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16. Abstract

A field and laboratory study was begun in 1995 to evaluate the engineering behavior of a stabilized soil from two test sections beneath a reconstructed roadway. This roadway is designated FM 1343 and is located in Medina County, south of SH 90, approximately 20 miles west of San Antonio, Texas. One test section was stabilized using Portland cement, and the other with lime. The primary intent of the research project was to make a comparison between the two stabilization methods in terms of their long-term effectiveness in controlling moisture susceptibility of pavement subgrade soils.

Soil samples were obtained from both test sites. For the comparison, similar tests were performed on the soil samples obtained from each site. Initial testing indicated that both stabilization methods are comparable in plasticity reduction, strength increase and durability. Instrumentation necessary for long-term monitoring was installed at both test sites, and initial data collected. It is expected that TxDOT personnel will continue this data collection over the next 5 years. Appropriate conclusions regarding the effectiveness of lime versus cement will be drawn at the end of the 5-year monitoring period after the data collected has been reviewed.

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EXPERIMENTAL PAVEMENT RECONSTRUCTION PROJECT TO DETERMINE LONG-TERM EFFECTIVENESS OF LIME AND CEMENT FOR STABILIZATION OF PAVEMENT SUBGRADE SOILS

by

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Research Report Number 7-2967-F

conducted for

Texas Department of Transportation

by the

CENTER FOR MULTICISIPLINARY RESEARCH IN TRANSPORTATION
TEXAS TECH UNIVERSITY

September 1998

IMPLEMENTATION STATEMENT

The Primary objective of this research study was to set up an experimental pavement construction project that can be monitored subsequently to determine the long-term effectiveness of lime and cement as subgrade stabilization agents. The characterization of the site, documentation of the cement and lime stabilization process, instrumentation of the pavement section and initial monitoring was completed during this one-year research study. TxDOT personnel will accomplish the monitoring of the experimental project over the next several years. Conclusions concerning the long-term effectiveness of lime versus cement stabilization will be drawn at the end of the long-term monitoring period upon review of the data collected. This report provides guidelines that must be followed by TxDOT personnel in collecting long-term monitoring data.

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

AUTHOR'S DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official view of policies of the Department of Transportation or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

PATENT DISCLAIMER

There was no invention or discovery conceived or first actually reduced to practice in the course of or under this contract, including any art, method, process, machine, manufacture, design or composition of matter, or any new useful improvement thereof, or any variety of plant which is or may be patentable under the patent laws of the United States of America or any foreign country.

ENGINEERING DISCLAIMER

Not intended for construction, bidding, or permit purposes. The engineer in charge of the research study was Warren K. Wray, P.E., Texas 51199.

TRADE NAMES AND MANUFACTURERS' NAMES

The United States Government and the State of Texas do not endorse products or manufacturers. Trade or manufacturers' names appear herein solely because they are considered essential to the object of this report.

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: Volumes greater than 1000 l shall be shown in m^3.				
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9 or (F-32)/1.8	Celcius temperature	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.71	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

* SI is the symbol for the International System of Units. Appropriate [illegible]

(Revised September 1993)

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I. INTRODUCTION

Many clayey soils have a potential to shrink and swell in response to varying moisture content. The resulting volume changes in the soil can be detrimental to buildings, roadways, retaining walls, utilities, and other structures built on such soil (Ingles and Metcalf, 1973; Snethen, Townsend, Johnson, Patrick, and Vedros, 1975). Roadways constructed on these types of soils suffer a reduction in serviceability due to the shrinking and swelling of the soil which is associated with moisture changes. Eventually the roadway becomes rough, causing discomfort for drivers and creating a potentially hazardous condition. The problem of shrink/swell soils can be found throughout Texas and beyond. In the United States, alone, damage has been estimated to range from $2 to 9 billion (Austin, 1987), and approximately $1.1 billion of this amount can be attributed to the repair costs of streets and highways (Snethen et al., 1975). Furthermore, problems due to shrink/swell soils are found not only in the United States, but throughout the world (Ingles and Metcalf, 1973; Snethen and Huang, 1992).

A number of stabilization methods have been used to control this shrink/swell behavior and thus prevent the potential damage. Two of these methods that involve treatment of soil with lime and cement have been successfully used to stabilize shrink/swell soils by lowering their plasticity index (PI). However, there has been no systematic, established method to determine which stabilization method works best. For example, one of the advantages of lime is that it is easier to incorporate into a soil that has a high PI, but there are conflicting reports as to its effectiveness in plasticity reduction in the long term. Likewise, adding cement to a soil normally results in a greater increase in strength than lime, but there has been some concern with respect to the durability of some soils stabilized with cement.

1.1 BACKGROUND

The use of lime to overcome shrink/swell behavior in soils is well documented. The first project in modern times in which lime was used as a soil stabilizer was completed in 1948 at Fort Sam Houston in Texas. Army roadways had failed and needed to be reconstructed within a limited budget. Incorporation of lime in subgrade soils was proposed as a cost-effective measure to achieve better performance in reconstructing roadways. The Army constructed the two block experimental section of roadway while the Texas Highway Department conducted the initial laboratory tests (National Lime Association, 1977). Because of the success of the initial project, both the Army and Texas Highway Department have used lime for soil stabilization.

Although technical literature indicates that concrete paving began to develop in the 1900's (Lesley, 1924), the first mention of cement as a stabilizing agent does not appear until the late 1920's and early 1930's (Christensen, 1969; Portland Cement Association, 1992b; Portland Cement Association, 1995). The first known soil-cement project that was scientifically-controlled was a 20,000 sq. yd. job completed in Johnsonville, South Carolina in the year 1935 (Norling, 1963). The reasons for this late development are only speculation. The inferior quality of the earlier cement could account for it not being utilized as a stabilizing agent, but more likely there was not a great need or demand for stabilization until the development and more common usage of the automobile.

1.2 OBJECTIVES AND SCOPE OF RESEARCH

The primary objective of this research is to compare lime and cement as subgrade soil stabilizing agents in pavement construction. The study includes a preliminary investigation, construction monitoring, and post-construction testing. During the preliminary investigation, two pavement reconstruction projects with identical soil conditions were identified in the San Antonio District. Cement was used as the stabilizing agent for subgrade soils in one of these projects, and lime was used in the other. Soil samples were collected from each of the two sites and subjected to detailed geotechnical characterization tests in the laboratory. Laboratory tests were also conducted to determine the optimum additive content according to established procedures as well as to evaluate strength and durability of stabilized soil mixtures at various additive content levels.

Construction monitoring involved the documentation of the entire construction process including specific procedures and equipment used in the construction. The actual additive contents used, the field moisture contents, and densities during compaction were also recorded. Post-construction evaluation included the installation of Thermocouple Psychrometers and monitoring of moisture changes within subgrade soils after the pavement construction has been completed. Falling weight deflectometer measurements, Profilometer measurements, and a visual condition survey were also conducted on each of the two test pavements. These monitoring efforts are expected to be continued by TxDOT personnel for the next 5 years. At the end of the 5-year monitoring period, the data collected in this research, as well as data from long-term monitoring, will be used in a comparative evaluation of the long-term effectiveness of lime versus cement in stabilizing subgrade soils with shrink/swell potential.

II. LIME AS A SOIL STABILIZING AGENT

2.1 INTRODUCTION

Many methods and chemical agents have been used in the past to successfully treat soils to increase their workability. Of these, lime has been the most effective in stabilizing subgrade. Subgrade includes the existing material or materials used as the foundation to support the pavement base course. Subgrade includes mainly the fine-grained soils, whereas base deals primarily with coarse grained soils. Lab tests and empirical methods are used to determine the optimum lime content to be used to stabilize the subgrade. Lime modification is another term used when treating the subgrade. Modification, however, usually refers to small amounts of lime being added to increase the workability or to reduce the plasticity index enough to subsequently use fly ash or cement to stabilize the soil. This section describes in detail the use of lime as a stabilizing agent for fine-grained soils and the subsequent benefits of using lime.

2.1.1 Formation of Lime. Lime, in general, refers to a form of burned lime, either quicklime (calcium oxide) or hydrated lime (calcium hydroxide). The production of quicklime involves the burning of limestone of a high quality at high temperatures and then removing the carbon dioxide. Hydrated lime forms as the result of a chemical reaction between enough water and quicklime which produces a white powder (Little, 1995).

2.1.2 Types of Lime Produced. Many types and forms of lime are available and used in soil stabilization. Higher quality limestone produces a higher quality lime, such as high calcium lime. Another type of lime, dolomitic lime, refers to lime which has magnesium oxide and is, therefore, of a lower quality and is less reactive with the soil.

Many forms of lime are available to stabilize soils. For example, lime slurry is hydrated lime with the addition of free water. Quicklime, on the other hand, is available in different forms and sizes from coarse to very fine and from dolomitic to high calcium. The following forms of quicklime range from coarse (2-3 inches in diameter) to fine (passing #100 sieve): lump lime, crushed or pebble lime, granular lime, ground lime, pulverized lime and pelletized lime. However, some forms of lime are preferred over others. Quicklime only accounts for ten percent of all lime used in soil stabilization. The following are forms of lime that are commonly used: high calcium lime, dehydrated dolomitic lime, monohydrated dolomitic lime, calcitic quicklime and dolomitic quicklime (Little, 1995).

2.1.3 Quantity of Lime Produced. According to the National Lime Association, approximately 19 million metric tons of lime is produced each year (Bulletin 214, 1992).

2.2 TYPES OF SOILS SUITABLE FOR LIME STABILIZATION

Lime, in general, is an effective stabilizing agent for many different types of soil. The addition of lime to a medium, moderately-fine, and fine-grained soils reduces the plasticity index and swell while increasing the workability and strength (Epps et., al.; Air Force Manual, 1982). Soils with heavy clay and clayey gravel are proven to be more affected by lime than other soils. Montmorillinitic clays also tend to react with lime more quickly than kaolinitic clays (Ingles and Metcalf, 1973). Little (1995) identifies the following soil types from the Unified Soil Classification System as soils suitable for stabilization with lime; CH, CL, MH, SC, SM, GC, SW-SC, SP-SC, SM-SC, GP-GC, and GM-GC. Little also states that soils with plasticity indices (PI) between 10 and 30, with 25 percent passing the U.S. No. 200 sieve, are highly reactive with lime. Robnett and Thompson (1969) report that soils with as little as 7 percent passing the No. 200 sieve and a PI of 8 can be stabilized with the addition of lime. Soils that have plasticity indices higher than 30 should be modified with lime until the PI is at least 30 and then stabilized with cement.

The literature also suggests that a soil containing more than about 1-2 percent organic matter may not be suitable for stabilization (Eades and Grim, 1966). However, these percentage estimates seem to vary with the molecular weight of the organic matter. The higher the molecular weight of the organic matter, the less detrimental organic matter is to the stabilizing process. Therefore, organic compounds such as nucleic acid or dextrose, with low molecular weights, could adversely affect stabilization.

The pH of a soil plays an important role in stabilization. Soils having insitu pH's of 7 or greater are more reactive to lime than soils with pH's less than 7 (Robnett and Thompson, 1976). The Road Research Laboratory found that soils having an initial pH of 7 or greater had a corresponding increased compressive strength response. An hour "quick test" can determine the minimum percent lime required to stabilize a soil (Eades and Grim, 1966; ASTM, 1984).

2.3 MECHANICS OF LIME-SOIL REACTIONS

The lime soil reaction is not fully understood; however, the rate of reaction between soil and lime seems to be dependent upon many characteristics of the soil. One such characteristic is the water content of the soil.

The water environment within the soil is a complex system in which the water molecules are bonded with the negative charge on the clay surface to form what is known as a diffused layer. Different clays exhibit various degrees of bonding (e.g., a montmorillinitic clay has a larger diffused layer than a kaolinitic clay). The shrink and swell potential of clays is clearly defined by the diffused layer. This diffused layer represents the amount of water that a clay material can retain and, in some instances, the clay can retain an amount of water seven times its dry weight (Jury, Gardner, and Gardner, 1991).

The reaction between lime, water, and clay occurs with the cation exchange, where the calcium cations replace the free cations available in water. This exchange is explained by the Lyotropic series, which states that higher valence cations will replace those of a lower valence and that larger cations will replace smaller cations. Figure 2.1 shows the relationship of exchangeable cations to the size of the diffused water layer. Adding lime to soil creates a more stable, diffused water layer, which is dramatically reduced in size because of the cation exchange. Once the water layer size decreases, clay particles attract one another more closely through flocculation (edge-to-face attraction).

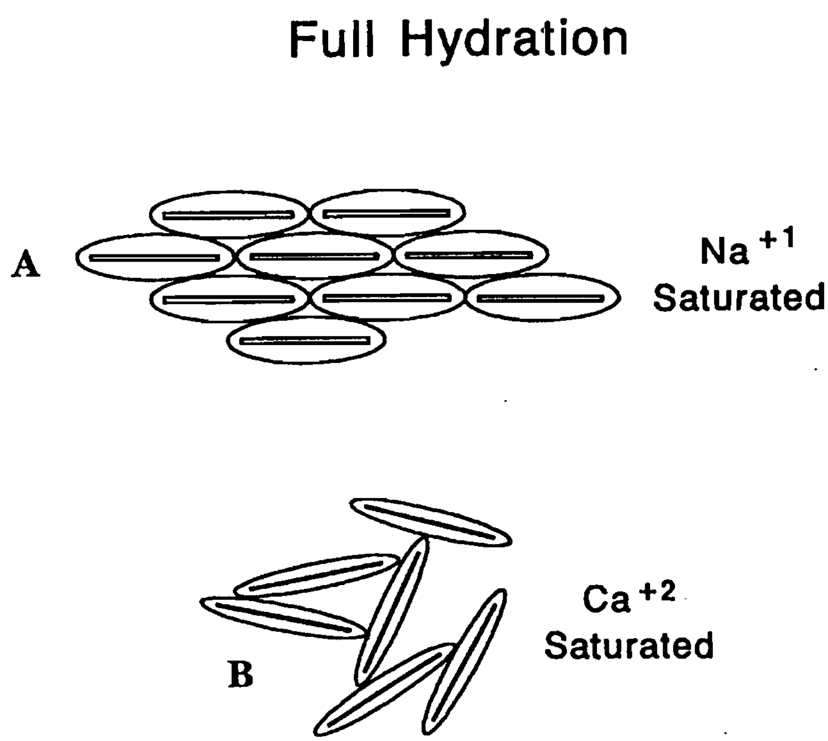


Figure 2.1: "The reason for the textural change is due to the Phenomenon of cation exchange followed by flocculation and agglomeration. (A) Illustrates low strength clay soil where particles are separated by large water layers. The addition of lime (calcium) shrinks the water layer (B) allowing the plate-like particles to flocculate." (Little, 1995)

Flocculation occurs, in part, because of the attraction created by "broken bonds at the edge of the clay particles to the oppositely charged surfaces of the neighboring clay particles" (Little, 1995).

The cation exchange and flocculation within soil creates the following results:

1. Substantial reduction in size and stabilization of the adsorbed water layer,
2. Increased internal friction among the agglomerates and greater aggregate shear strength and
3. Much greater workability due to the textural change from a plastic clay to friable, sand-like material. (Little, 1995)

Clay soils posses two pozzolans: alumina and silica. These pozzolans react with lime to form a bond much like that of cement. The bond between the lime, soil, and water forms calcium-silicate-hydrates and calcium-alumina-hydrates, which are also found in Portland cement after hydration. The addition of lime raises the pH level of the soil to approximately 12.4, which allows more silica and alumina to become available and increases the pozzolanic reaction. The bond that forms between lime, clay-silica, and clay-alumina provides long term strength gain to the stabilized soil (Little, 1995).

2.4 ENGINEERING PROPERTIES

For fine-grained soils, lime is an effective stabilizing agent, which has been used successfully to reduce plasticity, increase workability, and decrease the shrink-swell potential. Strength gain is important when the subgrade is to support the overlaying base course. The amount of strength a soil does show is dependent upon the pozzolanic reaction. However, all soils treated with lime do not show an increase in strength over time. Curing periods also enhance engineering properties, although most soil-lime mixtures are affected immediately.

Plasticity of soils is dramatically reduced with the addition of lime; in fact, the soils may become nonplastic. The most effective quantities of lime added to achieve this reduction are in the first increments shown in Figure 2.2. Less substantial reduction is evident with additional increments in lime percentages (TRB State of the Art Report No. 5, 1987). Curing periods are negligible due to immediate reactions (Little, 1995). Changes in Atterberg limits, such as PI and liquid limit, resulting from using different percentages of lime are shown in Table 2.1.

Moisture-density relationships are also immediately affected by the addition of lime. The addition of lime results in a decrease in the optimum density and an increase in the optimum moisture content. For lime-treated soils, the reduction in maximum dry density is around 3 - 5 pounds per cubic foot, and the increase in optimum moisture content is around 2 - 4 percent. Figure 2.3 shows the effects of lime treatment for a clayey soil. Research also indicates that if curing takes place, additional decreases in dry densities and increases in optimum moisture contents may be noticed (Little, 1995).

Swell potential of fine-grained soils can also be controlled with the use of lime. The amount of swell is a function of the PI of the soil. Seed, Woodward and Lundgren developed this relationship, which is shown in Figure 2.4 (1962).

When the swell potential is decreased, so is the swell pressure; this is shown in Figure 2.5, where the swell pressure decreases with additional percentages of lime. The amount of curing time also aids in the reduction of swell pressure. Finally, Table 2.2 illustrates the effectiveness of lime to reduce swell over a range of soils, as measured by the California Bearing Ratio (CBR) procedure (Little, 1995).

Strength properties in soil-lime mixtures are not as evident in immediate reactions as they are with long periods of curing. The curing period is usually between 7 to 28 days. Table 2.3 shows the compressive strength of soils with different percentages of lime and increased curing periods, while Table 2.4 illustrates the stress-strain curves for different percentages of lime. The strength gain is attributed to the pozzolanic reaction between the lime and the soil. Shen and Li noted from their study that strength gain is related to a ratio of the Fine Grain Fraction per Lime (FGF/Lime). Hence, lower strengths are associated with lower FGF/lime. Their study also found that curing increased the strength of various types of clays when using 5 percent lime (1970). Long term strength gain has been documented to last over ten years in field conditions (McDowell, 1966). The unconfined compressive strength is the best way to select the optimum lime content and is also a good measure of shear strength (Little, 1995).

Permeability is increased in soils treated with lime. In a study conducted by McCallister and Petry (1991), they found that the permeability of the soil increased by as much as 7 to 300 times with the addition of lime. They also looked at the leaching effects on permeability and found that permeability decreased over time with leaching. Finally, McCallister and Petry concluded that in order to reduce leaching and moisture increases, which affect permeability, it is important to use optimum lime contents from the maximum compressive strength test (1990).

Durability of lime treated soils is evaluated either with cyclic freeze-thaw conditions or wet-dry conditions. These tests are important because they examine and simulate actual field conditions which might be encountered. Long term studies have been conducted at Dallas Fort Worth International Airport (Long, 1989) and Friant-Kern Canal in California. The Friant-Kern Canal undergoes many wet-dry conditions within a year and still maintains a very strong slope stability (Little, 1995).

Other engineering properties attributed to soil lime mixtures include: triaxial strength, tensile strength, fatigue strength, and deformation or modulus properties.

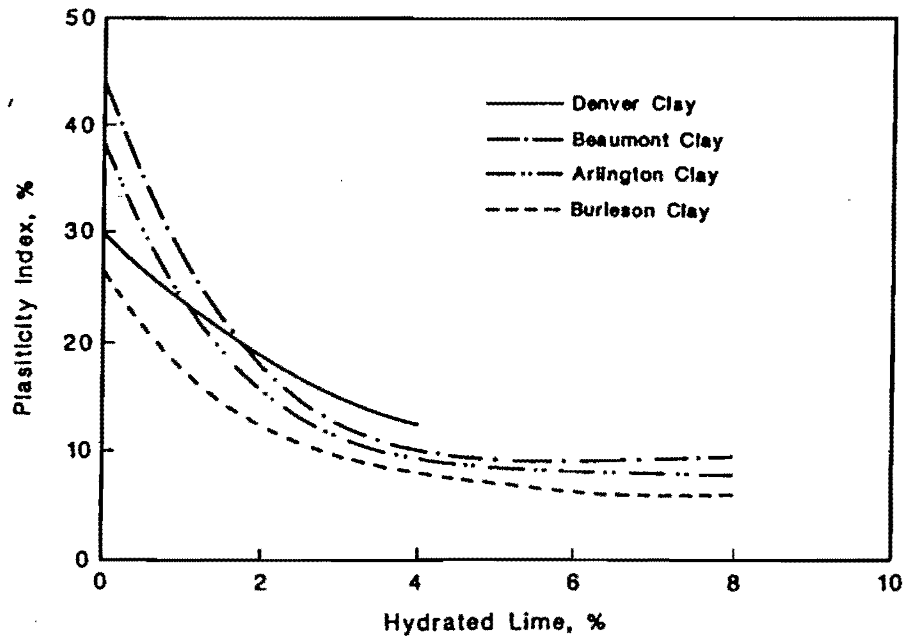


Figure 2.2: Degree of Reduction in Plasticity Index as Increasing Percentages of Lime are Added (Little, 1995)

Table 2.1: Atterberg Limits for Natural and Lime - Treated Soils (After Little et al., 1987).

