maximum size tire shreds and a 30 in. soil cover. Section 2, 3 and 4 contain 3 in. maximum size tire shreds and an overlying soil cover of 42 in., 54 in. and 30 in. respectively. A 20 ft long transition was built between each section. A control section was built with



North Yarmouth-typical cross section



TWP31-MD typical cross section

Figure 2.4 North Yarmouth and TWP31-MD typical cross sections (Nickels, 1995) standard MDOT specifications to serve as a baseline for comparison with the data collected from the tire shred sections. The travel lanes were finished with a 5 in. bituminous pavement overlay.

In Sections 2, 3 and 4, the 3 in. tire shreds were placed and compacted in two 12 in. thick lifts. Spreading of the tire shreds was done with a wide-track mounted bulldozer which was able to achieve the specified grade within ±1 in. In Section 1 the 12 in. size tire shreds had to be spread in a single 17 to 24 in. lift because the large size shreds tended to interlock due to exposed steel belts. The final grade was reached by placing 3 in. tire sheds.

The effectiveness of the following four types of compaction equipment was investigated:

1. Dynapac CA-25 vibratory smooth drum roller with a static weight of 10 US short tons or 138 lb/in. of drum width
2. Komatsu D41P track mounted bulldozer with a contact pressure of 4.48 psi
3. Loaded 14 cu·yd dual rear axle dump truck
4. Caterpillar CP-443B self-propelled vibratory tamping foot roller with a total operating weight of 7.4 US short tons or 258 lb/in. of drum width.

The effectiveness was determined by measuring the settlement of the tire shred surface on a 50 point grid in each section after every two passes of one of the compaction equipment listed above. One pass was defined as one complete coverage of the entire width of the section by using the specified machine traveling parallel to the center line. The settlement grid points were located at the center line, 11 ft to the right and left and 16 ft to the right and left of the center line at 10 ft stations. The settlement was calculate as the average of these points. Compaction continued until a negligible effect on settlement was achieved. The number of passes required per tire shred section did not exceed the specified maximum of 10. The compaction sequence varied for each section to test the effectiveness of each machine at different stages.

The performance of each compaction device was discussed based on visual observations and field data:

1. Vibratory smooth roller: the compaction data indicated the vibratory smooth roller to be somewhat more effective than the other devices in compacting tire shreds. A

small rebound was observed when used on tire shreds that have already achieved their maximum density.

2. Bulldozer: the bulldozer appeared to be most effective for the first and second passes on the first lift, however its efficiency appears to decline if used later in the sequence. The bulldozer did not cause any rebound.
3. Tamping foot roller: the tamping foot roller had an adequate performance but tended to fluff up the upper 3 in. of the tire shred lift which decreased compacted density.
4. Loaded dump truck: The loaded dump truck proved to be ineffective since its tires sank deeply into the tire shred and fluffed up the tire shreds instead of compacting them. The truck also had several flat tires due to exposed steel belts in the tire shreds and was eliminated from the compaction sequence.

Compacted in-place unit weight was computed for both tire shred sizes and the compacted laboratory unit weight was determined for the 3 in. tire shreds only. Field densities were calculated by measuring the cross section elevation at 10 ft stations in the tire shred sections before and after placing the tire shred lift. The weight of the tire shred fill was obtained from the weight tickets provided by the delivery truck drivers. The resulting compacted in-place unit weights were 43 pcf for the 3 in. tire shreds and 38 pcf for the 12 in. tire shreds. It was estimated that the 3 in. tire shreds had a field water content of 2% resulting in a dry field unit weight of 42 pcf. Laboratory densities were determined in a 12 in. high, 10 in. diameter mold. A modified hammer was used to apply 60% standard Proctor energy to the sample. A four sample average resulted in 37 pcf which is slightly lower than the compacted field unit weight.

The TWP31-MD project is located on a section of Route 9 near Calais, Maine. The average daily traffic (AADT) is 3000 of which 10% correspond to heavy trucks. The tire shred course was encased in a non-woven geotextile which was used as a separator and did not provide reinforcement. This project also measures 400 ft but has a wider section. Four sections were constructed with 2 ft thickness of tire shreds and approximately 200,000 waste tires were used. The soil overlay and bituminous asphalt thicknesses was larger due to the presence of high truck traffic. Sections 1, 2 and 3 contained 3 inch maximum size tire shreds with soil covers of

49 in., 73 in. and 97 in. respectively. Tire shreds with 12 in. maximum size were used in Section 4 with a soil cover of 73 inches.

A control section built under standard MDOT specifications was used as a baseline. Nine inches of bituminous pavement overlay the travel lanes in the entire project. In sections 1, 2 and 3 the 3 in. tire shreds were placed and compacted in two 12 in. thick lifts. The 12 in. tire shreds used in section 4 were also placed in two lifts. A smaller Caterpillar D-3 bulldozer used in this section was better suited to spread the larger size tire shreds and bring it within ±1 in. of the specified grade. The effectiveness of three types of compaction equipment was investigated:

1. Raygo 400 vibratory smooth drum roller with a static weight of 10 U.S. short tons or 238 lb/in. of drum width
2. Caterpillar D6D track mounted bulldozer with a contact pressure of 8.5 psi
3. Ingersoll-Rand SPF-60 self propelled vibratory tamping foot roller with a total operating weight of 21.6 U.S. short tons or 270 lb/in. of drum width.

The effectiveness was determined in a similar manner as in the North Yarmouth project. The compaction sequence was varied for each section. A specified piece of equipment was used for the first six passes, or until negligible settlement was measured and the vibratory smooth roller was used for an additional two passes since it appeared to perform the best in the North Yarmouth project. The data obtained from this project showed that the vibratory smooth drum roller performed equal or better than the dozer for both lifts.

It was also observed that for one lift the vibratory sheepsfoot performed better than the vibratory smooth drum and for the other lift the contrary was observed. This suggests that the effectiveness of these two compactors is similar. The effectiveness of all compactors generally decreases after 6 to 8 passes.

Compacted unit weights were only measured in the laboratory due to difficulties obtaining accurate weight tickets. The average compacted unit weight obtained for the 3 in. tire shreds was 41 pcf.

The final recommendation was that a 12 in. layer of tire shreds should reach final compacted density with 6 passes of a vibratory smooth drum roller or a vibratory tamping foot roller similar to those used in these projects (Nickels, 1995).

2.5.6.5 Pavement Deflections

One of the main objectives of the North Yarmouth and TPW31-MD projects was to determine the amount of soil cover required between the top of the tire shred course and the bottom of the pavement to limit pavement deflections and have acceptable levels of tensile strains.

The performance of the pavement structure under an 18 kip axle load was monitored with a modified Benkelman Beam because it was conjectured that the deformation basin for tire shreds is shallower and more extense than for conventional materials (Manion and Humphrey, 1992). Among the modifications made were:

1. A beam length of 16 ft in front of the fulcrum.
2. Dial gages were mounted along the arm of the beam to define the shape of the deflection basin.
3. A piece of angle iron was used instead of a bolt for the fulcrum to reduce friction induced by the additional weight of the longer beam.
4. A counterbalance weight was added behind the fulcrum point to balance the beam to ensure that the wheel probe barely rests on the pavement surface.
5. The steel components were painted white to minimize thermal expansion effects due to sunlight during testing.

The Benkelman beam was used in 8 locations in each section. The maximum deflection measured in the North Yarmouth project was 0.127 in. and occurred in Section 1 where the 12 in. tire shreds with a 30 in. soil cover was used. The maximum deflection measured in the control section (no tire shreds) was 0.032 in. Figure 2.5 shows the magnitude and extent of the deflections in Section 1 and the Control Section. Most of the deflection in the Control Section occurs within the first three feet of a very small diameter deflection basin. The deflection basins in tire shred sections are broad and flat and extend 10 to 15 ft from the point of loading. This characteristic supports the hypothesis that event though the magnitude of the deflections in the

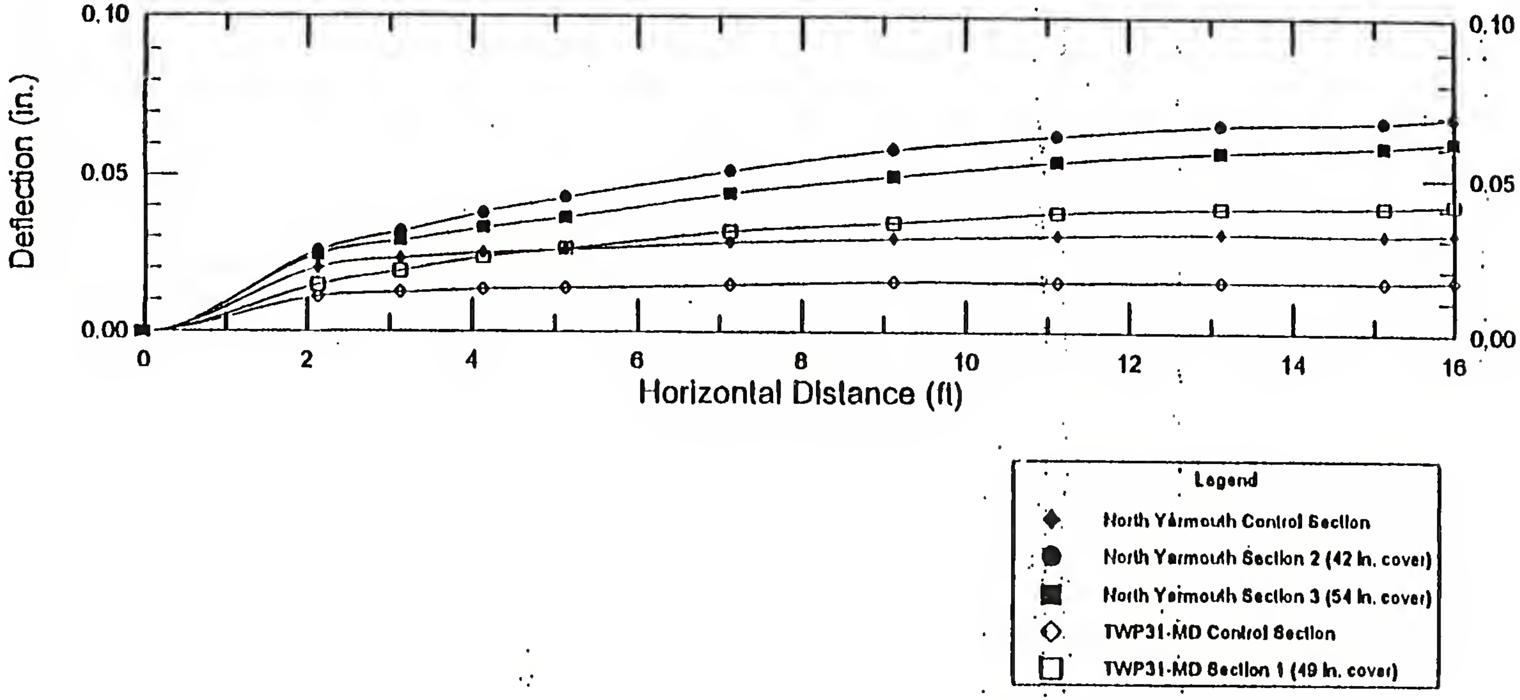


Figure 2.5 Comparison of deflection basins in sections with similar amounts of soil cover (Nickels, 1995)

tire shred courses is greater, the smooth gradual spread over a larger distance will generate similar tensile strains to those found in the control section.

The pavement section in the TWP31-MD project has a 12 ft wide truck lane, two 12 ft wide travel lanes and a 10 ft wide breakdown lane. The travel and truck lanes have pavement thickness of 9 in. and the breakdown lane has 2 in. thickness of pavement underlain by recycled milled pavement that has a greater strength than conventional granular material. The maximum deflection was 0.042 in. and was measured in Section 1 where 12 in. tire shreds with 49 in. of soil cover were used. The maximum deflection in the Control Section (no shreds) was 0.017 in. For all sections, most of the deflection appears to occur within the first 2 to 3 ft and a general trend of decreasing deflection with increasing soil cover was observed.

The design differences between the two projects determined the maximum deflections observed. The North Yarmouth project had 5 in. of asphalt pavement while the TWP31-MD project had 9 in. of asphalt pavement overlying a thicker soil cover. The normalized deflection was computed by dividing the centerline deflection, measured in a tire shred section by the centerline deflection in the corresponding tire section. The normalized deflections for both projects are plotted versus thickness of overlying soil cover in Figure 2.6. The following observations can be made. For both types of tire shreds the normalized deflection decreases with increasing soil cover. The normalized deflection approaches one for a 96 in. soil cover, indicating that the effect of tire shreds is negligible.

The deflection of 12 in. tire shreds is larger than the deflection of 3 in. tire shreds under the same soil thickness. This effect should increase for thinner soil covers.

A comparison of pavement deflections for the control sections and three tire shred sections with similar soil covers is made in Figure 2.7 The deflection basin of the TWP31-MD project has a smaller slope up to 2 ft and tends to be flatter after 5 ft than the deflection basins from the North Yarmouth project. This suggests that the 4 in. difference in asphalt pavement affects the shape of the deflection basin. The shape of the control sections deflection basins are similar beyond 2 ft but their magnitude is also affected by the difference in asphalt thickness.

The deflectometer results were compared with estimates from MICHPAVE, a finite element program that also generates tensile strain data along the bottom of the pavement. It was found

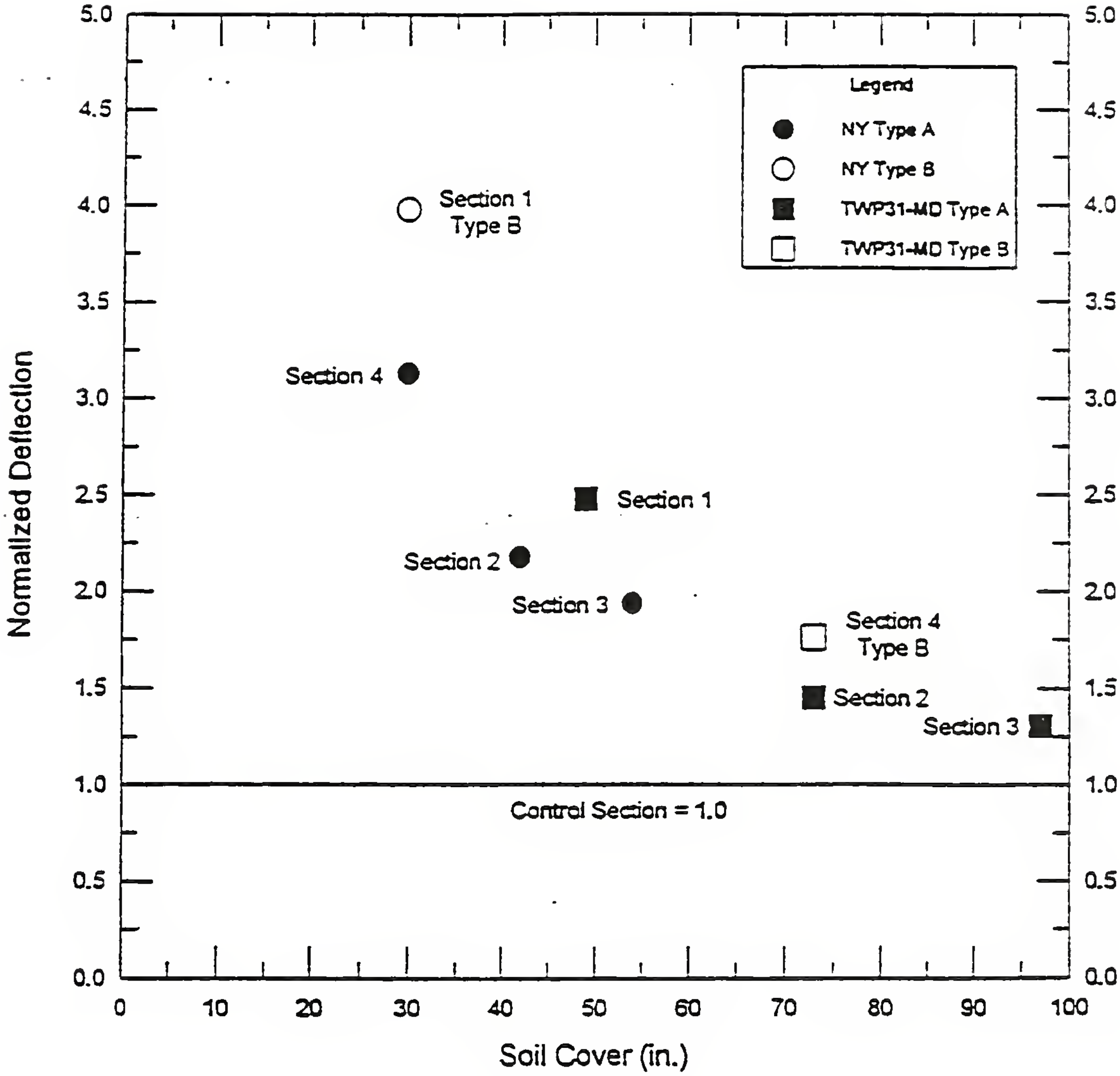
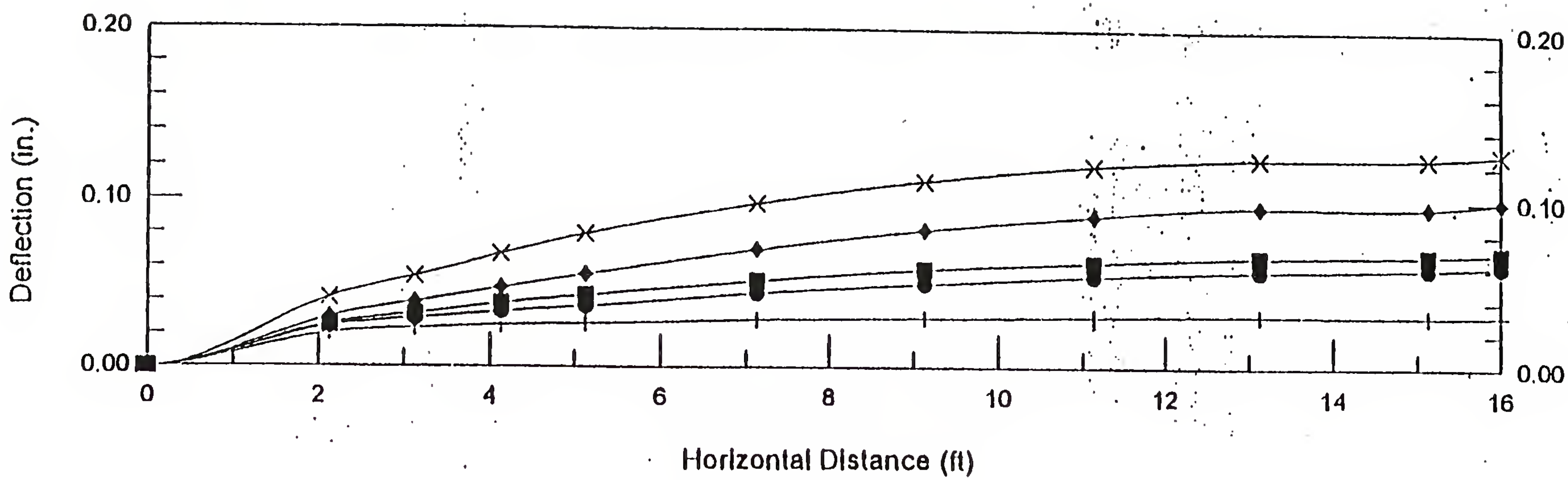


Figure 2.6 Normalization of North Yarmouth and TWP31-MD pavement deflection data (Nickels, 1995)



NOTES:

1. Section 1 - 30" gravel cover, type B
2. Section 2 - 42" gravel cover, type A
3. Section 3 - 54" gravel cover, type A
4. Section 4 - 30" gravel cover, type A

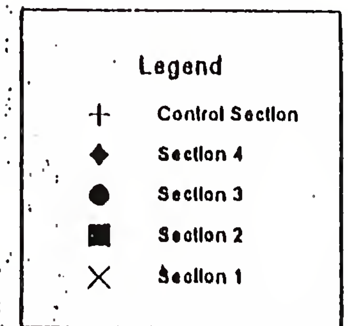


Figure 2.7 Comparison of deflection for sections with similar soil cover (Nickels, 1995)

that MICHPAVE severely overestimated the deflections because the program fixes the lateral boundary at ten wheel radii from the center of the wheel load and the deflection basin measured for tire shred sections extends beyond this limit.

Another finite element program called ALGOR was used to estimate the maximum tensile strains at the bottom of the pavement by inputting the profile of the observed deflection basin into the program. Tensile strains are a critical factor in pavement durability. The maximum tensile strains obtained were 0.0566 for the North Yarmouth project and 0.03 for the TWP31-MD project. The maximum tensile strains were normalized with respect to the tensile strains calculated for the control section in each project. There is a general trend for decreasing normalized strain with increasing cover thickness (see Figure 2.8). For cover thicknesses above 70 in. tire shreds have a negligible effect on tensile strains. The maximum normalized strain of 1.399 for 3 in. tire shreds was found in North Yarmouth section 4; the corresponding normalized deflection in this site was 3.125. This points out that tire shreds have a greater effect on deflection than on tensile strains. It would be expected that a 2 ft thick layer of 3 in. tire shreds with a 30 in. soil cover would have a small effect on pavement durability and a negligible effect for soil covers larger than 70 in. The maximum normalized strain was 2.117 for 12 in. tire shreds with a 30 in. soil cover. This indicates that 12 in. tire shreds should not be used in sections with thin soil covers and thin pavement overlays.

2.5.6.6 Construction Procedures and Design Guide Lines

Based on the construction observations from the North Yarmouth and TWP-31-MD project the following recommendations were made (Nickels, 1995):

1. Compactor type : For 3 in. maximum size tire shreds either a vibratory smooth roller or a vibratory tamping foot roller with a minimum operating weight of 10 US short tons be used. The same compactors are recommended for 12 in. tire shreds, even though the field data to support this is limited. Six to eight passes applied to 12 in. lifts for either type seems to be sufficient.
2. Grade tolerance: the specifications require that tire shreds be brought to ±1 in . of the specified grade. This tolerance was difficult to meet by the contractor, and is

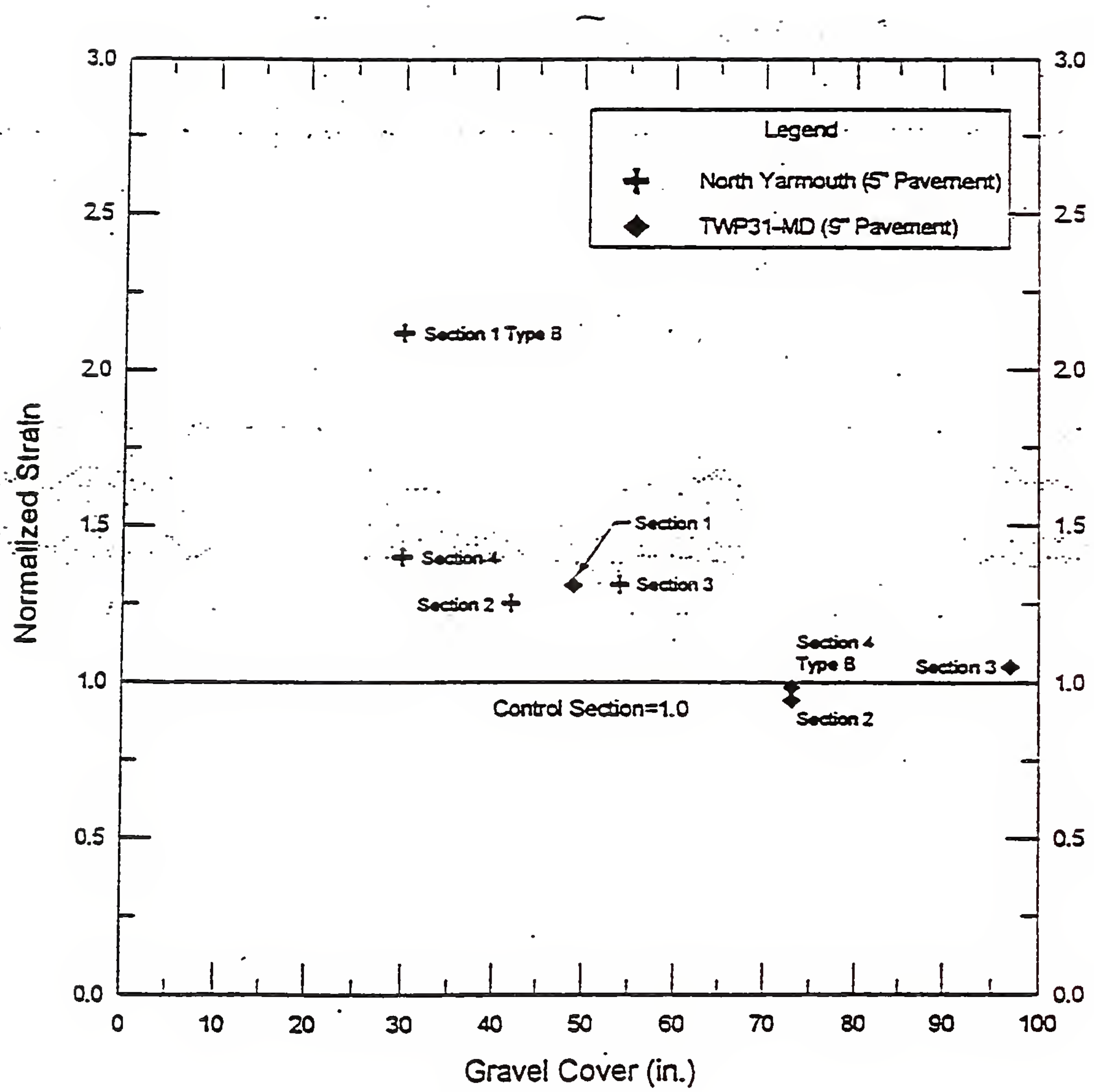


Figure 2.8 Normalized elastic strain (Nickels, 1995)

not critical for a layer located 2 to 6 ft from the pavement surface, therefore a ±3 in. tolerance for each lift and the final grade is recommended.

3. Method of payment: the recommended method of payment should be tons delivered and not in-place volume. It is difficult for a contractor to estimate the in-place unit weight of tire shreds at the time of bidding.
4. Tire shred size: 3 in. maximum size tire shreds performed better in both projects. The equipment operators objected to the 12 in. tire shreds due to the difficulty in spreading them and achieving final grade. Larger deflections and higher tensile strains can be expected from 12 in. tire shreds, therefore they should be used in the lower part of deep fills and if they are available at a lower cost than 3 in. tire shreds.
5. Soil cover thickness: soil covers greater than 70 in. reduced tensile strains and had a negligible effect on service life. Tensile strains increased by only 40% for 30 in. soil cover . Further research on the service life of pavement sections with thinner soil covers are currently underway (Nickels, 1995).

2.5.6.7 University of Wisconsin-Madison

Edil and Bosscher (1994) have conducted a laboratory study on tire shreds and rubber-soil mixtures of five different type of soils. Four of these soils were granular materials including a gravelly sand and three uniformly graded clean sands. The fifth soil was a low plasticity clay (LL=42, PI=22). The average specific gravity of tire shreds from five sources was found to be 1.22 (ranging from 1.13 to 1.36 depending on metal content) and 1.15 for tire shreds without metal. Compaction analyses were done on mixtures with varying soil-rubber ratios. Compacted densities similar to those obtained by other researchers were observed (Ahmed, 1993, Humphrey et al, 1993).

Compressibility tests ran in a 6 in. Proctor mold showed 37% compression for pure tire shreds under a vertical pressure of 100 psi with an initial porosity of about 0.67 and a final porosity of about 0.50. For rubber-sand mixtures beyond a sand content of about 40% by volume the compressibility is significantly reduced to less than 20%.

Resilient modulus tests on rubber-sand mixtures indicated almost one magnitude change between 100% sand and 30% tire shred content. Similar results (see Table 2.4) were obtained by Ahmed (1993).

Poisson's ratio was measured by compacting tire shreds in a 12 in. mold to a 10 inch height. At an axial pressure of 0.85 psi the mold was removed and four segments of a PVC membrane were placed along the sides of the standing specimen. The segments were attached together using latex rubber to allow lateral expansion with negligible lateral confinement. The axial pressure was increased in 0.45 psi steps to 2.65 psi and the lateral expansion was measured by tape at midheight. Three loading cycles were applied to check the stability of the results. Poisson's ratio varied between 0.2 and 0.3 during the three loading cycles. These values correspond to K_0 values of 0.3 to 0.4.

The angle of repose of loose tire shreds varied between 37° and 43° and was as high as 85° for compacted tire shreds. Direct shear tests on a rubber-sand mixtures with more than 10% tire shreds ratio indicate higher shear strengths than those obtained for dense sand at moderate normal stresses (less than 6 psi). At the highest normal stresses (around 11 kPa) the effect of tire shreds on shear strength is not as dramatic. Edil and Bosscher (1994) conclude that it is clear that randomly mixed tire shreds can reinforce sand to a strength greater than that of dense pure sand and result in a lighter material. The reinforcement effect was analyzed by inserting 10 vertical tire shreds in the shear plane of the direct shear device. The strength envelope shows a friction angle of 55° up to 40 kPa normal stress and 41° thereafter. Ten tire shreds represent only 3% of the sample weight.

Permeability tests on a specially designed rigid wall permeameter demonstrated a nearly constant $qh^{1/2}$ (q is flow and h is head) which indicates that flow through large, open pores of tire shreds is essentially turbulent. It was also observed that flow through the smaller sand pores was laminar. Permeability results are similar to those obtained by Ahmed (1993).

Compaction specifications should not be based on a final unit weight, but on an optimum number of passes determined on a test section in the field. Edil and Bosscher (1994) do not recommend vibratory compaction for tire shreds, this suggestion is refuted by the field work by Humphrey (1995) and the laboratory observations by Ahmed (1993).

Humphrey is currently involved in the development of standards for scrap tire use in civil applications for the American Society for Testing Materials, in conjunction with the Scrap Tire Management Council (ENR, 1996).

2.6 Other considerations

Another concern is using tires in embankments is the combustible nature of tires. To reduce the possibility of fire, a protective earth cover must be placed on the top and side slopes of tire embankments. A similar soil cover is recommended for other lightweight materials, like wood shreds, sawdust, slags, ashes, expanded clay or shale, etc. for protection against fire or to prevent leaching of undesirable materials into groundwater. During construction, caution is required to avoid any fires in stockpiled tires or embankment tires that have not yet been capped.

Roadbeds of two separate highway projects in Washington state where millions of scrap tire shreds were used to fill started to burn (ENR, 1996). The Federal Highway Administration assigned Professor Dana Humphrey from the University of Maine to study the problem.

Humphrey (1996) investigated a site in Garfield near Pomeroy, in eastern Washington, where half a million tires were shredded into 4 by 8 in. pieces to fill a 350 ft long by 50 feet deep ravine on a rural gravel bypass. The fill was constructed with 45 fit of tire shreds covered by 5 ft of soil, mainly run of pit fill, and a non uniform layer of 1 ft of gravel (one area was covered with only 0.5 in. of gravel). The side slopes were covered with 18 in. of top soil. The project was completed in Spring, 1995.

The tire shreds used were produced by a Hammer mill that impacts the scrap tires to tear them into pieces. This procedure creates tire shreds with a large amount of exposed steel belt and a larger exposed area in the tire rubber due to nicks and scratches.

Steam started rising from cracks in the roadway during the fall of 1995 and flames emerged in a spot 30 ft down the embankment on the side slope. It was established that the area had been flooded under 30 ft of agricultural runoff which contained fertilizers (mostly nitrates) during the summer.

The other site is located near Ilwaco in western Washington where an asphalt road bed was finished in October, 1995. The roadbed was constructed on a 25 ft tire shred fill (4 in. maximum size tire shreds) with a 4 ft rock fill drain. The side of the fill has a slope of 1.75:1 and is covered by a geotextile and 2 ft of soil. The area is located next to a Cranberry bog that has an acidic leachate.

The road began to crack within a month of completion. A temperature increment was detected in the fill during mid-December, 1995. In January, 1996 steam emerged from long cracks along the centerline of the road. Certain areas of the embankment have reached a temperature of 165°F. Steam, smoke and petroleum leachate have continued to emerge but no flames have been detected. The temperature has remained constant and the settlement rate of the embankment is also constant.