Unified Soil Classification	*Natural Soil*		*3% Lime*		*5% Lime*	
	LL	*PI*	*LL*	*PI*	*LL*	*PI*
Bryce B CH	53	29	48	21	NP	
Clay Till CL	49	27	51	12	59	11
Cowden B CH	54	3	47	7	NP	
Drummer B CH	54	31	44	10	NP	
Fayene C CL	32	10	NP			
Hosmer B2 CL	41	17	NP			
Piasa B CH	55	36	48	11	NP	
Illinoian Till CL	26	11	27	6	NP	

LL-Liquid Limit
NP-Nonplastic
PI-Plasticity Index

(Note data in Table 2.1 were provided by Dr. M. R. Thompson of the University of Illinois at Champaign - Urbana).

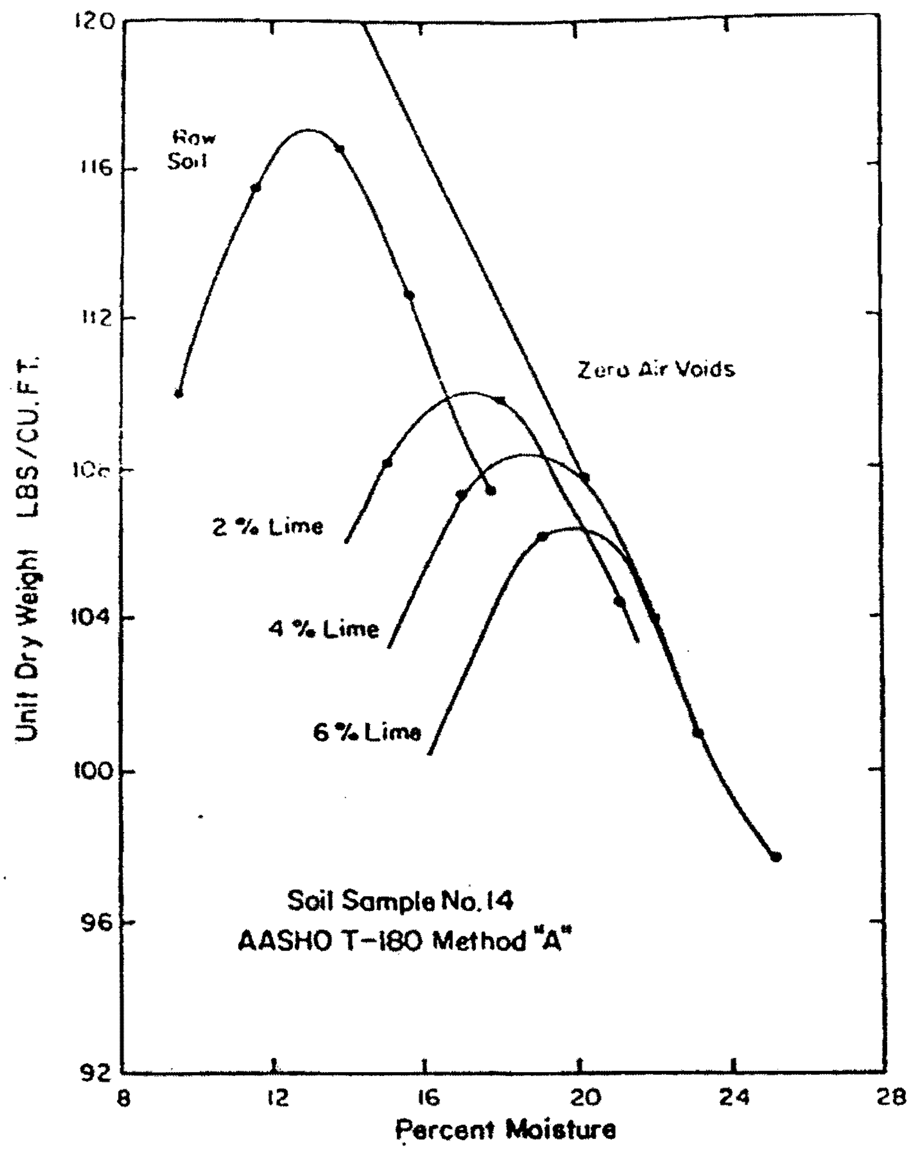


Figure 2.3: Moisture-Density Curve for a Natural Soil and a Lime Treated Soil (Haston and Wholgemuth, 1985).

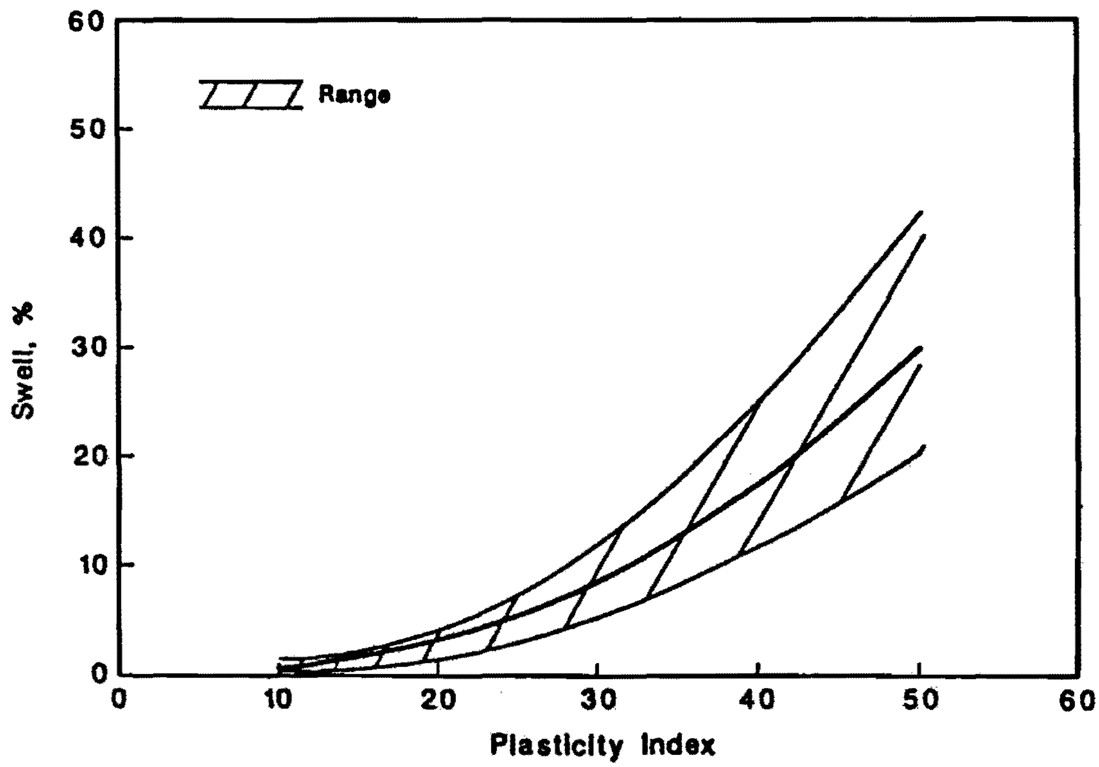


Figure 2.4: Predicting Swell Percent as Related to PI (Seed, Woodward, and Lundgren, 1962)

Figure 2.5: Swell Pressure Related to Different Lime Percentages (Little, 1995).

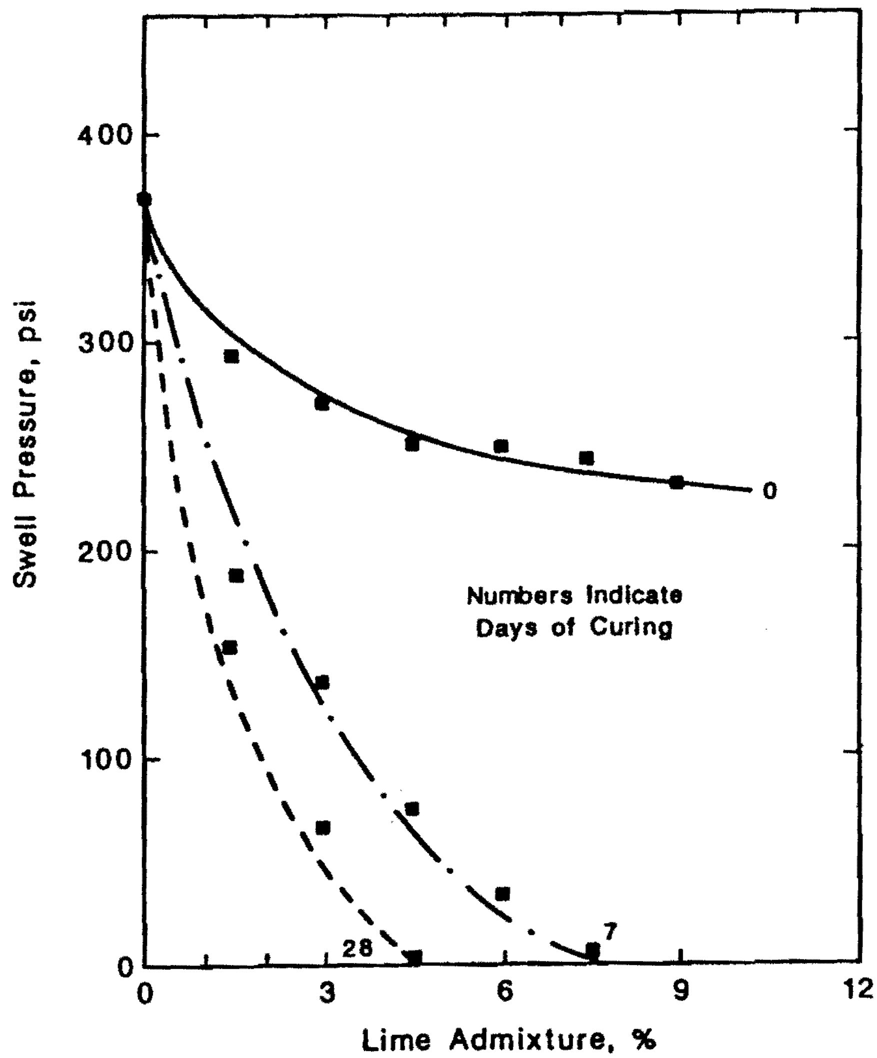


Table 2.2: Swell Percent for Different Soils Based on CBR Procedures (Little, 1995).

					Soil - Lime Mixtures			
	Unified Classifi -	Natural Soil			No Curing		48 Hrs Curing @ 120 ^{0}F	
Soil	cation	CB R%	Swell %	% Lime	CBR %	Swell %	CBR %	Swell %
Accretion Cley 2	CL	2.6	2.1	5	15.1	0.1	351.0	0.0
Accretion Gley 3	CL	3.1	1.4	5	88.1	0.0	370.0	0.1
Bryce B CH	1.4	5.6	3	20.3	0.2	197.0	0.0	
Champaign Co. Till	CL-ML	6.8	0.2	3	10.4	0.5	85.0	0.1
Cisne B CH	2.1	0.1	5	14.5	0.1	150.0	0.1	
Cowden B CH	7.2	1.4	3	—	—	98.5	0.0	
Cowden B CH	4.0	2.9	5	13.9	0.1	116.0	0.1	
Cowden C CL	4.5	0.8	3	27.4	0.0	243.0	0.0	
Darwin B CH	1.1	8.8	5	7.7	1.9	13.6	0.1	
East St. Louis Clay	CH	1.3	7.4	5	5.6	2.0	17.3	0.1
Fayette C CL	1.3	0.0	5	32.4	0.0	295.0	0.1	
Illinoian B CL	1.5	1.8	3	29.0	0.0	274.0	0.0	
Illinoian Till CL	11.8	0.3	3	24.2	0.1	193.0	0.0	
Illinoian Till CL	5.9	0.3	3	18.0	0.9	213.0	0.1	
Sable B CH	1.8	4.2	3	15.9	0.2	127.0	0.0	
Non-Reactive Soils								
Fayette B CL	4.3	1.1	3	10.5	0.0	39.0	0.0	
Miami B CL	2.9	0.8	3	12.7	0.0	14.5	0.0	
Tama B CH	2.6	2.0	3	4.5	0.2	9.9	0.1	

' Specimens were placed in 96 hours soak immediately after compaction.

Table 2.3: Compressive Strength Results on Various Clays (Little, 1995).

Note: Curing Conditions of 28 days at 22°C (73°F).Data for all Illinois soils provided by M. R Thompson (1982), 1 psi = 6,894 Pa

		Compressive Strength psi		
	Unified Classification	*Percent Lime*		
Soil		*3*	*5*	*7*
Arlington, TX	CH	250	350	650
Beaumont, TX	CH	70	100	200
Burleson, TX	CH	150	220	310
Victoria, TX	CH	100	190	260
Denver, CO	CL	300	400	350
Bryce A, IL	MH	43	58	53
Bryce B. IL	CH	201	212	193
Cisne B. IL	CH	107	190	189
Drummer A, IL	ML	29	49	32
Drummer B. IL	CH	186	152	146
Fayette A, IL	ML	37	46	49
Fayette B. IL	CL	109	114	113
Fayette C, IL	CL	137	185	125
Accretion-Cley, IL	CL	263	247	283
Huey B. IL	CL	223	216	233
Huey D, IL	CL	222	179	197
Illinoian Till, IL	CL	150	186	143
Loam Till, IL	MH	172	184	174
Davidson B. IL	MH	198	268	324
Greenville B. IL	CL	455	517	551
Norfolk B. IL	SC	347	421	332
Clalitos B. IL	MH	114	133	132
Nipe B. IL	ML	87	220	311
Cecil B. IL	CH	168	163	224

Table 2.4: Strength Gain from Long Term Curing on Different Soils (Little,1995).

			Unconfined Compressive Strength, psi					
			3 Percent Lime			*7 Percent Lime*		
Soil	*AASHTO Classification*	*Plasticity Index*	*28 Day*	*180 Day*	*360 Day*	*28 Day*	*180 Day*	*360 Day*
1	A-6 (10)	14	160	210	220	120	210	610
2	A-6 (10)	11	390	410	510	400	120	1410
3	A-7-5 (20)	30	280	360	310	550	1190	1210
4	A-2-4	NP	100	100	100	110	150	180
5	A-7-6 (20)	30	350	430	640	260	1200	1650
6	A-7-5 (13)	15	70	60	70	220	200	220
7	A-4 (5)	7	80	160	240	120	210	400
8	A-6	14	540	700	750	550	1200	1530
9	A-4	7	420	920	1100	550	1250	1900
10	A-7-5 (20)	22	400	760	830	300	950	1200
11	A-4 (2)	10	275	410	900	210	800	1110
12	A-7-5 (20)	22	360	490	320	510	810	1010

1 psi = 6,894 Pa

2.5 DESIGN PROCEDURES

Many design procedures are used to determine the optimum lime content to treat a soil. Design criteria vary between states and, therefore, may dictate the procedure to be used. For instance, the United States Air Force has developed a soil stabilization index system, which is illustrated in Figure 2.6. This system generalizes in order to identify the soil properties and the most effective stabilizing agent for that soil (Epps, Dunlap, Galloway and Curing). In this paper, a detailed description of the Eades and Grim Test will be given as well as a general overview of the Texas Procedure and Thompson Procedure.

<u>2.5.1 Eades and Grim Test</u>. Eades and Grim developed a procedure based on the pH of a soil. The pH quick test is effective for predicting optimum lime contents which correspond to maximum strength tests. A summary of the pH quick test is given:

1. Representative samples of air-dried, minus No. 40 soil to equal 20gm of oven dried soil are weighed to the nearest 0.1gm and poured into 150-ml (or larger) plastic bottles with screw tops.

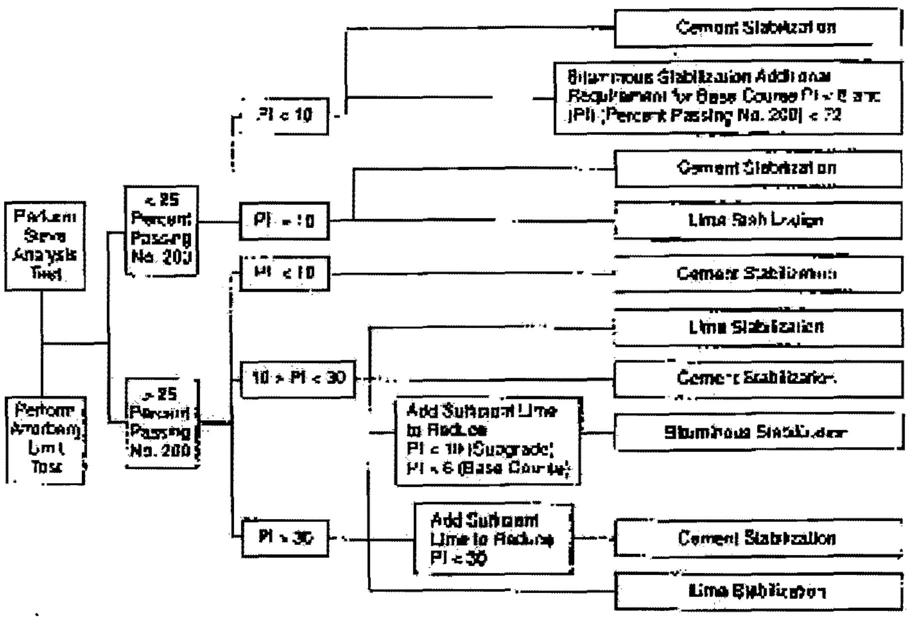


Figure 2.6: Air Force Soil Stabilization Index System (Epps, Dunlap, Galloway, and Curing, 1976)

2. Since most soils will require between 2 and 5 percent lime, it is advisable to set up five bottles with lime percentages of 2, 3, 4, 5, 6. In most cases, this will ensure that the percentage of lime required can be determined in one hour.

3. Weigh the lime to the nearest 0.01gm and add it to the soil.

4. Shake to mix soil and dry lime.

5. Add 100 ml of CO_2-free distilled water to the bottles.

6. Shake the soil-lime and water until there is no evidence of dry material on the bottom (a minimum of 30 seconds).

7. Shake the bottles for 30 seconds every 10 minutes.

8. After one hour, transfer part of the slurry to a plastic beaker and measure the pH. The pH meter must be equipped with a Hylalk electrode and standardized with a buffer solution having a pH of 12.00.

9. Record the pH for each of the lime-soil mixtures. If the pH readings go to 12.40, the lowest percent lime that gives a pH of 12.40 is the percent required to stabilize the soil. If the pH did not go beyond 12.30 and 2 percent lime gives the same reading, the lowest percent which gives a pH of 12.30 is that required to stabilize the soil. If the highest pH is 12.30 and only 1 percent lime gives a pH of 12.30, additional test bottles should be started with larger percentages of lime.

2.5.2 Thompson Procedure. Other procedures include the Thompson Procedure and Texas Procedure as well as many others not mentioned here. The Thompson procedure incorporates many tests to determine the optimum lime content. The test and test methods for each are as follows: Moisture-Density (ASTM D-698 or ASTM D-1557), Atterberg Limits (ASTM D-4318), Swell Potential (ASTM D-3668) and Unconfined Compression Test (ASTM D-5102).

2.5.3 Texas Procedure. The Texas procedure requires the subgrade and the base to meet certain strength criteria, 50 psi and 100 psi respectively. In the design it incorporates the PI, grain size of the material, moisture-density (controlled by Tex-113-E) and unconfined compression (Tex-117-E).

III. CEMENT AS A SOIL STABILIZING AGENT

3.1 INTRODUCTION

This chapter will provide an overview of the use of Portland cement as a soil stabilizing agent. It will describe the mechanism by which cement stabilizes a soil and some of the properties of a modified soil after stabilization. Additionally, the types of soils that are suitable for cement stabilization are identified. Finally, design procedures for both a soil-cement and cement-modified soil will be discussed.

3.2 MECHANICS OF SOIL-CEMENT REACTIONS

There are basically four mechanisms by which soil is stabilized using cement. The two most important mechanisms are hydration and cation exchange, with carbonation and pozzolonic reactions playing a less significant role.

3.2.1 Hydration. Cement is a complex mixture comprised of many compounds. The prominent compounds that play a major role in hydration and account for 90 percent or more of the weight of portland cement are tricalcium silicate, C_3S, dicalcium silicate, C_2S, tricalcium aluminate, C_3A, and tetracalcium aluminoferrite, C_4AF. However, there are others which play a significant role (Kosmatka and Panarese, 1988). C_3S and C_2S make up approximately 75 percent of the weight of portland cement and react with water to form two important new compounds: calcium hydroxide and calcium silicate hydrate (Kosmatka and Panarese, 1988). Calcium hydroxide increases the pH of the pore water and creates a favorable environment for stabilization (Sherwood, 1962; Christensen, 1969; Air Force Manual 88-6, 1982). Calcium silicate hydrate contains mostly lime, CaO, and silicate, SiO_2, and is the most critical component in establishing the engineering properties of concrete such as setting and hardening, strength, and dimensional stability (Kosmatka Panarese, 1988).

3.2.2 Cation Exchange. Cation exchange is the second most important mechanism in the stabilization of a cohesive soil. In this mechanism, a cation from the cement fills a vacant position or exchanges positions with another cation in the clay mineral crystalline structure. This exchange usually results in a reduction in the net surface charge of the clay particle and a consequent lesser attraction for free water molecules. Also, a flocculated soil structure is produced, which is considerably easier to manipulate during construction.

Cohesive soils that require stabilization have negatively charged sites that are occupied by exchangeable cations. There are three reasons for the presence of such sites with exchangeable cations: (1) broken bonds around the edges of the clay mineral, (2) substitution within the lattice structure of the clay mineral, and (3) replacement of the hydrogen of exposed hydroxyls by cations which may be exchangeable. Other factors affecting a soil's capacity for exchanging cations are particle size, temperature, availability and concentrations of ions in solution, clay mineral structure, and isomorphic substitution.

3.2.3 Carbonation. Lime is generated during the hydration of the cement. This lime will react with carbon dioxide present in the surrounding air and form cementitious materials of calcium carbonate. These materials contribute to the strength improvement in cement stabilized soils (Christensen, 1969).

3.2.4 Pozzolonic Reactions. A portion of the lime that is generated during the hydration process reacts with silica or alumina ions from the clay structure. A mild cementitious material results strengthening the bonds within the stabilized soil (Christensen, 1969; Air Force Manual 88-6, 1982). This reaction occurs over a long period and contributes little to initial increase in strength.

3.3 TYPES OF SOILS SUITABLE FOR CEMENT STABILIZATION

It is important to have a basic understanding of the major soil types and their properties. This knowledge can serve as a starting point for proper selection and use of a stabilizing agent. Though cement is capable of stabilizing a wide range of soil types, it is most effective in sands, sandy and silty soils, and clayey soils of low to medium plasticity (Air Force Manual 88-6, 1982). Cement may be used in highly plastic clays, but generally, it is considered to be more effective when lime is added initially to lower the plasticity index (Ingles and Metcalf, 1973; Air Force Manual 88-6, 1982).

Organic soils pose special problems for effective cement stabilization. While some organic compounds have little effect on the stabilization process, those with lower molecular weight may prevent or retard hydration (Air Force Manual 88-6, 1982). As a result, strength gains may not be realized. However, if the organic matter does not exceed about two percent, the soil can be adequately stabilized (Ingles and Metcalf, 1973).

Sulfates may also cause cement stabilization to be less effective. Sulfates are known to be detrimental to cement used in concrete because they interfere with hydration and disrupt the soil-cement by crystallizing and expanding (Ingles and Metcalf, 1973). Sulfates' affect on cement stabilization, however, is not as well understood. There is evidence that the adverse effect of sulfates may be due to a reaction with the clay rather than the cement (Ingles and Metcalf, 1973). Therefore, a sulfate-resistant cement would be of little benefit in a clay soil. For this reason, using cement stabilization for fine-grained soils should not be considered if the soil contains more than about one percent sulfates (Air Force Manual 88-6, 1982). In granular soil-cements, however, a sulfate-resistant cement would still be of benefit (Air Force Manual 88-6, 1982).

In general, cement requirements increase as the silt and clay content increase (Air Force Manual 88-6,1982). A rule of thumb is "use lime for clays and cement for sands." However, Ingles and Metcalf (1973) state that this "rule of thumb" leaves a lot to be desired in many cases. Table 3.1 shows the increase in cement required for a soil-cement as the soil becomes finer. Table 3.2 shows the applicability of various stabilization methods, and Table 3.3 shows stabilization response if major soil components are known. Tables 3.1-3.3 are generalizations and the information should be supported by laboratory testing.

3.4 ENGINEERING PROPERTIES OF CEMENT-STABILIZED SOILS

Addition of cement to a soil usually results in a modification of the engineering properties of that soil. Not all soils will be modified in the same way or to the same degree, though there are enough similarities that some strong generalizations can be made. Some of these properties will be discussed in this section. It should be noted that the modified properties are strongly dependent upon compacted density, moisture content, and confining pressure (Air Force Manual 88-6, 1982).

3.4.1 Plasticity Index. The plasticity index of a soil is an indication of its potential to change in volume in response to changes in moisture. A soil having a plasticity index of 35 or greater would be expected to have a very high degree of expansion, while a plasticity index of less than 18 would be considered to have a low degree of volume change (Portland Cement Association, 1992a). Normally, soils with a plasticity index of less than 18 would pose few problems but a preferred range is usually specified between 12 and 15 (Portland Cement Association, 1992a).

Usually cement has a great affect on reducing the plasticity index even at low cement percentages. However, as the plasticity index increases above 30 (Air Force Manual 88-6, 1982) or the liquid limit increases above 50 (Ingles and Metcalf, 1973),
mixing may become difficult. It is common to add 2 to 3 percent of either lime or cement as a pre-treatment (Ingles and Metcalf, 1973). After several days of curing, cement stabilization can continue (Ingles and Metcalf, 1973).

Table 3.1: Cement Requirements for Various Soils.

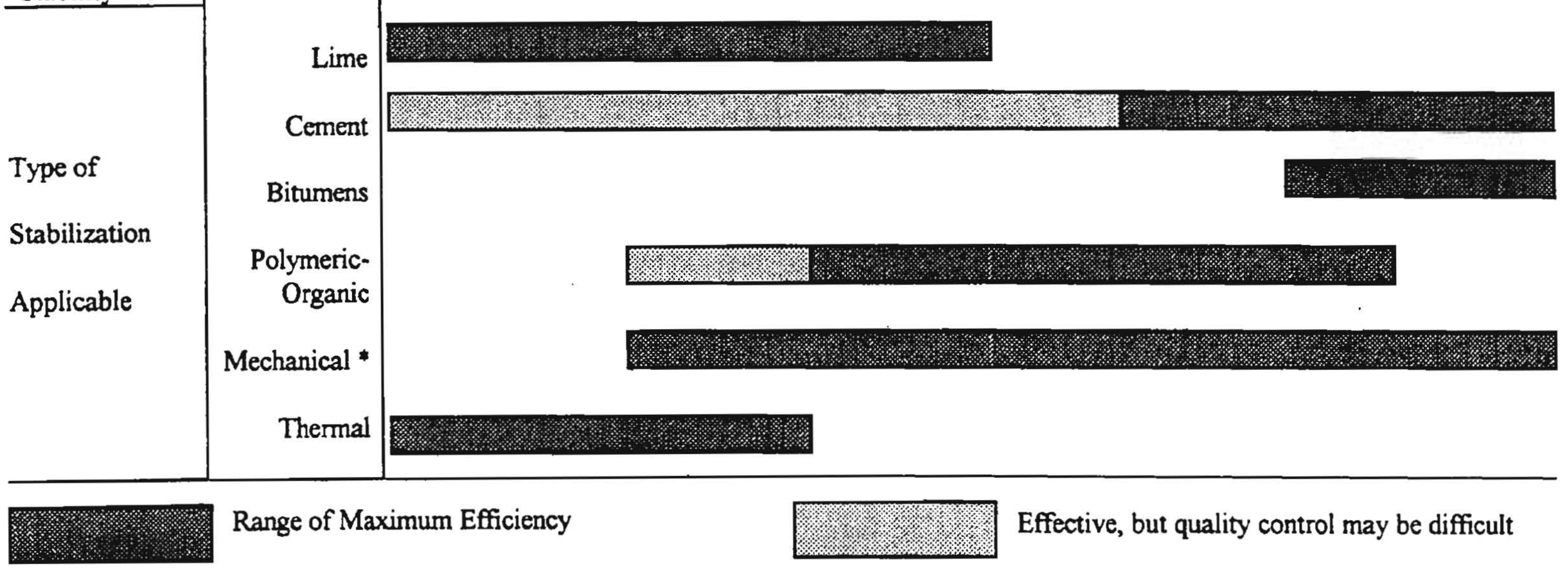
AASHTO Soil Classification	Unified Soil Classification*	Usual Range in Cement Requirement** Percent by Volume	Usual Range in Cement Requirement** Percent by Weight	Estimated Cement Content and That Used in Moisture-Density Test, Percent by Weight	Cement Contents for Wet-Dry and Freeze-Thaw Tests, Percent by Weight
A-1-a	GW, GP, GM, SW, SP, SM	5 - 7	3 - 5	5	3 - 5 - 7
A-1-b	GM, GP, SM, SP	7 - 9	5 - 8	6	4 - 6 - 8
A-2	GM, GC, SM, SC	7 -10	5 - 9	7	5 - 7 - 9
A-3	SP	8 - 12	7 - 11	9	7 - 9 - 11
A-4	CM, ML	8 - 12	7 - 12	10	8 - 10 - 12
A-5	ML, MH, CH	8 - 12	8 - 13	10	8 - 10 - 12
A-6	CL, CH	10 - 14	9 - 15	12	10 - 12 - 14
A-7	OH, MH, CH	10 - 14	10 - 16	13	11 - 13 - 15

*Based on correlation presented by Air Force. ** For most A horizon soils the cement should be increased four percentage points, if the soil is dark grey, and six percentage points if the soil is black.

Source: Air Force Manual 88-6, 1982.

Table 3.2: Applicability of Stabilization Methods.

Designation	Fine Clays	Coarse Clays	Fine Silts	Coarse Silts	Fine Sands	Coarse Sands
Soil Particle Size (mm)	<0.0006	0.0006-0.002	0.002-0.01	0.01-0.06	0.06-0.4	0.4-2.0
Soil Volume Stability	V. poor	Fair	Fair	Good	V. good	V. good



* i.e., improvement of soil grading by mixing-in gravels, sands or clays as appropriate

Source: Ingles and Metcalf, <u>Soil Stabilization: Principles and Practices</u>, 1973.

Table 3.3: Stabilization Response of Major Soil Components.

Dominant Soil Component	Recommended Stabilizers	Reasons
Organic matter	mechanical	other methods ineffective
Sands	clay loam	for mechanical stability
	cement	for density and cohesion
	bitumens	for cohesion
Silts	none known	———
Allophanes	lime*	for pozzolanic strength and densification
Kaolin	sand	for mechanical stability
	cement	for early strength
	lime	for workability and later strength
Illite	cement	as for kaolin
	lime	as for kaolin
Montmorillonite	lime	for workability and early strength
Chlorite	cement	theoretical (reported stabilization experience is sparse)

* Lime/gypsum mixtures with gypsum contents up to 40 percent may be especially favorable.

Source: Ingles and Metcalf, Soil Stabilization: Principles and Practices, 1973.

3.4.2 Compressive Strength. Normally, compressive strength of the stabilized soil is determined by using an unconfined compression test. Strengths will increase with increasing cement content (Davidson, 1962). The amount of curing also has a significant affect on strength and can be estimated by the following equation (Air Force Manual 88-6):

$$(UC)_d = (UC)_{do} + K \log (d/d_o) \qquad (3.1)$$

where, $(UC)_d$ = unconfined compressive strength at d days, psi

$(UC)_{do}$ = unconfined compressive strength at d_o days, psi where $d_o < d$

K = 70C for granular soils and 10C for fine-grained soils

C = cement content, in percent by weight

In general, fine-grained soils with low cement content may have an unconfined compressive strength as low as 200 psi in 7 days. At the other extreme, granular soils with higher cement contents may have unconfined compressive strengths over 2,000 psi Davidson, 1962). Air Force Manual 88-6 suggests that the 28-day strength can be predicted to be approximately 1.5 times the 7-day strength. Normal ranges of unconfined compressive strengths of soil-cements are shown on Table 3.4.

3.4.3 Durability.

Durability is generally considered to increase with increasing cement contents. This increase, though, may not be that significant. Petry and Wohlgemuth (1988) have found that cement stabilized specimens have performed poorly when subjected to the wet-dry test method. They observed that fine-grained soils stabilized with cement did perform somewhat better than coarse-grained soils. However, it could not be found in the technical literature where durability was cited as a concern or that lack of durability caused significant problems in actual usage in the field. Ingles and Metcalf (1973) suggest that durability is one of the most difficult properties to evaluate due to the lack of relevant test procedures. An example of this difficulty is suggested by Packard and Chapman (1963) who contend that the high temperatures used in laboratory testing may unduly benefit specimens that might not otherwise pass the test; specimens dried at lower temperatures did not perform nearly as well. Two standardized tests used to evaluate the durability of a molded specimen are ASTM Designation: D559, "Methods of Wetting and Drying Test of Compacted Soil-Cement Mixtures" and ASTM Designation: D560, "Methods of Freezing and Thawing Test of Compacted Soil-Cement Mixtures."

Table 3.4: Ranges of Unconfined Compressive Strengths of Soil-Cement.

Soil Type	Wet Compressive Strength[a] (psi) 7-day	28-day
Sandy and gravelly soils: AASHTO groups A-1, A-2, A-3 Unified groups GW, GC, GP, GF, SW, SC, SP, SF	300 - 600	400 - 1,000
Silty soils: AASHTO groups A-4, A-5 Unified groups ML and CL	250 - 500	300 - 900
Clayey soils: AASHTO groups A-6, A-7 Unified groups MH and CH	200 - 400	250 - 600

[a]Specimens moist cured 7 or 28 days, then saturated in water prior to strength testing.

Source: Air Force Manual, 1982.

3.4.4 Permeability. Permeability is a measure of a soil's ability to transmit the flow of water through it. Permeability of sand and gravel is influenced by particle size and is normally very high, as much as ten million times that of clay (Head, 1994). In a fine-grained soil it is not the particle size that affects the permeability as much as it is the mineralogy (Head, 1994). The mineralogy of a clay determines how thick the adsorbed water layer will be and, consequently, controls the effective pore size (Head, 1994).

The addition of cement to a sand or gravel will reduce its permeability by bonding the particles together during hydration and reducing the void spaces and the interconnectivity of these void spaces. With the addition of cement to clay, hydration also occurs. However, an additional mechanism that occurs is cation exchange. In this mechanism, a cation from the cement fills a vacant position or exchanges positions with another cation in the clay mineral crystalline structure. This exchange usually results in a reduction in the net surface charge of the clay particle and a consequent lesser attraction for free water molecules. A flocculated soil structure is produced. This flocculation causes an increase in permeability, but is tempered by the resulting cementitious material blocking the pores (Fam and Santamarina, 1996). Review of the technical literature did not reveal any case studies where permeability was a problem after the addition of cement to clay soils.

3.4.5 Compaction. Addition of cement to a soil usually causes a change in optimum moisture content and maximum density. The higher specific gravity of cement generally produces a higher density, but this is offset by the flocculating effect of the cement on a clay (Air Force Manual 88-6, 1982). Flocculation generally results in a slight decrease in density along with a slight increase in optimum moisture content (Air Force Manual 88-6, 1982).

A delay in compaction after addition of cement to a soil will give the cement an opportunity to begin hydration. This delay will make compaction more difficult which in turn will lower strength and density (Ingles and Metcalf, 1973; Air Force Manual 88-6, 1982). If no significant hydration has occurred, this delay can be overcome if the compactive effort is increased (Air Force Manual 88-6, 1982). Lightsey, Arman, and Callihan (1970) suggest that moisture content be increased above optimum by two to four percent if there is a delay in compaction, a frequent occurrence in the field, to help counteract strength loss.

3.4.6 Tensile Strength. Tensile strength of cement-modified soils can range from 20 to 33 percent of their unconfined compressive strength (Ingles and Metcalf, 1973; Air Force Manual 88-6, 1982). If the compressive strength is known then a good approximation of the flexular strength may be obtained by the following equation (Air Force Manual 88-6, 1982):

$$f = 0.51(UC)^{0.88} \tag{3.2}$$

where, f = flexular strength, psi

UC = unconfined compressive strength, psi.

An empirical relationship developed by Wang and Huston (1972) can estimate the direct tensile strength of a cement stabilized soil. This is shown in Eq. 3.3 (Wang and Huston, 1972):

$$\sigma = \left\{\frac{1}{2} + \frac{60C^{4/3}}{\left(32 + C^{4/3}\right)}\right\}\left\{1 + \left(2\log\frac{t}{92}\right)^{8/3} + \left(\log t\right)^{8/3}\right\} \tag{3.3}$$

where, σ = direct tensile strength, psi

C = cement content by weight of solid, percent, and

t = curing time, days.

Wang and Huston (1972) also noted that decreasing temperatures for curing decreased the strength of the cement treated soil. Moore, Kennedy, and Hudson (1970) identified nine factors thought to be the most important in affecting the tensile strength of a cement-treated soil: molding water content, curing time, aggregate gradation, type of curing, aggregate type, curing temperature, compactive effort, type of compaction, and cement content.

3.4.7 Shrinkage Cracking. Soil-cements commonly exhibit shrinkage and associated cracking. This is a natural occurrence and does not represent structural failure (Norling, 1973). While this characteristic may be inevitable, many factors determine the severity and frequency of this occurrence. Much depends upon the amount of cement, soil type, water content, compaction and curing (Air Force Manual 88-6, 1982). The primary reason for shrinkage is the loss of water through evaporation, self-desiccation during hydration, and temperature changes (Norling, 1973; Wang, 1973). When the resulting tensile stresses exceed the tensile stresses of the soil-cement, then cracking will occur (Wang, 1973). Because of the lower cement contents, this is generally not a factor in a cement-modified soil. Stress can be calculated to predict cracking and crack propagation (George, 1973). In general, severity will increase with higher water content, soils with higher clay contents, certain types of clay such as montmorillonite, and improper curing (Air Force Manual 88-6,1982). Clean, well graded gravels and crushed rock do not normally need to be stabilized, and addition of cement may cause serious shrinkage cracking (Ingles and Metcalf, 1973). If shrinkage cracking should occur, measures should be taken to prevent moisture from entering shrinkage cracks.

3.4.8 Fatigue Characteristics. Fatigue in flexure is evident by cracking of the pavement. This allows moisture into the support system of the roadway and can cause the roadway to fail. It should be noted that fatigue life is shorter when subjected to tensile stresses rather than compressive stresses (Air Force Manual 88-6, 1982). Also, flexural fatigue is unlikely to occur if repeated stress levels can be kept below 50 percent of the maximum flexural strength (Air Force Manual 88-6,1982). Flexural fatigue can be related to radius of curvature by the equation (Air Force Manual, 1982):

$$R_c/R = aN\text{-}b \tag{3.4}$$

where, R_c = the radius of curvature causing failure under static loading,

R = radius of curvature leading to failure under N load applications,

a = $(h^{3/2})/(2.1h\text{-}1)$,

h = pavement slab thickness, inches,

b= 0.025 for granular soil-cement and 0.050 for fine-grained soil-cement,

N = number of loaded applications.

3.5 DESIGN PROCEDURES

For any stabilization project there are four basic soil properties to be considered: volume stability, strength, durability and permeability. The design process basically involves selecting the appropriate cement content to obtain the desired engineering properties. There are various design procedures, and they differ for a hardened, soil-cement and an unhardened, cement modified soil.

3.5.1 Soil-Cement. In a hardened soil-cement the primary controlling factor is durability, with strength being of secondary concern (Ingles and Metcalf, 1973; Air Force Manual, 1982; Portland Cement Association, 1995). The reasoning is that if a soil cement mixture is able to resist the elements it will possess adequate strength. The two primary tests for durability are the wet-dry and freeze-thaw tests. In 12 cycles of wet-dry or freeze-thaw, the loss by weight must not exceed 14 percent for gravel or 7 percent for clays (Christensen, 1969; Air Force Manual 88-6, 1982). Some agencies, however, do base design criteria only on compressive strengths. For example, the Texas Department of Transportation specifies a compressive strength of 750 psi and 500 psi, designated Strength L and Strength M, respectively, depending on the strength requirements of a specific project (Texas Department of Transportation, 1995b).