A wall in Glenwood Canyon, Colorado was constructed using tire shreds as fill and tire shred blocks as facing. The tire shred blocks were produced by mixing and adhesive and pouring the mixture in molds to dry. These blocks are an ideal facing in areas where rocks fall from the slopes because the falling rocks bounce back without damaging the wall. The wall had a soil cover of compost and top organic soil. During the summer of 1995, steam and high temperatures in the facing were detected. The dark color of the facing acted as a heat absorbent and increased the temperature of the fill.

Tire shreds have a high insulation value and in such thick fills heat can be stored and promote chemical reactions. The tire shred fill had easy access to oxygen through the rock fill with a thin gravel cover and through the side slopes. The three processes that have been tentatively assumed to be occurring are oxidation reactions, bacterial activity and combustion. Oxidation affects the exposed steel belts and the rubber and can be accelerated by the increasing temperature. The agricultural runoff in one case and the leachate from the acidic bog in the other may promote the presence of bacteria. Three types of bacteria that consume iron (steel belts), petroleum products (spilled on tires or leaching in an acidic medium) and sulfur (vulcanized tire rubber) may be present. The bacterial activity is exothermic and increases the temperature in the fill. Combustion could occur on petroleum products from leachates and spills with low ignition temperatures.

Some recommendations for construction of future tire shred fills include: avoid the use of top soil as cover (mineral soil such as clean sand and gravel is recommended), limit oxygen intake to the fill by providing a good compact soil cover on all sides, limit amount of exposed steel belts in tire shreds (do not use Hammer mill tire shreds), remove crumb rubber and free steel wires from the tire shreds (by using a magnet and a ½ in. sieve) and prevent flooding and leaching into the tire shred fill. Professor Humphrey's recommendations are tested in more detail under 8.8 Recommendations (page 147).

The measurement of compacted field densities of tire shred and rubber-sand fills is not an easy task due to the size and form of the tire shreds. Common field methods such as the sand cone, the balloon method and the nuclear gage are not applicable due to the high void ratio of tire shreds and the interlocking effect of exposed steel belts which would not allow to remove part of the compacted fill without disturbing the surrounding area. An approximate method such as the one used by Humphrey is currently the best option. The method requires that known quantities of material be compacted in closed areas so that the compacted density can be calculated by measuring the height of the compacted fill.

Compacted tire shreds (about 2x2 in. nominal size) have permeability values equivalent to typical values for coarse gravel (Bressette, 1984). This property of shreds renders them suitable for use in subdrainage as an alternate permeable aggregate. As a highly permeable material, pore pressure development is prevented in tire fills and backfills. Use of tire shreds in alternate layers with non-select fills, like clays, silty clays, etc., will provide a shorter drainage path and thus help accelerate consolidation of the layer.

The use of shredded tires in embankments offers the potential benefit of disposing of large volumes of tires in short sections of highway. For example, the use of an asphalt-rubber pavement overlay utilizes only about 3600 tires per mile of a 2 lane road while a mile of 2 lane embankment 20 feet high would utilize about 5 million tires (one tire equals approximately one cubic foot loose bulk density before compaction (Read, et al., 1991).

CHAPTER 3
STRESS-STRAIN AND VOLUMETRIC BEHAVIOR OF TIRE SHREDS AND RUBBER-SAND

3.1 Introduction

The testing program conducted by Ahmed (1993) in the triaxial apparatus to establish the stress-strain and strength behavior of tire shreds and rubber-sand has been expanded to study the volumetric behavior of tire shreds and rubber-sand mixtures during shear. The 6 inch diameter triaxial cell used for testing was adequate for 1 inch nominal size tire shreds and minimizes scale effects that would occur in smaller triaxial cells. The samples were tested dry, and the tests were conducted under drained conditions at a low axial strain rate.

Subsequent sections of this Chapter contain a brief description of the testing equipment, experimental procedures, presentation of data, and a brief discussion of the deviatoric stress and volumetric strain results. The test results from this study are also compared to the published data available.

3.2 Description of Testing Apparatus

A 6 inch diameter internal chamber triaxial cell was used for the measurement of the shear strength parameters of rubber-sand. The cell can accommodate triaxial samples with a height of up to 12 inches. A 6 inch diameter vacuum split mold was used for the preparation of the samples. The samples were tested using an MTS Soil Testing System connected to a data acquisition system. A variety of loading conditions in a stress or strain controlled mode to simulate field conditions can be applied.

3.3 Testing materials

3.3.1 Ottawa sand

The sand used in this study was similar to that used by Ahmed (1993). The test sand is manufactured by U.S. Silica (Ottawa, Illinois) and sold under the trade name Ottawa sand. The desired gradation was obtained by mixing three different types of Ottawa sands including Flintshot (AFS Range 26-30), #17 Silica (AFS Range 46-50) and F-125 (AFS Range 115-130) in equal proportions. This sand is a white, medium to fine sand and has been classified under the United Soil Classification System as SP (poorly graded sand) and under the AASHTO Soil Classification System as A-3(0), and has a maximum dry unit weight obtained through the vibratory method of 115.6 pcf. The grain size distribution of the test sand can be seen in Figure 3.1.

3.3.2 Tire Shreds

One inch nominal size tire shreds with no exposed steel belting were used to avoid damage to the rubber membranes during the testing program. The average value of specific gravity for 1-inch tire shreds was computed as 1.02 by Ahmed (1993). Water absorption ranged between 1% and 2.5% depending on the amount of exposed fibers. The grain size distribution of the tire shreds can be seen in Figure 3.1

3.3.3 Rubber-sand

The rubber-sand samples have a tire shred to mix ratio of about 40% by weight. This weight ratio is equivalent to a 50% tire shred to mix ratio by volume. A homogeneous mix was obtained throughout the sample by following the experimental procedures described below.

3.4 Experimental Procedures

3.4.1 Tire Shreds

The tire sheds samples were tested dry and were compacted using a vibratory method of compaction. Two rubber membranes were attached to the vacuum split mold and placed

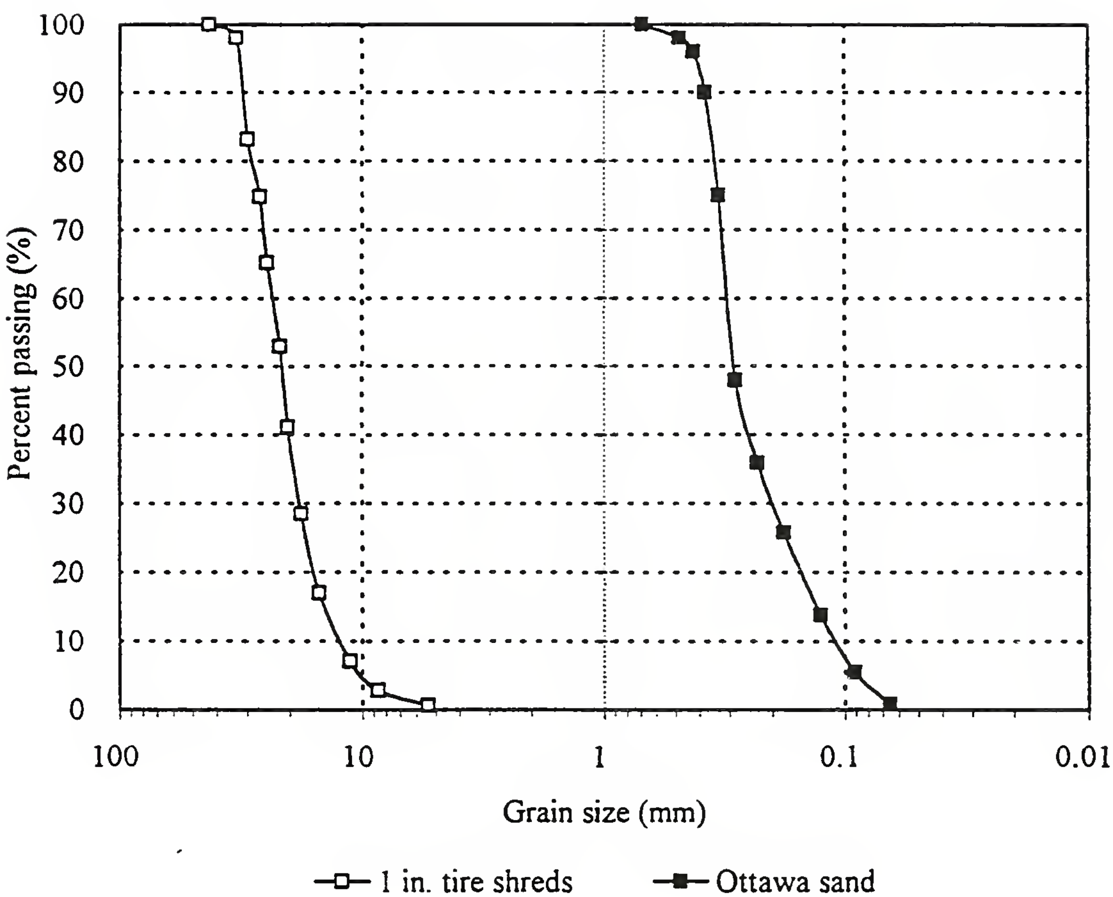


Figure 3.1 Grain size distribution of tire shreds and Ottawa sand

on the 6 inch base of the triaxial cell. The complete setup was then attached to the vibratory table. Tire shreds were weighed and poured in three inch layers into the vacuum-split mold. Each layer was vibrated at 60 Hz under weights that provided a 2 psi vertical pressure. The setup was placed on the MTS loading system and the split mold was removed. The height and diameter of the compacted sample were measured and the triaxial cell was assembled in the loading frame of the MTS System.

The initial displacement (measured by a LVDT) and the initial load (measured by a load cell) were zeroed. The confining pressure was provided by deaired water that completely filled the volume in the triaxial cell around the sample. The desired confining pressure was applied to the water with a pressure regulator and was measured with a pressure transducer located at the bottom of the triaxial cell. The pressure transducer was also zeroed after the triaxial cell was filled. The amount of water used to reach the required confinement pressure was carefully measured with a burette to calculate the pre-test volume and unit weight of the sample. The pre-shear load and LVDT readings are taken to establish the sample height. The sample was then sheared at a constant rate of strain of 1% per minute. The load, volume change (measured by a burette), and deformation were recorded throughout the test.

3.4.2 Rubber-sand

The sample preparation procedure was similar to the one used for tire shreds. The rubber-sand samples were tested dry and were compacted using a vibratory method of compaction. Tire shreds and Ottawa sand were poured in three inch layers in the vacuum-split mold attached to the 6-inch base of the cell. Each layer was vibrated at 60 Hz under 2 psi confining pressure. The prepared sample was enclosed in double rubber membranes. The weight, height and diameter of the samples were measured after compaction and the sample was assembled in the loading frame of the MTS System. The LVDT and load zeros are recorded. Water was used to completely fill the triaxial cell around the sample. The desired confining pressure was applied through the control panels with a pressure regulator and was measured with a pressure transducer. The amount of water used to reach the required confinement pressure was carefully measured to calculate the pre-test volume of the sample. The pre-shear load and LVDT readings are taken to establish the sample height. The sample was then sheared at a constant rate of 1% per minute. The load, volume change and deformation were recorded throughout the test.

3.5 Laboratory Testing Program

The testing program was planned to produce the results required to complete the other areas of this study. Two or more triaxial tests were conducted at various confining pressures to ensure the repeatability of the results. The three confining pressures at which the tests were run were: 4, 14 and 28 psi.

3.6 Presentation of Shear and Volumetric Strain Results

3.6.1 Tire Shreds

The results of the triaxial tests are presented graphically. Figures 3.2(a) to 3.4(a) show the deviatoric stress versus axial strain and Figures 3.2(b) to 3.4(b) present the volumetric strain versus axial strain under the three confining pressures (4, 14 and 28 psi).

The general shape of the stress-strain curves shows a linear behavior with increasing deviatoric stress under increasing axial strain. The material did not reach a peak deviatoric stress under the different confining pressures.

Tire shreds show an almost linear decrease in volume with increasing axial strains. The volume change under 4 psi confining pressure is linear up to 5 percent strain and stabilizes under the higher deformations.

The volume change for 14 psi is linear up to 15 percent strain and shows a declining rate at higher strains. The volume change for tire shreds under a confining pressure of 28 psi is almost linear throughout the test.

The voids within the tire shreds are reduced as the axial strain increases. The tire shred sample tends to bulge under low strains and confining pressures. For higher confining pressures the sample initially deforms vertically and around 10 percent strain the bulge is apparent.

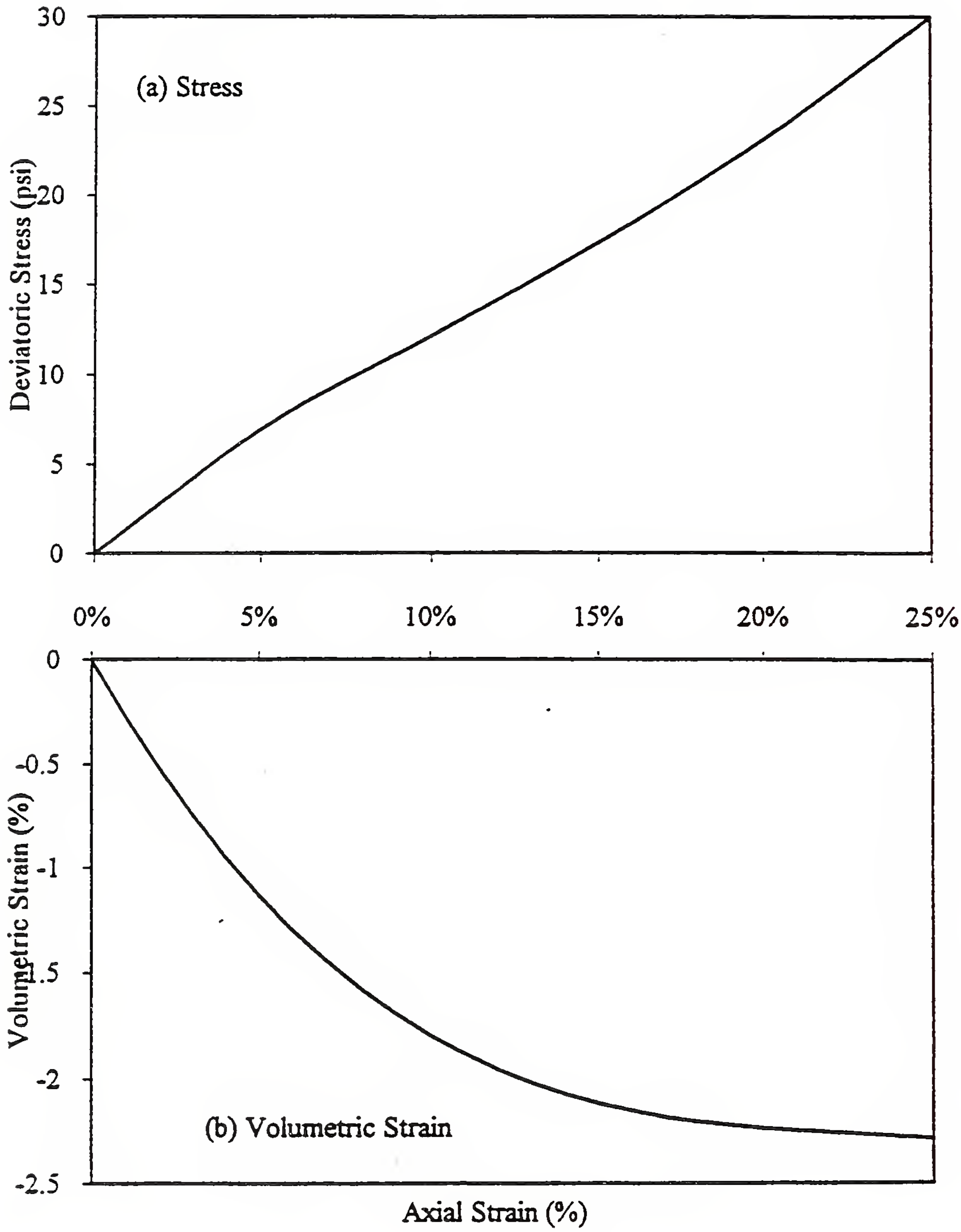


Figure 3.2 Triaxial test on tire shreds (4 psi confining pressure)

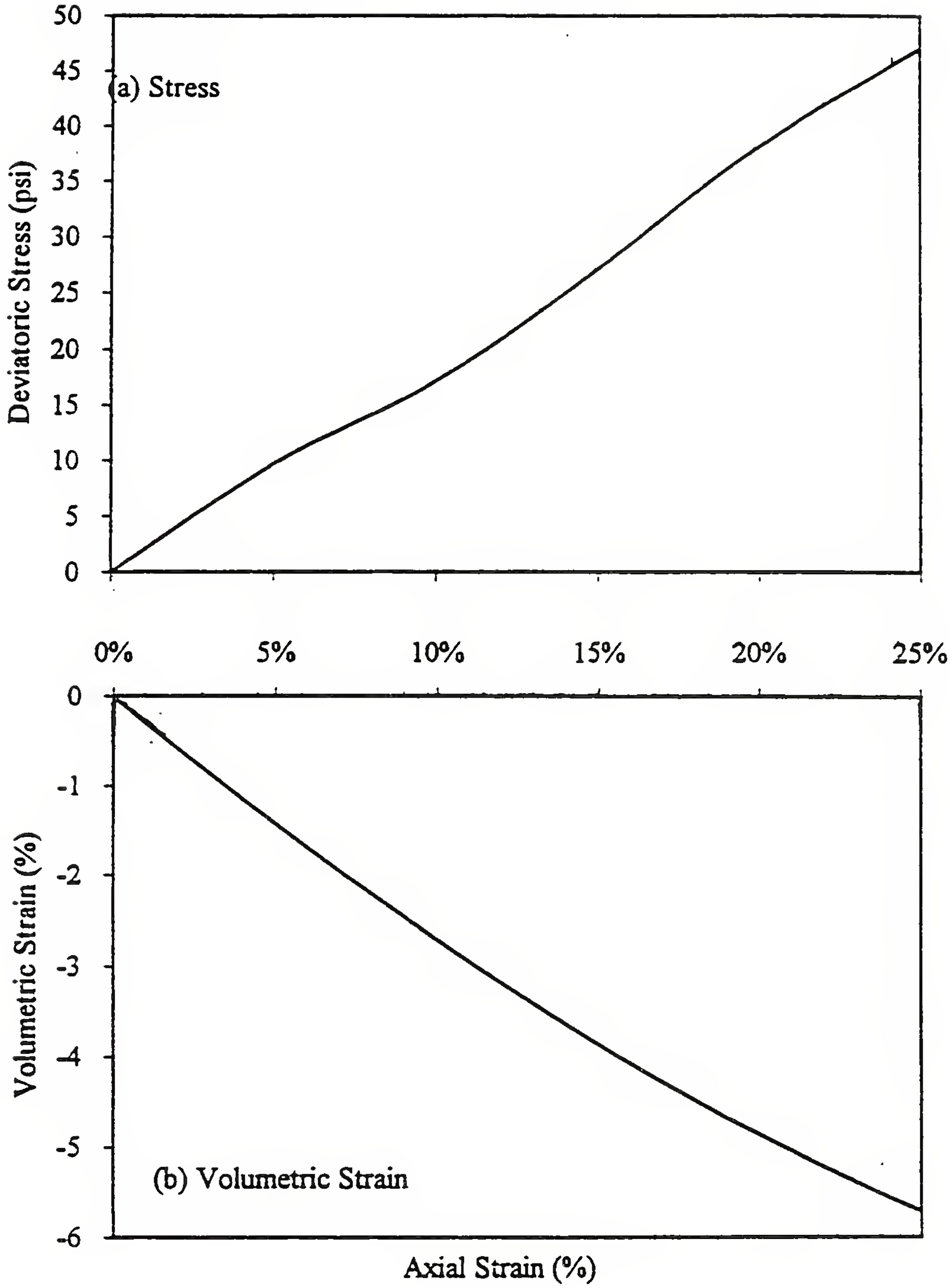


Figure 3.3 Triaxial test on tire shreds (14 psi confining pressure)

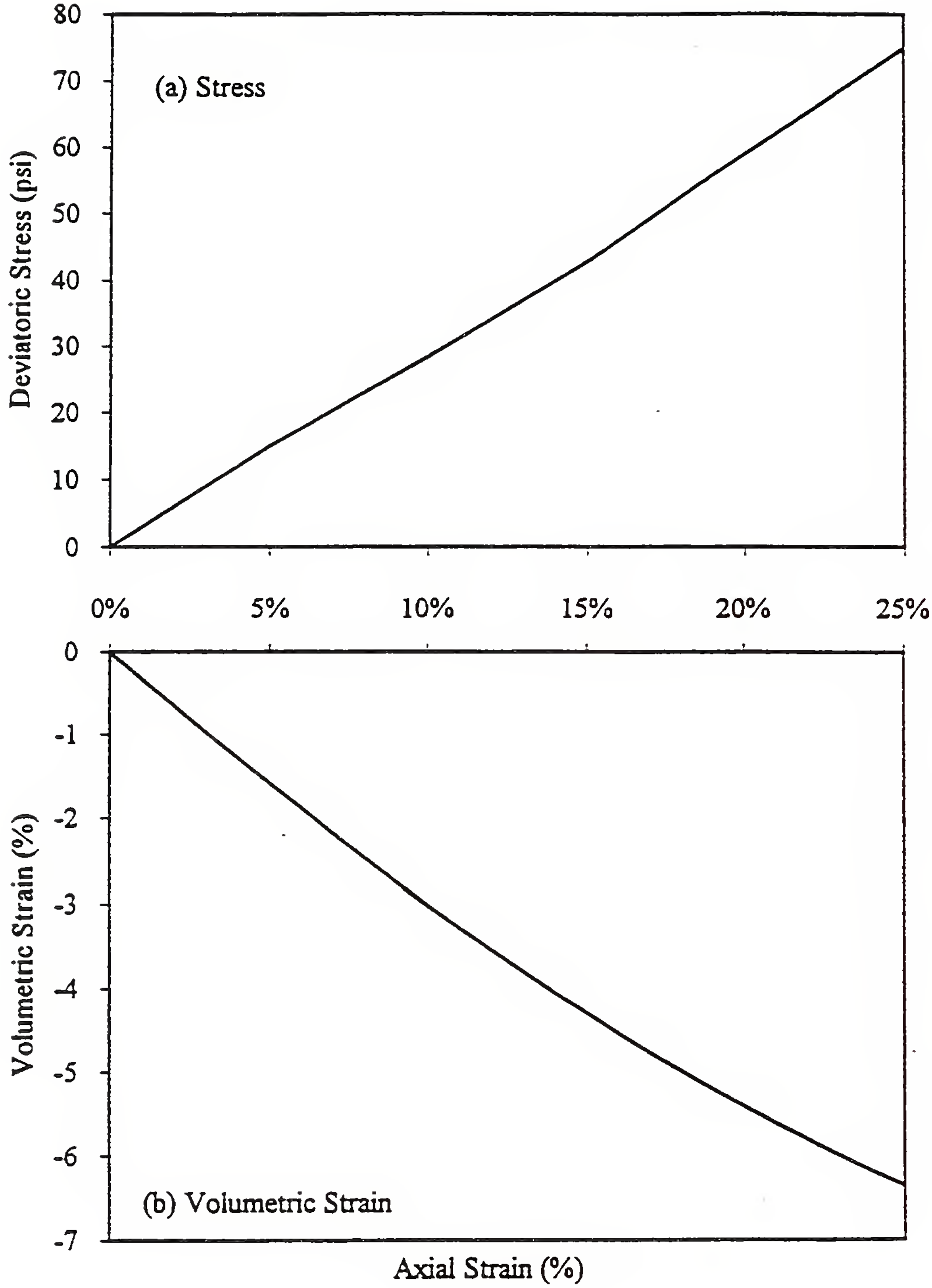


Figure 3.4 Triaxial test on tire shreds (28 psi confining pressure)

3.6.2 Rubber-sand

The results of the triaxial tests are presented graphically. Figures 3.5(a) to 3.7(a) show the deviatoric stress versus axial strain. Figures 3.5(b) to 3.7(b) present the volumetric strain versus axial strain under the three confining pressures (4, 14 and 28 psi).

The general shape of the stress-strain curves show a behavior similar to that observed for the direct shear tests that will be described in Chapter 5. The deviatoric stress tends to stabilize at increasing levels of axial strain with increasing confining pressure.

The volumetric strain show an initial loss of volume and varying levels of dilation have been observed for the three confining pressures.

The shear behavior of tire shreds under triaxial conditions has been studied by Ahmed (1993) and Masad et. al. (1995). The tests conducted by Ahmed (1993) did not measure the volumetric change during shear. The results obtained by Ahmed (1993) can be seen in Figure 3.8.

Masad et.al. (1995) conducted a series of tests on 0.25 inch tire shreds and a mixture of 50% tire shreds and 50% Ottawa sand by weight. Even though the exposed nylon belting in tire shreds of this size has a mayor influence in the behavior of the material, the trends observed for this material are similar to those for 1 inch tire shreds and rubber-sand mixtures (see Figures 3.9 and 3.10).

3.7 Discussion

The triaxial test results show good agreement with test performed by other researchers such as Ahmed (1993) and Masad, et. al. (1995).

The results from these tests will be used to determine the hyperbolic parameters to be used for the numerical modeling of the behavior of tire shreds and rubber-sand in embankments and wall backfills.

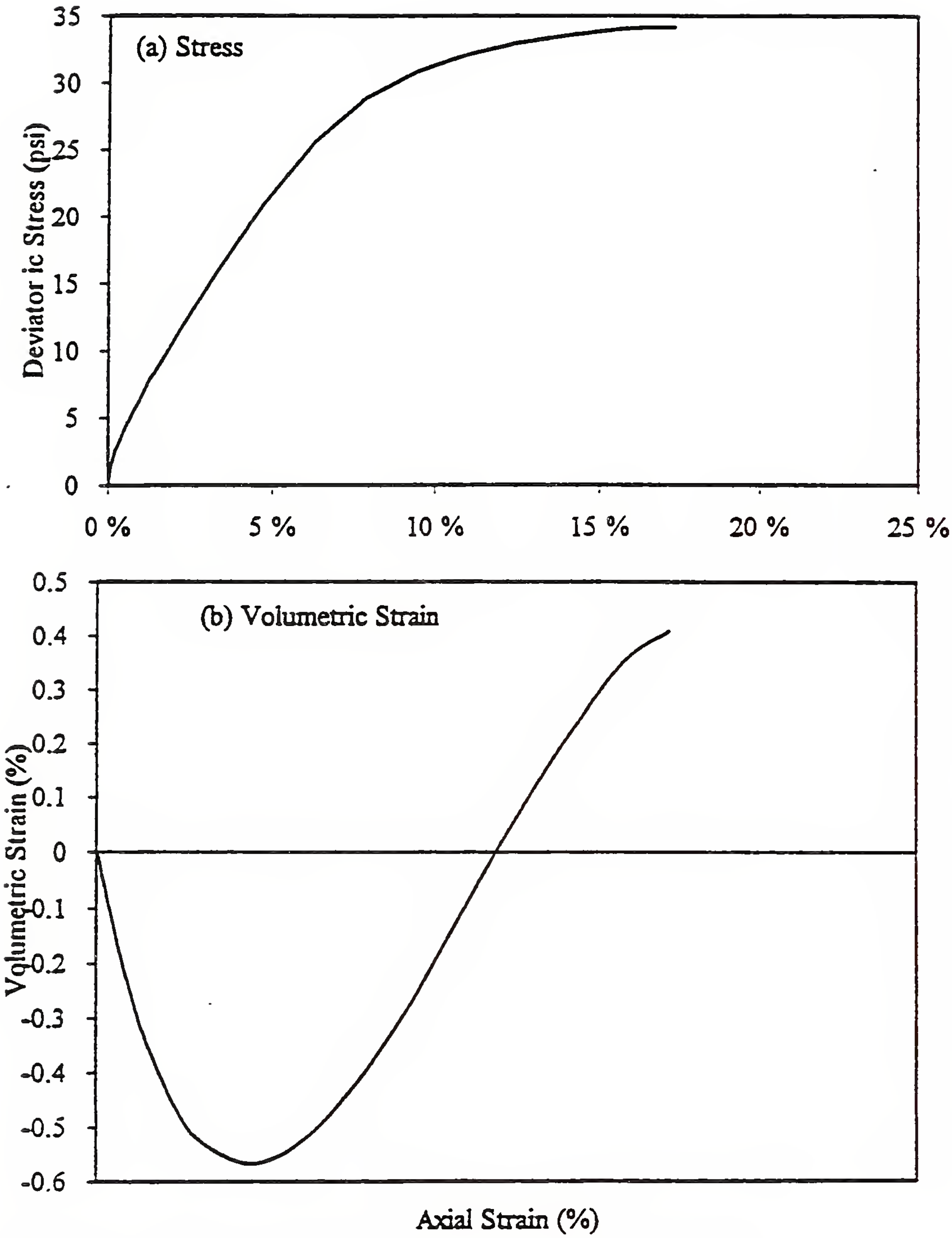


Figure 3.5 Triaxial test on rubber-sand (4 psi confining pressure)

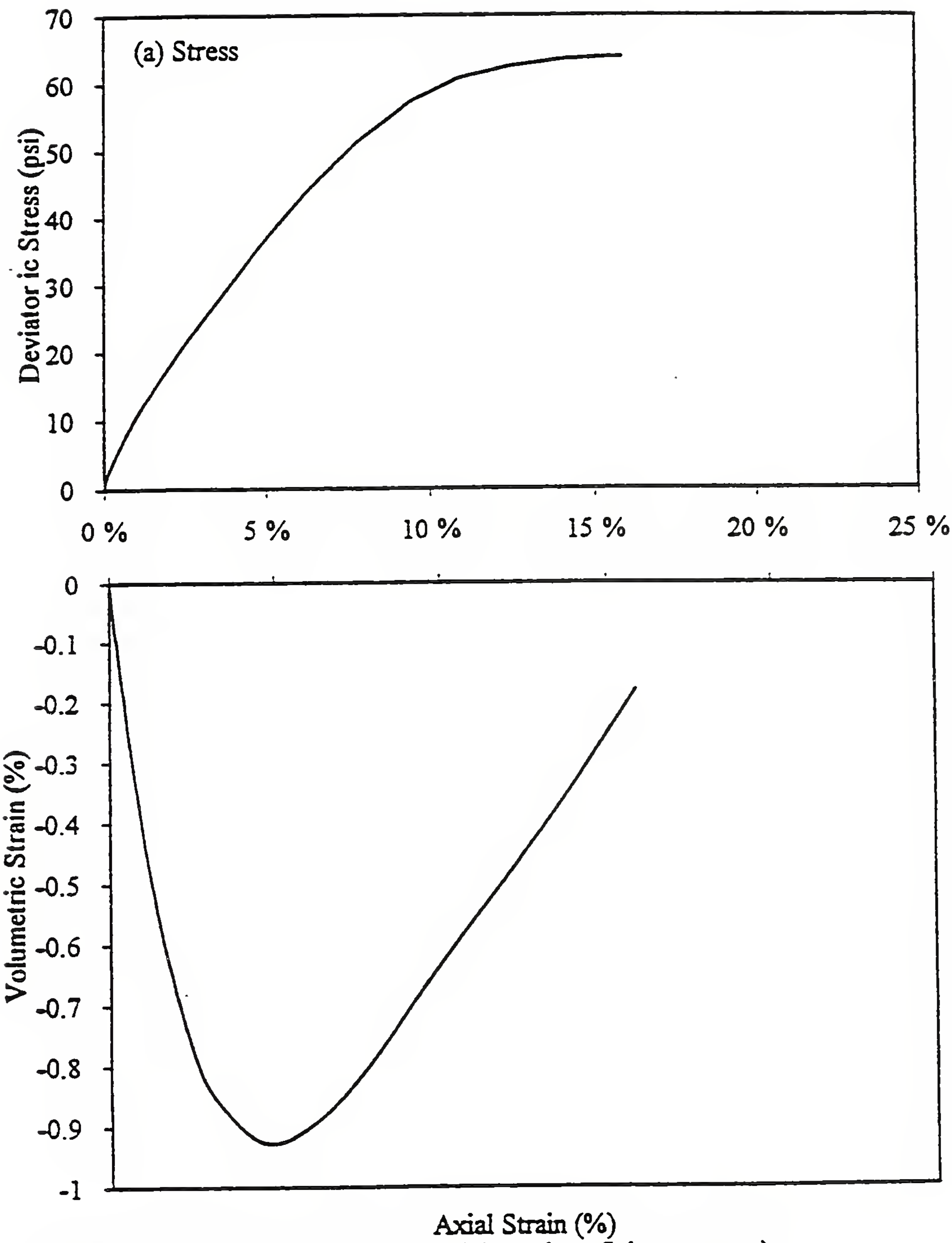


Figure 3.6 Triaxial test on rubber-sand (14 psi confining pressure)