In any design, the three fundamental control factors required are cement content, water content and density. Standard test methods often used are as follows (ASTM, 1992):

- *Methods of Test for Moisture Density Relations of Soil-Cement Mixtures*, ASTM Designation: D558; AASHTO Designation: T134

- *Methods of Wetting and Drying Test of Compacted Soil-Cement Mixtures*, ASTM Designation: D559; AASHTO Designation: T1135
- *Methods of Freezing and Thawing Test of Compacted Soil-Cement Mixtures*, ASTM Designation: D560; AASHTO Designation: T136.

The above procedures are quite valuable evaluation tools, but can be laborious and time consuming. At times it may be adequate to use less rigorous procedures. The Portland Cement Association has developed other methods for design of soil-cement. These methods are simple to perform, but may not lead to the most economical design (Portland Cement Association, 1992b). Figure 3.1 presents a flow diagram showing selection of the appropriate design method depending upon the magnitude of the project.

3.5.2 Cement-Modified Soil. For an unhardened, cement modified soil the criterion can be quite different. It seems there is no standard design, but rather the design is based on a specific engineering property or properties to be modified. For example, the Texas Department of transportation Standard Specification Item 275 (Texas Department of Transportation, 1995b) allows the engineer to specify a range of compressive strengths for a cement-treated soil, as shown in Table 3.5.

Cement content selection may also be based on a reduction in the plasticity index. ATEC Associates, Inc. (City of Dallas, 1989) consultants for a project at Love Field in Dallas, Texas, recommended a reduction in the plasticity index to a value of 12 or less, with a minimum cement content of 4 percent, regardless of the initial plasticity index of the soil samples. Taubert (1996), at an on-site demonstration project in San Antonio, Texas, indicated that reduction in the plasticity index was the primary concern. For this project, a four percent cement content lowered the plasticity index from 37 to 11. Other factors may be considered depending upon the problems encountered during a specific project.

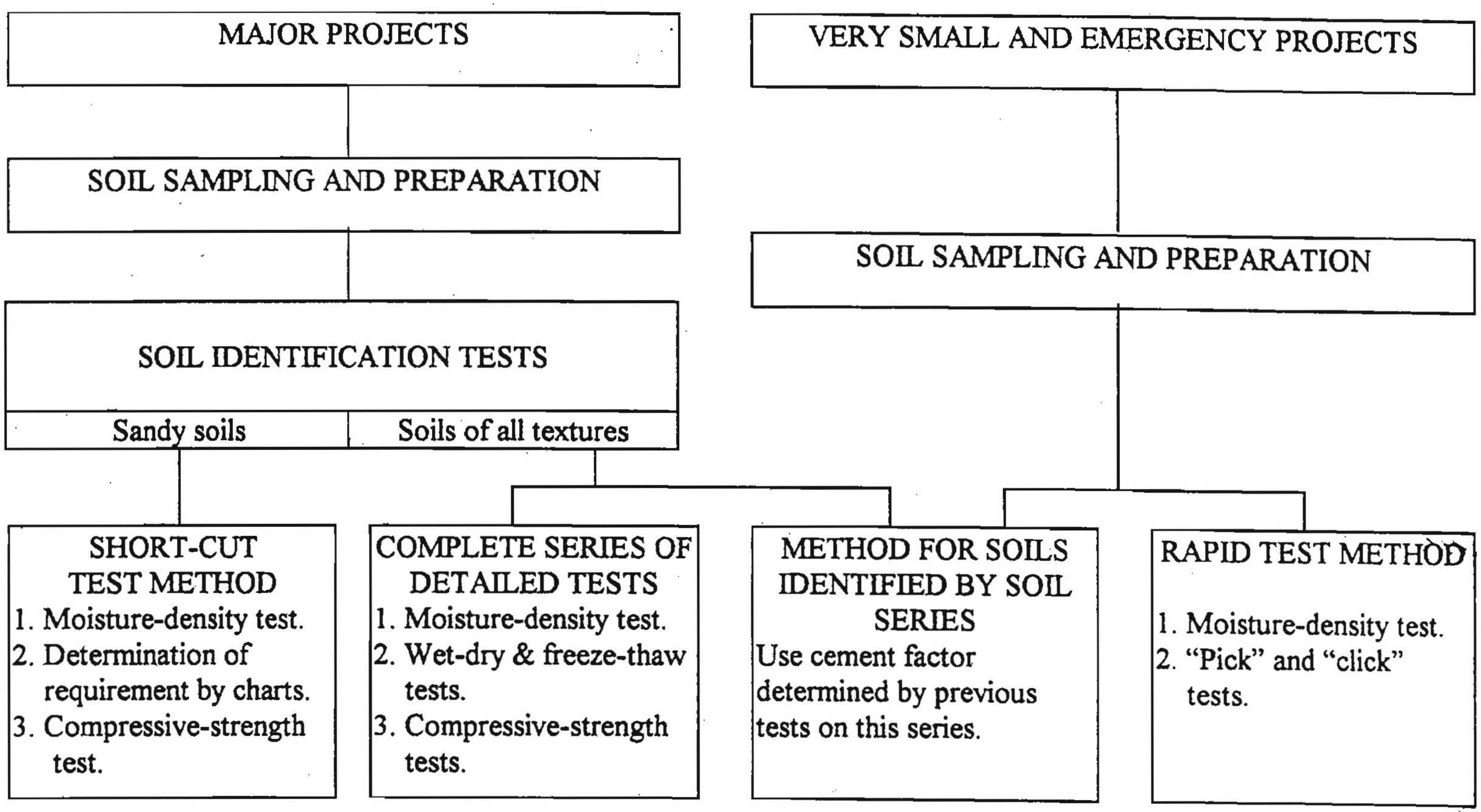


Figure 3.1: Flow Chart for Soil-Cement Laboratory Testing Methods.

Source: Portland Cement Association, 1992b.

Table 3.5: Selection of Strength Requirements for a Cement-Treated Soil.

Strength	Minimum Design Compressive Strength	Allowable Cement Content %
Strength L	5170 kPa (750 psi)	4-9
Strength M	3450 kPa (500 psi)	3-9
Strength N	As shown on plans	----
Strength O	No strength specified	As shown on the plans

Source: Texas Department of Transportation, 1995b.

SELECTION AND CHARACTERIZATION OF THE EXPERIMENTAL PROJECT SITE

4.1 INTRODUCTION

This chapter describes the process of selecting a project site as well as the tests performed to characterize the site. Before site selection could begin, appropriate selection criteria had to be established. These selection criteria would ensure that lime and cement stabilization test results could be compared on a common basis.

One of the primary criteria for selecting a test site was that the roadway needed to be scheduled for reconstruction within the duration of the research project. It was also important that soil conditions/types should be uniform along the lengths of roadway selected for lime and cement stabilization. Other factors taken into consideration during the selection of the experimental project site are as follows:

- Sections should be within close proximity to each other so that both the lime and cement sections would be subjected to the same environmental conditions.
- Sections of selected roadway should not have low areas that would encourage ponding.
- Sections should be free from bridges, culverts and other structures, which would influence the testing results.
- Sections should have similar types and amounts of vegetation along each section of roadway.
- Sections should have the same amounts of traffic.

Based on these criteria, a section of roadway near San Antonio was selected as the project site.

4.2 Project Description

After inspection of several possible locations, two sections on FM Highway 1343 in Medina County, Texas were selected (Figure 4.1). The two test sections, each 1,000 ft in length, are similar in that they are free from culverts, bridges and other structures. Also, both are in a slightly curved section of the roadway, which slopes to the south. There are no low areas that would encourage ponding. The center of the cement stabilized test section is located approximately 1,150 ft. south of the intersection of SH 90 and FM 1343. The center of the lime stabilized test section is located approximately 4,850 ft. from this same intersection. The roadway consists of two lanes, each 12 ft. wide, and 3 ft. shoulders. The average daily traffic on this roadway is 1,400 vehicles per day.

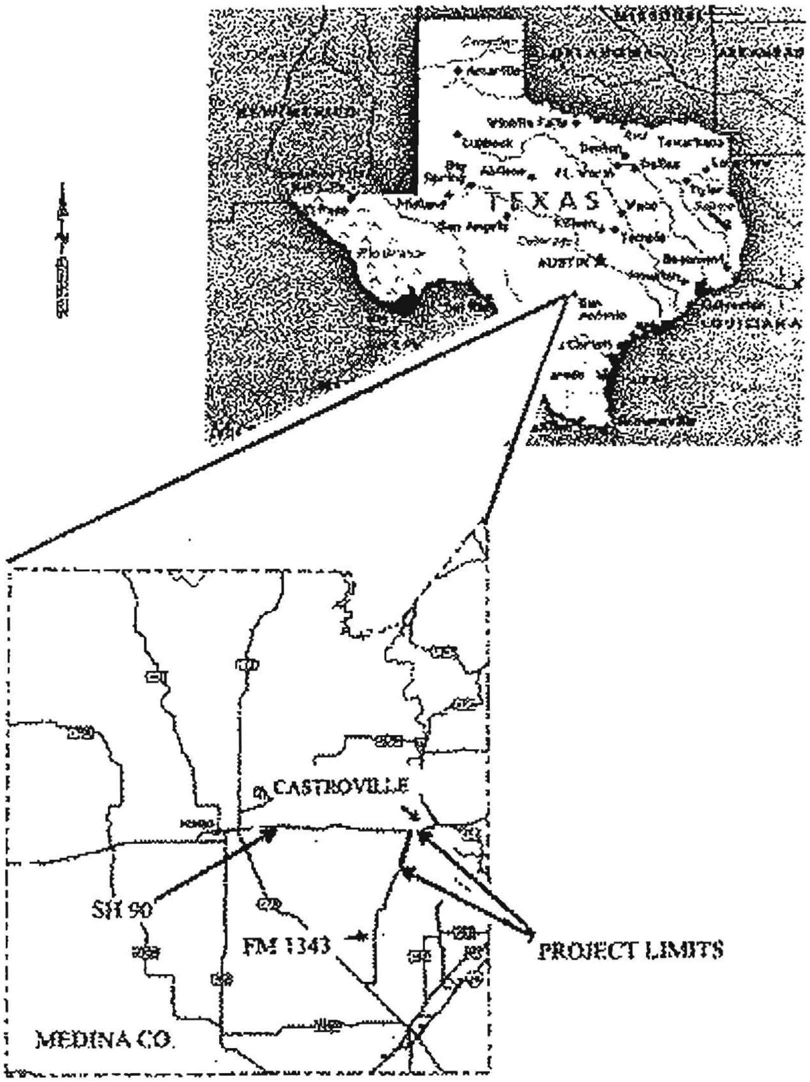


Figure 4.1: Location Map for the Test Site

4.3 SITE CHARACTERIZATION

4.3.1 Soil Sampling. Sampling of the soil was performed prior to any construction activity. Soil samples were obtained approximately 5 feet from the west edge of the roadway, at the center of the two, 1,000 ft. long test sections. Using a Shelby tube, undisturbed soil samples were obtained from two separate borings of each test section. Because the topsoil was mixed with rocks at the surface, an auger was used to a depth of about 1.5 to 3 ft. Shelby tube samples were obtained continuously to a maximum depth of 15.0 ft. in 2 ft. intervals. The soil was basically a stiff, tan clay. All of these samples were carefully wrapped in plastic wrap and then in tin foil to preserve their moisture contents. After labeling, they were placed in a rigid wooden box, with partitions for each sample, for transport back to Texas Tech University in Lubbock, Texas. Once at Texas Tech University, the samples in the wooden boxes were stored in the humid room in the soils laboratory of the civil engineering building.

Two 55-gallon drums were then filled with "near surface" material, one from each test section. Again, the topsoil, which was mixed with rocks, was removed. Because of previous highway construction and maintenance activities, this top layer would not be representative of the soil actually being stabilized during reconstruction. For this reason, the material directly below this top layer was used for laboratory testing. Both drums were sealed and transported back to Texas Tech University.

4.3.2 LABORATORY TESTING

4.3.2.1 Water Content. From the undisturbed soil samples, water contents for each foot of depth were determined using TxDOT test method Tex-103-E. Results of the cement test section are shown in Table 4.1. Water content values for the lime section with depths are reported in Table 4.2. Water content results from the first four feet of depth may be ignored because an auger was used. There is no doubt, however, that moisture movement does occur in this top portion. For the lime section, only one test was performed for each reported value.

4.3.2.2 Atterberg Limits. Atterberg limits (TxDOT test methods Tex-104-E, Tex-105-E, and Tex-106-E) were also determined for each foot of depth from the undisturbed soil samples. The results are shown in Table 4.1.

4.3.2.3 Soil pH. Identifying the pH of a soil helps to identify if the subgrade is suitable soil for stabilization. The standards for a suitable soil are a pH greater than 7 and a USCS classification of CL. The pH of the lime section of soil was determined by ASTM D4972 and was found to be 8.312. The soil was classified as a CL according to the USCS classification system and A-6 classification according to AASHTO. It was, therefore, determined that both soil classifications meet the standards of a suitable soil.

Table 4.1: Atterberg Limits and Moisture Content of Undisturbed Samples.

CEMENT STABILIZED TEST SITE									
Depth, ft	Boring No. 1			Boring No. 2			Average Values		
	Liquid Limit	Plasticity Index	Percent Moisture Content	Liquid Limit	Plasticity Index	Percent Moisture Content	Liquid Limit	Plasticity Index	Percent Moisture Content
0-1	39.9	18.6	----	----	----	----	39.9	18.6	----
1-2	47.2	21.9	----	----	----	----	47.2	21.9	----
2-3	----	----	----	----	----	----	----	----	----
3-4	----	----	----	----	----	----	----	----	----
4-5	25.5	7.3	20.6	25.4	7.2	----	25.4	7.3	20.6
5-6	32.3	12.5	21.4	30.1	10.1	----	31.2	11.3	21.4
6-7	33.6	14.1	20.8	31.5	11.6	----	32.6	12.9	20.8
7-8	24.5	5.8	19.7	24.4	6.0	----	24.5	5.9	19.7
8-9	27.5	8.7	22.1	28.1	9.0	----	27.8	8.9	22.1
9-10	52.6	28.7	22.3	52.0	29.3	----	52.3	29.0	22.3
10-11	45.9	25.6	21.1	44.8	25.6	----	45.4	25.6	21.1
11-12	47.4	27.0	21.9	49.0	27.6	----	48.2	27.3	21.9
12-13	66.5	40.9	23.1	64.3	38.7	----	65.4	39.8	23.1
13-14	57.6	36.3	21.6	55.6	32.6	----	56.7	34.5	21.6
14-15	63.2	36.3	20.5	60.8	34.1	----	62.0	35.2	20.5

Table 4.2: Raw Data for Water Content of Lime Stabilized Section

Depth, ft	Wet Weight, g	Dry Weight, g	Water Content %
2	20.39	16.73	21.88
3	22.34	19.86	12.49
4	31.58	26.10	21.00
5	26.63	22.99	15.83
6	26.81	22.47	19.31
7	28.99	24.37	18.96
8	25.21	20.73	21.61
9	30.46	24.69	23.37
10	26.18	21.75	20.37
11	28.63	24.81	15.40
12	27.43	22.05	24.40
13	32.93	28.61	15.10
14	25.67	19.80	29.65
15	33.91	26.73	26.86

4.3.2.4 Soil Mineralogy. Soil mineralogy or the potential shrink/swell in a given subgrade soil is important in determining the amount and kinds of minerals present within the soil. Two tests, sulfate testing (lime stabilized section only) and X-Ray Diffraction (XRD) analysis were performed on the project experimental section using an air-dried sample of the near surface material. The sulfate test, conducted in the environmental laboratory at Texas Tech University, followed EPA Method 300 and found sulfates to be negligible. Since clay minerals are so small that they can not be visually identified, X-ray diffraction techniques were utilized to determine the clay mineralogy. This technique is based on the knowledge that clays are made up of repeating crystalline sheets and will diffract X-rays (Holtz and Kovacs, 1981; Lancellotta, 1995). These diffraction patterns vary in quite distinctive ways for specific minerals, thus leading to their identification using X-ray diffraction techniques. This procedure was performed on an untreated soil sample from the test site.

The Texas Tech University Geosciences Department conducted the XRD using Phillips Defractometer. The results from this test (Lime—Table 4.3; cement—Table 4.4) show that the soil contains significant amounts of Smectite and calcite. Smectite, which was found to be 20 percent of the soil content mineralogy, is an expansive clay mineral and is responsible for the high plasticity and volume change capabilities in clay.

The significant amount of calcite found in the soil raises major concerns because this suggests that lime is already abundant in the soil. However, according to the chief inspector in the Hondo Area Office of TxDOT, Jerry Burrell, the subgrade of the existing road had not been previously stabilized with lime. Calcium is naturally present in many soils anywhere from two-thirds to three-quarters saturated, which may explain the 70 percent calcite found in the soil.

4.3.2.5 Filter Paper Test. A filter paper test (ASTM Designation D5298) was performed for each foot of depth on the undisturbed samples. This test yields the initial soil suction for each foot of depth down to 11.5 ft. Samples below this depth, from this bore hole, were disturbed during sample retrieval. Results of this test for the cement section are shown in Figure 4.2. The measurements for the initial soil suction profile were taken for the lime section from boring No. 1 and are reported in Figure 4.3.

4.3.3 Review and Analysis of Site Characterization Data.

4.3.3.1 Soil Classification. Based on the laboratory test results presented in Section 4.3.2, soil was classified according to the Unified Soil Classification System (USCS), ASTM Designation D 2487 as well as AASHTO Soil Classification Method, AASHTO Designation M-45. By the USCS scale, soil was classified as CL, and by the AASHTO model soil was classified as A-6.

Table 4.3: Mineralogy of the Untreated Soil from the Lime Test Section as Identified by X-Ray Defraction.

Mineral	Percent
calcite	70
quartz	8
smectite	20
kaolinite	2

Table 4.4: Mineralogy of the Untreated Soil from the Cement Test Section as Identified by X-Ray Defraction.