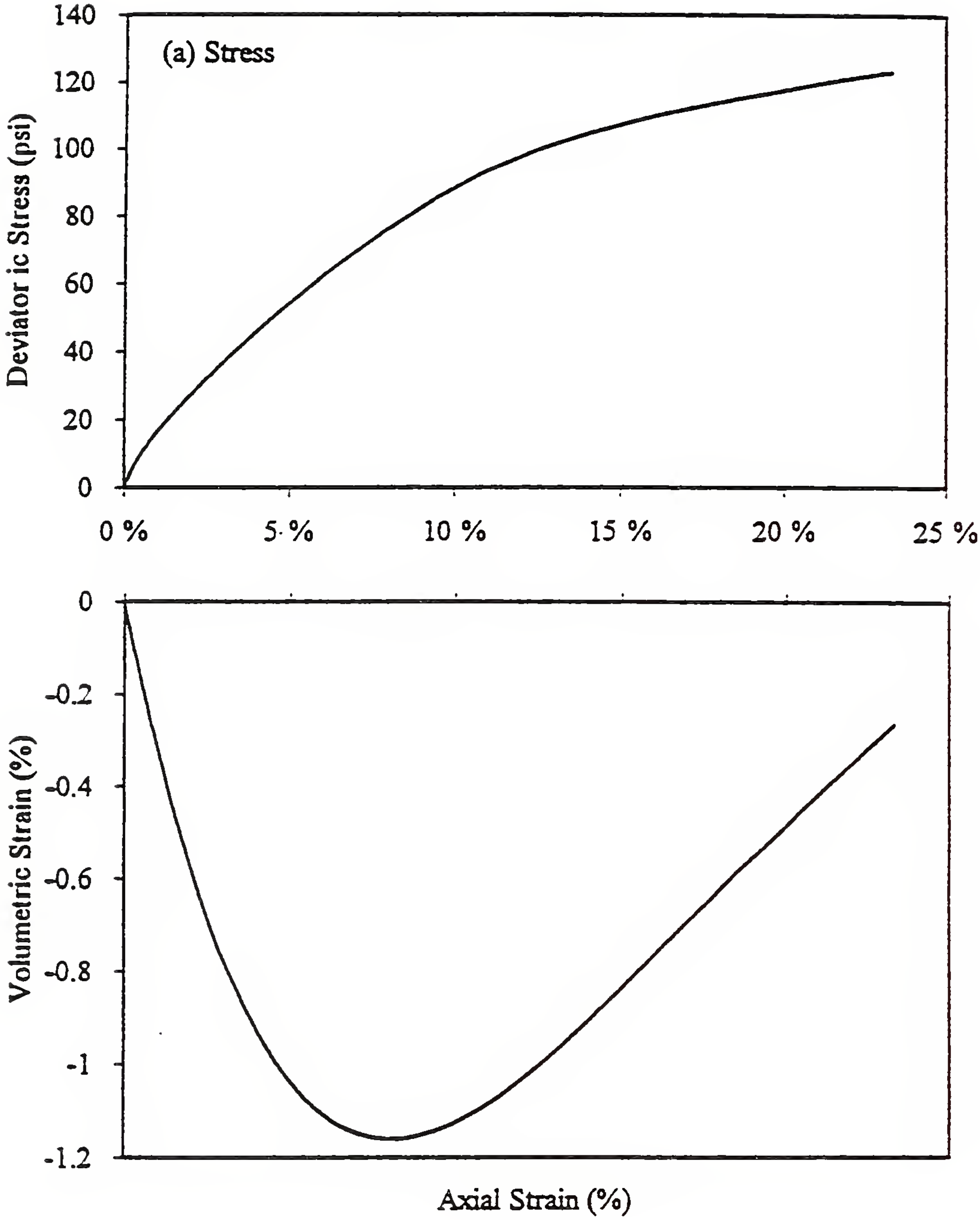


Figure 3.7 Triaxial test on rubber-sand (28 psi confining pressure)

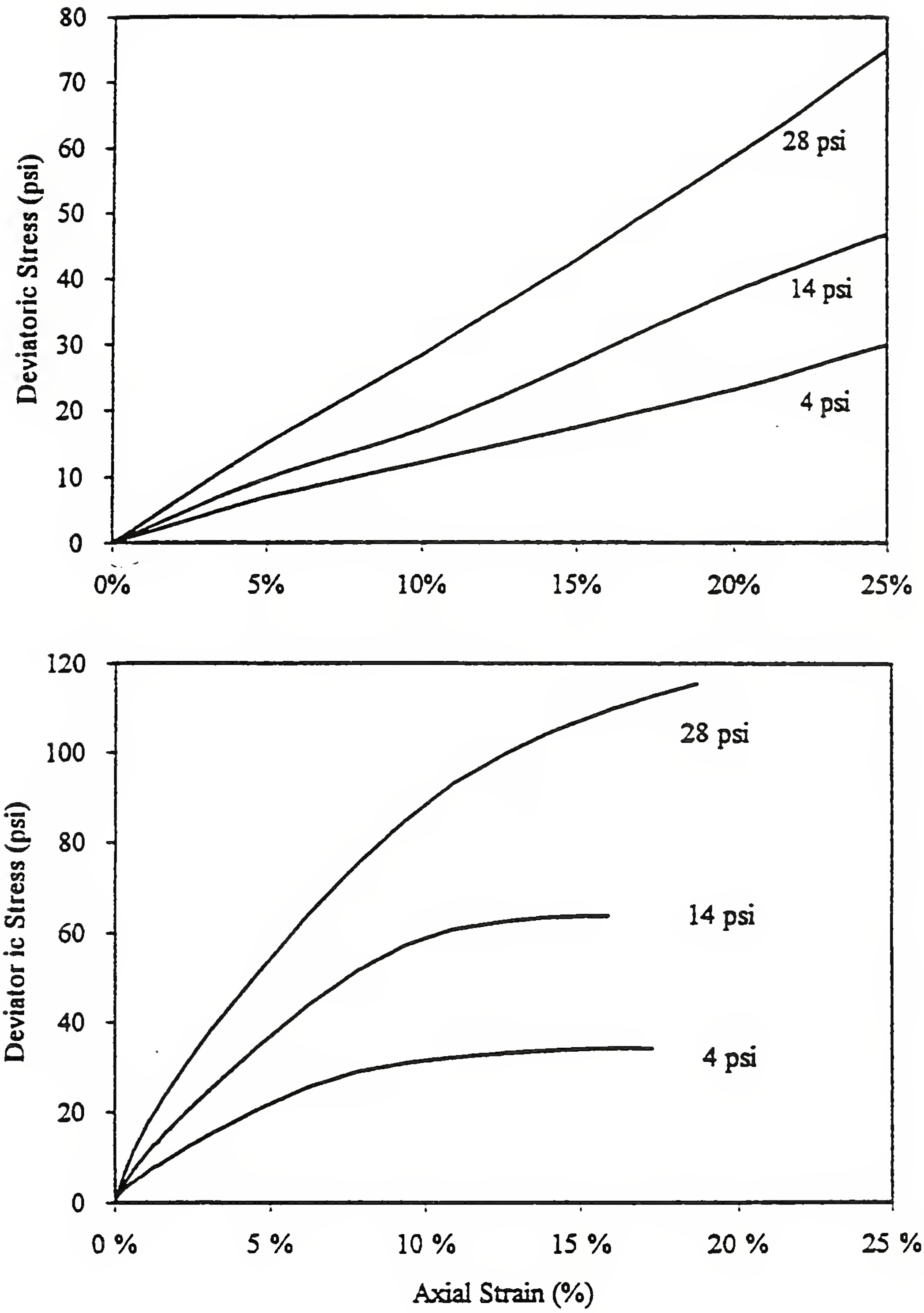


Figure 3.8 Triaxial tests on tire shreds and rubber-sand - no volumetric measurement (Ahmed, 1993)

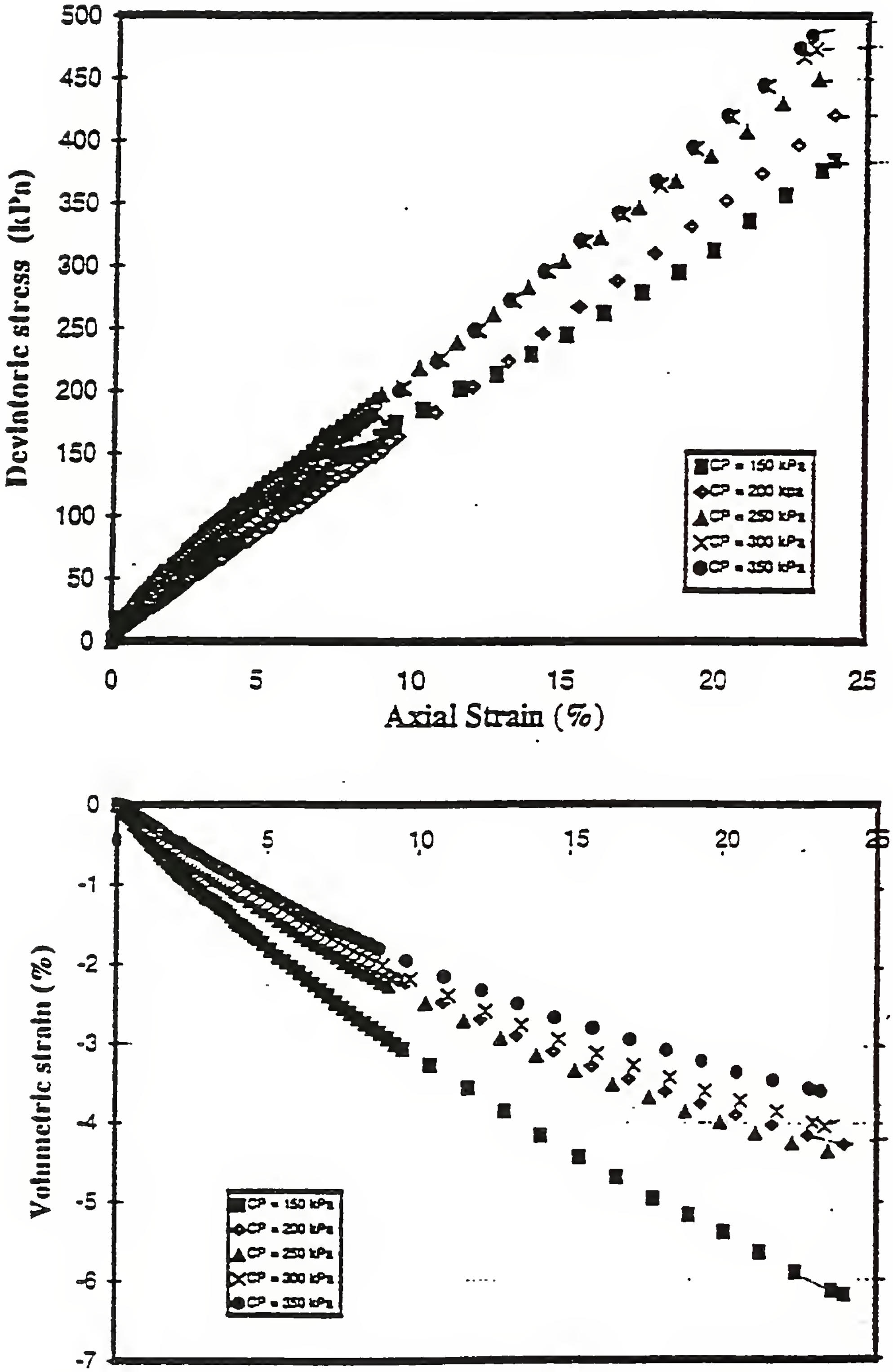


Figure 3.9 Triaxial tests on ¼ inch tire shreds (Masad et al, 1995)

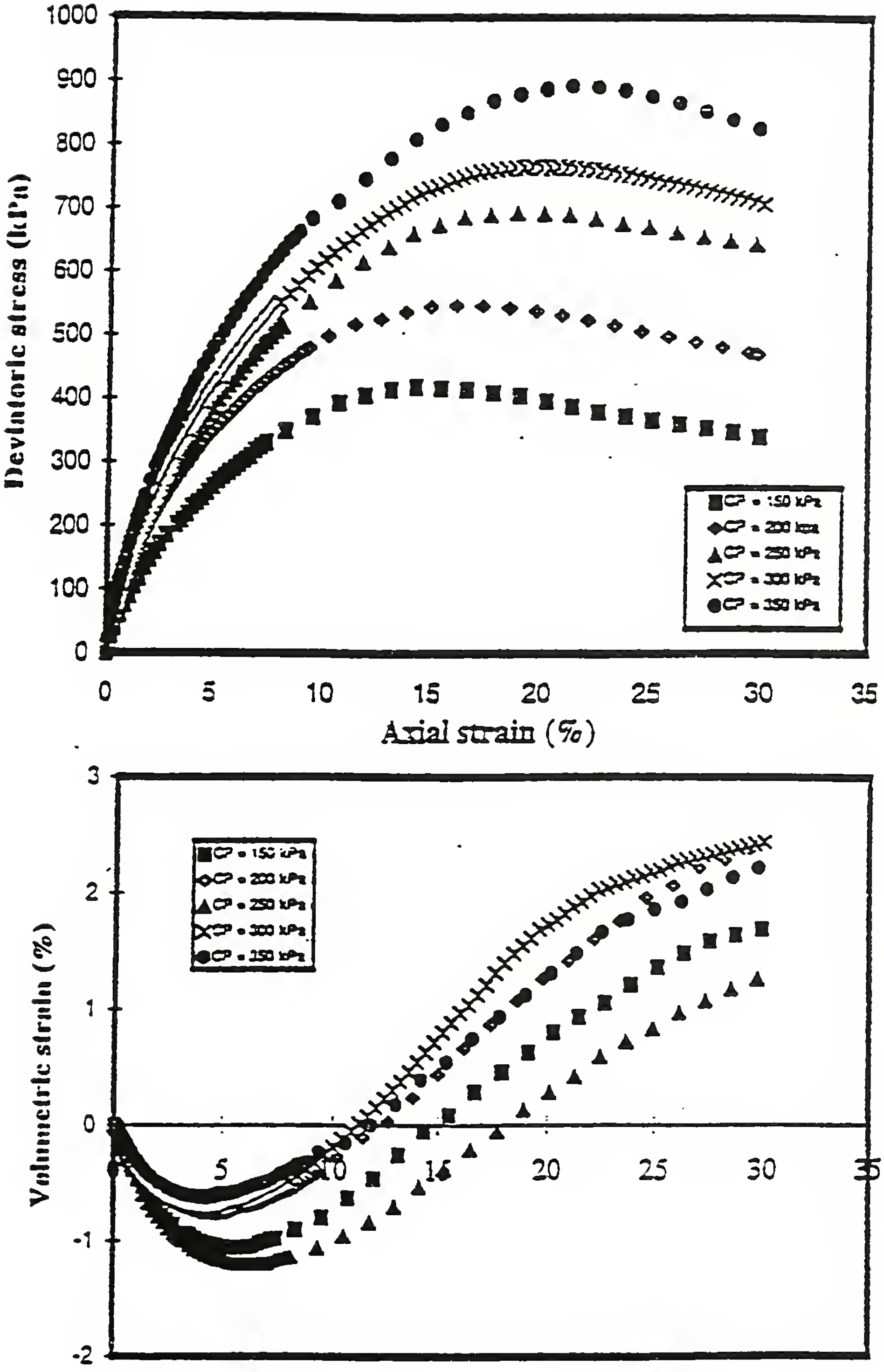


Figure 3.10 Triaxial tests on rubber-sand (Masad et al, 1995)

CHAPTER 4
COMPRESSIBILITY OF TIRE SHREDS AND RUBBER-SAND

4.1 Introduction

The high compressibility of tire shreds is probably the most important characteristic that will affect the performance of embankments and backfills constructed with this material. The measurement of the compressibility properties of tire shreds and rubber-sand require the use of a large scale apparatus. Ahmed (1993) performed tests in a 12 inch steel compaction mold subjected to vertical pressure. Manion and Humphrey (1992) ran tests under large vertical stresses (up to 80 psi) in a 12 inch PVC pipe and determined Young's modulus (E) and Poisson's ratio (μ). Nickels (1995) worked with a 13 inch diameter HDPE cylindrical tank under low vertical stresses (less than 10 psi).

4.2 Equipment

Vertical strains up to 50% in tire shreds (Ahmed, 1993) have been measured during compressibility testing. This characteristic and the large size of the tire shreds tested require the use of a large scale apparatus. The design and testing procedures adopted for this program were similar to those used by Manion and Humphrey (1992) and Ahmed (1993).

A schedule 40 PVC pipe with a 12 inch nominal internal diameter, 24 inch length and 0.43 inch thickness was used to measure the compressibility, the at rest lateral pressure coefficient (K_0), and to determine Young's modulus (E) and Poisson's ratio (μ) for tire shreds and rubber-sand.

Four strain gages were attached to the PVC pipe to measure the circumferential deformation and one gage was used to measure the vertical deformation. The strain gage selected for the testing program was a Micro Measurements strain gage model EA-13-250BK-10C. This strain gage is described as a general purpose Constantan strain gage with an open-face construction and a 0.001 in. tough flexible polyimide backing that is widely used in experimental stress analysis was selected for the testing program. This strain gage measures 0.25 in., has a 1000 Ohm resistance and can read up to 1800 μS (micro strains equivalent to 10^{-6} in./in. or 10^{-6} m/m). The strain gages were glued with a cyanoacrilate adhesive to the PVC pipe at 5.3 in. height from the bottom.

The PVC pipe was placed on a 1½ inch thick aluminum plate. Three steel rods were used to secure the pipe to the plate and prevent vertical displacements during unloading. The testing material was placed in 3 inch layers and compacted with a tamper in the pipe. A circular steel plate having a 1 inch thickness and 12 inch diameter circular steel plate was placed on the compacted sample to transfer the vertical load. The sample height was measured and the compacted unit weight was determined.

The vertical load was applied with the MTS loading system connected to a data acquisition system. The system can work on deformation and load controlled conditions and records both load and deformation continuously. The strains were measured by connecting the five strain gages to a Vishay Instruments Switch and Balance Unit model SB-1.

An additional compensating strain gage was used to eliminate temperature effects. The strain gage was attached to a separate piece of PVC pipe and connected with the Switch and Balance Unit to form a half bridge circuit with the strain gage that was being measured. The Switch and Balance Unit was connected to a Vishay Instruments Wide Range Strain Gage Indicator Model 3800.

Strain gages change their resistance as they deform. A Wheatstone bridge is capable of reading these minute resistance variations as long as the other resistances in the circuit remain constant. The half bridge circuit becomes a complete Wheatstone bridge when it is connected to the internal resistance located in the Strain Gage Indicator. An excitation voltage was input to the circuit and the variation in the output voltage was recorded. The Strain Gage Constant (2.135 at 24°C for the strain gage used) was recorded in the Strain

Gage Indicator and the readout of the indicator was given directly in μS. The strain gages were read manually and sequentially during the tests by turning the switch on the Switch and Balance Unit. The strains were recorded along with the time, and later this information was combined with the load and displacement data recorded by the computer. The test layout is presented in Figure 4.1.

The friction between the testing material and the pipe wall was measured during one of the tests by placing a SINCO model 51482 total pressure cell on the steel plate under the pipe. The compacted material was then placed in the PVC pipe. The 9 inch diameter pressure cell measured vertical stresses transmitted to the bottom of the sample with a low displacement, liquid filled flexible diaphragm. The fluid pressure in the pressure cell was converted into pneumatic pressure by force balancing the diaphragm with a continuous controlled gas source supplied by the SINCO model 222 pneumatic pressure indicator. The vertical pressure at the bottom of the sample was constantly measured by the pressure cell. Wall friction was determined by correlating the vertical pressure measured at the bottom with the vertical pressure applied on the top.

4.3 Analysis

The deformation of the 12 inch PVC pipe under vertical and horizontal loading was analyzed as the deformation of a thin walled cylinder (Poulos and Davis, 1974). The ratio of wall thickness to radius of the pipe is:

$$R = \frac{t}{r} = \frac{0.43}{6.37} = 0.0676 \qquad (4.1)$$

The vertical and horizontal stresses exerted on a thin walled cylinder are related to the measured vertical and horizontal strains in the following manner (see Figure 4.2):

$$\sigma_1 = \frac{\varepsilon_1 E + \mu E \varepsilon_2}{1-\mu^2} \qquad (4.2)$$

$$\sigma_2 = \frac{\varepsilon_2 E + \mu E \varepsilon_1}{1-\mu^2} \qquad (4.3)$$

where ε_1 is the hoop strain

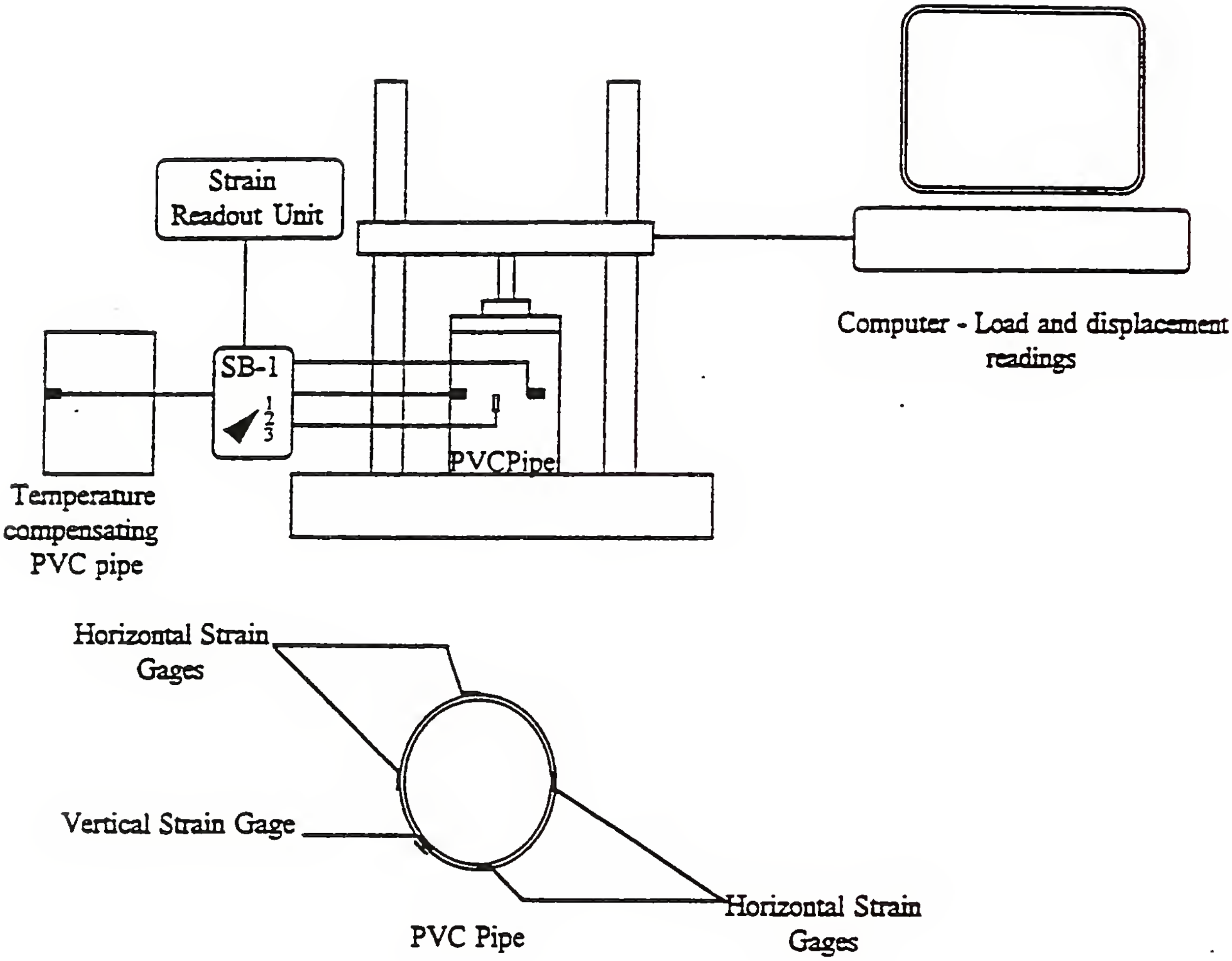
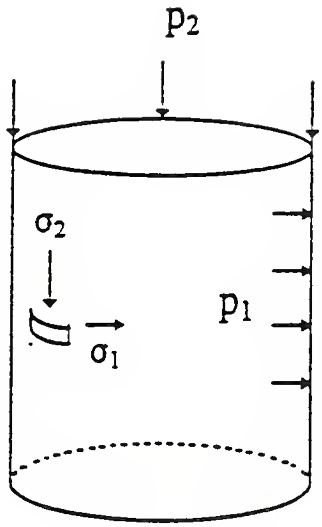


Figure 4.1 Compressibility Test Setup



$$\sigma_1 = p_1 r/t$$

$$\sigma_2 = p_2 r/2t$$

$$\epsilon_1 = (\sigma_1 - \mu\sigma_2)/E$$

$$\epsilon_2 = (\sigma_2 - \mu\sigma_1)/E$$

Figure 4.2 Vertical and Horizontal Stresses on ThinWalled Cylinders

Pressure was applied to the water in the tube through a hole in the top platen and the strain gage readings were recorded at various pressure levels. This calibration procedure was repeated to produce an average calibration curve. Equations 4.6 and 4.7 were used to obtain E=368231 psi and μ=0.3478 for the PVC pipe. Long term creep of the pipe under pressure was determined to be negligible by Manion and Humphrey (1992).

4.4.2 Vertical Pressure

A vertical stress calibration was also performed. The top platen was placed on the PVC pipe and used to apply a distributed vertical load on the empty cylinder. The deformation was recorded with the vertical and horizontal strain gages. This procedure was followed several times and the average results were used in Equations 6 and 7 to obtain E=363659 psi and μ=0.333 for the PVC pipe. The vertical and horizontal calibration setup can be seen in Figures 4.3 and 4.4.

There was a small discrepancy between the values obtained for both calibrations that can be attributed to inhomogeneities in the PVC pipe. The average values of E=365945 and μ=0.3404 will be used for the following tests.

Typical values of E vary between 420000 and 520000 psi and for μ vary between 0.26 and 0.34 for PVC pipes (MGI, 1992).

4.4.3 Wall Friction

The friction between the testing material and the PVC pipe was determined with the use of the pressure cell during one of the tests. The friction coefficient was calculated as:

$$F = \frac{p_v - p_{bottom}}{p_v} \qquad (4.8)$$

where p_v is the applied vertical pressure and p_{bottom} is the pressure measured at the bottom of the sample. The graphs show that the relationship varies initially but stabilizes at the higher normal stresses. The values of friction to be used for the analysis were determined by using a regression to calculate the values at different vertical pressures (see Figure 4.5).

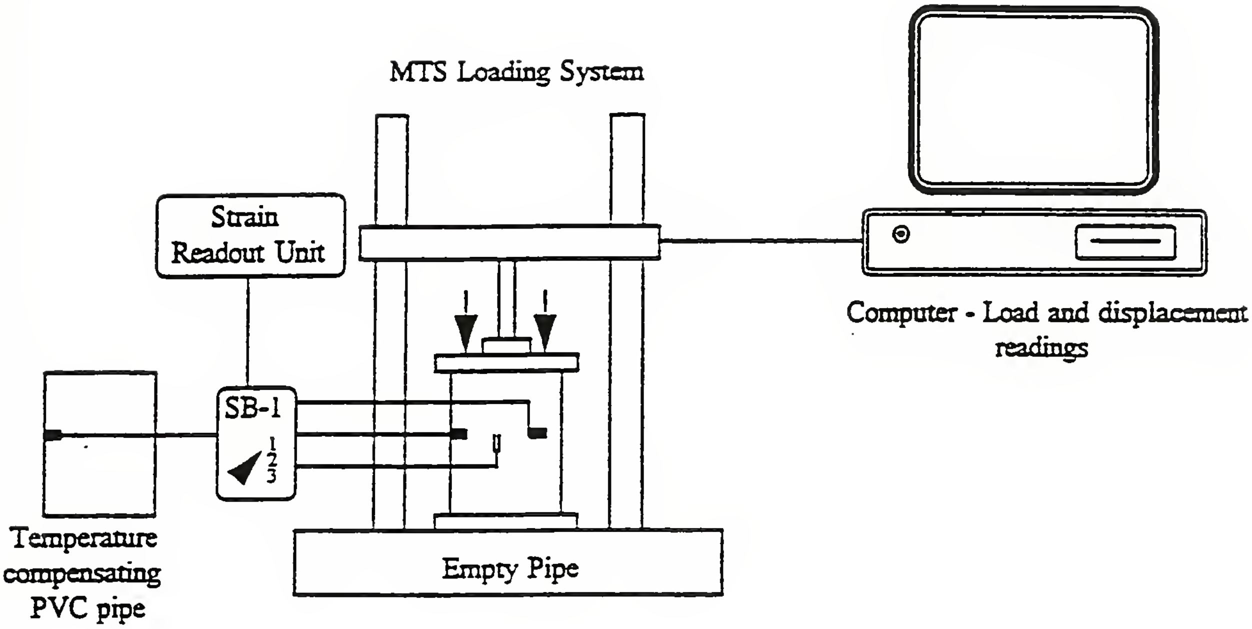


Figure 4.3 Vertical Pressure Pipe Calibration

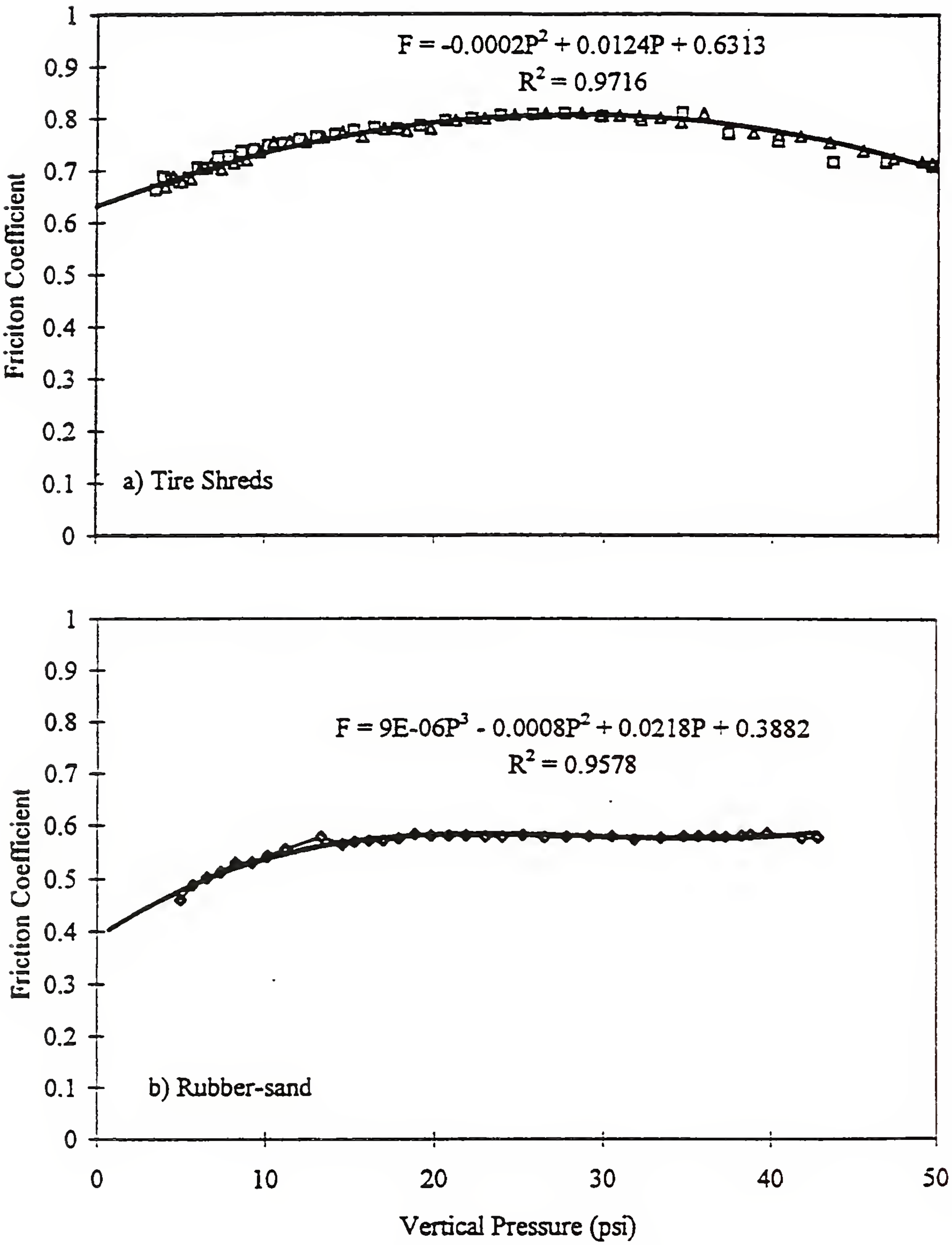


Figure 4.5 Friction Coefficients

4.5 Compressibility Testing Procedure

The PVC pipe was attached to the bottom platen with clamps on the three steel rods. The testing material was placed in three inch layers and compacted with a tamper. The sample type and initial height were recorded. The sample was placed on the MTS testing system and the load frame was positioned. The maximum displacement possible with the load frame was 4 inches.