CLAY MINERAL IDENTIFIED	PERCENTAGE OF SOIL
Calcite	70
Smectites	20
Kaolinites	4
Quartz	6

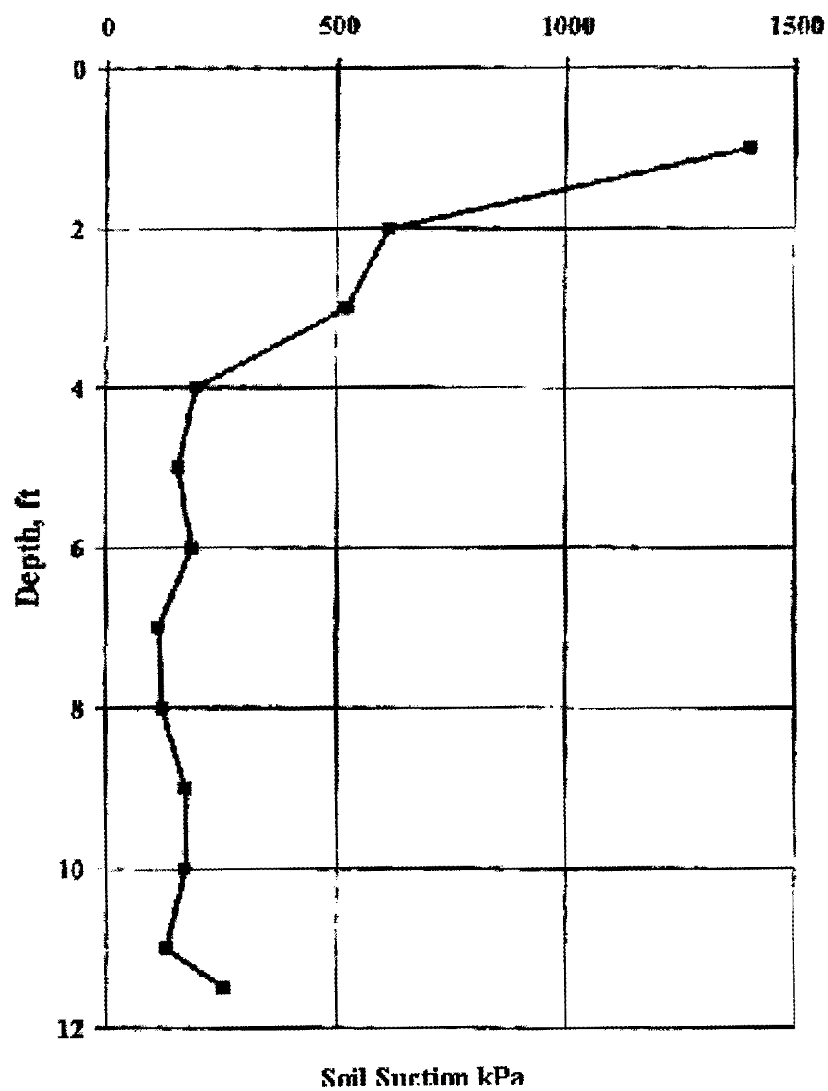


Figure 4.2: Initial Soil Suction Profile from the Cement Test Section

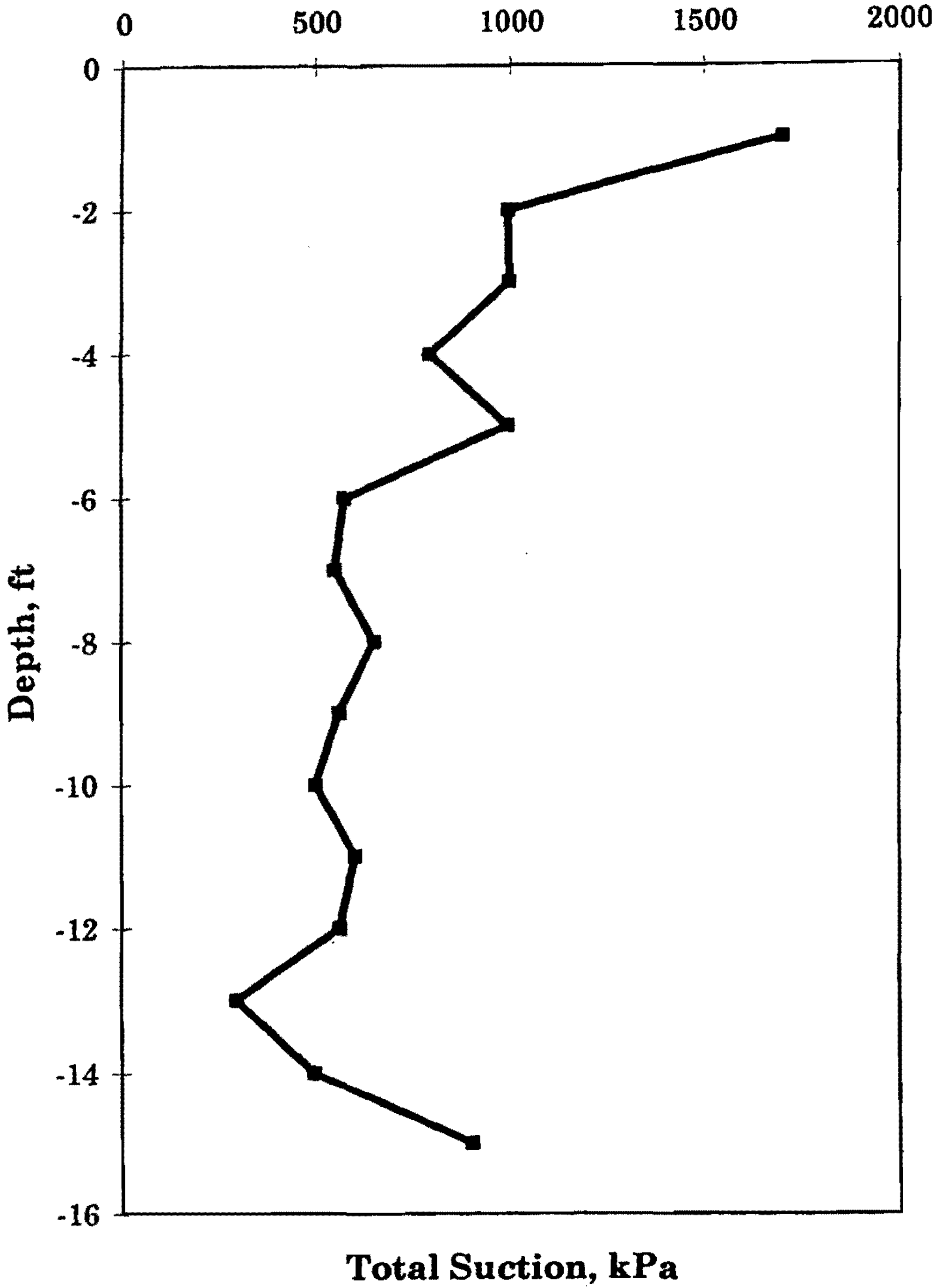


Figure 4.3: Soil Suction Results from LimeTest Section

4.3.3.2 Depth of Active Zone. The moisture content at each foot of depth was divided by the corresponding plasticity index (PI) and, plotting the resulting ratio against depth, the depth of the active zone was estimated. This active zone indicates the depth to which fluctuations in moisture content could be expected. Figure 4.4 shows that depth of the cement section's active zone is approximately 10 feet, the depth at which the plotted data forms an approximately vertical line. The active zone in expansive clays is the depth to which the moisture content changes greatly. Below this depth the moisture tends to remain constant. In expansive clays, this is significant for predicting the heave which can occur due to changes in moisture content (Coduto, 1994).

4.3.4 Design of the Cement Stabilized Subgrade

4.3.4.1 Determination of the Optimum Cement Content. Design of a soil-cement is typically more involved and requires more cement than a cement-modified soil. This project required only a modification of the soil with the sole
criterion being a reduction in the plasticity index. Accordingly, the design cement content was selected as 4 percent.

The average plasticity index is 18.3 for the near-surface material. Since the criterion for selecting a cement content was based only on a reduction in the plasticity index, different percentages of cement, by weight, were mixed with soil samples and the corresponding plasticity index determined (Figure 4.5). As cement content is increased, up to 4 percent, there is a significant reduction in plasticity; therefore, even if the cement content was doubled to 8 percent, only a nominal further reduction in plasticity would occur. Based on this information, cement content of four percent was selected for further testing.

4.3.4.2 Moisture-Density Relationships. Several of the tests required specimens be molded at maximum density and optimum moisture content. Moisture-density relationships were determined using ASTM Designation D558. This procedure allowed the use of a smaller mold and thus, required much less material than the TxDOT test method. Therefore, this method was beneficial because of the limited quantity of soil. While this method did not yield the same result as the TxDOT test method due to the difference in compactive effort, it was still adequate for purposes of this research.

Moisture-density curves were developed for both zero percent and four percent cement contents (Figures 4.6 and 4.7). The addition of cement caused a slight reduction in the maximum dry unit weight. This can be attributed to the flocculating reaction of the clay exposed to cement. Also, the optimum moisture content is slightly higher with the addition of cement. This can occur due to the fineness of the cement and subsequent increase in surface area.

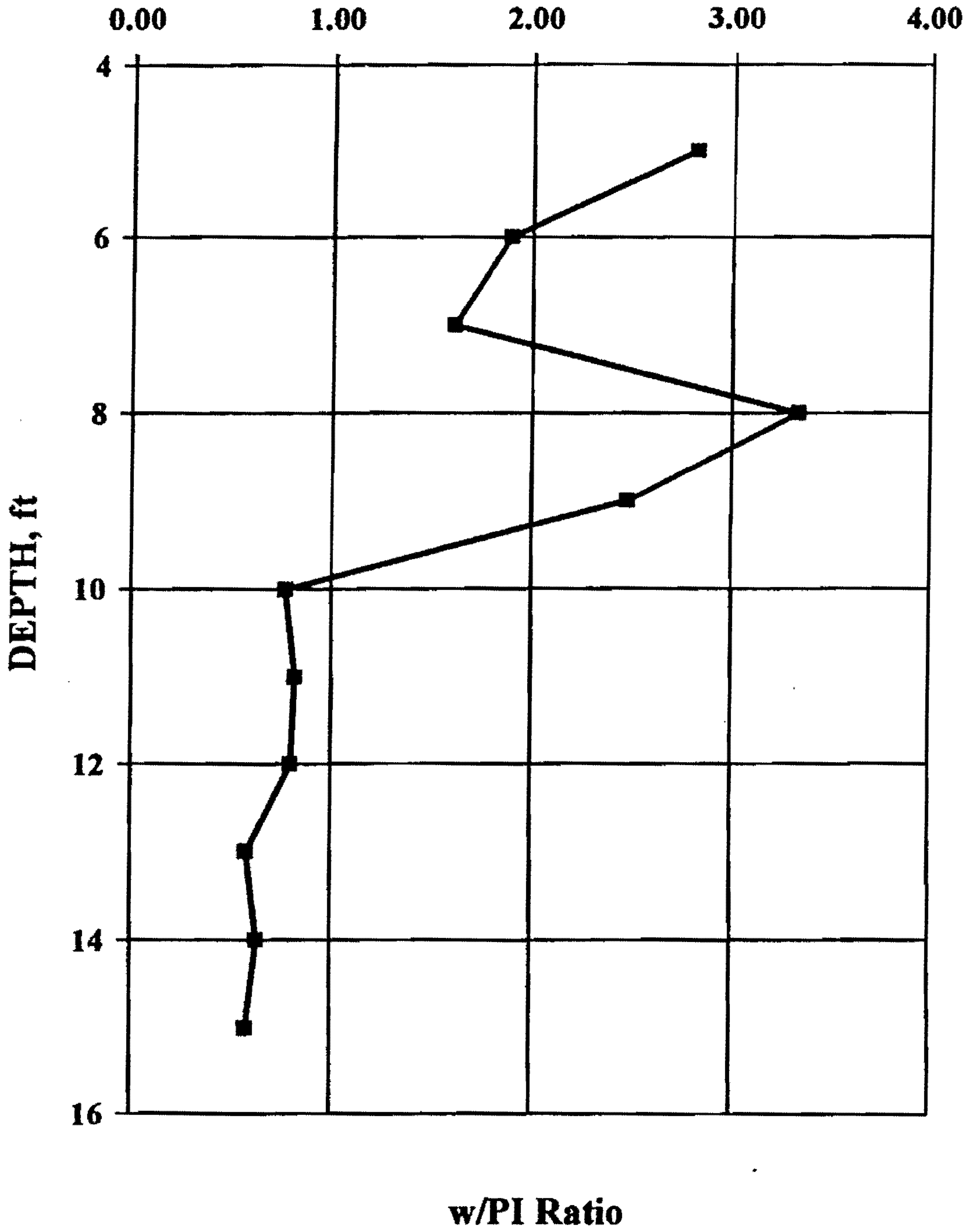


Figure 4.4: Determination of the Active Zone

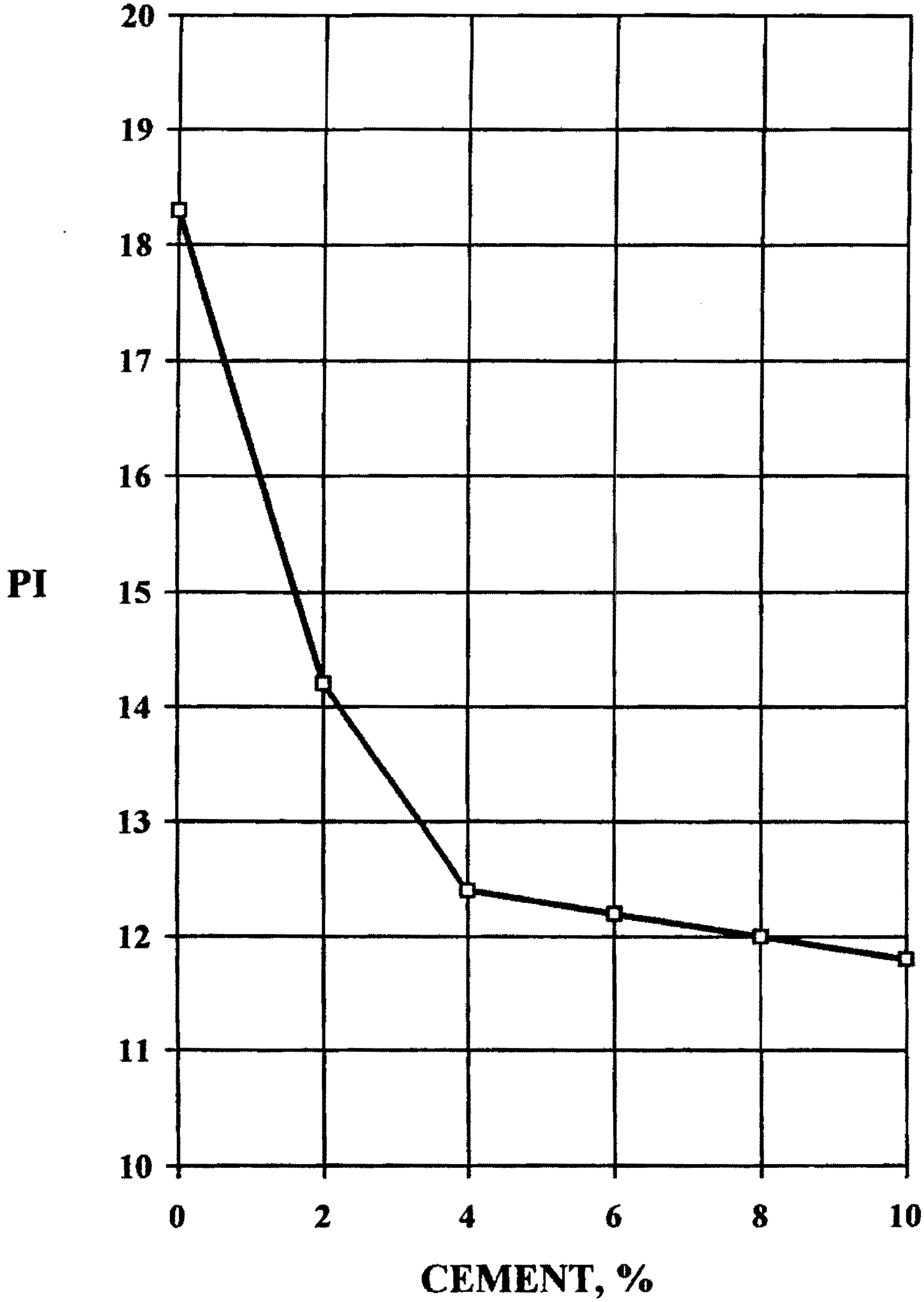


Figure 4.5: Reduction of Plasticity by the Addition of Cement

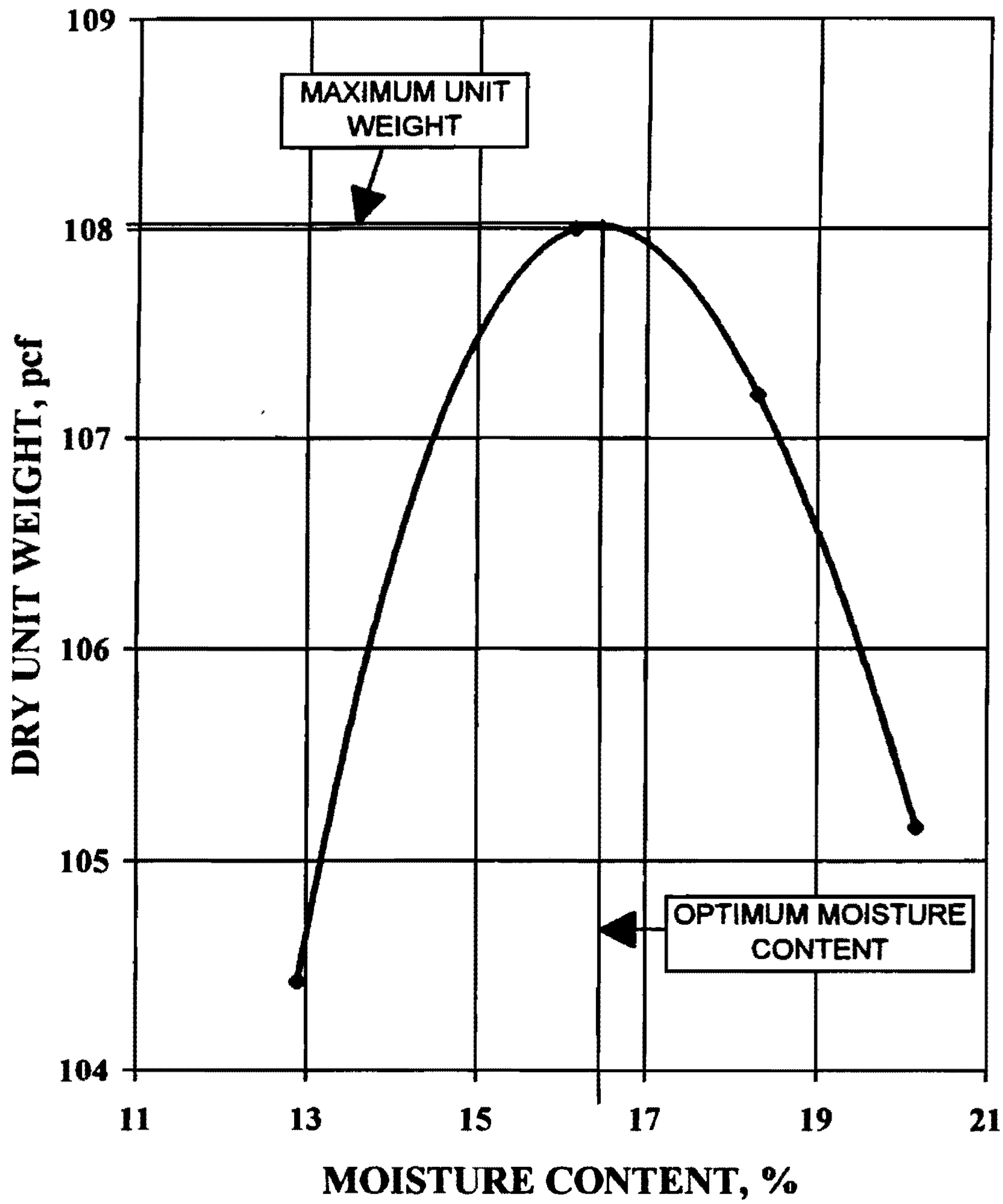


Figure 4.6: Moisture-Density Relationship of Soil with 0% Cement.

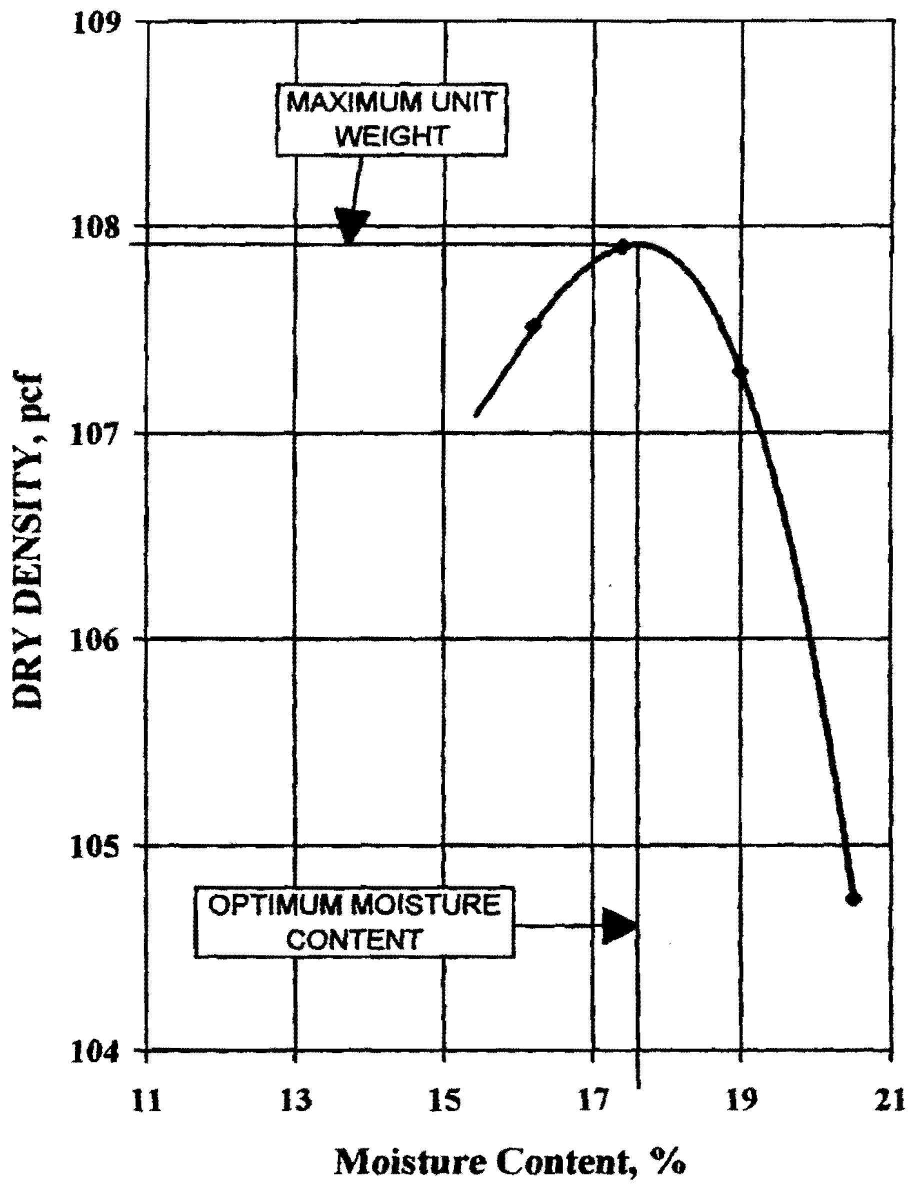


Figure 4.7: Moisture-Density Relationship of Soil with 4% Cement.