The clamps were unfastened and the strain gages were connected to the to the Strain Indicator and the initial readings were taken. The test was run under displacement controlled conditions with a rate of 0.02 in./sec.

The strain was read sequentially for all the gages and the time of each reading was recorded. The circumferential strain was calculated as the average strain from the four circumferential gages. Equations 4.2 to 4.4 are used to determine the horizontal pressure at gage height.

The vertical stress at gage height was calculated by taking friction into account. The friction force was assumed to increase linearly from zero at the top of the sample to a maximum at the bottom. The vertical stress at gage height (5.3 in.) was calculated as follows:

$$\sigma_{gage} = \sigma_v \times \left[1 + \frac{F \times (H - 5.3)}{H}\right] \qquad (4.9)$$

where

σ_{gage} is the average stress at midheight

σ_v is the vertical stress applied at the top

H is the sample height at the time of the reading

F is the friction coefficient

The test continues until the maximum load of 5000 lb or the maximum displacement (4 inches) was reached by the MTS loading system. The clamps were fastened to prevent the PVC pipe from separating from the bottom platen during the unloading cycle. The piston is moved upwards at the same rate. One or two unload-reload cycles were performed on each sample.

4.6 Compressibility of Tire Shreds

The average vertical strain (ε_v) versus average vertical stress at gage height ($\sigma_{v\ gage}$) for compacted tire shreds is shown in Figures 4.6 and 4.7 for tire shreds and rubber-sand respectively.

The at-rest lateral pressure coefficient (K_0) was calculated as the ratio between the horizontal and vertical stresses. The results of the tests run for tire shreds and rubber-sand indicate that this value stabilizes after the normal stress goes beyond 5 psi. These values are not significantly changed in the reload cycle . The values obtained for tire shreds are presented in Figure 4.6 and for rubber-sand in Figure 4.7.

It was observed that the lateral pressure coefficients for tire shreds show a large variation and that the average value was around 0.5. The lateral pressure coefficients for rubber-sand present a smaller variation and show an almost linear decrease from 0.72 at low pressures to 0.65 at the larger pressures.

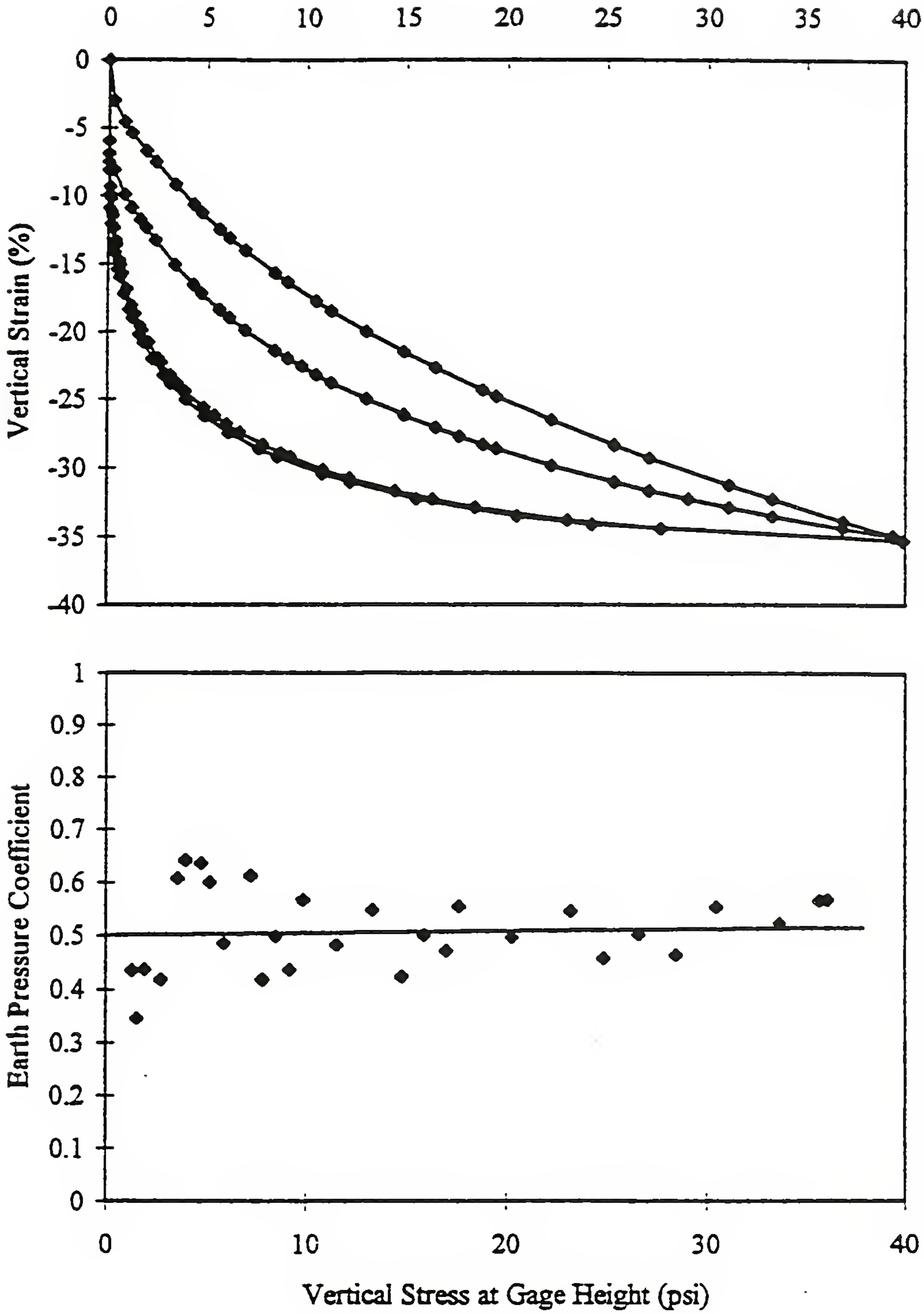


Figure 4.6 Compressibility and lateral pressure coefficients for tire shreds

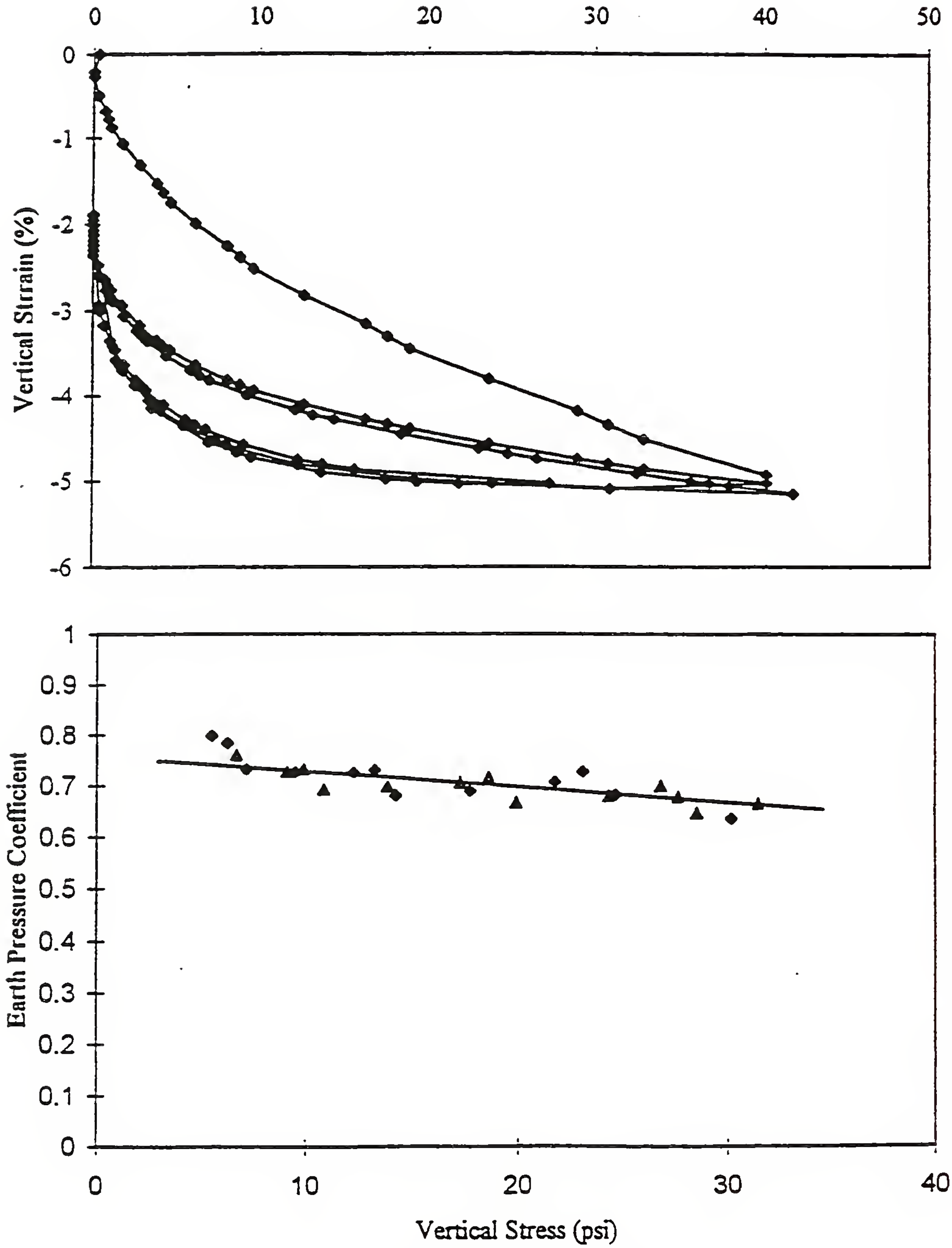


Figure 4.7 Compressibility and Earth Pressure Coefficients for Rubber-sand

CHAPTER 5

REINFORCED SOIL APPLICATIONS OF TIRE SHREDS AND RUBBER-SAND

5.1 Introduction

Several beneficial uses for scrap tires have been proposed in the past and some have been put into practice in various highway and non-highway applications. The use of tire shreds as lightweight fill can sharply reduce the tire disposal problem. The engineering properties of tires shreds have led to their use in a variety of applications (Ahmed, 1991).

The growing interest in utilizing waste materials in civil engineering applications has opened the possibility of constructing reinforced soil structures with non-conventional backfills. Scrap tires are a high profile waste material for which several uses have been studied, including the use of shredded tires as backfill. This interest raises the need for development of testing procedures to evaluate the interaction properties of tire shreds and rubber-sand mixtures with geogrids through pullout testing and direct shear testing.

This Chapter presents the results of a direct shear and pullout testing program conducted to evaluate the interaction properties of four kinds of geosynthetics which include a woven geotextile and three types of flexible geogrids having 0.8 in. (2 cm), 2 in. (5 cm) and 4 in. (10 cm) square apertures, within two types of backfill materials. The first backfill material was a 2 in. (5 cm) nominal size tire shred fill and the second backfill material was a rubber-sand mixture consisting of a blend of 2 in. tire shreds and a masonry sand.

Direct shear testing was conducted using a large direct shear box having plan dimensions of 12 in. by 12 in. (0.3 m by 0.3 m) and a total depth of 9 in. (0.23 m). Pullout tests were carried out in a large pullout box having plan dimensions of 4 ft (1.2 m) in length by 3 ft (0.9 m) in width and a total depth of 20 in. (0.50 m).

5.2 Materials Tested

The direct shear and pullout tests were conducted using two kinds of backfill materials: tire shreds and a rubber-sand mixture. Tire shreds have a high degree of compressibility because rubber is its main component and their void ratio is relatively high. Compressibility can be decreased by mixing tire shreds with sand to reduce the void ratio.

The first backfill material consisted of tire shreds with a nominal maximum size of 2 in. (5 cm) processed by BFI Tire Recyclers, a tire shredder operator from Jackson, GA. The second backfill material consisted of a rubber-sand mixture prepared by combining 40% by weight of 2 in. tire shreds with 60% by weight of a medium grain masonry sand having the following soil properties, Unified Soil Classification (USCS) of SP (poorly graded sand), coefficient of uniformity (C_u of 2.75, coefficient of curvature (C_c) of 1.1, maximum particle size of 0.08 in. (2 mm) and a strength angle (ϕ) of 31° determined from direct shear tests under normal stresses of 1, 5, 8 psi (7, 35 and 56 kPa). Figure 5.1 presents the grain size distribution of the tire shreds and the masonry sand.

The rubber sand mixture was prepared by mixing the tire shreds and the sand in a separate container by pouring tire shreds in a 35 gallon barrel and then adding sand in adequate proportion. The material was then thoroughly blended. A small moisture content was present in the sand (w=4%) and helped to prevent segregation of the mix since the sand grains tended to stick to the tire shreds.

Pullout testing was carried out on three types of flexible geogrids identified as FORTRAC 55/30-20, FORTRAC-OM 35/35-50 and FORTRAC-OM 35/35-100S (a special product developed for this testing program) manufactured by Huesker Inc.. The first number is the ultimate strength in kN/m in the warp direction, i.e., 3700 lb/ft (55 kN/m) and 2650 lb/ft (35 kN/m); the second number is the ultimate strength in kN/m in the fill direction, i.e., 2020 lb/ft (30 kN/m) and 2650 lb/ft (35 kN/m) and the third number is the square aperture size in mm, i.e., 0.8 in. (20 mm), 2 in. (50 mm) and 4 in. (100 mm).

The information provided by Huesker Inc. states that FORTRAC geogrids are manufactured from high tenacity polyester yarns which are woven into a stable, gridlike pattern and then coated with polyvinylchloride (PVC) which makes the final product very pliable.

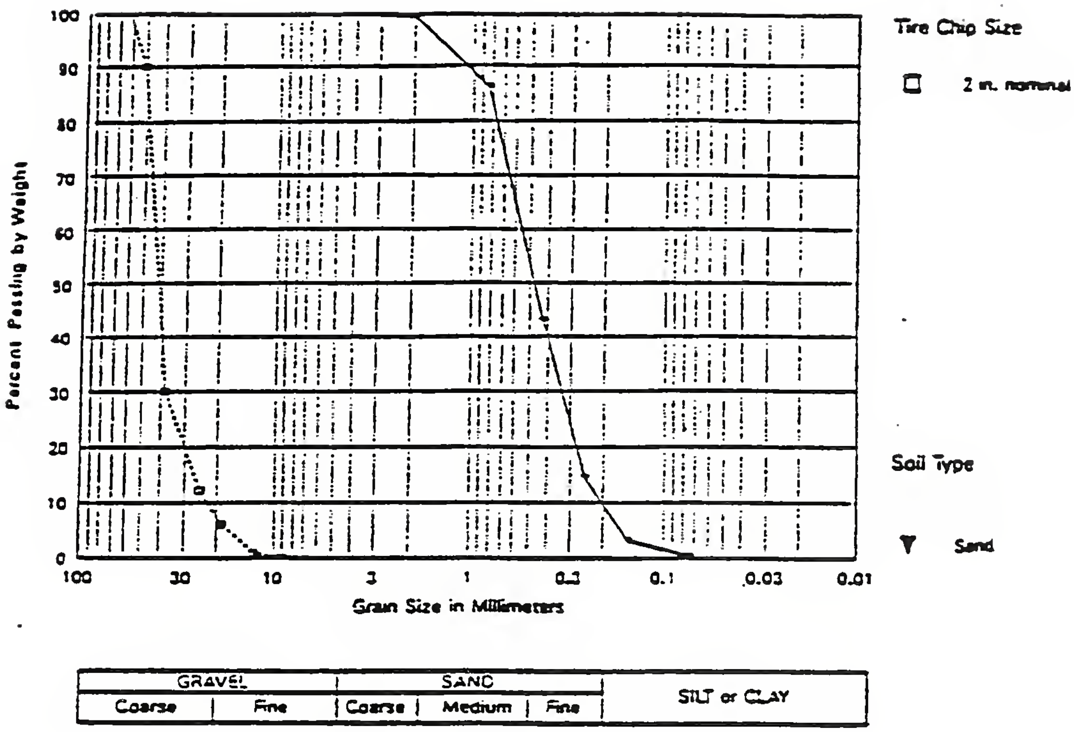


Figure 5.1 Grain size distribution of tire shreds and masonry sand

The multifilament polyester (PET) fibers are chemically similar to fibers used in the manufacture of high-performance automobile tires. Some of the physical properties of the FORTRAC-OM 35/35-50 include: unit weight - 10 oz/sq. yd; open area - 90%; tensile strength at 5% strain in both directions - 1020 lb/ft; elongation at break - 11%; and long term design load - 1347 lb/ft (sand, silt and clay), 1010 lb/ft (2.5 in. crushed stone and gravel).

The geotextile used in the pullout tests was also provided by Huesker, Inc. and is identified as COMTRAC R 200.45, a woven geotextile made of high tenacity polyester filament yarns. Some of the physical properties of the COMTRAC R 200.45 include: unit weight - 13 oz/sq. yd; tensile strength at 6% strain in the machine direction - 560 lb/ft; elongation at break - 9% and ultimate tensile strength - 1125 lb/ft (machine direction); and elongation at break - 20% and ultimate tensile strength - 250 lb/ft (cross direction).

5.3 Test Equipment Description

The direct shear and pullout testing was conducted at the GeoSyntec Consultants Soil-Geosynthetic Interaction Testing Laboratory located in Atlanta, Georgia. The test equipment used in the testing program included a large direct shear box, a large pullout box and various electronic instrumentation and data acquisition systems designed and built by GeoSyntec Consultants.

A large pullout box was utilized in the evaluation of the pullout resistance of geogrids in tire shreds and rubber-sand. The pullout box has plan dimensions of 4 ft in length (1.2 m) by 3 ft in width (0.9 m) and a total depth of 20 in. (0.50 m). Figure 5.2 shows a longitudinal cross section of the pullout box. Sleeve plates were placed above and below the front wall slot to minimize the lateral load transfer to the rigid front wall on the test specimen during pullout. The sleeve plates measure 6 in. (0.15 m) in length by 3 ft (0.9 m) in width and provide for soil layers thicknesses of 10 in. (0.25 m) above and below the geogrid. One end of each of the geogrid specimens tested was cast in epoxy resin to form a rigid specimen clamp. The epoxy specimen clamp was bolted between two plates which extended inside the fill to ensure that the geogrid remained confined and aligned during the test.

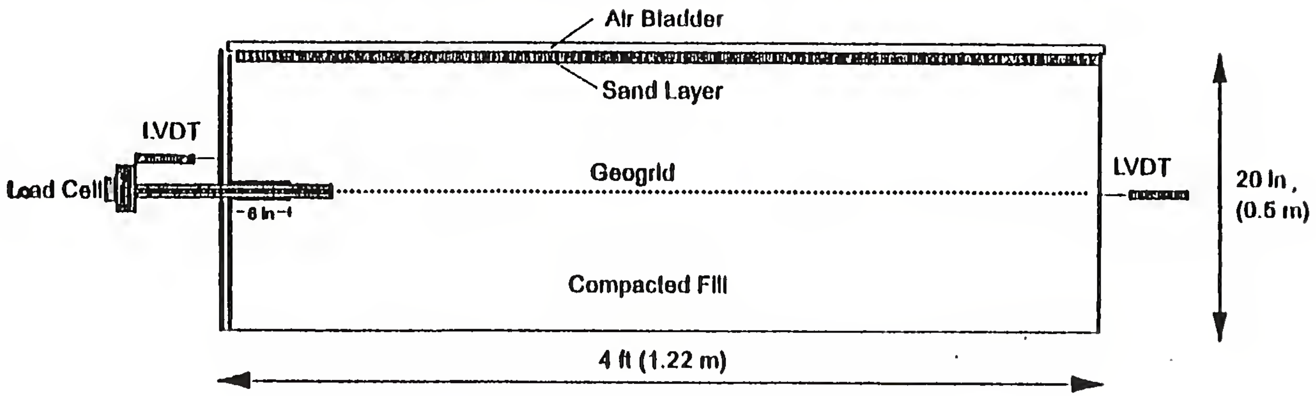


Figure 5.2 Large pullout box (side view)

A hydraulic loading system composed of two hydraulic cylinders mounted on each side of the pullout box with a common pressure supply (hydraulic pump) applies the pullout force on the test specimen . The pullout test runs under a constant displacement rate controlled mode. Vertical pressure was applied through an air bladder. The air bladder rests on a 1 in. (2.5 cm) sand layer placed above a geotextile to prevent the possible puncture of the bladder due to exposed steel belts in the tire shred or rubber-sand fill.

A large direct shear box was utilized in the evaluation of the shear strength of the two backfill materials. The direct shear box has plan dimensions of 12 in. by 12 in. (0.3 m by 0.3 m) and a total depth of 9 in. (0.23 m). Figure 5.3 shows a diagram of the large direct shear box. The normal stress was applied to the test specimen using a mechanical advantage lever arm system loaded by dead weights or an air cylinder. The shear load was applied to the test specimen through the use of a screw advance drive system driven by an electric motor and a gear reduction system which was electronically controlled to maintain a constant rate of shear displacement.

The backfill-geotextile interface properties were measured in a setup similar to the large direct shear box described above where the upper box contains the fill material to be tested and the lower box is filled with sand and covered with the geotextile. The geotextile is fixed to the end opposite to the direction of displacement. The normal pressure is applied by an air cylinder resting on the upper box material.

5.4 Electronic Instrumentation

A load cell and a linear variable differential transformer (LVDT) were mounted on the pullout loading system to measure the pullout load and the test specimen clamp displacement. These instruments were connected to a computer data acquisition system which consisted of a Validyne Engineering UPC-608 data acquisition card and Labtech Notebook data acquisition software that monitored the electronic instrumentation throughout the test.

A SINCO model 51482 total pressure cell similar to the one used for compressibility testing was placed on the bottom of the pullout box under the backfill material to evaluate the actual normal pressure transmitted by the air bladder onto the geogrid specimen.

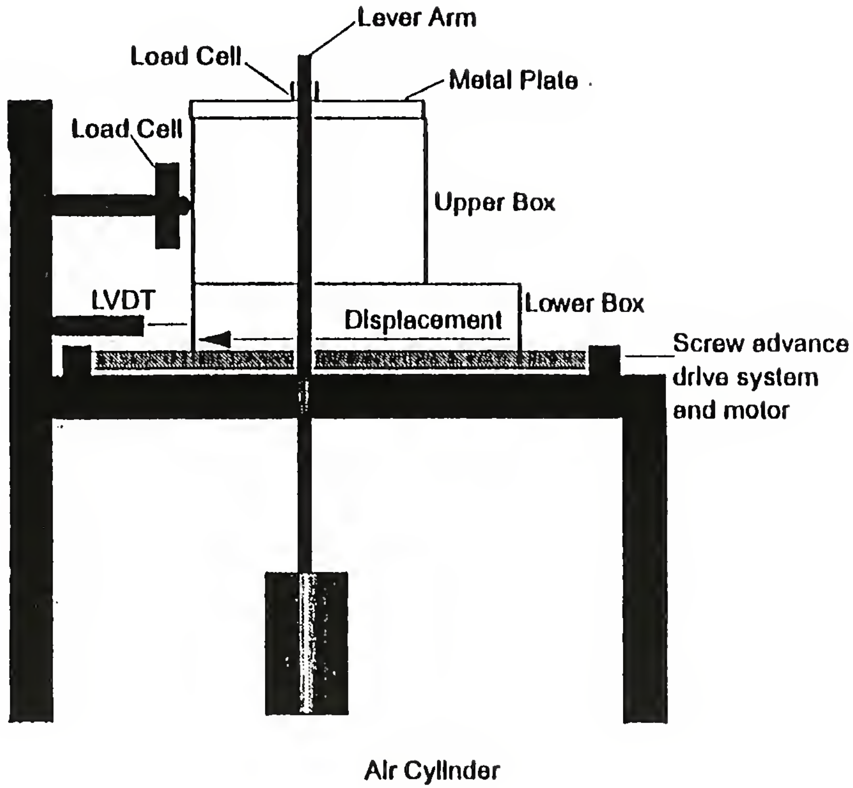


Figure 5.3 Directshear apparatus

The displacements along the geogrid were monitored using three LVDT's mounted at the rear of the pullout box. The LVDT's were connected at different points of the geogrid specimen through a "telltail" wire system protected by small diameter aluminum tubes to avoid friction with the fill material and prevent stretching. The "telltail" wire LVDT's were also monitored by the computer data acquisition system throughout the test.

The applied normal load and shear force were measured with load cells and the shear displacement was measured with a LVDT during the direct shear test and interface tests. These instruments were also monitored by the computer data acquisition system throughout the tests.

5.5 Testing Procedure

5.5.1 Pullout Tests

The pullout tests were performed following the ASTM Draft Standard Test Method D 35.01.87.02, "Measuring Geosynthetic Pullout Resistance in Soil". The tire shreds fill was placed in 3 in. (7.5 cm) layers in the pullout box and compacted with a hand compaction tamper. Tire shreds reach their final compacted unit weight with compaction energies as low as 50% Standard Proctor (Ahmed, 1993). The final compacted unit weight was approximately 37 pcf (2300 N/m^3). The geogrid specimen was placed at midheight in the box and was connected to the "telltail" cables and LVDT's, and the final lifts of the tire shred fill were placed and compacted. Figure 5.4 shows the placement of the geogrid specimen in the pullout box.

The rubber sand mixture was prepared by mixing the tire shreds and sand in a separate container and pouring the mix in 3 in. (7.5 cm) layers in the box. Each layer was compacted through hand tamping and a new layer was added. The recorded compacted unit weight was approximately 73 pcf (4500 N/m^3). Eight lifts were required to fill the pullout box and the amount of material used was controlled carefully. The depth of fill was measured in various areas of the pullout box to obtain an accurate value of fill height in order to compute the compacted unit weight before placing the air bladder.

Pullout tests were performed on both materials under various confining pressures ranging between 0.3 and 9.8 psi (2.1 kPa - 68 kPa) to simulate conditions at different depths in a

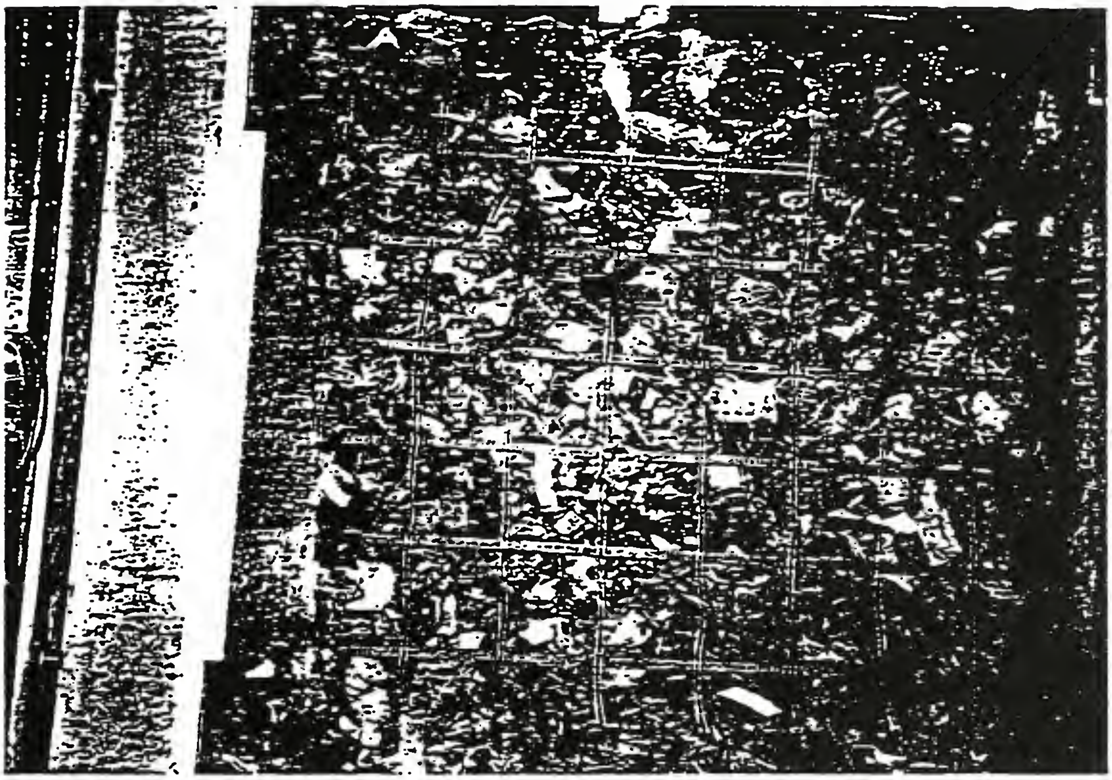


Figure 5.4 Placement of geogrid specimen in pullout box (tire shred fill)

geogrid reinforced backfill. A constant displacement rate of 0.04 in./min (0.1 cm/min) was used for all tests. Figure 5.5 presents the results of 4 tests performed on the 4 in. aperture geogrid within the tire shred fill under various confining pressures. Figure 5.6 shows the results of three pullout tests conducted on the 2 in. aperture geogrid within the tire shred fill. Figure 5.7 displays the results of three tests performed on the 0.8 in. geogrid and Figure 5.8 shows the results of four tests on the geotextile within the tire shred fill. Figures 5.9 to 5.12 present the corresponding tests in the rubber-sand fill.

5.5.2 Direct Shear Tests

The direct shear tests were performed with the large direct shear box. The samples were prepared by placing and compacting with a small tamper the initial layer of material in the lower part of the shear box. The lower part of the shear box has 3 in. (0.07 m) in depth and has plan dimensions of 14.5 in. (0.36 m) in length by 12 in. (0.3 m) in width which allows for 2.5 in. (0.06 m) maximum displacement during shear. The upper part of the shear box has 6 in. (0.15 m) in depth and plan dimensions of 12 in. by 12 in. (0.3 m by 0.3 m). The upper box was placed and filled with 2 in. layers of material that were compacted with a small tamper until the final height was reached and the sample was covered with a metal plate. The unit weights obtained during the placement and compaction of each sample were similar to those in the pullout box. The normal pressure was measured by a load cell mounted on the lever arm system.

5.5.3 Interface Tests

Interface tests are performed to measure directly the interface resistance between the geotextile and the two backfill materials used. The interface tests were performed in a setup similar to the large direct shear box. The lower part of the box has 2 in. (0.05 m) in depth and has plan dimensions of 13.5 in. (0.36 m) in length by 12 in. (0.3 m) in width which allows for 1.5 in. (0.04 m) maximum displacement during shear. The lower box was filled with a fine compacted sand and a geotextile sample was placed on top. The geotextile was fixed to the end opposite to the direction of displacement and the upper box was placed on top of it.