4.3.4.3 Unconfined Compressive Strength. Eight specimens were molded at the maximum dry unit weight and optimum moisture content for unconfined compressive strength testing. Three tests were conducted in accordance with ASTM Designation D559. Two of the specimens were molded with no cement added. One was cured for 24 hours and the other for 28 days. The longer curing period was not expected to result in a significant increase in the strength because there would be no hydration occurring. This assumption was verified with the 24 hour curing specimen yielding a maximum strength of 65 psi and the 28-day curing specimen producing nearly the same maximum strength of 63 psi.

The other six specimens were molded with four percent cement content. Two specimens were tested after curing periods of 24 hours, 7 days, and 28 days. The average unconfined compressive strength for each pair of specimens was 129 psi, 219 psi, and 393 psi for the curing periods of 24 hours, 7 days, and 28 days, respectively. It can be seen that a significant increase in strength resulted in an increase in curing time after the addition of cement. All were significantly higher than the untreated soil. Results for each pair of specimens are shown in Figures 4.8 - 4.11. Compressive strengths of the specimens containing both zero and four percent cement content are illustrated in Figure 4.12.

4.3.4.4. Durability. Using ASTM Designation D559 (Wet-Dry), two durability tests were performed. One test was performed on the natural soil and the other using the soil mixed with four percent cement. For the first test, two molded specimens were compacted to maximum density with four percent cement. Specimen "A" was optional, but was made to determine volume changes after 12 wet-dry cycles. Specimen "B" was also subjected to 12 wet-dry cycles, but was brushed with a wire brush after each drying. Soil-cement losses of specimen "B" were recorded. Both specimens endured the entire twelve cycles, though specimen "A" suffered a loss in volume of 8.4 percent after twelve cycles, while specimen "B", after the wire brushing, suffered a dry, corrected, weight loss of 15.7 percent after twelve cycles. Figure 4.13 shows the uncorrected weight loss for specimen "B" after each cycle. A rise in the weight of the specimen at cycle 8 may be attributed to the malfunction of the oven's thermostat. Apparently the specimen did not get completely dried. How much this variation in temperature affected the overall weight loss of the specimen is not known. However, the final data point at cycle 12 fell in line with the data points in the earlier cycles, suggesting the overall weight loss was not greatly affected.

For the second test, only one, untreated specimen was molded. It disintegrated shortly after its first submersion in water. While the soil treated with four percent cement may not meet the requirements of a soil-cement, it did represent a significant improvement over the untreated soil. The two specimens treated with four percent cement were also subjected to an unconfined compression test after the twelve cycles were completed. Specimen "A" failed at 318 psi and specimen "B" failed at 145 psi. (Figure 4.14). Even after twelve wet-dry cycles both specimens exceeded the unconfined compressive strength of the untreated soil, which was only 65 psi.

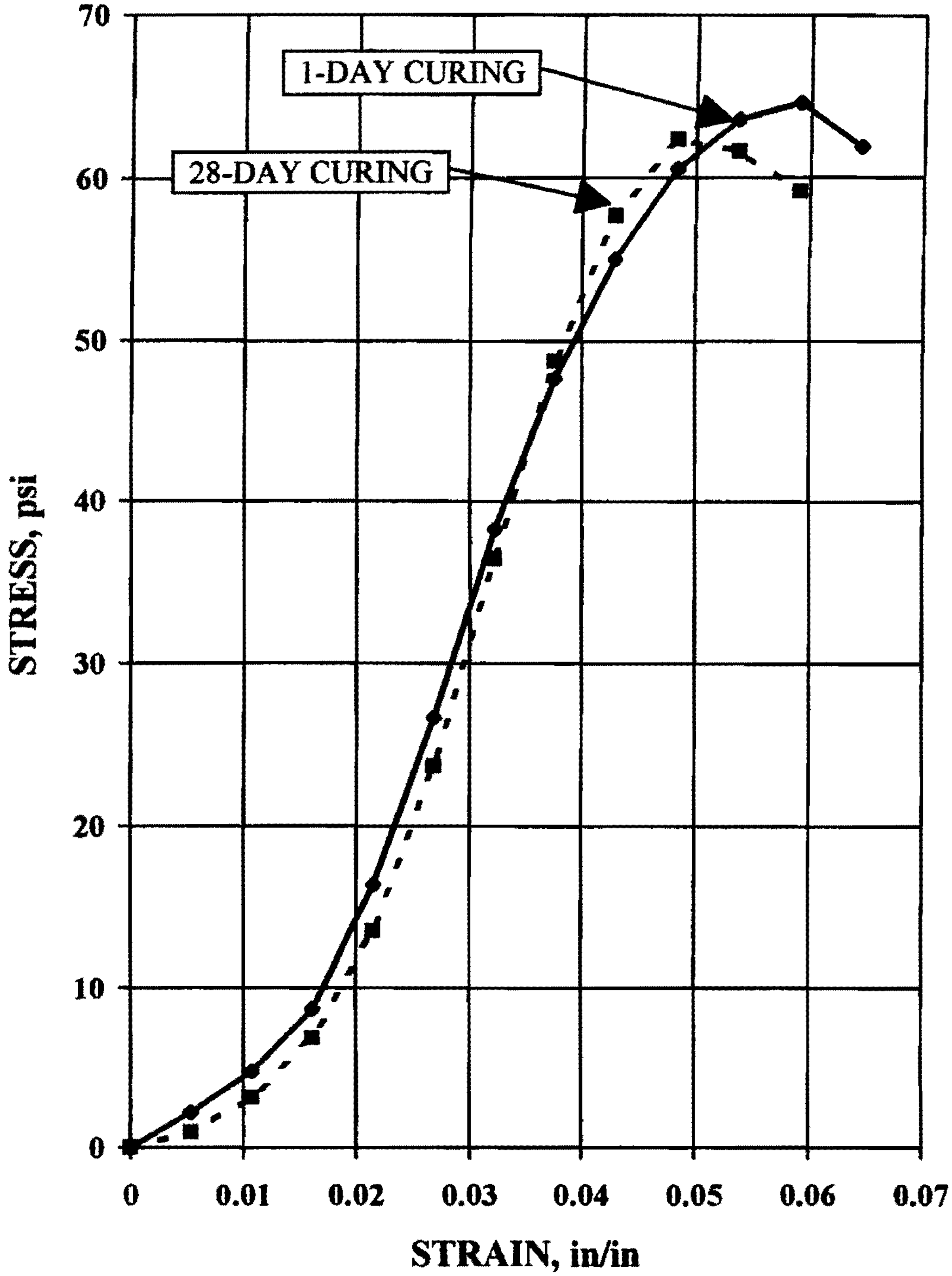


Figure 4.8: Unconfined Compressive Strengths with 0% Cement
After 1 Day and 28 Days of Curing.

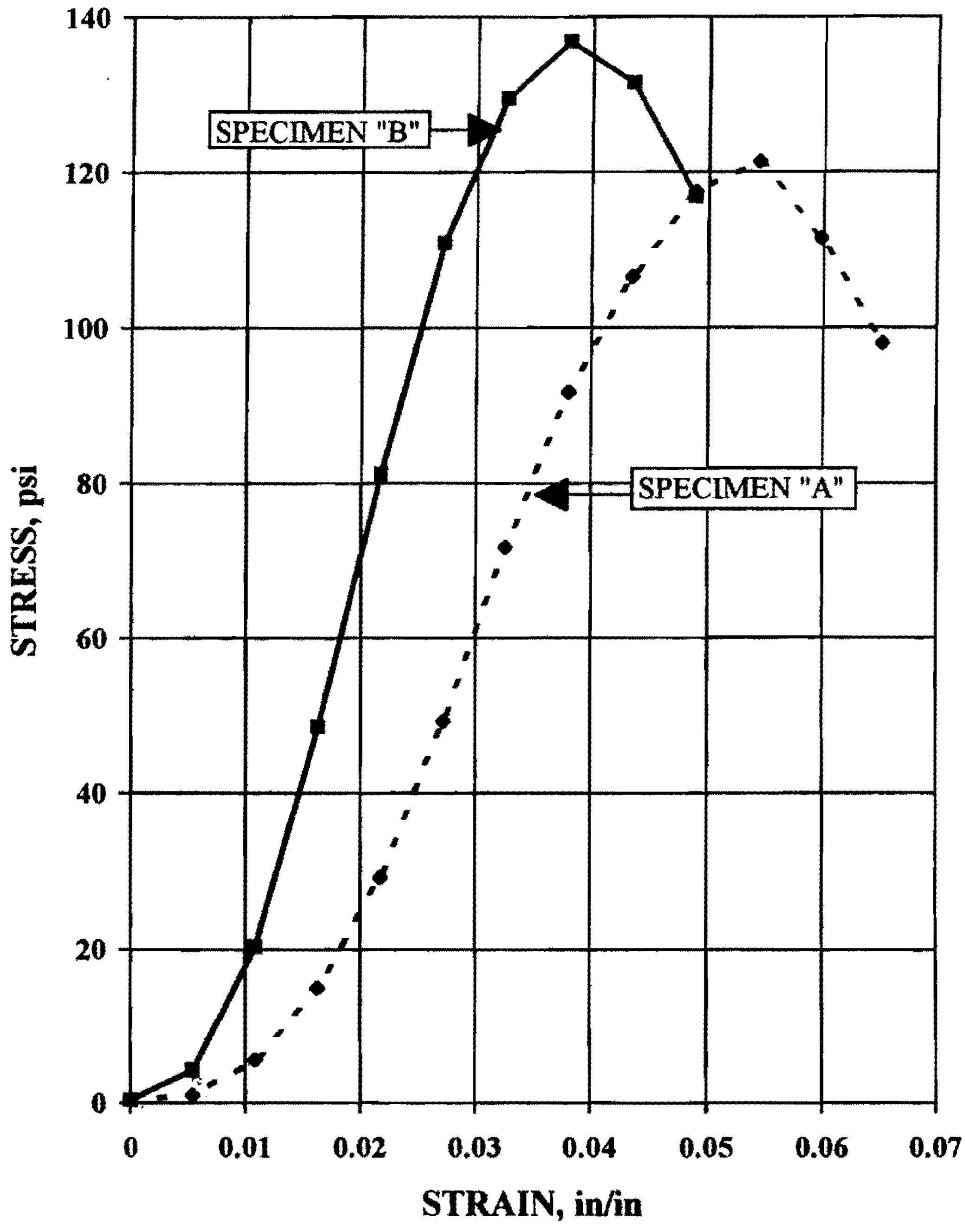


Figure 4.9: Unconfined Compressive Strengths with 4% Cement After 1 Day of Curing.

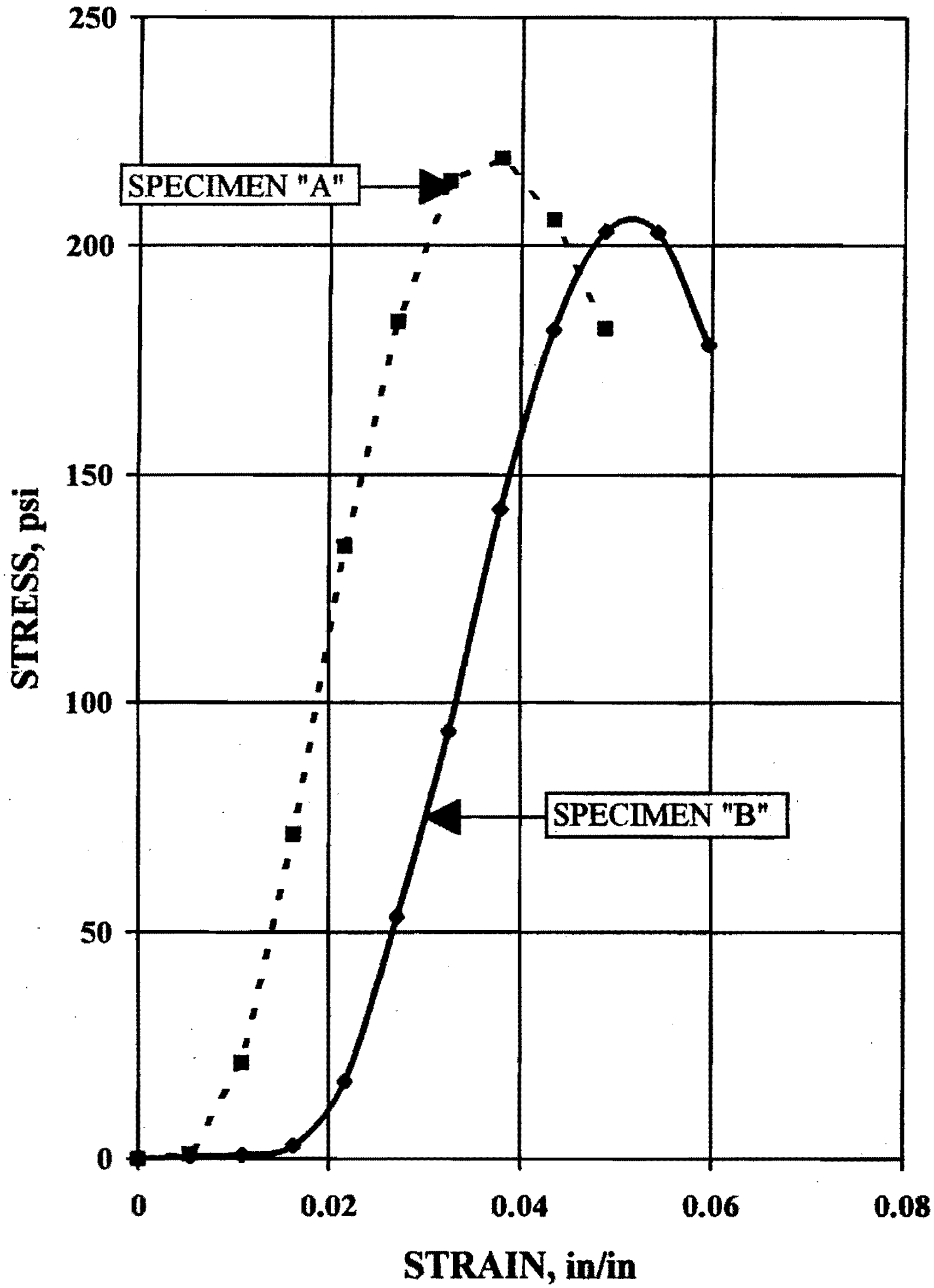


Figure 4:10: Unconfined Compressive Strength with 4% Cement after 7 Days of Curing

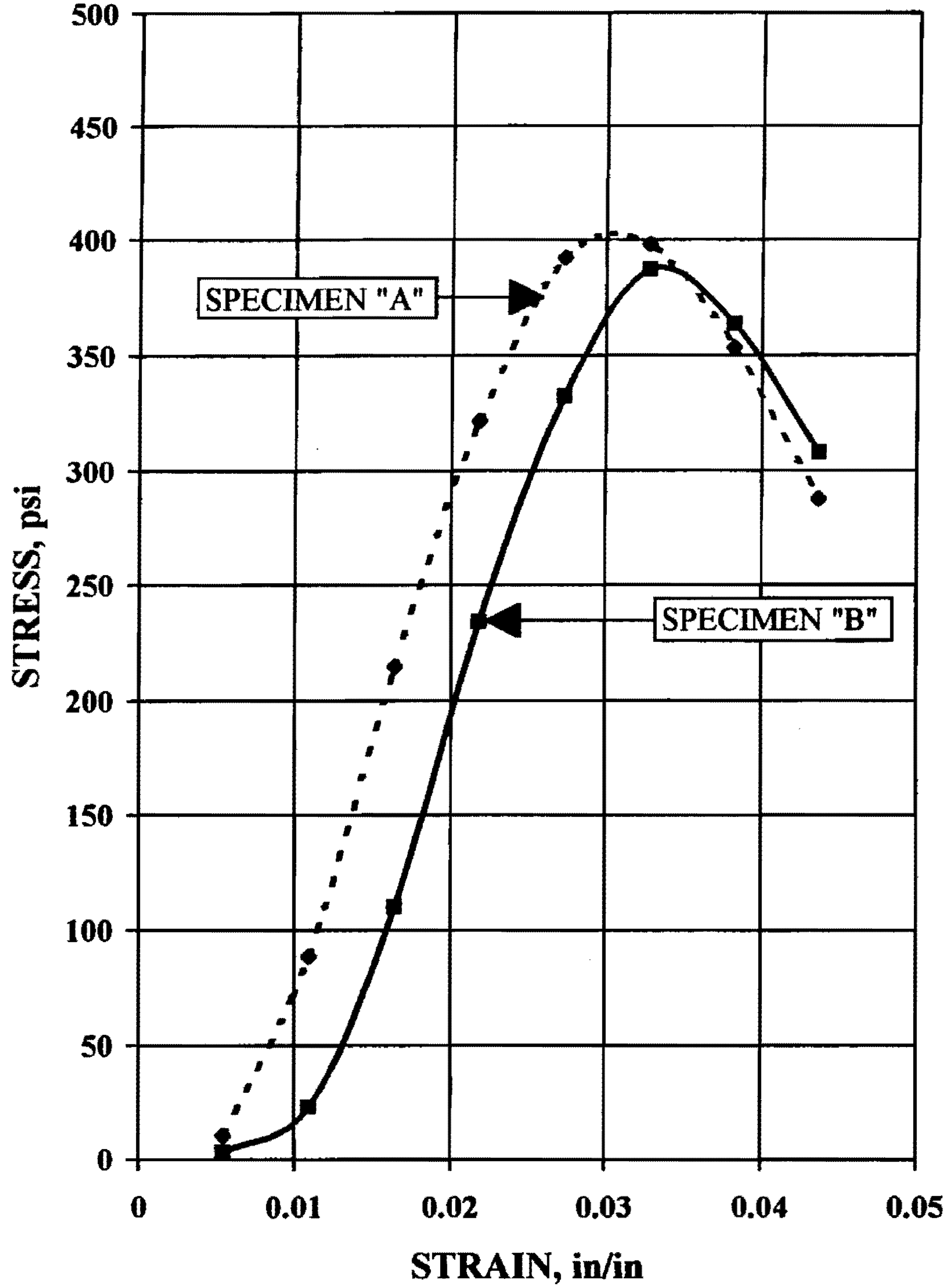


Figure 4.11: Unconfined Compressive Strength with 4% Cement after 28 Days of Curing