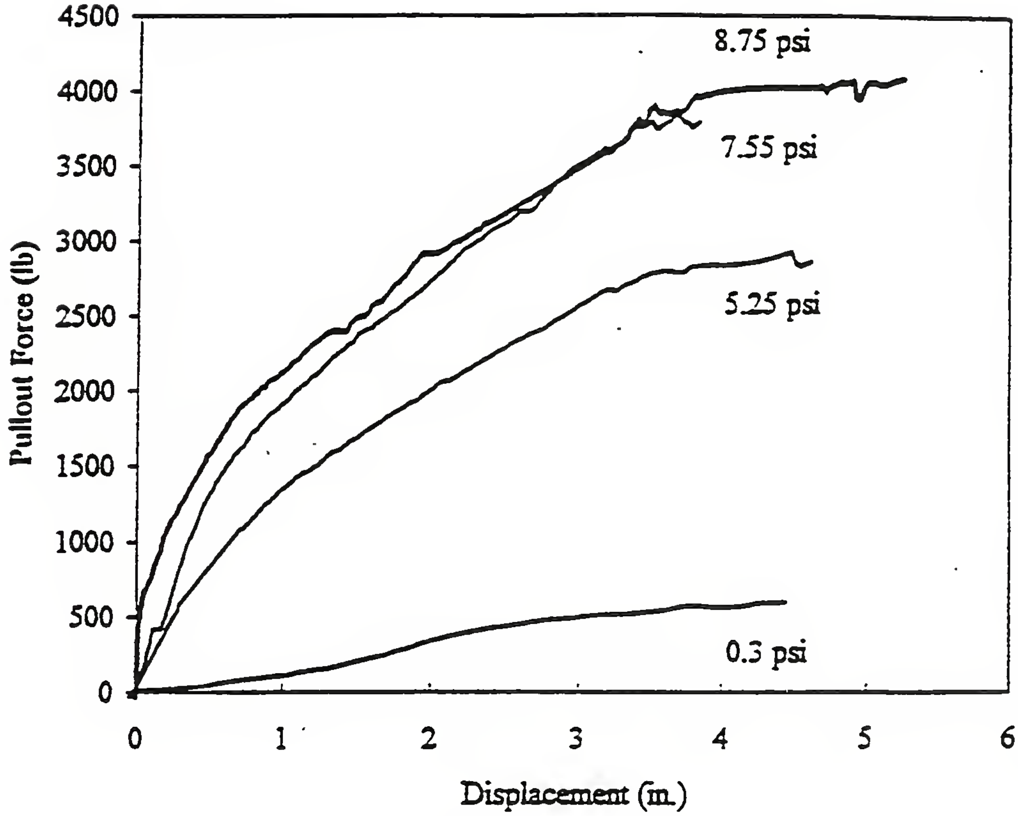


Figure 5.5 Pullout of 4 in. geogrid in tire shreds

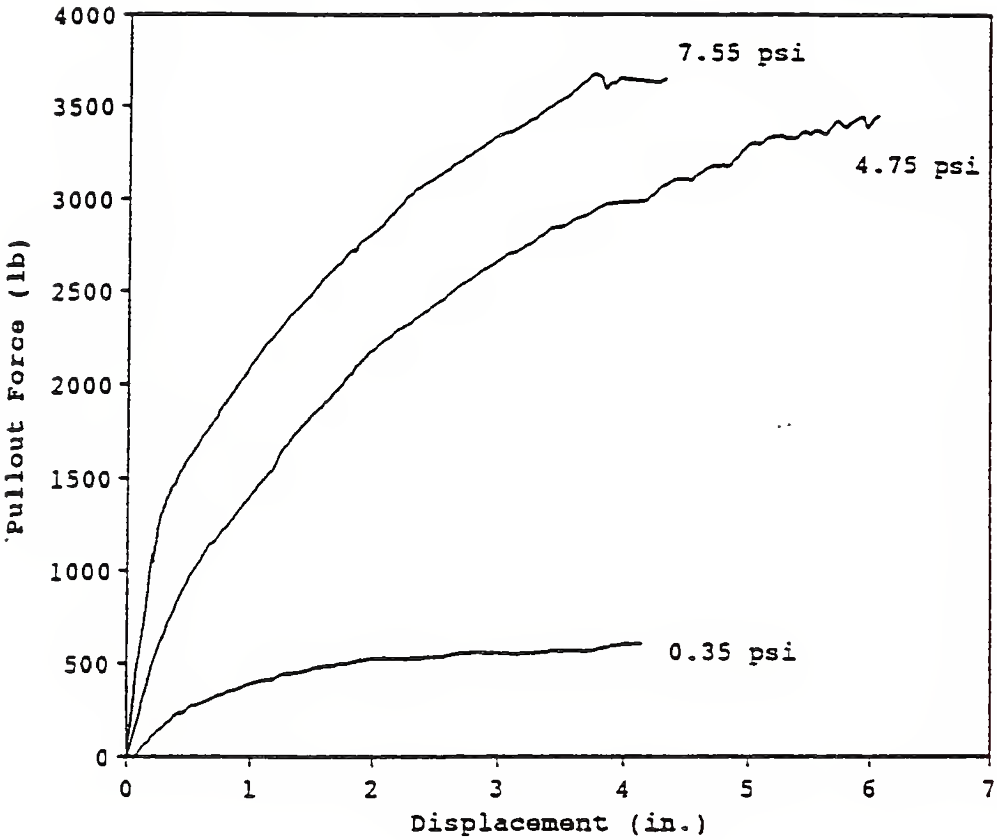


Figure 5.6 Pullout of 2 in. geogrid in tire shreds

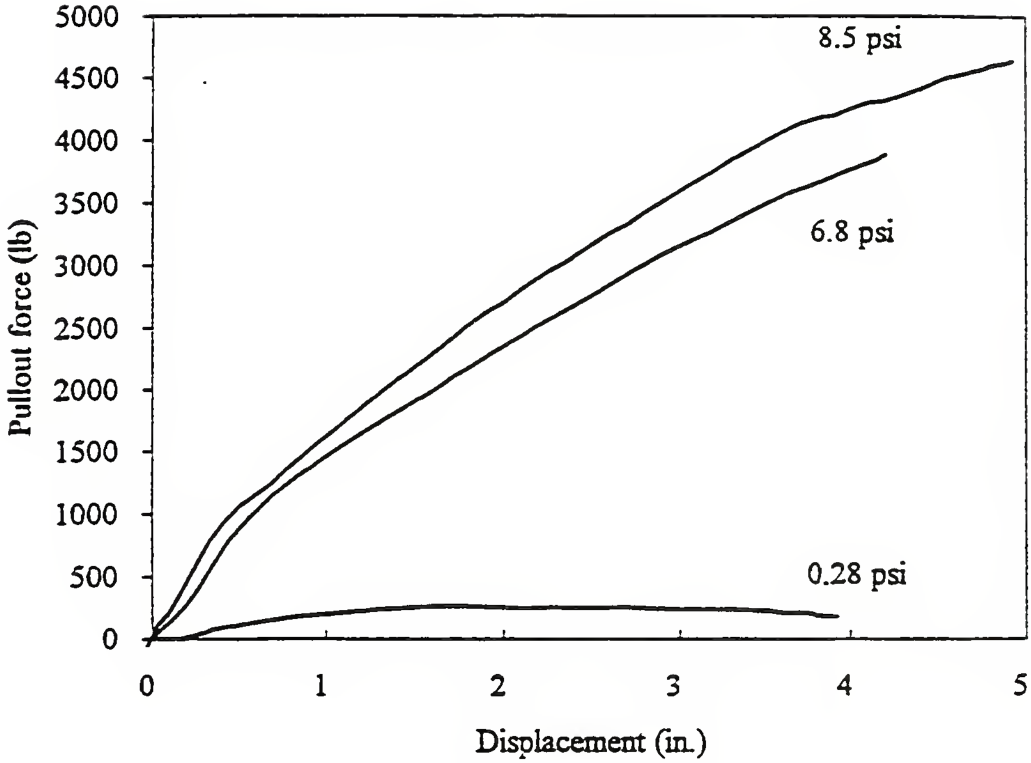


Figure 5.7 Pullout of 0.8 in. geogrid in tire shreds

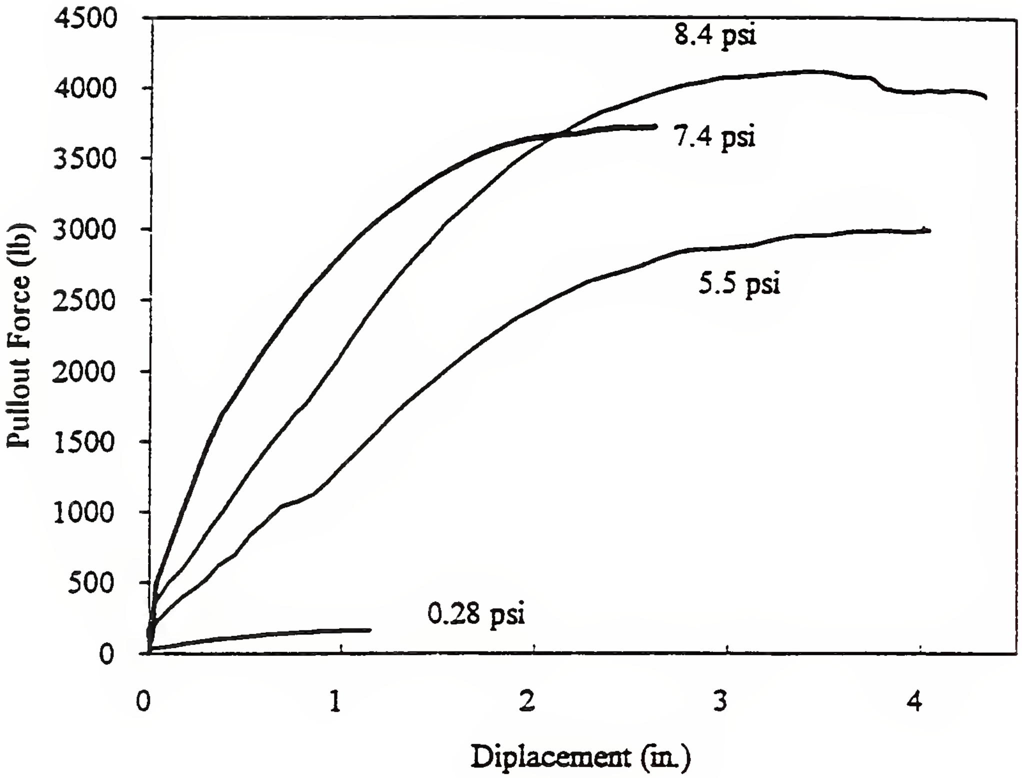


Figure 5.8 Pullout of geotextile in tire shreds

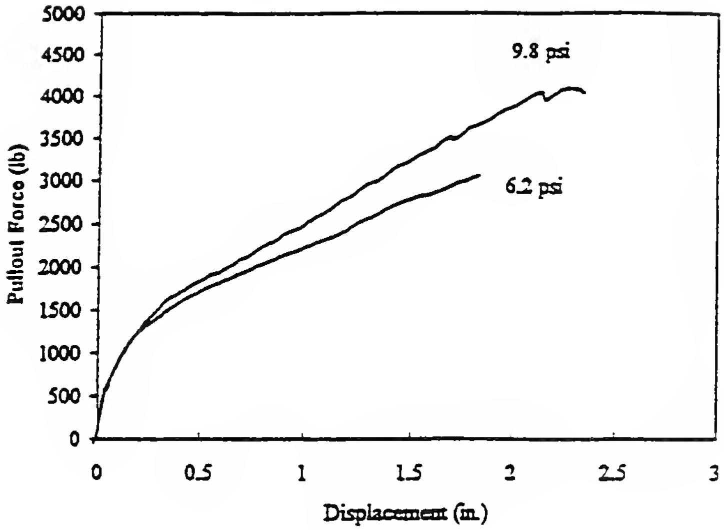


Figure 5.9 Pullout of 4 in. geogrid in rubber-sand

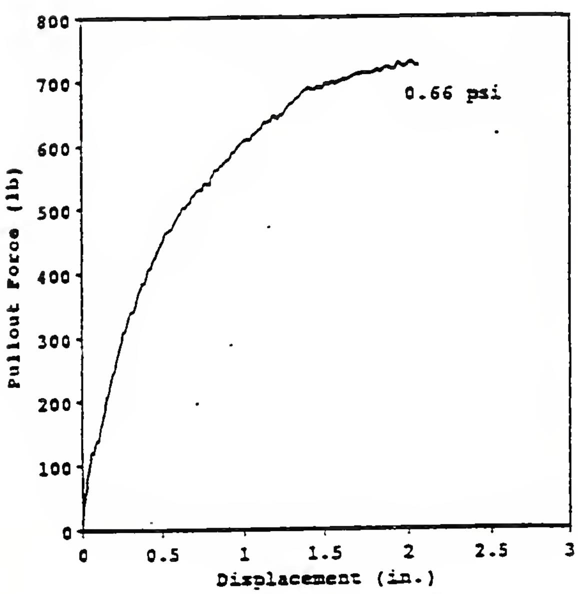


Figure 5.10 Pullout of 2 in. geogrid in rubber-sand

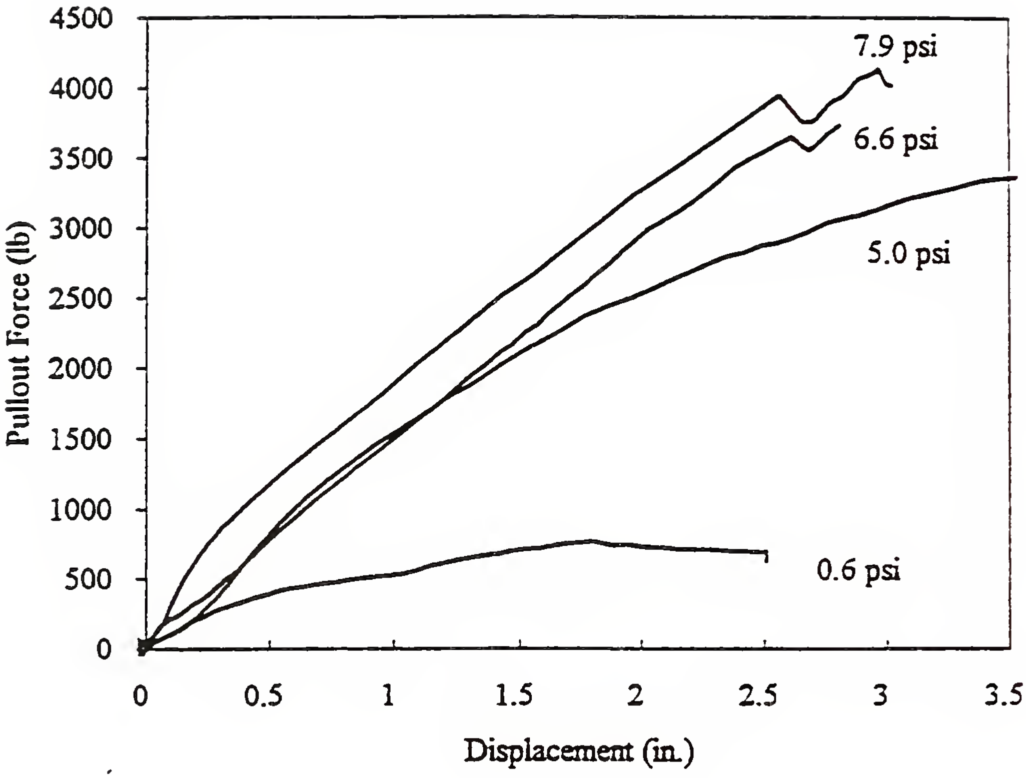


Figure 5.11 Pullout of 0.8 in. geogrid in rubber-sand

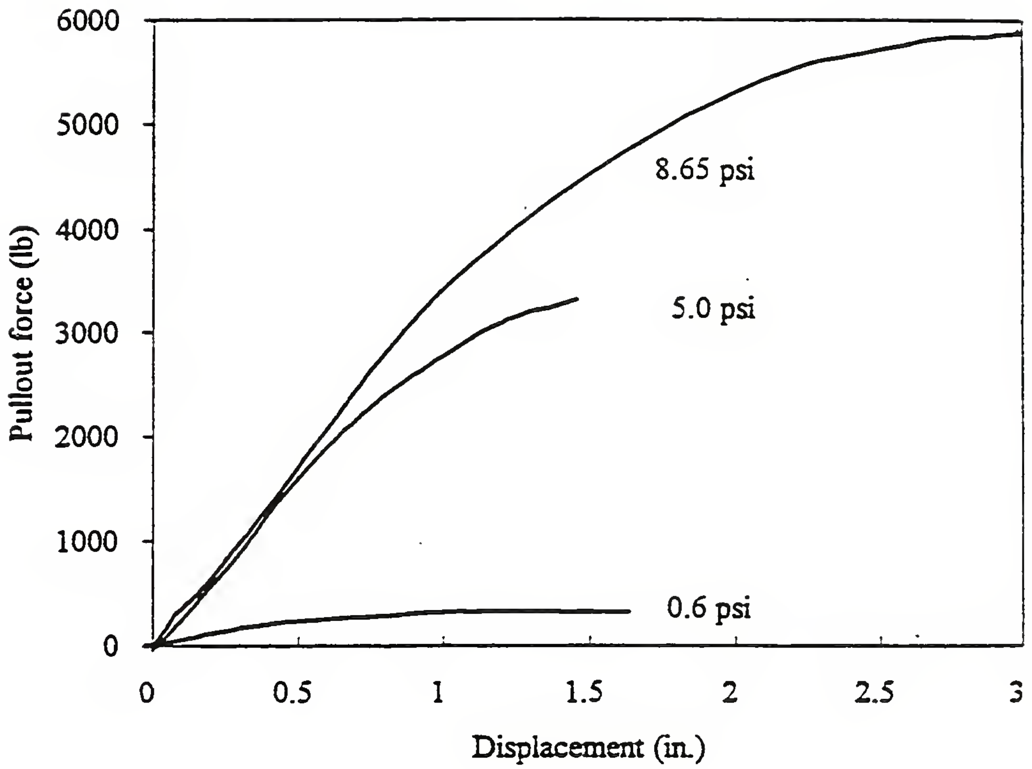


Figure 5.12 Pullout of geotextile in rubber-sand

The upper part of the box has 6 in. (0.15 m) in depth and plan dimensions of 12 in. by 12 in. (0.3 m by 0.3 m). The upper box was placed and filled with 2 in. layers of material that were compacted with a small tamper until the final height was reached and the sample was covered with a metal plate. The unit weights obtained during the placement and compaction of each sample were similar to those in the pullout box. The normal pressure was applied by an air cylinder and was measured by a load cell placed on the setup. The upper box is connected to two hydraulic cylinders similar to those used for pullout testing. The upper box is pulled by the hydraulic cylinders and moves the backfill material over the geotextile attached to the lower box.

The results of the interface tests showed that the interface friction angle between tire shreds and the geotextile was 30° and between rubber-sand and the geotextile was 32°for confining pressures of 1, 5 and 9 psi.

5.6 Analysis of Results

The factors that influence the measured properties during pullout testing are generally related to the testing equipment, boundary effects, testing procedure, rate of loading, geosynthetic characteristics, backfill properties (e.g., dry unit weight, moisture content, relative density, particle shape and size distribution, etc.), placement procedure and confining pressure. The analysis of pullout test results requires that direct shear tests be performed on the two types of backfill materials to determine their strength parameters under the confining pressures used for the pull-out tests. The results of the direct shear tests can be seen in Figure 5.13 for tire shreds and in Figure 5.14 for rubber-sand.

These backfill materials do not present a well defined peak shear strength as the sample is sheared. The shear strength parameters for tire shreds and rubber-sand must be defined at pre-established levels of deformation. The values for the strength angle and the strength intercept were obtained from the direct shear tests on tire shreds and rubber-sand agree with results obtained by other researchers (Ahmed, 1993; Humphrey et. al., 1993).

The mobilized strength angle for various displacements is shown in Figure 5.15 for tire shreds and Figure 5.16 for rubber-sand. This behavior could be attributed to the densification of the

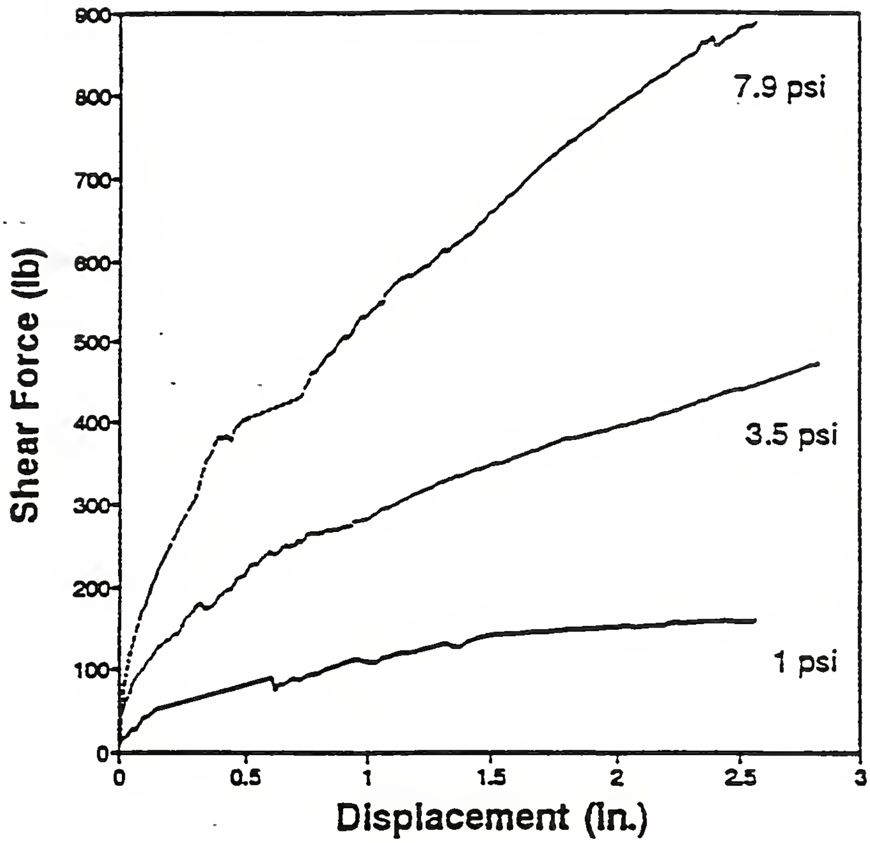


Figure 5.13 Direct shear of tire shreds

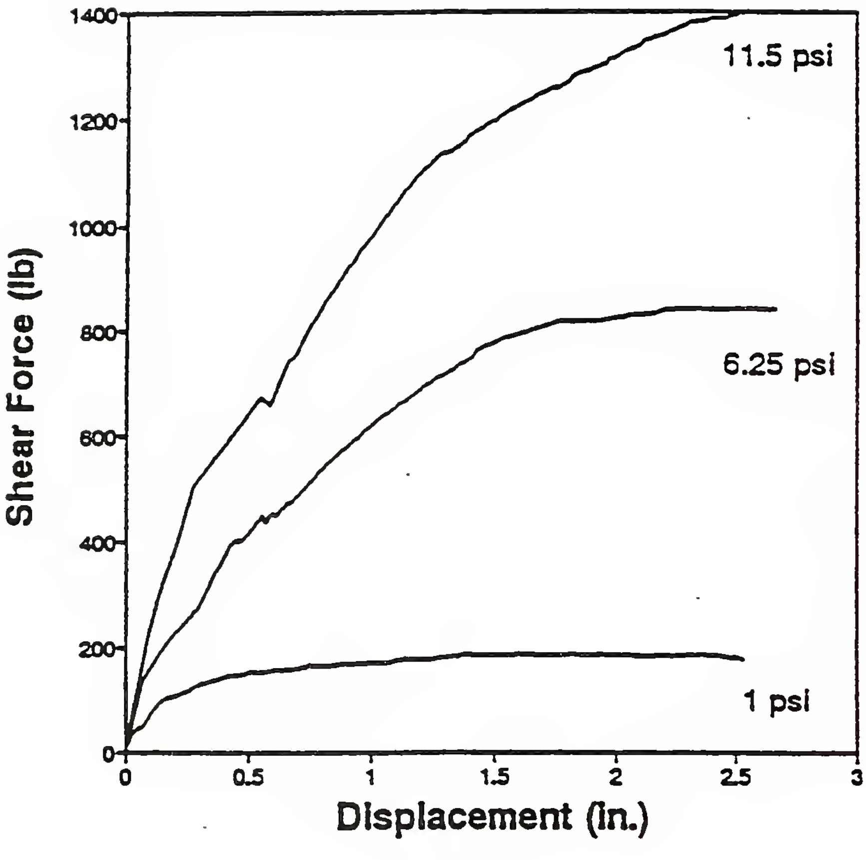


Figure 5.14 Direct shear of rubber-sand

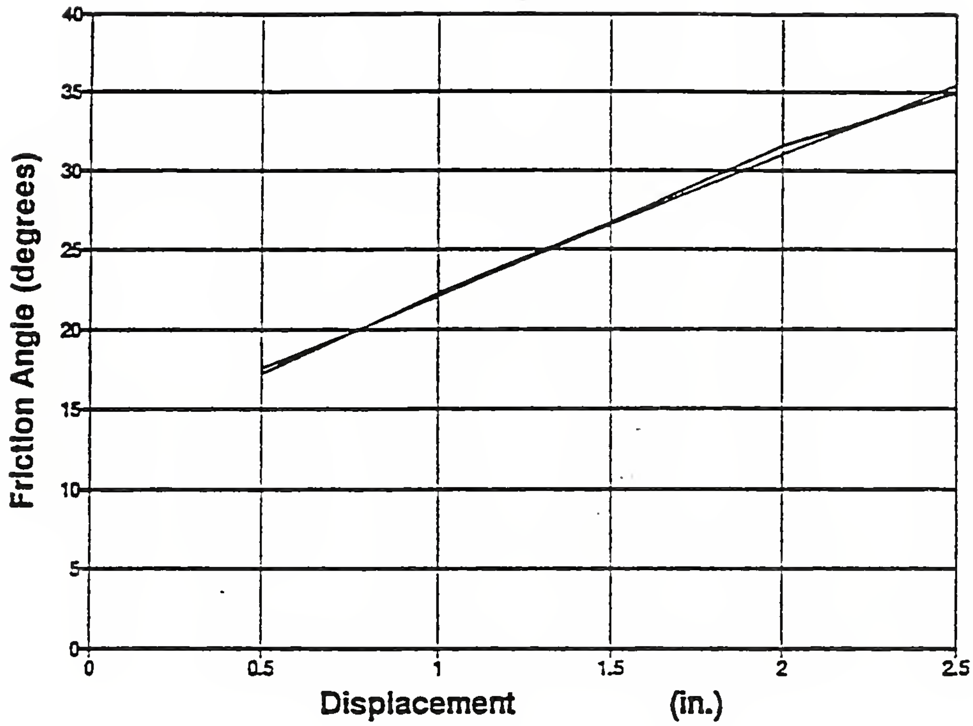


Figure 5.15 Mobilized ϕ for tire shreds

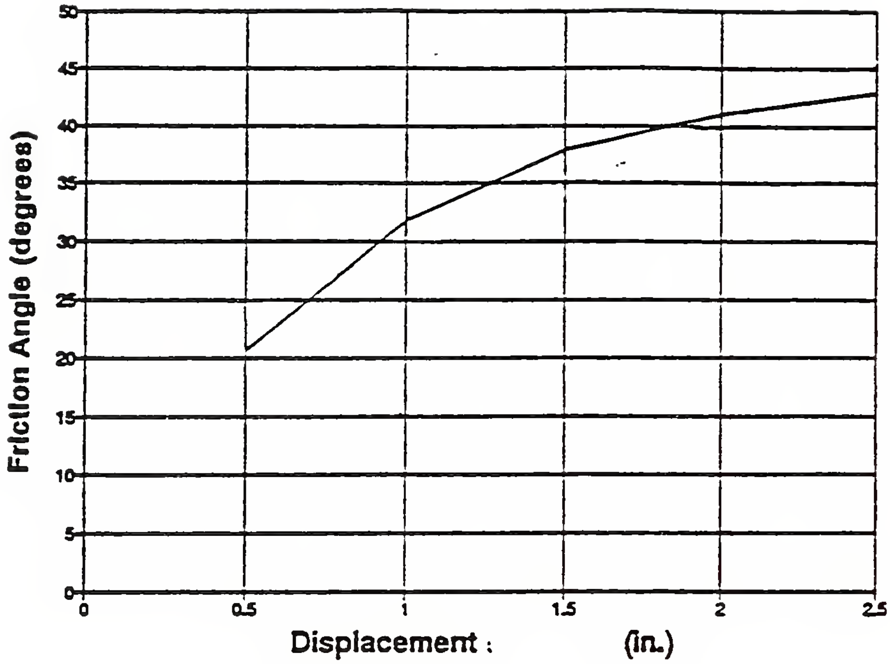


Figure 5.16 Mobilized ϕ for rubber-sand

test sample as it was being sheared. Compacted tire shreds tend to interlock with each other due to the presence of exposed steel belting and their high compressibility. During shear, the tire shreds in the lower box are pulled towards the moving front of the direct shear box, possibly causing densification of the material in the area that will be sheared.

The large apertures in the geogrid allow for the backfill material to pass through and generate pullout resistance through two separate mechanisms. The first mechanism is the shear resistance between the top and bottom area of the longitudinal and transverse ribs of the geogrid and the backfill material. The second mechanism is passive resistance of the backfill material against the front of the transverse ribs of the geogrid. The backfill material goes into a state of passive resistance and opposes the geogrid pullout by means of bearing capacity (Koerner, 1993).

Pullout results are analyzed by envisioning what happens during the test. As the geogrid is pulled from within the backfill, the material directly above and below the geogrid is sheared forming two shearing surfaces. The coefficient of interaction (C_i) is specific for the type of geosynthetic and backfill material tested and is determined by comparing the measured pullout force with the shear strength of the backfill material under the same confining pressure (see equation 5.1):

$$C_i = \frac{F_p}{2(L)(W)(\sigma_n \tan\phi + c)} \qquad (5.1)$$

where:

C_i = Coefficient of interaction

F_p = Measured pullout force

L = Initial length of geogrid test specimen

W = Initial width of geogrid specimen

σ_n = Applied normal stress on test specimen

ϕ = Strength angle of backfill material

c = Strength intercept of backfill material

Table 5.1 presents a summary of the test results. Since the pullout force of geogrids in tire shreds and rubber-sand does not exhibit a clear peak, it is necessary to define a displacement level under which both the pullout force and shear strength of the materials tested can be compared. This level was established at 2.5 in. (6.25 cm) shear displacement for tire shreds and 2.0 in. (5 cm) shear displacement for the rubber-sand mixture, since the latter material shows a stiffer response. Figure 5.17 shows the relationship between C_i and σ_n for tire shreds and Figure 5.18 for rubber-sand.

From the limited test data some trends can be observed. The aperture size of the geogrid seems to have a strong influence on C_i; as the aperture size decreases the value of C_i increases and reaches its maximum level for geotextiles (geotextiles can be thought of as geogrids with a null aperture size). This observation supports the idea that for the materials tested the greatest effect on mobilized pullout force comes from shearing resistance and not from passive resistance of the fill particles located in the apertures.

The 0.8 inch aperture geogrid and the geotextile produce small values for C_i at low confining pressures due to slipout of the geosynthetic (very small shearing resistance is mobilized and their passive resistance is negligible since the tire shred particles cannot pass through the apertures).

It has been observed in most pullout testing programs that C_i decreases with increasing confining pressure and it is recommended to determine the normal pressure on each layer and use the appropriate C_i for a geogrid reinforced wall design (Swan, 1995).

The values in Table 5.2 for tire shred and rubber-sand backfill may be considered low when compared to C_i values published by Huesker, Inc. for sand and this type of geogrid i.e. C_i ~0.9-1.1, but the direct shear and triaxial tests show that these backfill materials do not behave as a sand.

One explanation for the low C_i values observed could be due to the possibility that the two shearing areas above and below the geogrid are not fully developed and the displacement of the geogrid during the test only affects the material in the immediate vicinity of the geogrid impeding full shear strength mobilization.

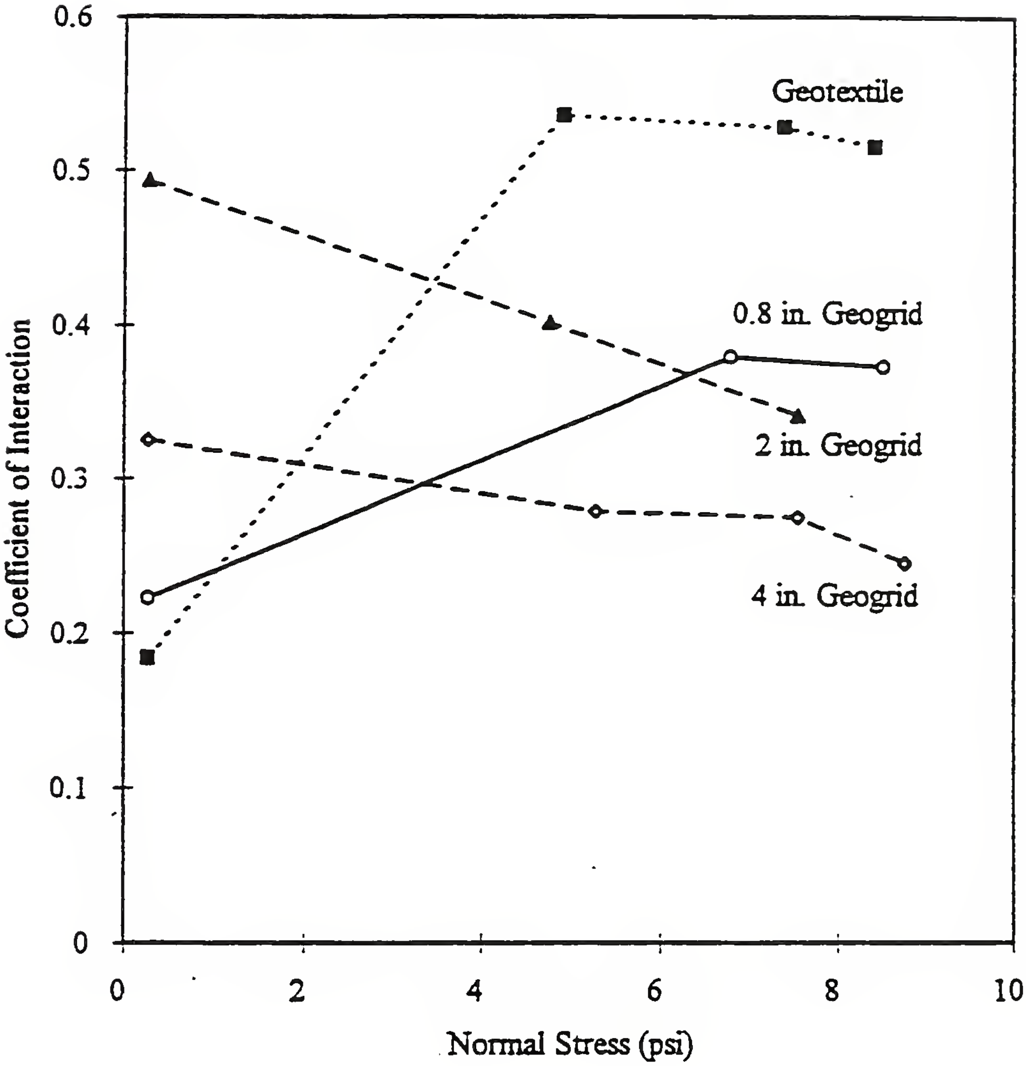


Figure 5.17 Tire Shreds Coefficient of Interaction

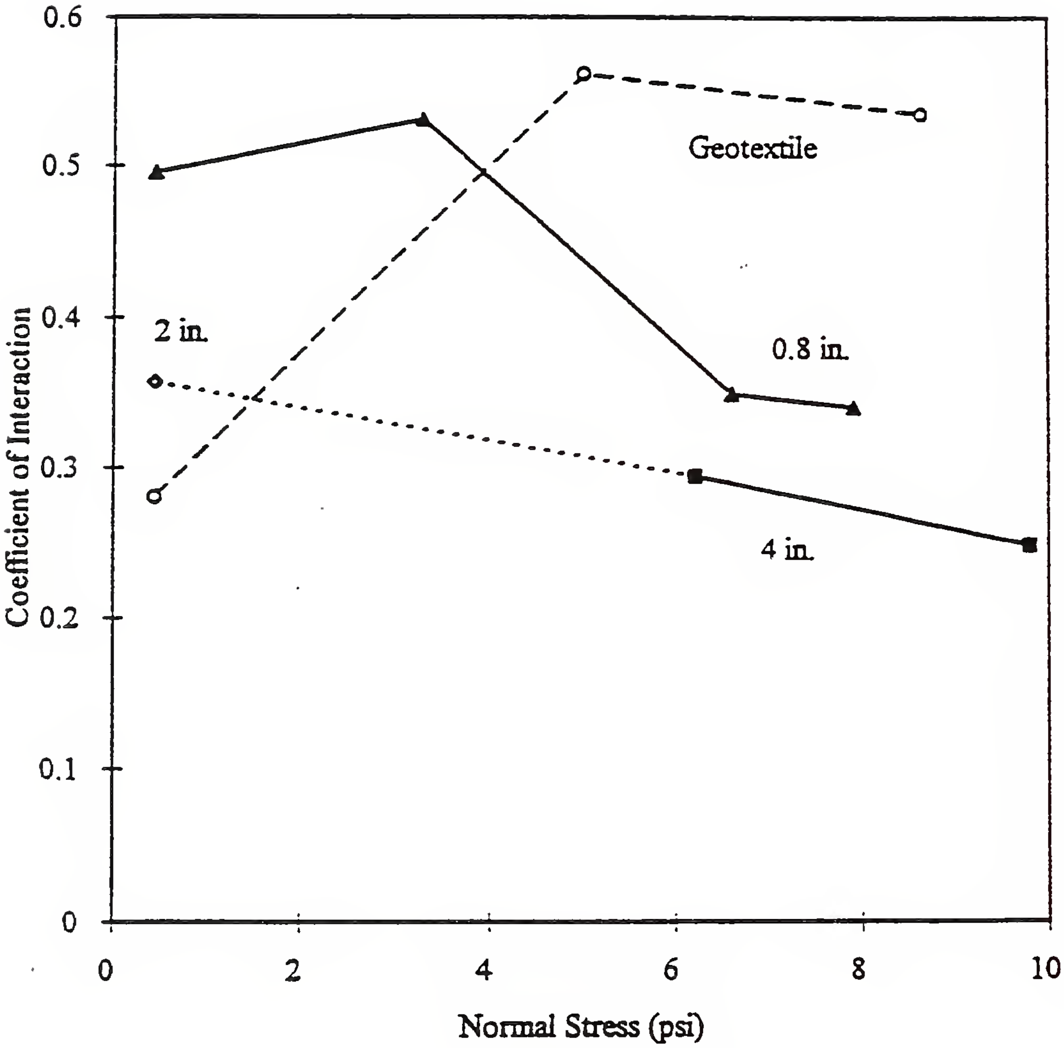


Figure 5.18 Coefficient of Interaction for Rubber-sand

Table 5.1 Pullout test results

Test	Geosynthetic	Material	σ_n (psi)	F_p (lb)	C_i	Failure
1	4 in. geogrid	Tire shreds	0.28	434.4	0.33	No
2	4 in. geogrid	Tire shreds	5.25	2273.9	0.28	No
3	4 in. geogrid	Tire shreds	7.55	3112.6	0.28	Rows 1-7
4	4 in. geogrid	Tire shreds	8.75	3176.5	0.25	Rows 1-4
5	2 in. geogrid	Tire shreds	0.28	532.7	0.49	No
6	2 in. geogrid	Tire shreds	4.75	2425.7	0.40	All rows
7	2 in. geogrid	Tire shreds	7.55	3116.6	0.34	All rows
8	0.8 in. geogrid	Tire shreds	0.28	264.9	0.22	No
9	0.8 in. geogrid	Tire shreds	6.80	2751.0	0.38	No
10	0.8 in. geogrid	Tire shreds	8.50	3150.0	0.37	Row 1
11	Geotextile	Tire shreds	0.28	161.8	0.18	No
12	Geotextile	Tire shreds	4.90	2713.0	0.53	No
13	Geotextile	Tire shreds	7.40	3707.2	0.52	No
14	Geotextile	Tire shreds	8.40	3897.6	0.51	No
15	4 in. geogrid	Rubber-sand	6.20	3048.7	0.36	Rows 1-7
16	4 in. geogrid	Rubber-sand	9.80	3873.0	0.29	Rows 1-3
17	2 in. geogrid	Rubber-sand	0.44	724.0	0.25	No
18	0.8 in. geogrid	Rubber-sand	0.44	772.5	0.49	No
19	0.8 in. geogrid	Rubber-sand	3.30	2520.0	0.53	No
20	0.8 in. geogrid	Rubber-sand	6.60	2940.0	0.35	Rows 4-5
21	0.8 in. geogrid	Rubber-sand	7.90	3260.0	0.34	Rows 1-5
22	Geotextile	Rubber-sand	0.44	328.0	0.28	No
23	Geotextile	Rubber-sand	5.00	3302.7	0.56	No
24	Geotextile	Rubber-sand	8.65	5300.0	0.53	No

Table 5.2 Recommended C_i values

Backfill Material	Geotextile	0.8 in. Geogrid	2 in. Geogrid	4 in. Geogrid
Tire Shreds	0.18-0.53	0.22-0.37	0.34-0.49	0.25-0.33
Rubber-Sand	0.28-0.56	0.34-0.53	0.25-0.36	0.25-0.36

5.7 Applications

Geogrids and woven geotextiles have been used effectively to improve the performance of embankments and backfills by reducing deflections, settlement and earth pressures and by increasing bearing capacity and adding confinement. The geogrids should be placed within the tire chip or rubber-sand fill to increase the lateral confinement of the system, improve the shear modulus due to vertical confinement and spread the vertical stresses due to the tensioned membrane effect. The lateral confinement should resist the tendency of the fill to "walk out" under repetitive surface traffic loads (Koerner, 1993).

It has been shown through cyclic loading tests on unreinforced and geogrid reinforced conventional soil embankment sections under dry (strong) and saturated (weak) subgrade conditions that failure occurs later in reinforced sections than in unreinforced sections for both subgrade conditions. The elastic strain and angle of curvature are reduced by 50% in the reinforced section indicating a load-spreading effect. Permanent deformations were also reduced. At a 20-mm vertical deformation failure assumption, the nonreinforced section carried 110,000 load repetitions and the reinforced section carried 320,000 (Abd El Halim et.al., 1983). The ratio of load repetitions is called a geogrid effectiveness factor (GEF) and equals 2.9. These results should be applicable to reinforced tire chips and rubber-sand courses, but the effectiveness should be determined.

Geogrids have also been used to reinforce unpaved roads. The mechanisms of reinforcement are increased soil strength , load spreading, and membrane support via controlled rutting. The difference in required thickness of stone base is compared to the cost of the installed geogrid. If the later is less expensive (as is usual for soil subgrades with CBR values lower than 3 to 5) it is recommended to use a geogrid (Koerner, 1993). The performance of embankments on soft subgrades would be improved by using a lightweight fill such as tire chips or rubber-sand.

Geotextiles have been used to separate the tire chip course from the borrow cover material in several projects in Maine (see Chapter 2). Woven geotextiles could perform the double function of separation and reinforcement in this type of situations. It is recommended that the road remains unpaved for some time to allow the traffic loads to deform the geotextile around the fill and generate the confining effect through tension (pre-tensioning the fabric could be advisable).

Conventional gravity wall systems resist lateral pressure by virtue of their large mass. Reinforced fills act by reducing the lateral pressure on the wall face by transmitting it to the geotextile or geogrid layers in the wall backfill. These walls are relatively flexible compared to massive gravity structures. The construction sequence followed by the U.S. Forest Service for geotextile walls is applicable for tire shreds and rubber-sand fills. Compaction of each layer will produce displacements of the wall facing that will increase the tension in the geotextile, provide more confinement of the fill material and mobilize a larger pullout force within the fill.

5.8 Summary and Conclusions

The present chapter introduces the parameters necessary for the design of a geogrid reinforced tire shred or rubber-sand backfill structure. Tire shreds have approximately one third and rubber-sand has approximately two thirds of the compacted unit weight of conventional backfill materials. The strength parameters of these lightweight materials make them ideal for use in areas of weak soils. Tire shreds and rubber-sand, to a lesser degree, are compressible and a soil cover could be used to avoid negative effects.

The advantages of constructing geogrid reinforced fills with these types of materials would include ease of construction (because material handling is as simple as with conventional materials), limited deformation of the facing and reduction of earth pressures.

CHAPTER 6
NUMERICAL MODELING

6.1 Introduction

Finite element analysis of embankments or retaining walls require an accurate model of the stress-strain behavior of the subgrade, foundation soil and backfill. The formulation of a stress-strain model for soils must take into account the nonlinear, inelastic and stress dependent behavior of soils. Furthermore, factors such as density, water content, drainage conditions and stress history influence the stress-strain behavior of soils. A number of constitutive models have been proposed in recent years. One of these, the hyperbolic model, has been widely used and is readily available in several FE programs. The input parameters for the hyperbolic model can be obtained from conventional soil tests.

6.2 The Finite Element Program (SSCOMPPC)

The finite element program SSCOMPPC (Boulanger et al., 1991) is a general, plane-strain finite element code for incremental modeling of soil placement and compaction. The features of this finite element program include: (1) interface elements to model the interaction between different soil types or between structural elements, or between soil and structural elements, (2) the ability to model compaction induced stresses and deformations, and (3) the incremental placement of structural elements in reinforced soil walls.

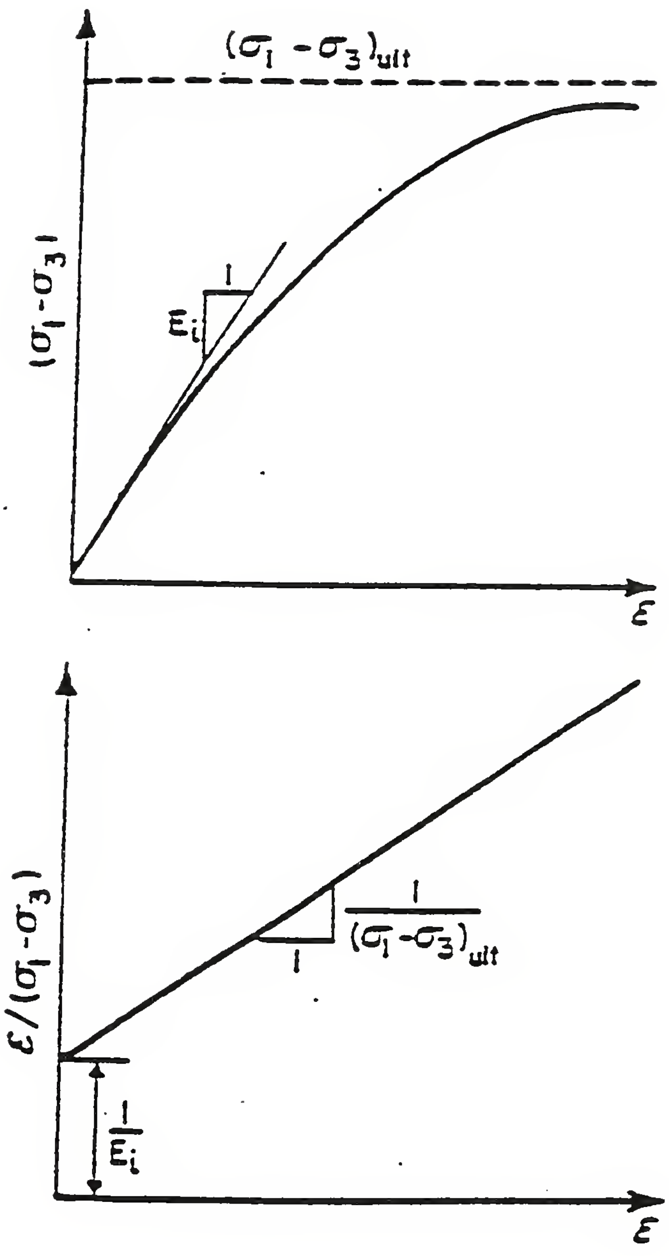
Four types of elements are used in the program to model each component of reinforced soil structures discretely and to model soil-structure interaction effects. They are: (1) soil elements, which are four-node, two-dimensional isoparametric elements, (2) bar elements, which are two-node, one-dimensional elastic elements with axial stiffness only, (3) beam elements, which are two-node, one-dimensional elastic elements with axial and bending stiffness, and (4) interface elements, which are four-node elements with zero thickness and normal and tangential stiffness. The PC version of SSCOMP accepts a maximum of 500 nodes and 300 elements.

The finite element program calculates stresses, strains, and displacements in the soil elements, as well as the internal forces and displacements in structural elements, by means of analyses that simulate the actual sequence of construction operations in a number of steps. The nonlinear stress dependent stress-strain properties of the soil are approximated by varying the values of the modulus and Poisson's ratio. Stress estimation is done in a double iterative process for every analysis increment. The first iteration uses the modulus and Poisson's ratio corresponding to the stress condition at the beginning of the increment, and the second iteration uses adjusted soil properties based on the average stresses during the increment.

6.3 Soil Elements

Soil response to loading is highly nonlinear, inelastic and extremely dependent on the magnitude of stress. This behavior has a significant influence on the stresses and displacements developed within the structure. Nonlinear elastic (hyperbolic) models can be expected to provide acceptable prediction of the soil behavior at relatively low shear stress levels. The soil stiffness modeled in this manner increases with increasing confining pressure and decreases with increasing shear stress level (see Figure 6.1). A very low stiffness is assigned to elements with stress condition at failure.

The hyperbolic model is relatively simple, well validated and it reliably represents soil behavior. In general, previous studies have shown that the model is also appropriate for modeling reinforced soil behavior. The parameter values can be determined from the results of conventional triaxial compression tests. Consolidated drained triaxial compression test data are considered the best tool to determine the soil parameters. However, as long as only total stresses are considered in the analysis, the model can handle consolidated undrained or unsaturated soil behavior.



REAL

$$(\sigma_1 - \sigma_3) = \frac{\varepsilon}{\frac{1}{E_i} + \frac{\varepsilon}{(\sigma_1 - \sigma_3)_{ult}}}$$

TRANSFORMED

$$\frac{\varepsilon}{(\sigma_1 - \sigma_3)} = \frac{1}{E_i} + \frac{\varepsilon}{(\sigma_1 - \sigma_3)_{ult}}$$

Figure 6.1 Hyperbolic model for stress-strain curve for primary loading (after Duncan et al., 1980)

The simple hyperbolic relationship has significant limitations: (1) the nonlinear elastic (hyperbolic) model assumes that stress-strain curves for soils can be approximated by a hyperbolic equation, hence it cannot model post-peak drop in strength or strain softening behavior; (2) it is based on elastic theory, thus it cannot correctly model plastic failure and plastic strains within the soil mass, and (3) bulk modulus modeling requires that the volumetric strain behavior of soil be compressive only, and it therefore cannot represent dilatant behavior.

The parameters are developed for axial compression conditions and may not reflect the true behavior in axial extension or lateral compression. Furthermore, the values of the parameters depend on the soil density, water content, the range of pressures used in testing, and the drainage conditions. The laboratory testing conditions should correspond to field conditions.

The material properties during any solution increment are calculated based on the hyperbolic model described by Duncan et. al. (1980). It is formulated as:

$$(\sigma_1 - \sigma_3) = \frac{\varepsilon}{[(\frac{1}{E_i} + (\sigma_1 - \sigma_3)_{ult}]} \qquad (6.1)$$

where ε is the axial strain; E_i is the initial tangent modulus represented by the initial slope of the stress strain curve, and $(\sigma_1-\sigma_3)_{ult}$ is the asymptotic value of stress difference that is always greater than the compressive strength of the soil. The Mohr-Coulomb criterion is used to define the failure condition $((\sigma_1-\sigma_3)_f)$ expressed as:

$$(\sigma_1 - \sigma_3)_f = \frac{2c\cos\phi + 2\sigma_3\sin\phi}{1-\sin\phi} \qquad (6.2)$$

where c and ϕ are the strength intercept and the strength angle of the soil respectively.

The model allows the variation of the strength angle (ϕ) as a function of confining pressure (σ_3). The decrease of ϕ corresponding to the change of one order of magnitude of σ_3 is defined as $\Delta\phi$.

The instantaneous slope of the hyperbolic stress-strain curve or tangent modulus (E_t) is related to the stress level (SL) by

$$E_t = (1-R_f \cdot SL)^2 E_i \tag{6.3}$$

The stress level (SL) and failure ratio (R_f) are defined as follows:

$$SL = \frac{(\sigma_1-\sigma_3)}{(\sigma_1-\sigma_3)_f} \tag{6.4}$$

$$R_f = \frac{(\sigma_1-\sigma_3)_f}{(\sigma_1-\sigma_3)_{ult}} \tag{6.5}$$

The variation of E_i with confining stress (σ_3) is represented by the following equation:

$$E_i = K p_a \left(\frac{\sigma_3}{p_a}\right)^n \tag{6.6}$$

in which p_a is the atmospheric pressure expressed in the same unit as σ_3 and E_i, whereby K and n are the modulus number and the modulus exponent, both of which are dimensionless.

A loading-unloading modulus (E_{ur}) is used for a loading-unloading situation (Figure 6.2). This value is related to the confining pressure (s_3) as

$$E_{ur} = K_{ur}\ p_a \left(\frac{\sigma_3}{p_a}\right)^n \tag{6.7}$$

where K_{ur} is the loading-unloading modulus number whose value is always greater than the value of K for primary loading. For stiff soils, such as a dense sand, K_{ur} may be 20% greater than K, while for soft soils like a loose sand, K_{ur} can be three times as large as K.

Many soils exhibit non-linear and stress dependent volume change characteristics. The bulk modulus of the soil (B) is assumed to be independent of stress level, but varies with confining pressure (σ_3). The variation of the bulk modulus with the confining pressure is approximated by an equation of the form:

$$B = K_b p_a \left(\frac{\sigma_3}{p_a}\right)^m \tag{6.8}$$

in which K_b is the bulk modulus number and m is the bulk modulus exponent.

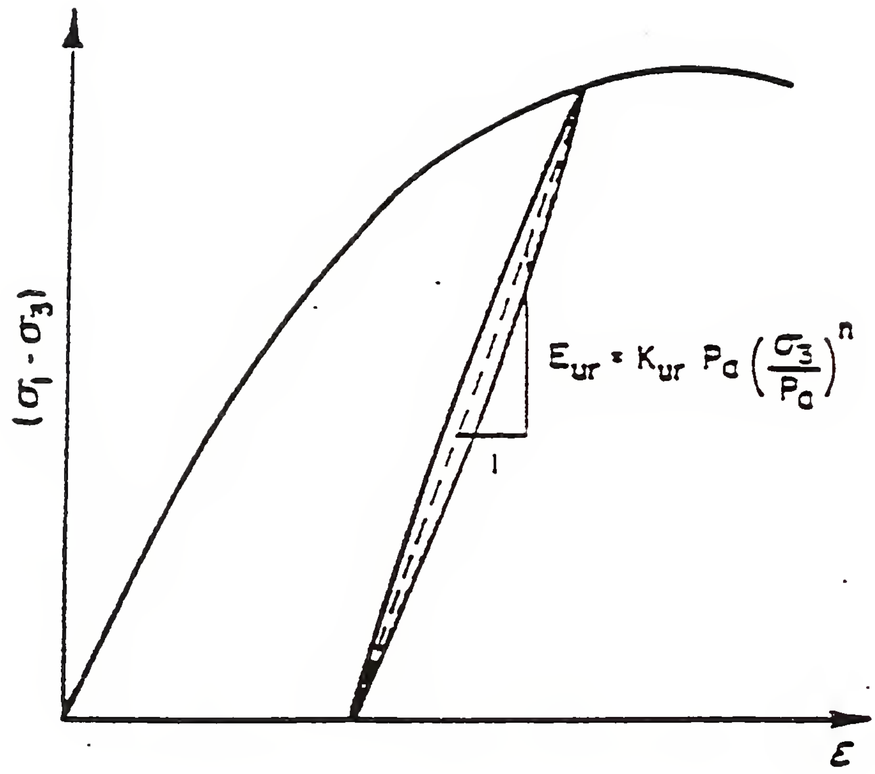


Figure 6.2 Linear stress-strain relationship for unloading-reloading (after Duncan et al., 1980)

The bulk modulus (B) is related to tangent Poisson's ratio of the soil (μ_t) by $\mu_t = (3B-E_t)/6B$. Therefore, in order to keep the corresponding value of μ_t between 0 and 0.5, B should be greater than $B_{min} = (E_t/3)((2-\sin\phi')/\sin\phi')$. In summary, nine parameters are employed in the hyperbolic stress strain relationship including:

c = strength intercept

ϕ = strength angle

$\Delta\phi$ = change in strength angle per log cycle change in confining pressure

K = modulus number

K_{ur} = unloading-reloading modulus number

n = modulus exponent

R_f = failure ratio

K_b = bulk modulus number

m = bulk modulus exponent

6.3.1 Determination of Hyperbolic Parameters for Tire Shreds and Rubber-sand

The hyperbolic parameters for tire shreds and rubber-sand were calculated by following the procedure presented by Duncan, et. al. (1980). The results of the triaxial tests were used for this purpose. The comparison between the hyperbolic model and the laboratory data is presented in Figure 6.3 for tires shreds and Figure 6.4 for rubber-sand. The calculations in Figures 6.5 and 6.6 were used to determine the hyperbolic parameters for tire shreds and in Figures 6.7 and 6.8 for rubber-sand. The parameters used for the finite element analysis are presented in Tables 6.1 and 6.2 for tire shreds and rubber-sand respectively.

Table 6.1 Hyperbolic parameters for tire shreds

$c =$	6 psi	$K =$	15	$K_{ur} =$	45
$\phi =$	29°	$n =$	0.49	$K_b =$	19.4
$\Delta\phi =$	0°	$R_f =$	0.05	$m =$	-0.02

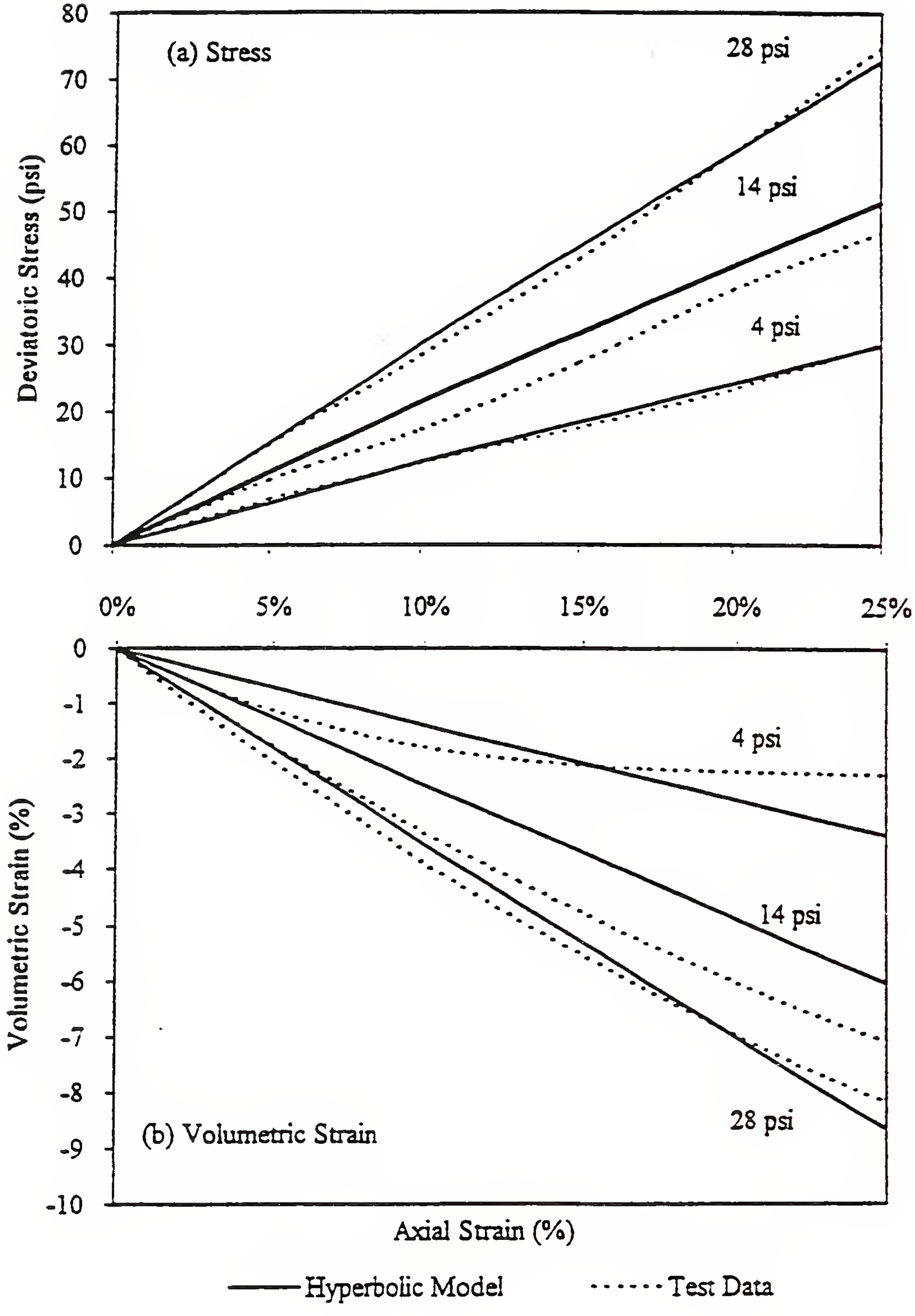


Figure 6.3 Hyperbolic model vs laboratory data for tire shreds

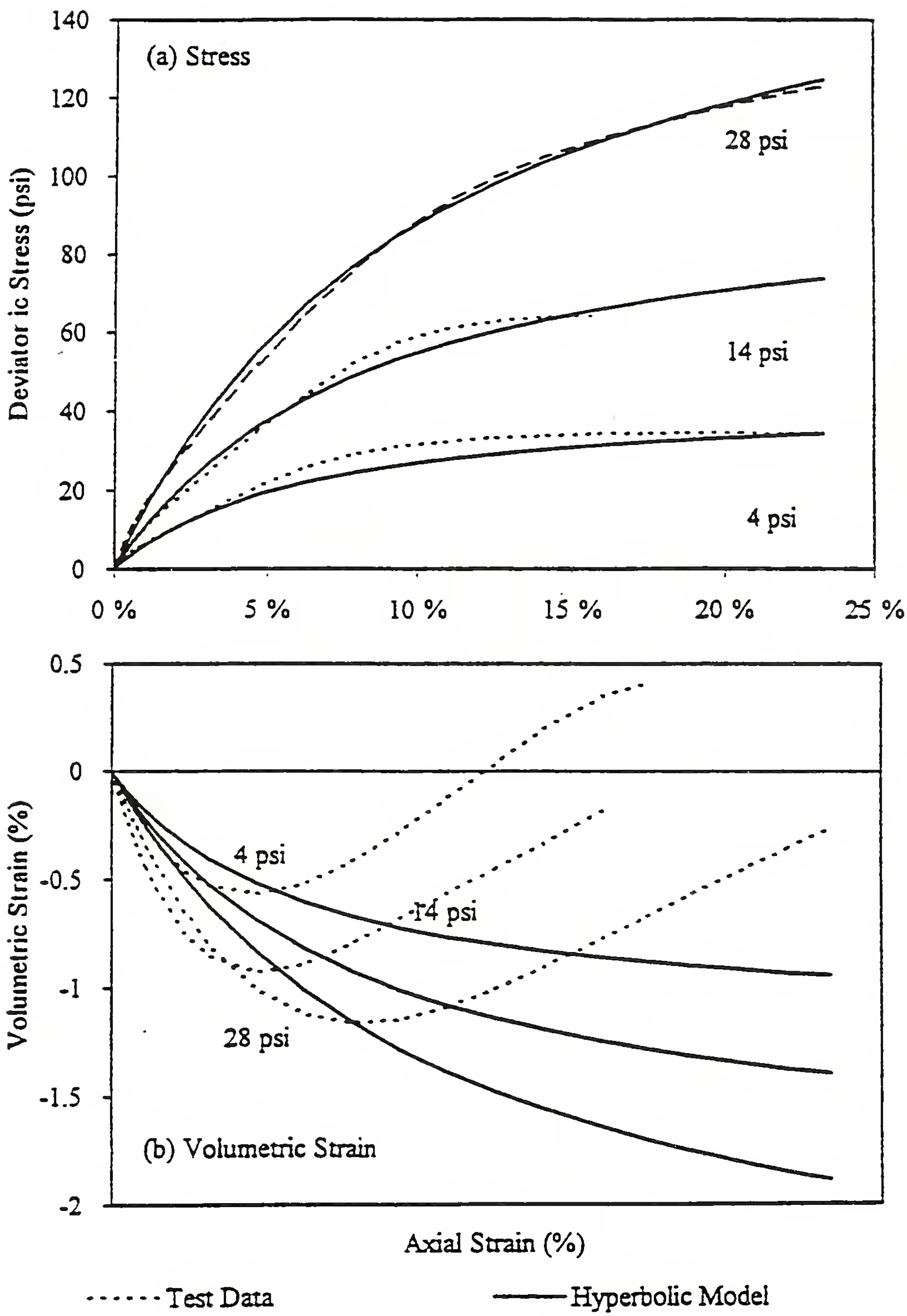


Figure 6.9 Hyperbolic model vs laboratory data for rubber sand

Tire Shreds CD Test $\qquad p_a =$ 14.70 psi

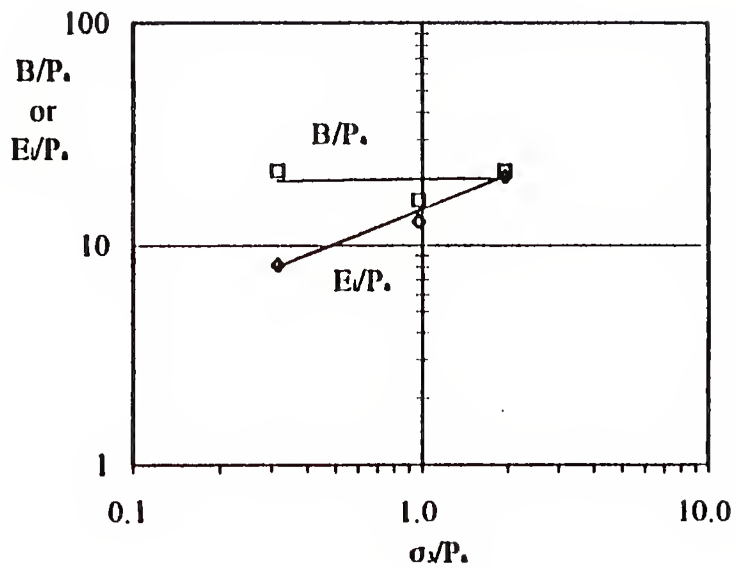
		70% Stress Level			95% Stress Level			Bulk Modulus			
σ_3	$(\sigma_1-\sigma_3)_f$	$(\sigma_1-\sigma_3)$	ε_a	$\varepsilon_a/(\sigma_1-\sigma_3)$	$(\sigma_1-\sigma_3)$	ε_a	$\varepsilon_a/(\sigma_1-\sigma_3)$	$(\sigma_1-\sigma_3)_b$	ε_v	$(\sigma_1-\sigma_3)_b/3\varepsilon_v$	σ_3/p_a
I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII
4.64	30.0	21.0	0.18	0.008	28.5	0.24	0.008	21.0	0.022	318	0.32
14.36	47.0	32.9	0.18	0.005	44.7	0.24	0.005	32.9	0.047	233	0.98
28.86	75.0	52.5	0.18	0.003	71.3	0.24	0.003	52.5	0.055	318	1.96