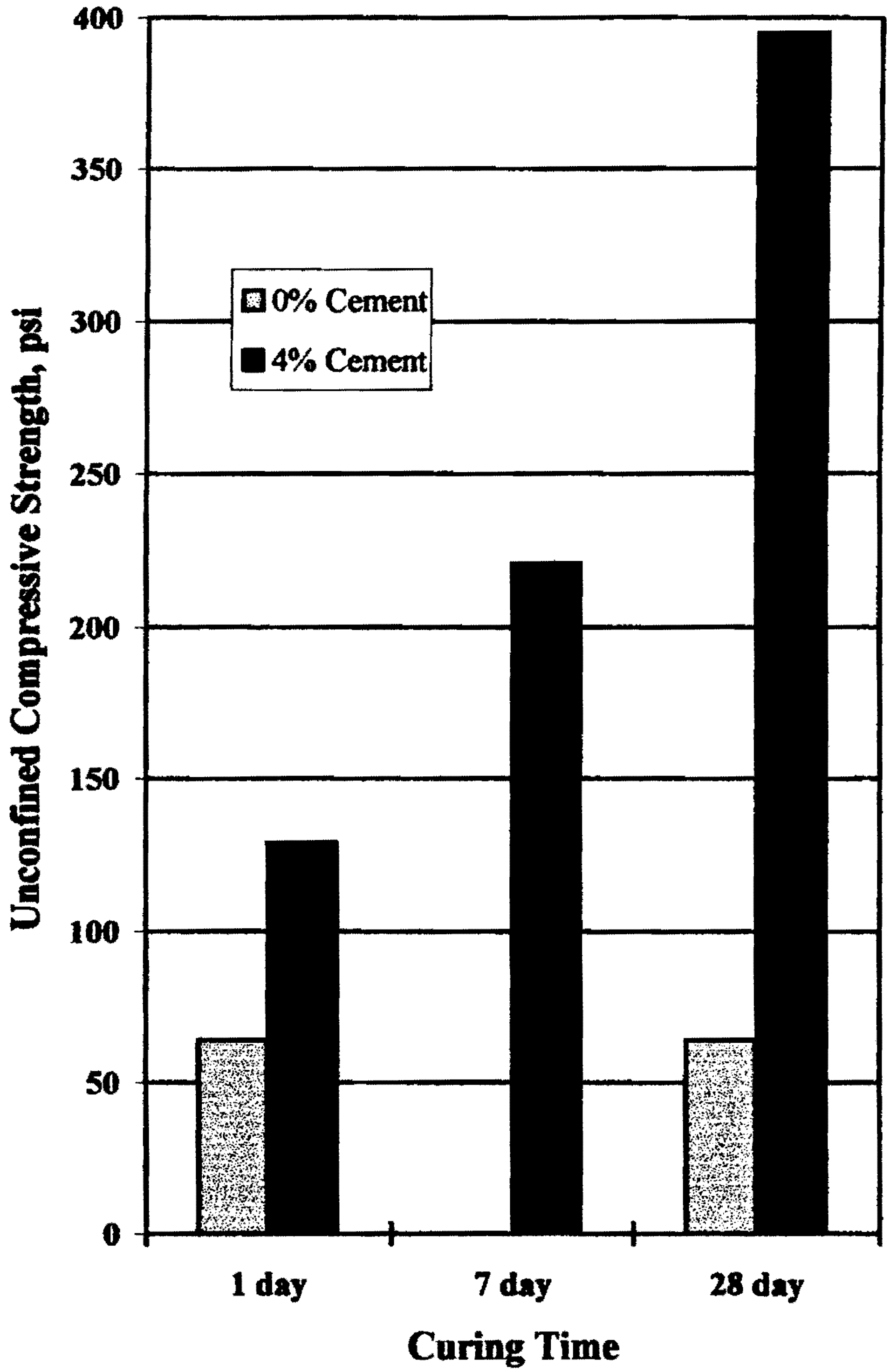


Figure 4.12: Average Unconfined Compressive Strengths of Specimens at the Three Curing Periods

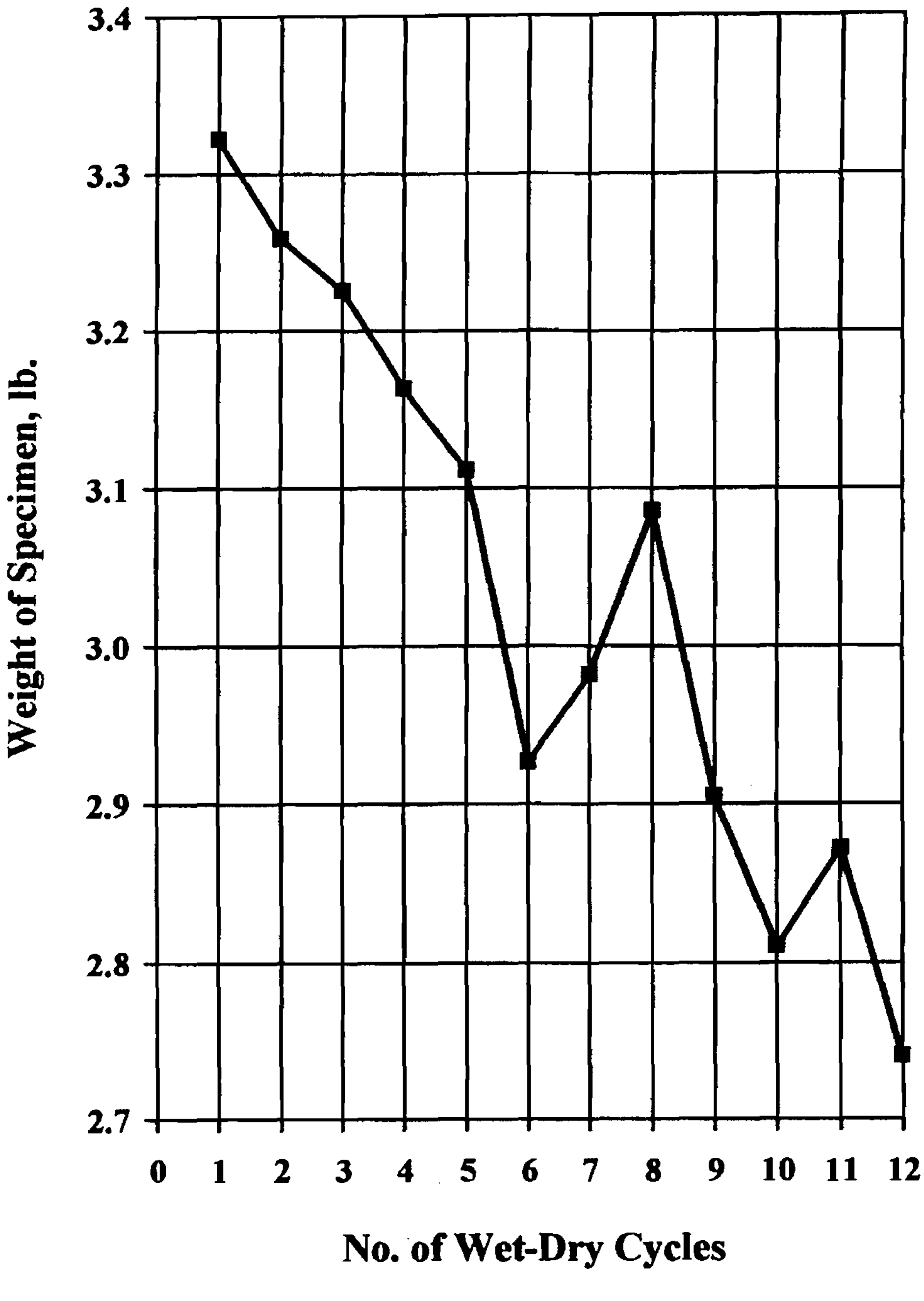


Figure 4.13: Uncorrected Dry Weight Loss During Wet-Dry Durability Testing with 4% Cement, Specimen B

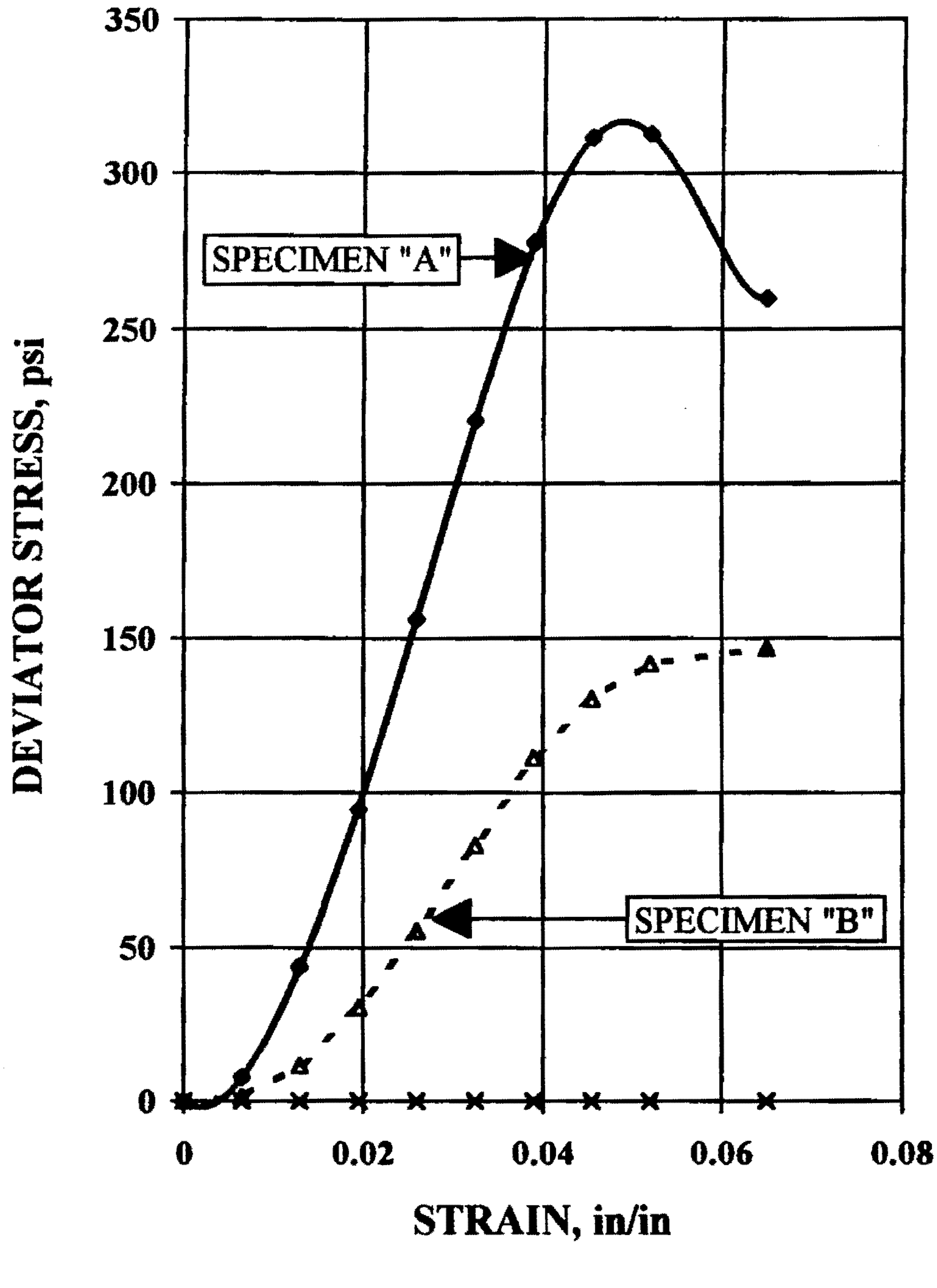


Figure 4.14: Unconfined Compressive Strengths after Durability Testing on Specimens with 4% Cement Content.

4.3.4.5 Permeability. Determination of permeability of the soil with four percent cement was first attempted using the falling-head test method. After 30 days, the specimen was removed from the mold. The specimen was 4.5 inches in height, and the water had barely penetrated the surface. Because of the slow progress of water intrusion, the falling head permeability test was discontinued in favor of employing a more indirect, but faster, way of determining the permeability. This test was performed in accordance with ASTM Designation D2435. The specific gravity of the soil solids was needed for the consolidation test calculations and it was determined using TxDOT test method
Tex 108-E.

Determining permeability from the consolidation test is a rather involved process and results can be somewhat variable, depending upon the method of determining the required parameters. The required parameters are the coefficient of consolidation (c_v), the coefficient of compressibility (a_v), the compression index (C_c), and the initial void ratio (e_o). These parameters may be determined from the test data. The permeability of the untreated soil was determined to be 2.85×10^{-7} in./min. After addition of 4 percent cement, the average permeability increased slightly to 8.25×10^{-7} in./min., due to flocculation of the clay. Table 4.5 shows the results from the additional permeability tests performed.

4.3.5 Design of the Lime Stabilized Subgrade

4.3.5.1 Determination of the Optimum Lime Content. The type of lime used in the design was a high calcium, hydrated lime produced by the APG Lime Corporation in New Braunfels, Texas. Two procedures were performed to determine the optimum lime content to be used in construction. The plans did not call for the subgrade to be treated as a structural layer; therefore, the unconfined compressive strength was not employed in determining the design lime content. The Eades and Grim test for pH provided the initial lime content to be used. Table 4.6, suggests that four percent lime by weight is the optimum lime content.

The other procedure used to determine the optimum lime content was to add increasing amounts of lime to the soil and find the PI of each soil lime mixture. Figure 4.15 shows the reduction in PI with the increasing amounts of lime. Four percent lime by weight yields the lowest PI with the least amount of lime. According to TxDOT, after stabilization the PI of the subgrade must be below 15; this is accomplished by adding four percent lime.

4.3.5.2 Moisture Density Soil samples were compacted according to ASTM D558 to determine the optimum moisture content as well as the optimum dry density. The size of the mold used to compact the samples was 4 inches in diameter and 4.5 inches in height. As predicted by the technical literature, the optimum moisture content was lower for zero percent lime than that of the soil mixed with four percent lime. The dry density for the zero percent lime was higher than the four percent lime, Figure 4.16. The optimums were used in remolding the material to perform the unconfined compressive strength, durability and consolidation tests.

Table 4.5: Summary of Permeability Data

		Permeability, in^2/min (x 10^{-7})	
Specimen	Cement Content, %	Equivalent Load Applied, tsf	
		4	8
A	0	9.16	3.10
A	4	8.25	5.18
B	4	0.17	3.81

Table 4.6: Percentage of Lime Versus pH to Determine Optimum Lime Content

pH	%Lime
8.435	0
12.115	2
12.164	3
12.208	4
12.200	5
12.221	6

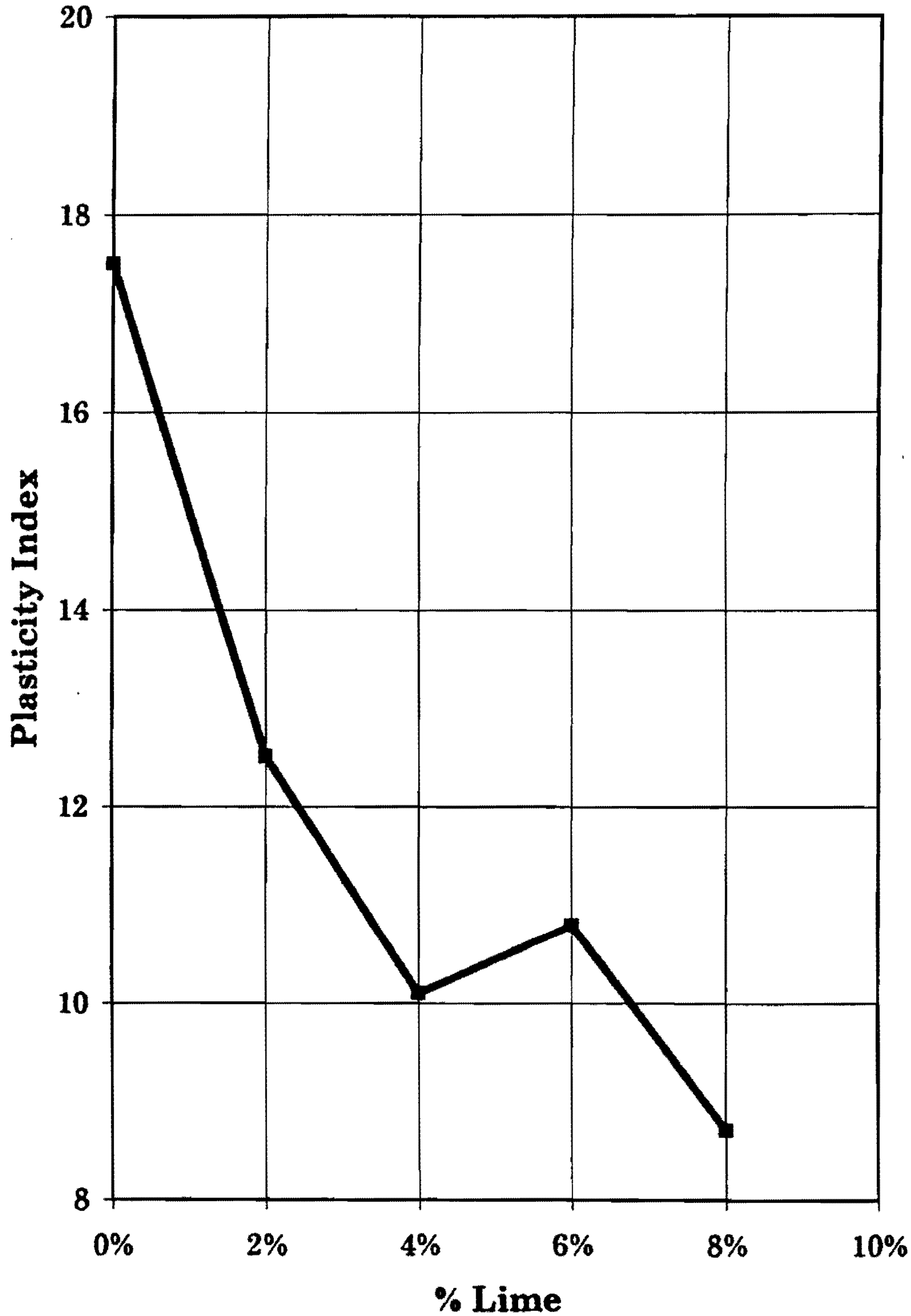


Figure 4.15: PI Versus Lime Percentages to Verify the Optimum Lime Content at 4%.