$\frac{1}{(\sigma_1-\sigma_3)_u}$	R_f	E_i/p_a	B/p_a
XIII	XIV	XV	XVI
0.000	0.00	8.2	21.6
0.000	0.00	12.8	15.9
0.000	0.00	20.4	21.6

$$\frac{1}{(\sigma_1-\sigma_3)_u} = \frac{\text{VIII-V}}{\text{VII-IV}} \qquad R_f = \text{II} \times \text{XIII}$$

$$\frac{E_i}{p_a} = \frac{2.0}{(\text{V+VIII-XIII(IV+VII)})p_a}$$

$$\frac{B}{p_a} = \frac{\text{XI}}{p_a}$$



$K_b = 19.43569$

$m = -0.026$

$K = 13.97012$

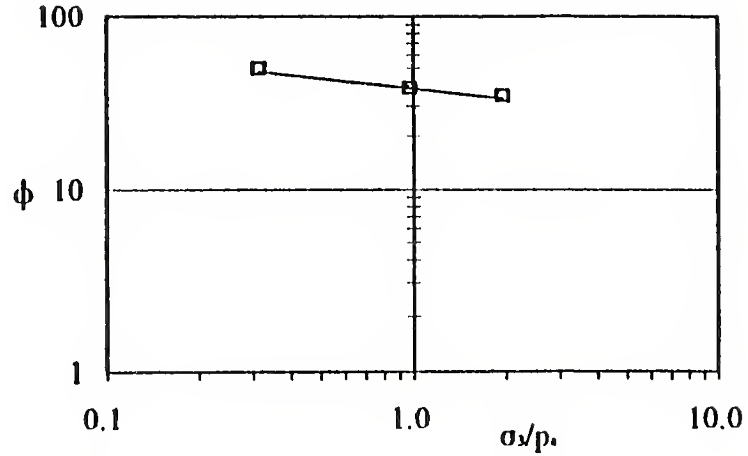
$n = 0.4914$

Figure 6.5 Computation of Hyperbolic Parameters for Tire Shreds

Units for σ, p, q and c (psi)

σ_3	σ_1	ϕ
4.6	34.6	49.8
14.6	61.4	38.4
28.9	103.9	34.4

$\phi_{at\,1\,atm.}$= 39.4 °
$\Delta\phi$= 19.8 °



p	q
19.64	15.00
37.86	23.50
66.36	37.50

Ψ= 25.8 °
α= 5.4 psi

ϕ= 28.8 °
c= 6.2

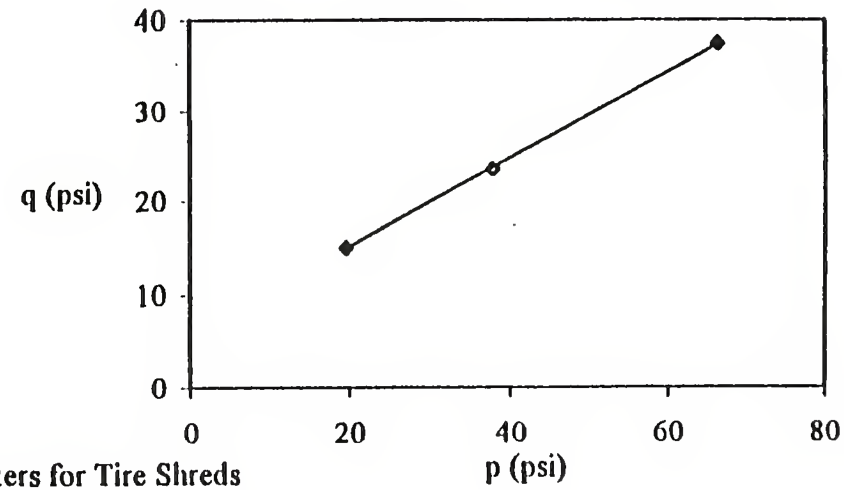


Figure 6.6 Computation of Strength Parameters for Tire Shreds

Rubber-sand CD Test p_a= 14.70 psi

		70% Stress Level			95% Stress Level			Bulk Modulus			
σ_3	$(\sigma_1-\sigma_3)_f$	$(\sigma_1-\sigma_3)$	ε_a	$\varepsilon_a/(\sigma_1-\sigma_3)$	$(\sigma_1-\sigma_3)$	ε_a	$\varepsilon_a/(\sigma_1-\sigma_3)$	$(\sigma_1-\sigma_3)_b$	ε_v	$(\sigma_1-\sigma_3)_b/3\varepsilon_v$	σ_3/p_a
I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII
4.64	34.1	23.9	0.06	0.002	32.4	0.11	0.003	23.9	0.006	1421	0.32
14.64	63.9	44.7	0.06	0.001	60.7	0.11	0.002	44.7	0.009	1657	1.00
28.86	130.0	91.0	0.10	0.001	123.5	0.24	0.002	91.0	0.011	2708	1.96

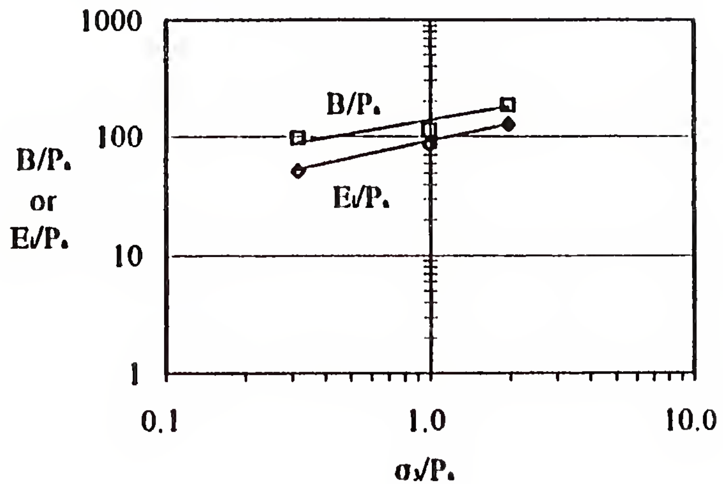
$\frac{1}{(\sigma_1-\sigma_3)_u}$	R_f	E_i/p_a	B/p_a
XIII	XIV	XV	XVI
0.019	0.66	51.8	96.7
0.009	0.59	86.6	112.7
0.006	0.75	126.1	184.2

$$\frac{1}{(\sigma_1-\sigma_3)_u} = \frac{\text{VIII-V}}{\text{VII-IV}}$$

R_f=II x XIII

$$\frac{E_i}{p_a} = \frac{2.0}{(\text{V+VIII-XIII(IV+VII)})p_a}$$

$$\frac{B}{p_a} = \frac{\text{XI}}{p_a}$$



K_b= 133.01

m= 0.3298

K= 83.734

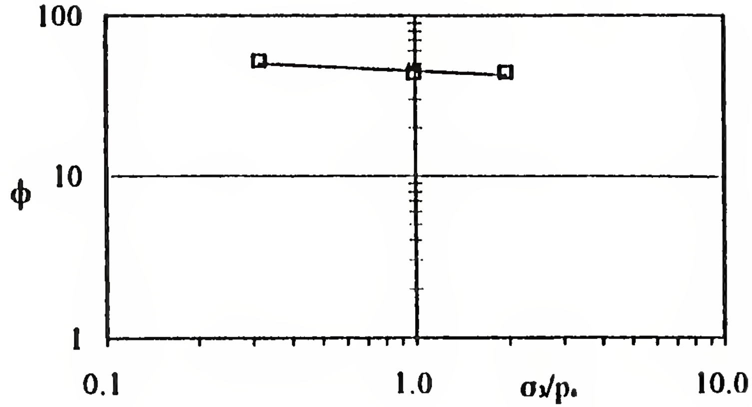
n= 0.4656

Figure 6.7 Computation of Hyperbolic Parameters for Rubber-sand

Units for σ, p, q and c (psi)

σ_3	σ_1	ϕ
4.6	38.7	51.8
14.6	78.5	43.3
28.9	158.9	43.8

$\phi_{at\ 1\ atm.}$= 45.6 °
$\Delta\phi$= 10.8 °



p	q
21.69	17.05
46.59	31.95
93.86	65.00

Ψ= 33.8 °
α= 1.8 psi

ϕ= 42.0 °
c= 2.5 psi

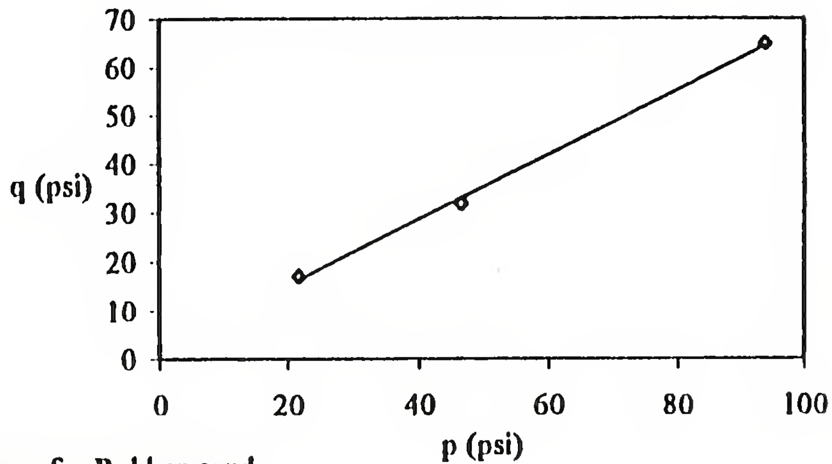


Figure 6.8 Computation of Strength Parameters for Rubber-sand

Table 6. 2 Hyperbolic parameters for rubber-sand

c =	2.5 psi	K =	83.7	K_{ur} =	217.62
ϕ =	42°	n =	0.47	K_b =	120
$\Delta\phi$=	0°	R_f=	0.7	m =	0.33

6.4 Reinforcing Elements

In SSCOMPPC, reinforcing elements can generally be modeled using a one-dimensional bar element. Non-linearity of the stress-strain behavior and yield or creep can be readily modeled by making the element stiffness a function of stress (or strain) level. However, breakage of the reinforcement cannot be modeled in this numerical analysis. It requires the redistribution of stresses developed in the reinforcement prior to breaking, or else an erroneous stress distributions can be obtained.

The bar elements are represented by two-node elements with axial stiffness only. These elements behave as elastic bars that are able to resist axial loading only. The following input parameters are required for bar elements:

E = elastic modulus

A = cross sectional area

g = the weight per unit area or per unit length of the bar

The elastic modulus is calculated as the secant modulus at a strain level of 9% from the results of the laboratory tensile strength test on the reinforcement material used (geogrid or geotextile). The information provided by Huesker Inc. indicate that the ultimate wide width tensile strength achieved at 9% strain (elongation at break) is 1125 lb/in (20 ton/m), the cross sectional area is 0.00064 m^2 and the weight is 13 oz/yd^2 (0.00044 ton/m^2).

6.5 Wall Facing

Beam elements or bar elements may be appropriate to use depending on the type of facing being considered. The use of beam elements to model relatively rigid facing is quite straightforward.

Beam elements are two-node elements capable of exhibiting axial, bending, and shear stiffness. Input parameters for beam elements include:

E = elastic modulus

I = moment of inertia

A = cross sectional area

g = weight per unit length

C_{top} = distance from the neutral axis to the top fiber of the beam

C_{bot} = distance from the neutral axis to the bottom fiber of the beam

The most difficult problem to model is wrap around facings, i.e., facings that consist of only geotextile or geogrid reinforcement wrapped around the soil and locked into place by the overlying fill.

This type of facing has been modeled by vertical bar elements. In any case, correctly modeling the stresses and deformations resulting from the form of construction is not a trivial exercise. The facing for this type of application is expected to be flexible to adjust to the deformations from the fill. The following parameters were used: E=0.28 psi, I=2.94 ft^3, A=1 ft^2, C_{top} = 0.66 ft and C_{bot} = 0.66 ft. Since shear deformations are not considered significant the other parameters are set to zero.

6.6 Interface Elements

Interaction between the soil mass and the reinforcement can be modeled by introducing soil-reinforcement interface elements. In this study, the interface elements are used to model the relative movement between adjacent elements such as the geosynthetic reinforcement (bars) and the soil, the facing (beams or bars) and the soil, the two layers of reinforcement, and also the soil and the loading plate.

If the reinforcement is in the form of a sheet, which completely separates the soil above and below reinforcement, the interface resistance can be readily determined by direct shear tests. On the other hand, if the reinforcement consists of geogrids, with openings that are large compared to the grain size of the soil, or if the reinforcement consists of separate reinforcing strips, then pullout tests are required.

The interface elements have zero thickness and are capable of modeling soil structure interface condition through normal and shear springs. The normal spring is assumed to be a linear elastic material. It controls the opening and compressing of the interface between two adjacent elements. The shear behavior of the interface is modeled by a hyperbolic relationship between the shear stress and the relative shear displacement at the interface. A hyperbolic representation similar to that used for soil (Duncan et al., 1980) is used to describe the non-linear behavior of the interface element (see figure 6.9). The properties of the interface elements include:

c_{int} = interface adhesion

ϕ_{int} = interface friction angle

$\Delta\phi$ = change in interface friction angle per log cycle of σ_3

K_n = normal spring coefficient (recommended value 100,000,000)

K_s = shear spring coefficient (recommended value 5,000 to 25,000)

K_u = unloading shear spring coefficient (recommended value 5,000 to 25,000)

n = shear exponent

R_f = failure ratio

The interface element parameters determined through direct shear interface tests indicate that: c_{int} =0, ϕ_{int}=30° for tire shreds/geotextile and 32° for rubber-sand/geotextile, $\Delta\phi$=0, K_n=100,000,000, K_s=17,500, K_u=17,500, n=1 and R_f=0.9.

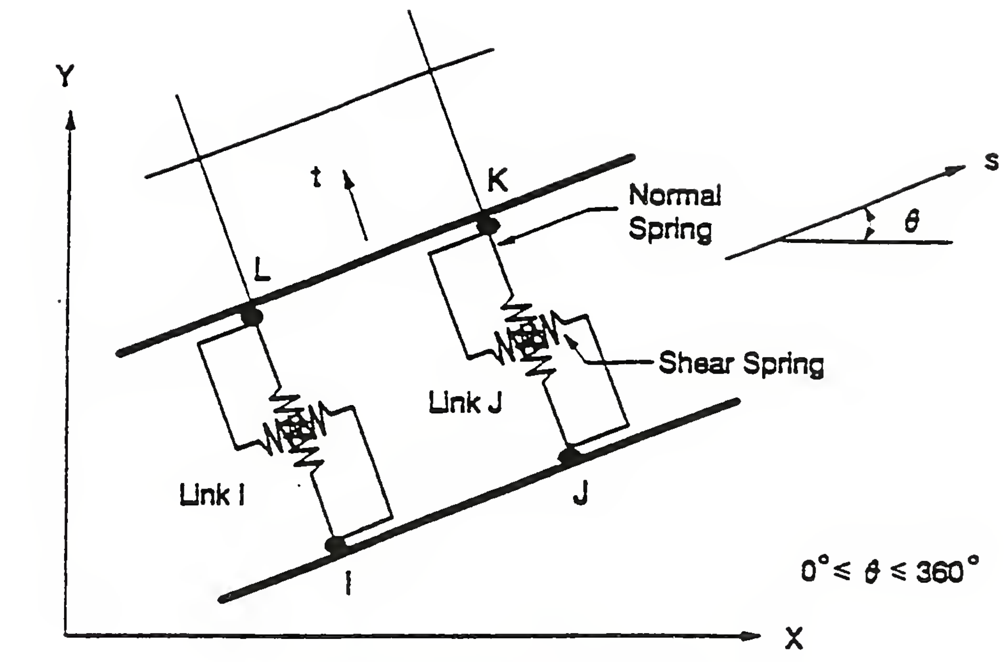
6.7 Nodal Links

An additional element type (nodal link) has been used in SSCOMPPC to control the relative displacement between two nodal points irrespective of the distance between the two. This element consists of an orthogonal pair of springs (see Figure 6.10). The properties of the nodal links include:

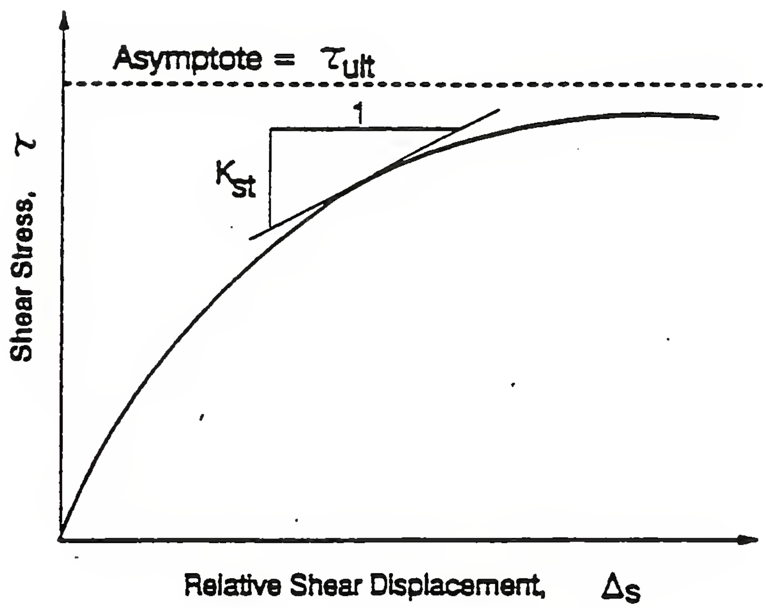
K_n = normal spring coefficient

K_s = shear spring coefficient

The parameters used for the analysis were: K_n=48,800 and K_s=48,800,000.



(a) Components of an Interface Element



(b) Hyperbolic Shear Stress-Relative Shear Displacement Relationship

Figure 6.9 Interface element and hyperbolic shear stress-relative shear displacement relationship (Duncan et al, 1980)

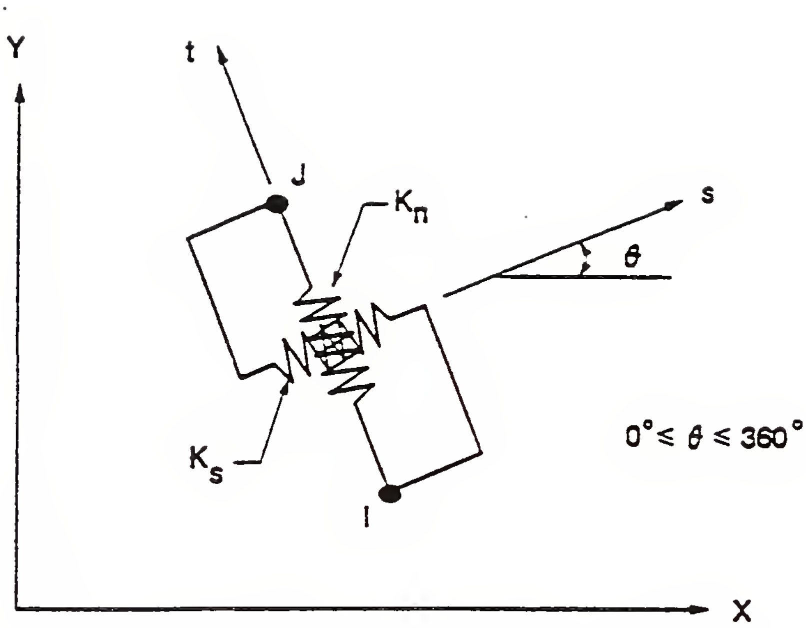


Figure 6.10 Components of nodal links

6.8 Finite Element Analysis

6.8.1 Wall Facility at the University of Maine

The facility is equipped to measure loads and pressures exerted on it in both the at rest and active conditions. Compressibility and settlement data were also recorded. Three different types of tire chips were tested. The first two types, Pine State Recycling and Palmer Shredding, were made up of 3 inch maximum size pieces and a mixture of steel and glass belted tires. The third type, produced by F & B Enterprises, consisted of 1 inch maximum size pieces with most of the belts removed.

The wall facing was not allowed any displacement during construction and loading, to simulate at rest pressure conditions. Horizontal forces and pressures were measured under the following loading: no surcharge, 250 psf (12.0 kPa), 500 psf (23.9 kPa), 750 psf (35.9 kPa). The maximum surcharge was also removed and then reapplied two to three times, to see the effects of repeated reloading. Figure 6.11 shows the horizontal stress distribution on the wall for Palmer Shredding tire shreds under all four loading conditions. This type of distribution is similar to the other two types of tire chips tested.

Settlement information was recorded for the four types of fill tested. Data were taken from different points at the fill surface, referred to as the settlement grid. Deflection was also recorded for settlement plates located at depths of 5 ft (1.52 m) and 10 ft (3.05 m) below the fill surface.

In Figure 6.12 the vertical strain for the Palmer Shredding settlement grid is plotted against the applied vertical stress, or surcharge. Notice that the zero reading is taken at 125 psf (6.0 kPa) during the period of no activity in the winter months from 11/21/94 to 5/31/95.

In Figure 6.13, the time rate of settlement is shown for the settlement grid for all the types of tire chips tested. On the vertical axis, "day 1 = 0% strain" means that a value of 0% for strain was given to the day that the maximum surcharge was applied. All subsequent strain values were determined in reference to day 1. The dip in the Palmer curve around day 180 coincides with readings taken during the winter months.

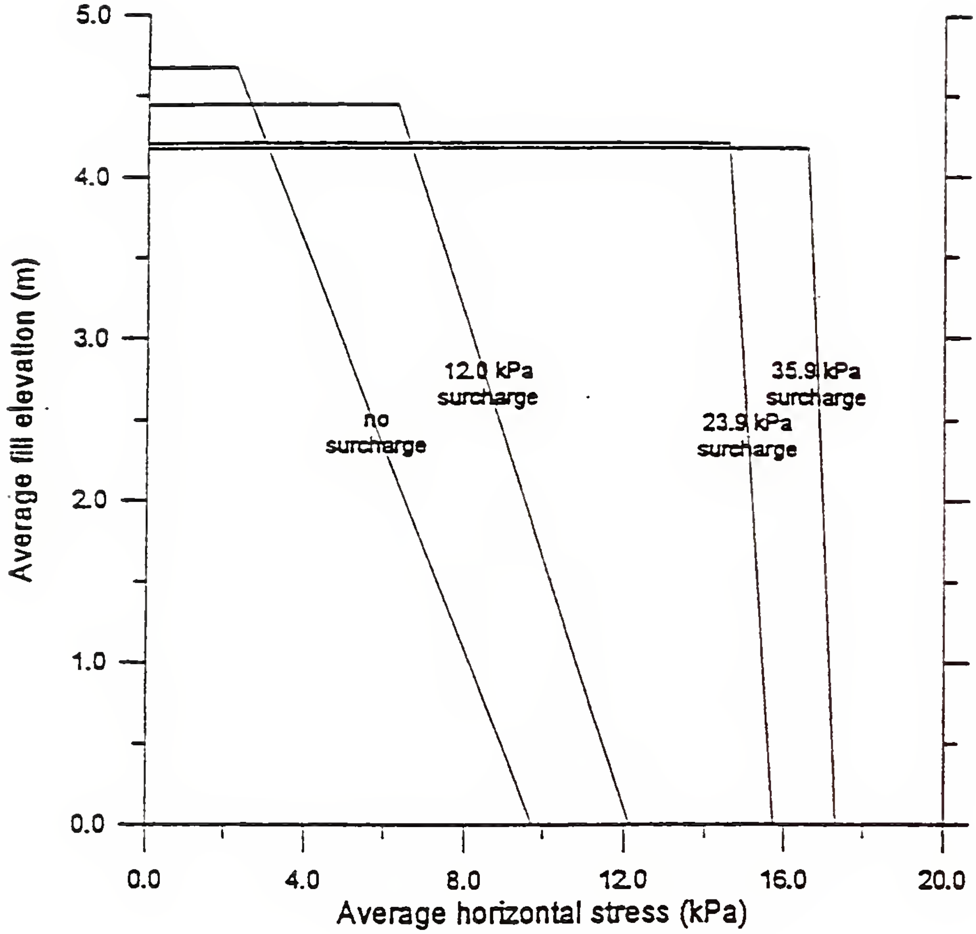


Figure 6.11 Average horizontal stress vs Average fill elevation - Palmer Shredding (Humphrey, 1996)

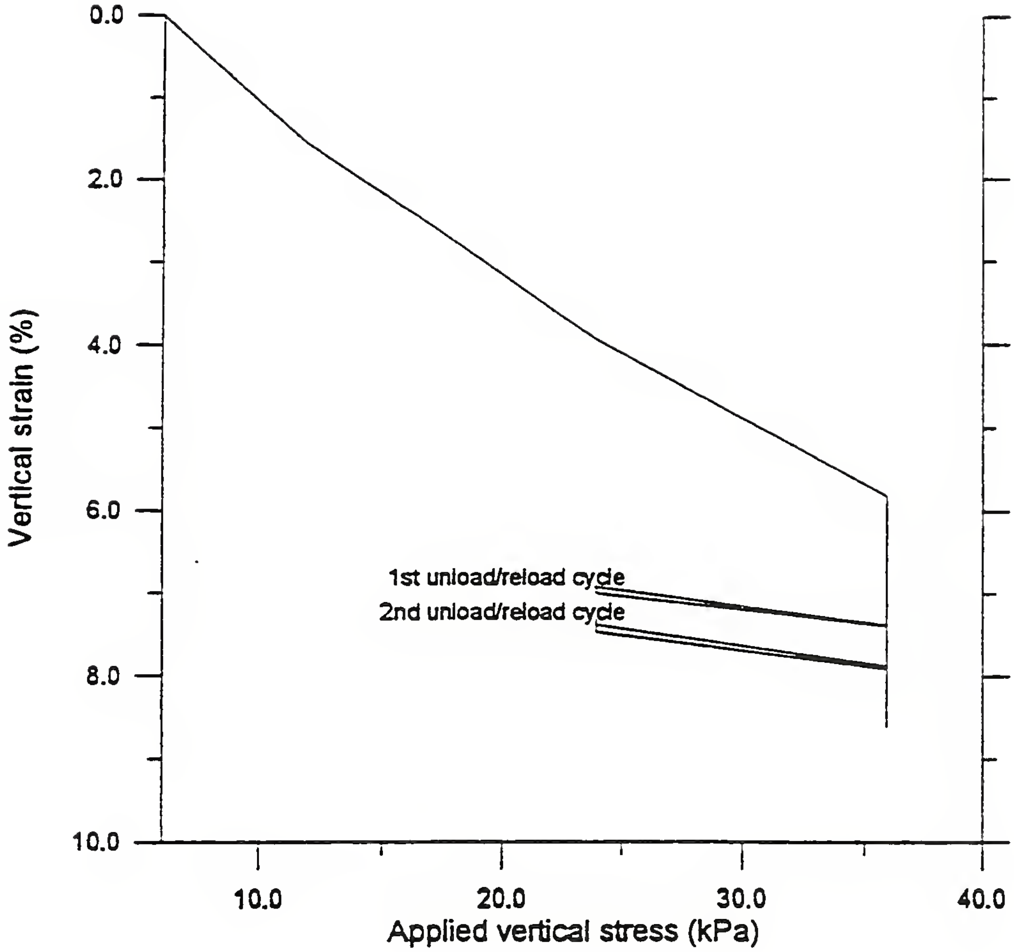


Figure 6.12 Applied vertical stress vs Vertical strain - Palmer Shredding (Humphrey, 1996)

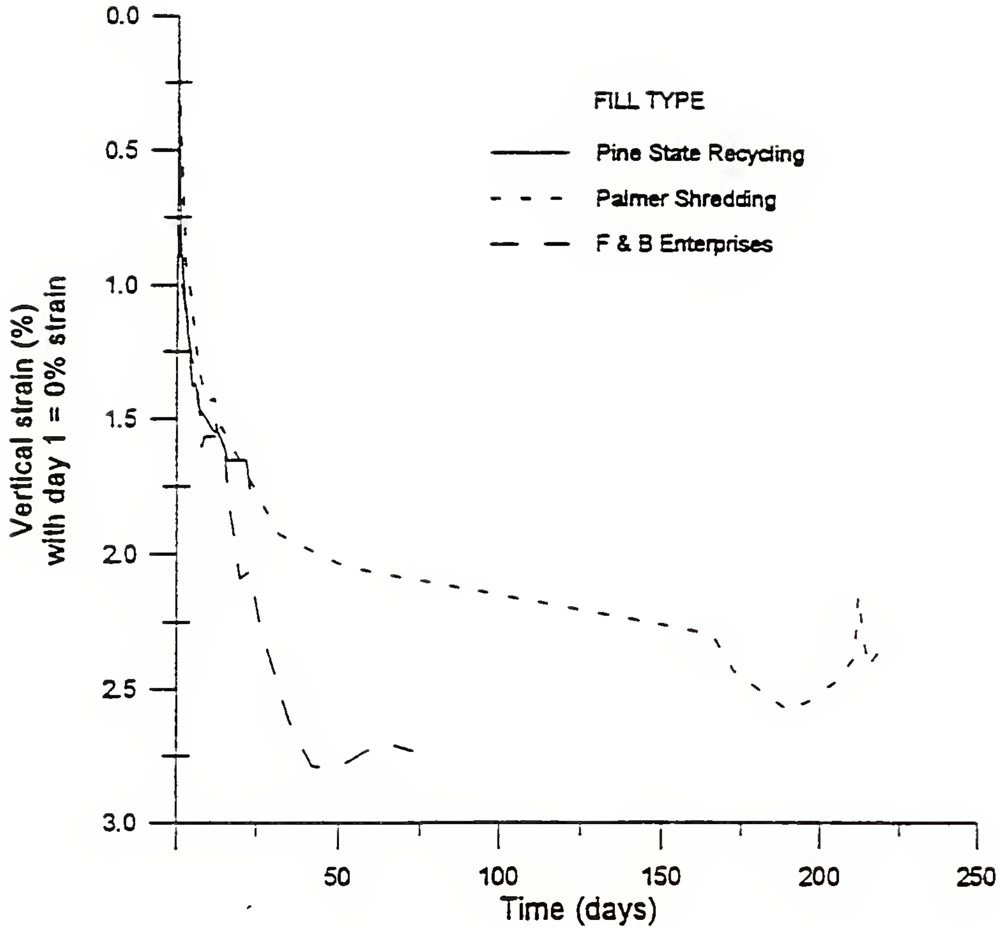


Figure 6.13. Time vs Vertical strain (Humphrey, 1996)

6.8.2 Wall Model

The finite element analysis of the tire shred fill wall was done with SSCOMPPC on the two meshes presented in Figure 6.14. The first mesh was used to model the behavior of tire shreds and rubber-sand without reinforcement. The mesh has 299 nodes and 252 elements with dimensions of 1.3 ft in height and 1 ft in length up to 15 ft from the wall facing and 2.5 ft in length from 15 ft to 30 ft from the wall facing.

The second mesh includes reinforcement and has 464 nodes and 216 elements with dimensions of 1.3 ft in height and 1.3 ft in length up to 15.6 ft from the wall facing and 2.3 ft in length from 15.6 ft to 29.4 ft from the wall facing. This mesh also has beam and interface elements used to model the geotextile reinforcement. Six layers of geotextile were used between fill lifts 1 and 2, 3 and 4 and so on up to the last geotextile layer which was placed between fill lifts 11 and 12.

The numerical models included the tire shred fill, the geotextile reinforced tire shred fill, the rubber-sand fill and a geotextile reinforced rubber-sand fill. The program was run under the same loading and boundary conditions for all cases and the results were compared.

The at-rest pressure conditions were modeled by fixing the wall facing during the analysis. A comparison of the pressure coefficients can be seen in Figure 6.15. The vertical strains observed in the top of the wall fill for the different cases are presented in Table 6.3.

Table 6.3 Vertical strains at the top of the wall fill

Type of Analysis		Vertical Strain (%)
Tire Shreds - Field Conditions	(Field TS)	9.2
Tire Shreds - Finite Element Analysis	(TS)	12.3
Geotextile Reinforced Tire Shreds - Finite Element Analysis	(TSRE)	10.6
Rubber-sand - Finite Element Analysis	(RS)	1.5
Geotextile Reinforced Rubber-sand - Finite Element Analysis	(RSRE)	1.2

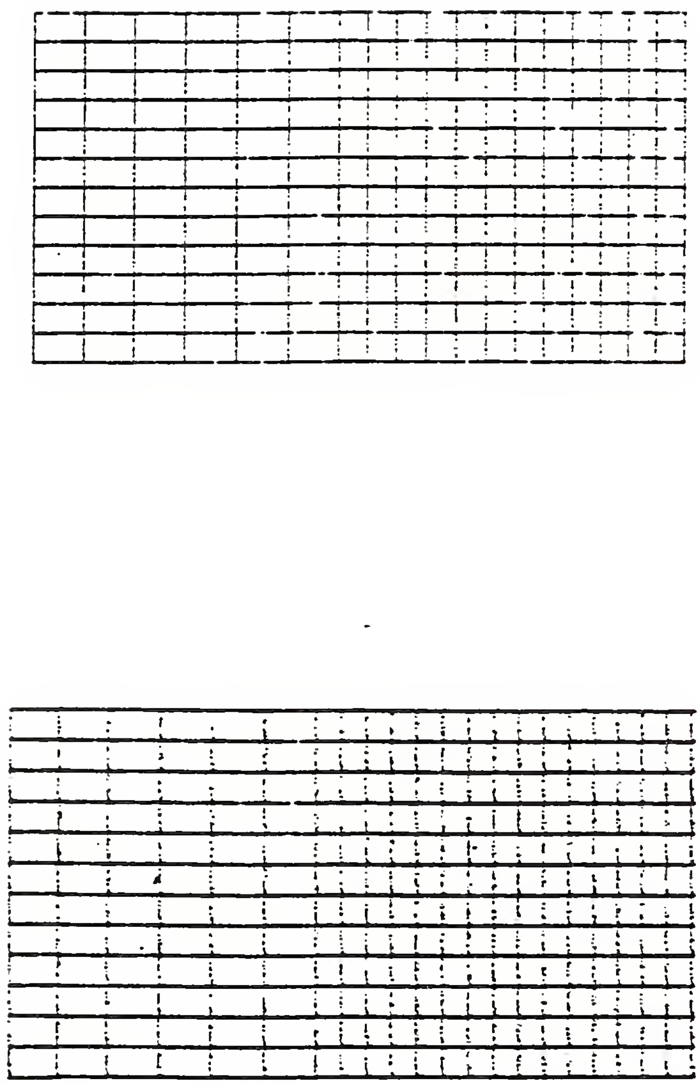


Figure 6.14. Finite Element mesh

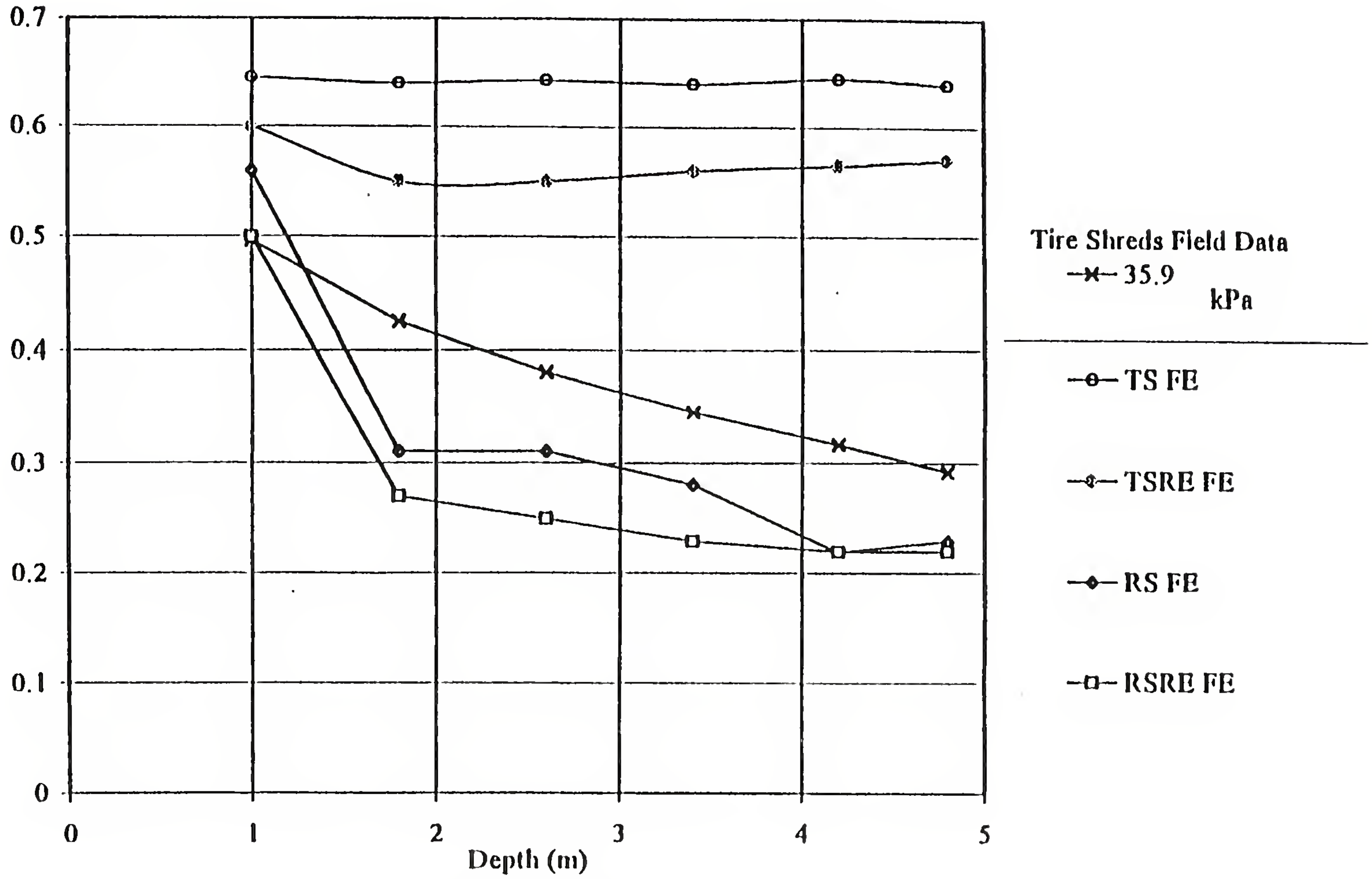


Figure 6.15 Field data K values vs FE analysis (Tire shreds - TS, Rubber-sand - RS, Reinforced - RE)

6.8.3 Discussion

The finite element analysis can be used to model the deformations and stresses of a tire shred or rubber-sand backfill wall. The comparison with the actual data show that SSCOMPPC overestimates the pressure on the wall with tire shreds backfill. This can be due to the fact that the interlocking effect of the exposed steel belts can not be appropriately reproduced by the hyperbolic model. This would produce a conservative wall design. It is conjectured that the interlocking effect might be lost with time due to decomposition of the steel belting.

The vertical deformation at the top of the tire shred wall is adequately modeled. The finite element analysis results show a good correlation with the field data and provide useful information about the performance of the wall.

CHAPTER 7
SPECIFICATIONS

7.1 Tire Shred Specifications

The following draft special provisions were proposed by INDOT for the use of tire shreds in embankments. The draft has been revised and some comments and suggestions are presented in the Section 7.1.1.

INDOT SPECIAL PROVISIONS FOR EMBANKMENT CONSTRUCTED OF SHREDDED TIRE:

DESCRIPTION: This work shall consist of using chipped or shredded tire as a lightweight fill if such material is in accordance with the Indiana Department of Environmental Management and INDOT as described herein. This material shall not be used as backfill for the Reinforced Earth wall (metallic strip reinforcement).

MATERIALS: Chipped or shredded tires shall be restricted to Type IV and Type III fills as defined by 329 IAC 2-9-3. The following table shall be used to determine the quality of material. Type I and II fill will not be permitted.

INDIANA ADMINISTRATIVE CODE RESTRICTED WASTE SITE TYPE CRITERIA

(1) For Parameters Using the EP Toxicity Test

PARAMETER	CONCENTRATIONS (milligrams per liter)			
	Type IV	Type III	Type II	Type I
Arsenic	≤0.05	<0.5	<1.25	<5.0
Barium	≤1	<10	<25	<100
Cadmium	≤0.01	<0.1	<0.25	<1.0
Chromium	≤0.05	<0.5	<1.25	<5.0
Lead	≤0.05	<0.5	<1.25	<5.0
Mercury	≤0.002	<0.02	<0.05	<0.02
Selenium	≤0.01	<0.1	<0.25	<1.0
Silver	≤0.05	<0.5	<1.25	<5.0
(2) For Parameters Using the Leaching Method Test:				
Barium	≤1	<10	<25	**
Boron	≤2	<20	<50	**
Chlorides	≤250	<2,500	≤6,250	**
Copper	≤0.25	<2.5	<6.25	**
Cyanide, Total	≤0.2	<2	<5	**
Fluoride	≤1.4	<14	<35	**
Iron	≤1.5	<15	<**	**
Manganese	≤0.05	<0.50	<**	**
Nickel	≤0.2	<2	<5	**
Phenols	≤0.3	<3	<7.5	**
Sodium	≤250	<2,500	≤6,250	**
Sulfate	≤250	<2,500	<6,250	**
Sulfide, Total	≤1 ***	<5	<12.5	**
Total Dissolved Solids	<500	<5,000	<12,500	**
Zinc	≤2.5	<25	<62.5	**
pH (Standard Units)	6-9	5-10	4-11	**

* The Indiana Department of Environmental Management will permit EP toxicity test or TCLP test.

** Testing will not be required.

*** If detection limit problems exist, the Indiana Department of Environmental Management's Office of Solid and Hazardous Waste shall be consulted for guidance.

Chipped or shredded tire shall comply with the following specifications:[1]

(a) The source of tires is to be determined (such as automobile, truck, tractor, etc.). The type of shredding process is also required to be determined.

(b) 80% of the shredded or chipped tires (by weight) must pass an 8 inch screen.

(c) A minimum mof 50% of the material (by weight) must pass a 4 inch screen.

(d) All the pieces must have at least one side wall severed from the face of the tires.

(e) The largest allowable piece shall be 18 inches or less in length.

(f) All metal fragments shall be firmly attached and 98% embedded in the tire sections from which they were cut. No metal fragmet will be allowed in the fill without being contained within a tire segment. Exposure of small metal pieces from belts and beads shall be allowed in cut faces of some of the tire chips. If metal fragments are found, the supplier will be asked to take back the whole lot of material along with any unused chips at no cost to INDOT.

(g) The tire chips supplied shall be free from any contaminates such as oil, grease, etc., that could affect the quality of ground water.

(h) The loose volume of shredded tires shall not weigh less than 600 pounds per cu-yd.

CONSTRUCTION REQUIREMENTS

On-Site Storage: The shredded tires shall be stored in stockpiles and each stockpile shall be duly approved by the Engineer. The contractor shall be

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[1]Additional requirements to reduce the risk of exothermic reactions may be revealed by the laboratory study recommended under 8.8 Recommendations (page 147).

responsible to secure the shredded tire chip stockpiles from vandalism and arson. Any exposed stockpile on the project not being used immediately, should be enclosed in a locked chain link fence. Each load shall be accompanied by a bill of lading verifying approved source and material requirement as described above.

Siting Criteria: Shredded tires to be used as lightweight fill to construct highway embankment shall not be placed in following cases:

(a) Within 3 vertical feet (0.9 m) of the seasonal high water table, unless an adequate drainage system is provided to prohibit saturation of the shredded tires.

(b) Within 100 horizontal feet (30 m) of a perennial stream, drainage channels, lake or reservoir, unless the embankment is protected by a properly engineered diversion or structure that is approved by the department.

(c) Within 300 horizontal feet (91 m) of a well, spring or other ground water source of potable water, unless it can be demonstrated and approved by Indiana Department of Environmental Management that no ground water contamination will occur.

(d) Within a wetland, floodplain or other protected environmental resource area, unless appropriate approval are obtained from federal, state, and/or local agency having jurisdiction.

(e) Within an area of karst topography or over mines, unless it is demonstrated that the integrity of the embankment will not be damaged by subsidence.

(f) Shredded tires shall not be used directly under hard surfaced pavement (rigid pavement.).

Placement and Compaction: Compaction shall be performed with a D-8 crawler roller or equivalent and it should move in a zigzag pattern. One pass is defined as one complete coverage of the entire width of the section by using the specified machine traveling parallel to the center line. It is expected that four passes shall be required to achieve the desired density. All the shredded tire lifts in the field shall be compacted to 95% of the maximum dry density (AASHTO T-99). Air dried chips are recommended to determine target density in laboratory.

(a) If necessary, a 12" thickness of compacted granular material (No. 53 or "B" Borrow) shall be placed at a specified elevation of the existing grade except in the area where leachate collection system is installed. The purpose is to provide stable ground for construction equipment. A two foot thick lift of shredded tires shall be placed and compacted over geotextile filter fabric which is spread on compacted granular material. The rest of the embankment shall be constructed with 1.5 ft thick lift.

(b) The tire chips shall be enveloped in geotextile filter fabric to keep the material together and to prevent the surrounding material from intruding into the tire chips.

(c) The design of geotextile filter fabric shall be based on surrounding soils. The geotextile shall be laid transversely and an overlap of 12" shall be provided. Also, the joints of Geotextile shall be pinned with "hog ring" clips.

Cover Material: Shredded tires shall be covered or sealed with a minimum of 3 ft (0.9 m) of non-erodible soils. Shredded tires shall not be used within 3 ft (0.9 m) of the pavement section. Encasement shall be placed and compacted at the same time as the shredded tire lift is placed. All cover materials shall be appropriately seeded and vegetated in accordance with 203.09.

The soil used under the tire chips embankment shall be "clay" or "silty clay" ("A-6" or "A-7-6") as classified under INDOT Standard Specification Section 902.1. This means that the soil particles must be more than 30% (by weight) smaller than 0.002 mm and less than 50% larger than 0.075 mm (by weight).

Leachate Monitoring: Leachates shall be monitored by installing a clay liner with a properly designed and approved leachate collection system under the part of the embankment that shall contain shredded tires. To check background water quality sampling shall be done before placement of any shredded tires. During embankment construction the sampling events shall be more frequent. After construction is complete, sampling shall be done on a quarterly basis at least five years to detect any leachate problems.

The leachate collection system shall be at least 50 ft. long covering the entire width of the embankment or as specified on the plans.

Instrumentation: Testing of the embankment material will be carried out by an Engineering firm under a separate contract. The CONTRACTOR shall provide the ENGINEERING FIRM (to be named at the preconstruction conference) at least two working days notice prior to placement of the various study sections. NO WORK SHALL CONTINUE PAST THE MEASUREMENT POINTS WITHOUT THE REQUIREMENT MEASUREMENTS BEING MADE.

To facilitate the testing the CONTRACTOR is required to provide time and assistance for the measurements as follows:

(a) Settlement Plates; shall have a base of at least 3 ft. X 3 ft. with a 2" pipe attached firmly (bolted) to the center at 90 degrees to the plate. The pipe shall be capped on the top end.

(b) A 4" diameter smooth wall PVC pipe with end caps shall be laid beneath the fabric near the middle of the shredded tire section near but not directly under the settlement plates. This pipe shall be installed for the full width of the embankment from toe of slope to toe of slope perpendicular to the centerline.

(c) Settlement plates with 4 ft. pipes shall be installed on top of the first fabric layer in the lightweight fill section in accordance with the layout shown on the attached drawings. The ENGINEERING FIRM SHALL DETERMINE elevations of the top of the pipe BEFORE any fill material is added to the fabric and referenced to a temporary benchmark (TBM) which is well outside the construction zone and well protected.

(d) Stakes made of re-bar (1/2" x 5') shall be placed at the toe of the fill, in a vertical manner (and plumbed), with 2' left above the final grade, at 50' intervals (all locations which receive settlement plates).

(e) Care shall be taken in filling not to disturb the settlement plates or the toe stakes. The engineer shall be notified immediately of any problems in this regard.

(f) After the lightweight fill material is placed, a second set of settlement plates, up to 3.5' pipes, shall be placed 1.5' from the location of the first set but on top of the fabric which covers the lightweight fill. The ENGINEERING FIRM SHALL DETERMINE the elevation of both sets of pipes in reference to the established TBM, BEFORE any further fill material is added to the section.

(g) When the select borrow material reaches the top of the pipes a steel plate 1' X 2' X 3/16" thick shall be placed just above the top of the settlement pipes, in a manner that will provide protection for the pipes, before the last 6" of material is added.

METHOD OF MEASUREMENT: shredded tire embankment and encasement will be measured by the cubic yard (cubic meter)

BASIS OF PAYMENT: Shredded tire fill shall be deposited in layers of 2 ft or less in thickness before compaction. Each layer of shredded tire fill shall be compacted and kneaded into place by at least four passes of a D-8 or equivalent type dozer.

Payment will be made under

Pay Item	Pay unit
Place shredded tire fill (complete in place)	C.U. yds. (cubic meters)

Payment for placing shredded tire fill (complete in place) shall be full compensation for all work and materials and equipment required to complete the item including haul, placement, compaction, and removal and disposal of unsuitable material.

Note:

1. In the event serious lateral movement or settlement develop during the construction of shredded tire embankment or within the required settlement period, the work will be suspended and corrective measures taken as directed.

2. All instrumentation at top or toe of embankment shall be protected and shall not be disturbed or damaged. Any damaged instrument shall be replaced by Contractor with no additional cost to INDOT.

7.1.1 Comments and Suggestions

The following comments and suggestions have been proposed for the Special Provisions on Embankments Constructed of Shredded Tires.

1. The available information on the Toxicity Tests for tire shreds (Radian, 1989 and Maryland, 1993) indicate that the concentrations of the various elements listed are well below those required for Type III materials and near those required for Type IV materials. It can be expected that tire shreds will comply with those limits. Other contaminants such as organic compounds are not included in the list and may be present in shredded tire leachates.
2. It is recommended that the maximum allowable tire shred size be reduced from 18 in. to 8-12 in. to improve the compactability and increase the final compacted density of the fill. It may not be necessary to specify the minimum loose density since the pay item deals with compacted fill.
3. The requirement of 98% embedment of metal fragments in tire sheds is difficult to measure and to achieve since very fine cuts would be required. It would be more practical to require firm embedment of the metal fragments in the tire shred and that a maximum of 1% free metal fragments be allowed. It is also recommended to avoid burned tire shreds in the fill.
4. It would be advisable to require proper cover for stockpiled tire shreds for fire protection and reducing runoff.
5. Four passes of a smooth vibratory compactor weighing 10 tons minimum has been shown to provide adequate compaction to tire shred fills (Nickels, 1995).
6. The 12 in. overlap and the use of the "hog ring" clips for the geotextile have worked adequately in other projects. The installation of the geotextile should follow INDOT

Standard Specifications 616.09 Installation of Geotextile under Riprap and 913.18 Geotextile for Use under riprap where 18 in. minimum overlap is required.

7. The following placement requirements are recommended (Maine Special Provisions - Nickels, 1995):

   Placing: The maximum compacted thickness of any tire shred layer shall not exceed 12 inches. Each layer of tire shreds shall be placed over the full width of the section. The tire shreds shall be spread with track mounted bulldozers, rubber tire motor graders, backhoes or other equipment as needed to obtain a uniform layer thickness. The tire shreds as spread shall be well mixed with no pockets of either fine or coarse tire shreds. Segregation of large of or fine particles shall not be allowed.

8. The minimum 3 ft cover of non-erodible soils should be specified for all sides of the embankment. It would be advisable to clarify if the soil used under the embankment is part of the clay liner.

9. Leachate monitoring can be done more economically with wells installed parallel to the embankment instead of using a clay liner and a leachate collection system.

10. The length of pipes on settlement plates should be adjusted according to the actual size of the embankment. It would be advisable to clarify the party in charge of the instrumentation and the monitoring program.

11. The Special Provision for Rubber-Sand embankments should be similar to the Tire Shred Special Provisions but should include the requirement that the rubber-sand mixture (60% sand, 40% tire shreds - by weight) present a homogeneous mix produced by various passes of a motor grader or similar equipment. It is advisable to spread the tire shreds before proceeding to spread the sand.

7.2 Tire shreds and rubber-sand as wall backfill

The construction of walls with tire shred or rubber-sand backfills should be similar to that of conventional material backfills with the provision that the recommendations mentioned in the section 7.1 are followed.

Reinforced backfills require a space between the backfill and the retaining structure to allow for the lower layers of fill to deform. This movement produces an initial mobilization of pullout resistance in the geogrid or geotextile and increases the confinement of the fill.

CHAPTER 8
SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

8.1 Background

Lightweight fill materials can be used to solve bearing capacity and settlement problems of walls and embankments on soft compressible soils. Some common lightweight materials used include sawdust and bark from the lumber industry, slags and ashes from the power generating industry and engineered materials such as expanded shales and Elastizell. These materials have intrinsic disadvantages that lessen their appeal as lightweight fill.

Field and laboratory studies indicate that the use of tire shreds and rubber-sand meets the requirements of durability, low unit weight, availability and relative cost required for lightweight fill material applications.

Millions of scrap tires are discarded annually and an even bigger number are currently stockpiled throughout the country consuming valuable landfill space, or are improperly disposed providing a breeding ground for mosquitoes and rodents. The use of tire shreds as lightweight fill can reduce the tire disposal problem in an economically and environmentally beneficial way.

8.2 Summary

Laboratory testing and evaluation combined with computer analyses support the feasibility of using shredded tires in embankments and wall backfill. The study has focused on the volumetric behavior, earth pressure coefficients, reinforced earth applications and addressed the environmental impact of tire shreds and rubber-sand mixtures. The findings of this study provide parameters for design of embankments and walls and their performance prediction and evaluation.

The research objectives presented in Chapter 1 were accomplished by following a detailed testing plan. The materials tested included:

1)Ottawa sand - classified as poorly graded sand SP - USCS or A-3(0) - AASHTO

2)Masonry sand - classified as poorly graded sand SP - USCS

3)One inch nominal size tire shreds

4)Two inch nominal size tire shreds

A six inch diameter triaxial cell and its pressure panel were modified to measure the volumetric behavior and a twelve inch instrumented PVC pipe was used to determine the lateral pressure coefficients of the materials tested. The MTS soil testing system was modified to accommodate the large size compressibility and triaxial shear apparatus.

The interface parameters between geosynthetics (geogrids and woven geotextiles) and tire shreds and rubber-sand were established in a large pullout box and a large direct shear device.

The test data were analyzed and presented in tables and figures. Correlations have been developed for design and performance evaluation of tire shred and rubber-sand fill embankments and walls.

The report is divided in eight chapters as follows: Chapter 1 describes the tire disposal problem, lists the research objectives, the approach followed to reach those objectives and presents an brief summary of the thesis; Chapter 2 presents the tire disposal problem in detail, presents and overview of the current recycling, reuse and disposal practices and updates the civil engineering applications and studies since the report presented by Ahmed in 1993; Chapter 3 states and analyses the results of the volumetric triaxial shear testing program ; Chapter 4 presents the compressibility and earth pressure coefficient tests; Chapter 5 describes the pullout and direct shear tests performed to determine reinforced earth design parameters; Chapter 6 contains the results of the numerical modeling and finite element analysis used to predict the performance of tire shred and rubber-sand embankments and walls; and, Chapter 7 combines the experimental work accomplished to propose a design/construction protocol for the use of tire shreds and rubber sand in embankments and backfills. The conclusions and recommendations of this study are presented in the following subsections.

8.3 Current Practice

Chapter 2 presented an overview of the current practice in recycling, reuse and disposal options for scrap tires and discussed the use of tire shreds as lightweight fill material in highway construction. It was established that the major market for scrap tires are industries that use this material as fuel (Tire Derived Fuel) and that by 1995 this market was consuming more than 50% of the annual generation and that 70% of the scrap tires have established markets. A calculated 800 million scrap tires are still present in stockpiles and landfills throughout the country. The major conclusions based on a critical analysis of the available information indicates that:

-Waste tires are a valuable raw material. · The factors that favor recycling include their high physical and chemical durability, elasticity, high tensile strength, low unit weight, high caloric value, low cost and positive impact of recycling on the environment. Some factors that are impediments for recycling include the complex chemical composition which makes them potentially combustible and leachates possibly generated under adverse environmental conditions.

-To reduce the possibility of fire, a protective earth cover must be placed on the top and side slopes of tire embankments. A similar soil cover is recommended for other lightweight materials, like wood shreds, sawdust, slags, ashes, expanded clay or shale, etc. to protect against fire or to prevent leaching of undesirable materials into groundwater. During construction, caution is required to avoid any fires in stockpiled tires or embankment tires that have not yet been capped.

-Compacted tire shreds (about 2x2 in. nominal size) have permeability values equivalent to typical values for coarse gravel (Bressette, 1984). This property of shreds renders them suitable for use in subdrainage as an alternate permeable aggregate. As a highly permeable material, pore pressure development is prevented in tire fills and backfills. Use of tire shreds in alternate layers with non-select fills, like clays, silty clays, etc., will provide a shorter drainage path and thus help accelerate consolidation of the layer.

-The use of shredded tires in embankments offers the potential benefit of disposing of large volumes of tires in short sections of highway.

8.4 Shear and Volumetric Strain Results

8.4.1 Tire Shreds

The results of the triaxial tests are presented graphically. The general shape of the stress-strain curves show a behavior similar to that observed for the direct shear described in Chapter 5: there is no peak stress and the shear stress tends to stabilize and at higher confining pressures tends to increase with deformation.

Tire shreds show an almost linear loss of volume as the deviatoric stress is increased. The volume change under 4 psi confining pressure is linear up to 5 percent strain and stabilizes under the higher deformations. The volume change for 14 psi is linear up to 15 percent strain and shows a declining rate at higher strains. The volume change for tire shreds under a confining pressure of 28 psi is almost linear throughout the test.

8.4.2 Rubber-sand

The general shape of the stress-strain curves show a behavior similar to that observed for the direct shear tests described in Chapter 5, there is no peak stress and the shear stress tends to stabilize and at higher confining pressures tends to increase with deformation.

The volumetric strain show an initial loss of volume and varying levels of dilation have been observed for the three confining pressures.

Masad et.al. (1995) conducted a series of tests on 0.25 inch tire shreds and a mixture of 50% tire shreds and 50% Ottawa sand by weight. Even though the exposed nylon belting in tire shreds of this size has a major influence in the behavior of the material, the trends observed for this material are similar to those for 1 inch tire shreds and rubber-sands mixtures.

The results from these tests were used to determine the hyperbolic parameters to be used for the numerical modeling of the behavior of tire shreds and rubber-sand in embankments and wall backfills.

8.5 Compressibility and Lateral Pressure Coefficients

The average vertical strain versus average vertical stress at gage height for tire shreds and rubber-sand were calculated after calibrating and studying the friction of these materials with the PVC pipe.

The at-rest lateral pressure coefficient (K_0) was calculated as the ratio between the horizontal and vertical stresses. The results of the tests run for tire shreds and rubber-sand indicate that this value stabilizes after the normal stress goes beyond 5 psi. These values are not significantly changed in the reload cycle .

It was observed that the lateral pressure coefficients for tire shreds show a large variation and that the average value was around 0.5. The lateral pressure coefficients for rubber-sand present a smaller variation and show an almost linear decrease form 0.72 at low pressures to 0.65 at the larger pressures.

8.6 Reinforced Earth

Chapter 5 introduces the parameters necessary for the design of a geogrid reinforced tire shred of rubber-sand backfill structure. The strength parameters of these lightweight materials make them ideal for use in areas of weak soils. Tire shreds and rubber-sand, to a lesser degree, are compressible and a soil cover could be used to avoid negative effects such as differential settlements and to prevent oxygen flow and diminish the possibility of a fire.

The advantages of constructing geogrid reinforced fills with these types of materials would include the ease of construction because material handling is as simple as with conventional materials; limited deformation of the facing and reduction of earth pressures; and the possibility of constructing the geogrid reinforced fill without a retaining structure provided that the material is confined, possibly with a geotextile as the facing.

8.7 Numerical analysis

Finite element analysis can be used to model the deformations and stresses of a tire shred or rubber-sand backfill wall. The comparison with the actual data show that SSCOMPPC overestimates the pressure on the wall with tire shreds backfill. This can be due to the fact that the interlocking effect of the exposed steel belts can not be appropriately reproduced by the hyperbolic model. This would produce a conservative wall design. It is conjectured that the interlocking effect might be lost with time due to decomposition of the steel belting.

The vertical deformation at the top of the tire shred wall is adequately modeled. The finite element analysis results show a good correlation with the field data and provide useful information about the performance of the wall.

8.8 Recommendations

The waste tire problem in the United States is of great magnitude and has strong environmental and economic consequences. It was found that civil engineering applications constitute an important area for use of scrap tires.

The environmental effects can be diminished by providing proper encapsulation of the tire shred fill and preventing the presence of water in the fill. The largely positive results observed in tire shred and rubber-soil embankments in different areas of the country support the feasibility of this application in Indiana. It is recommended that demonstration projects be immediately identified and built in Indiana.

In addition, laboratory research on the effect of various tire shred properties in producing exothermic reactions in situ should be initiated. The nature of the material properties research is recommended by Humphrey (1996).

The following material-specific factors should be studied in the laboratory to determine the temperatures at which they are at risk from initial exothermic reaction. Where these temperatures are found to be relatively low, use of the shreds is likely to constitute an unacceptable engineering risk, and must be avoided.

The factors to be studied are:

(1) Percentage of exposed steel belt at the edges of the shreds

(2) Age of the steel exposure and degree of rusting

(3) Minimization of exposed steel by reduced shredding (and larger shreds)

(4) Effect of fine rubber pieces (crumb size)

(5) Availability of free sulfur from tire rubber

(6) Petroleum contamination of the tire shreds (cleanliness of the tires which are shredded).

Data from these studies would allow INDOT to recommend/construct demonstration fills which would be monitored for internal temperature, as well as more usual performance criteria.

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