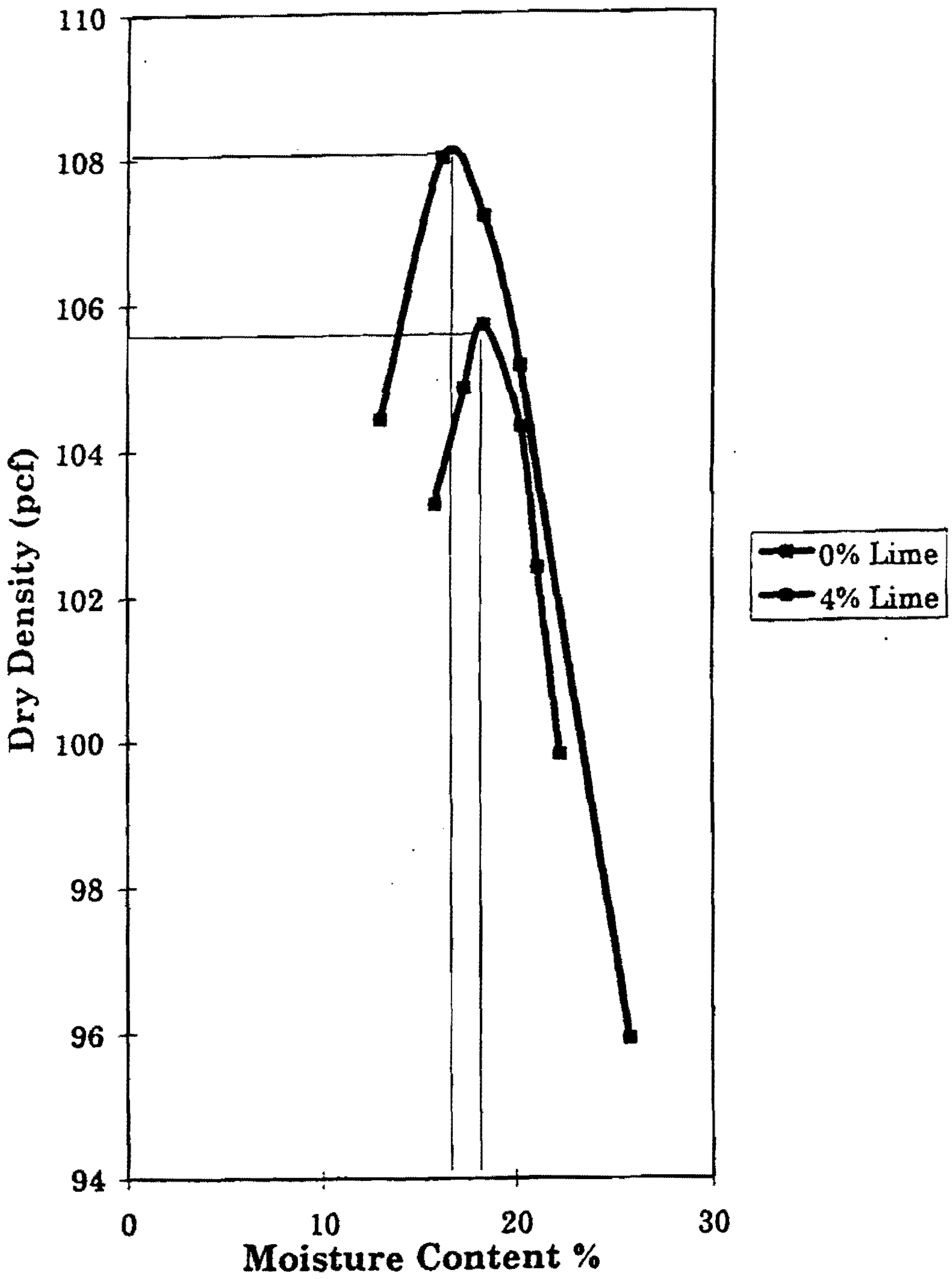


Figure 4.16: Moisture-Density Relationships

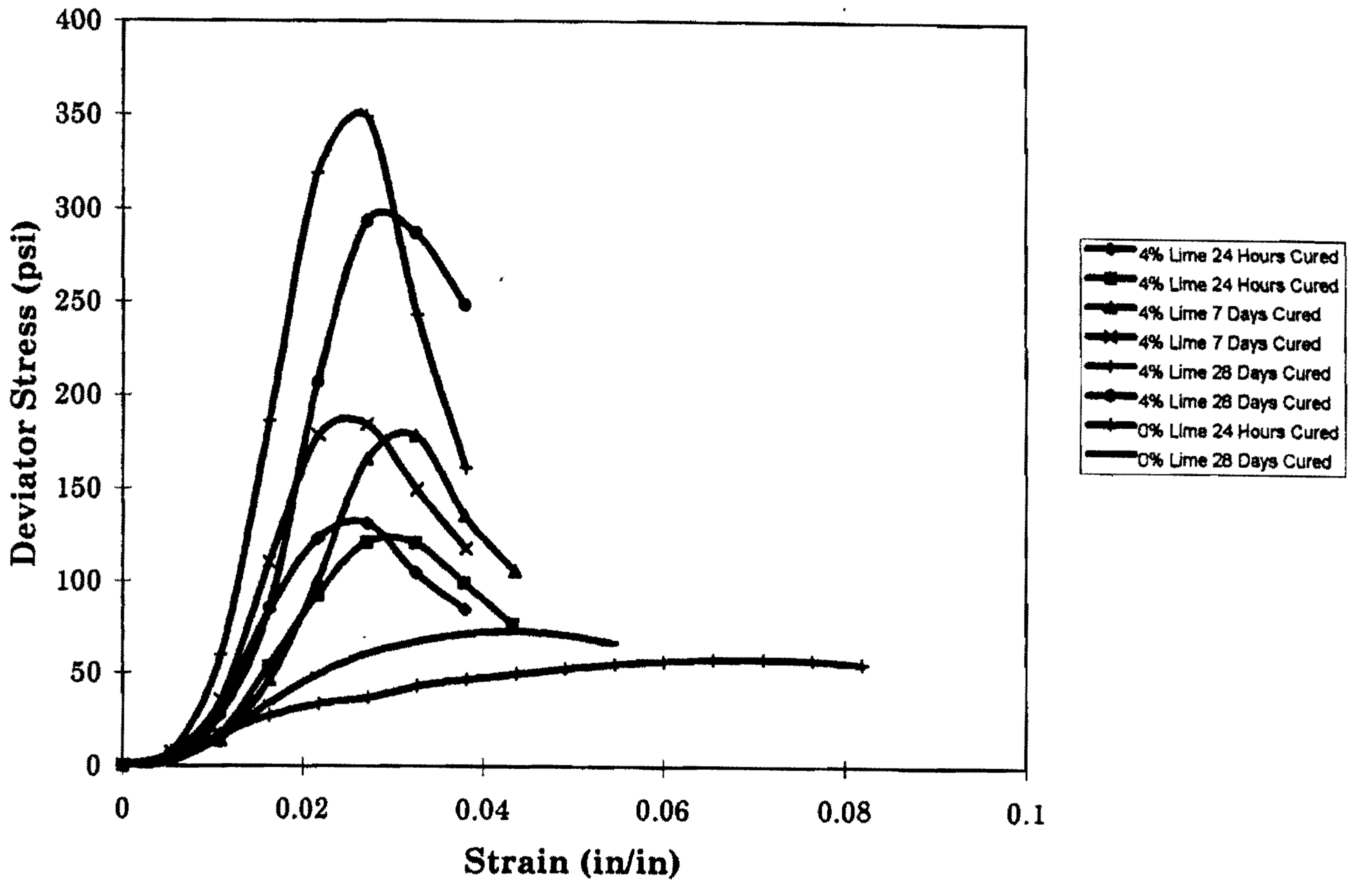


Figure 4.17: Deviator Stresses of the Natural Soil and Soils Treated with 4% Lime and Increased Curing Periods

4.3.5.3 Unconfined Compressive Strength. Unconfined compression tests were performed on remolded soil samples at optimum moisture content and dry density. The test method followed ASTM D5102 Procedure. Samples were molded with four percent lime and zero percent lime to observe the increase in strength. The dimensions of the mold were 4 inches in diameter and 4.5 inches in height. The recommended ratio of height to diameter is 2:1; however, the aforementioned dimensions are acceptable according to Procedure B and will yield higher strength values.

The molded samples were cured in a humid room for 24 hours, 7 days, and 28 days and then tested for compressive strength. Figure 4.17 shows the deviator stress versus strain for each mold tested. The maximum strength for each mold is equal to the maximum deviator stress. The maximum strength for 28 days curing is considerably higher for the four percent lime. In Texas, for a subgrade to be considered as a structural layer, the increase in strength must be at least 50 pounds per square inch (psi). The proposal did not specify that the subgrade meet this standard; however, this test indicates that the subgrade does meet the standard.

4.3.5.4 Consolidation Test. Consolidation tests were performed, in accordance with ASTM D2435 to determine the permeability of the natural soil and lime-treated soil. The soil-lime mixture and natural soil were compacted in a mold 4 inches in diameter and 4.5 inches in height. One mold was made for each the soil-lime mixture and natural soil. Two samples were taken from each mold, 2.5 inches in diameter and .79 inches in height and then placed in a water bath until fully saturated.

Once saturated, samples were placed in the consolidation apparatus and pressures of 4 tons per square foot (tsf) and 8 tsf were applied. For the purpose of calculating the permeability, only one or two loads were needed. Petry (1991) reports that with the addition of lime the permeability may increase by as much as 7 to 300 times that of the natural soil. The permeability for the natural soil is 6.826×10^{-9} cm/sec and the lime treated soil with four percent lime yielded a permeability of 4.942×10^{-9} cm/sec.

4.3.5.5 Durability. Durability of soil-lime mixtures consists either of wet-dry or freeze-thaw cyclic conditions. According to AASHTO, Castroville, Texas is in a non-freezing zone; therefore, it was necessary to only be concerned with wet-dry conditions. The wet-dry test followed ASTM D559. Two tests were attempted; however, after the fifth cycle in the second test, the specimens broke apart, thus, eliminating the validity of the second test. The volume change for specimen No.1 was 8.6% and the soil cement loss for specimen No.2 was 16.5%. Unconfined compression tests were also conducted with the specimens following the durability test. Figure strength after 4.18, depicts the stress-strain curves for the specimens after 12 cycles of wet-dry conditions. The 28 days was then compared to the strength of the natural soil after 28 days.

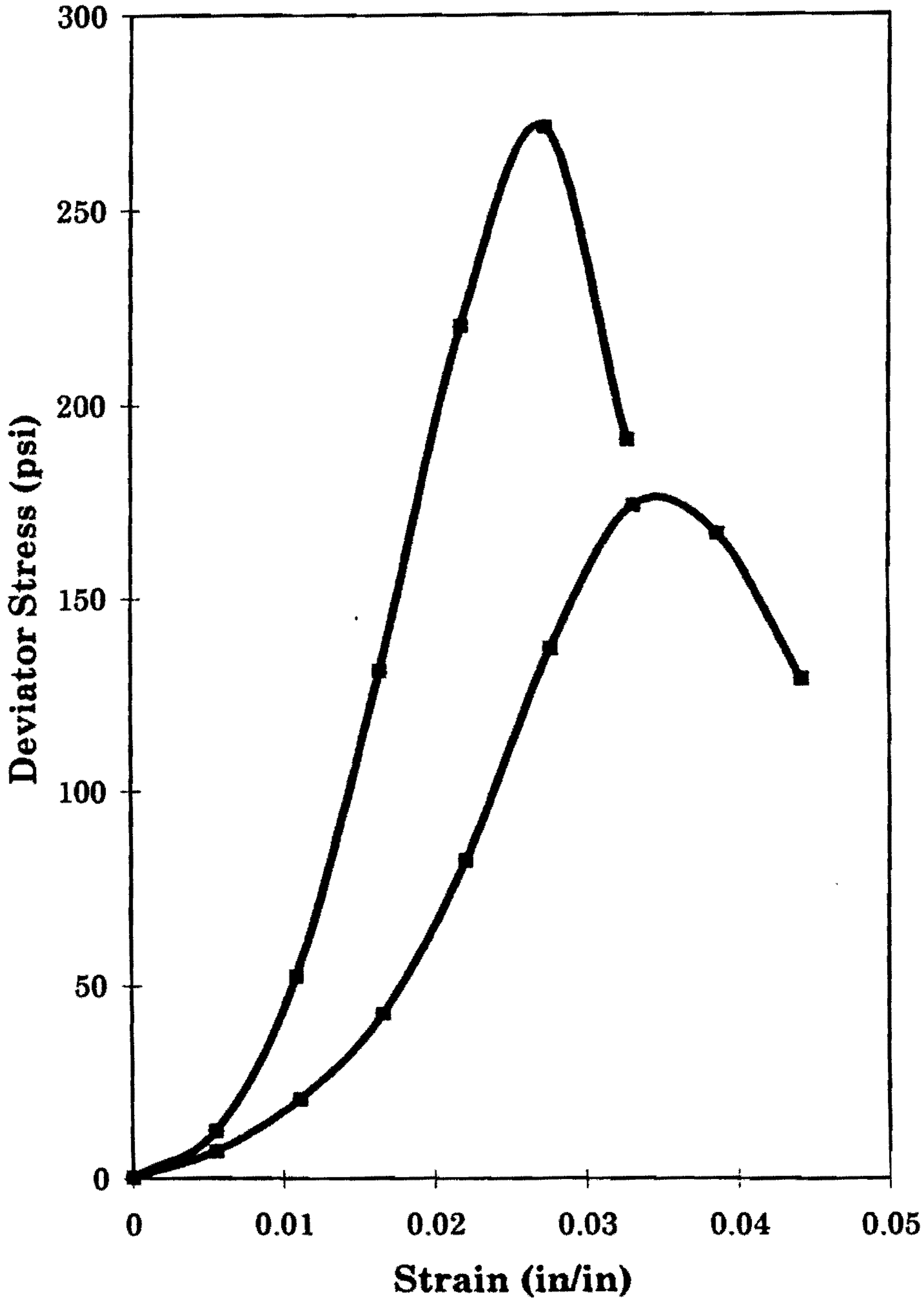


Figure 4.18: Deviator Stress Versus Strain after 12 Cycles of Wet-Dry Conditions

V. CONSTRUCTION OF THE TEST PAVEMENT SECTION

5.1 INTRODUCTION

This chapter will briefly discuss the procedures used in the construction of the test pavement sections for the lime and cement modified soils.

There are several primary requirements for field construction of a stabilized roadway subgrade that will help ensure a high quality product. The construction procedures are generally the same whether the subgrade or base is being stabilized or modified. In general, good construction practices are as follows: scarification and pulverization, soil-additive application, preliminary mixing, preliminary curing, final mixing, final compaction, and final curing.

The completed roadway structure at the cement stabilized test site consists of a stabilized subgrade, flexible base, and surface treatment. The new roadway is wider than the previously existing roadway, and paved shoulders have been added.

The subgrade (or sub- base) is a low plasticity clay stabilized to a depth of six inches. This depth was specified in the TxDOT plans and was determined empirically. The overlying base is to be 10 inches with a two course surface treatment applied to the base, Figure 5. 1. A cross sectional view of existing road is shown in Figure 5.2.

The flexible base, placed on top of the treated subgrade, is a Type A, Grade 6, (Texas Department of Transportation, 1995a). The "Type A" indicates that a crushed stone must be used and prohibits the use of any gravel (Texas Department of Transportation, 1995 b). "Grade 6" indicates that the gradation has to meet the requirements, which are shown in the plans. This requires that no aggregate should be larger than 1-3/4 inches, 45 to 70 percent should be retained on the No. 4 sieve, and 70 to 80 percent should be retained on the No. 40 sieve. The flexible base has a thickness of 10 inches, with a maximum dry density requirement of at least 98 percent of the laboratory maximum dry density. The width of the base is 31 feet.

The surface of the roadway is a two-course surface treatment, with each course consisting of a distribution of asphalt immediately followed by a thin layer of aggregate. The first course consists of a PE-3 aggregate, while the second course consists of a PE-4(MOD) aggregate. The "PE" indicates that the aggregates are to be pre-coated as shown on the plans.

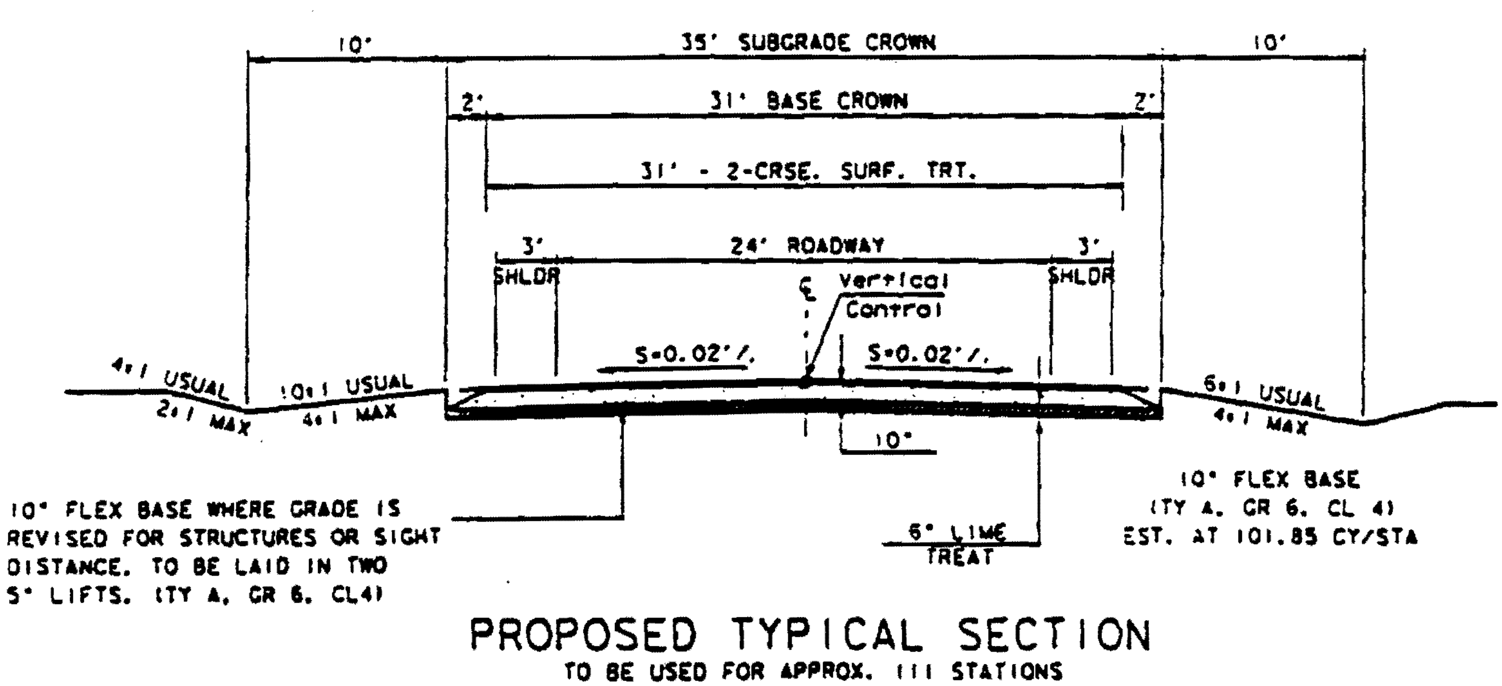


Figure 5.1: Proposed Pavement Cross Section

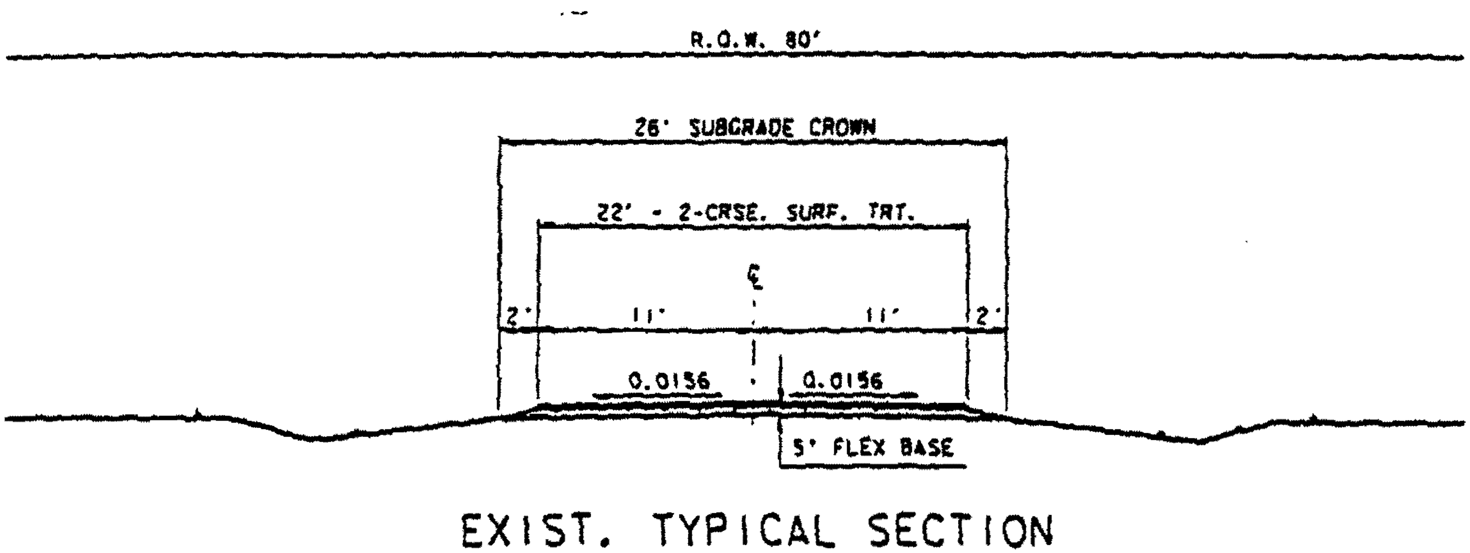


Figure 5.2: Existing Pavement Cross Section

The "3" indicates the gradation, with the "4(MOD)" indicating a smaller gradation than the "3," but has been modified from the standard specifications. The asphalt applied for the first course was an AC-10 with two percent latex.

At the time of completion of the research project, the second course had not been applied. It is intended that the same type of asphalt will be used if the weather is warm. For cold weather application, a HFRS 2-P will be used. If funds are available, a hot mix overlay may be placed in lieu of the second course. It not known which of the three possibilities will eventually be used.

5.2 CONSTRUCTION PROCEDURES: LIME STABILIZED PAVEMENT

The following steps outline and explain the construction of the lime section of the project. The lime used in the construction process was a type C quicklime manufactured by the Redland Lime Corporation.

Step 1: A belly dump truck delivered the lime to the construction site. A blade was then used to spread the lime evenly over the subgrade (Figure 5.3). The amount of lime used was 3%.

Step 2: A scarifier (Figure 5.4) tilled the subgrade and lime to a depth of six inches to prepare it for the watering and mixing.

Step 3: A water truck passed over the scarified lime and subgrade delivering enough water for the quicklime, water and soil to react (Figure 5.5). Preliminary mixing was accomplished using a rotary mixer (Figure 5.6). The blade was used after the rotary mixer to further mix the material then water was added. The mixing and adding of water was repeated several times.

Step 4: A pneumatic roller (Figure 5.7) achieved Compaction. A final compactive effort was accomplished with a vibrating steel roller (Figure 5.8).

Step 5: After a light compaction, the soil-lime mixture was allowed to cure for 7 days.

Step 6: The final mixing and compaction utilized the same equipment as in the preliminary stage. After the final mixing, all aggregates should pass the 1 inch-screen.

Step 7: Once the mixing was completed the mixture was compacted , the optimum moisture content was 10.8% and the maximum dry density was 115.2 pounds per cubic foot (pcf).

Step 8: The subgrade was allowed to cure for a period of 7 days. Curing can mean one of two things: (1) a moist curing where water is lightly added and then a light compaction or (2) a membrane curing where the subgrade is sealed with an overlying bituminous layer. The lime section was cured according to the first option (1).

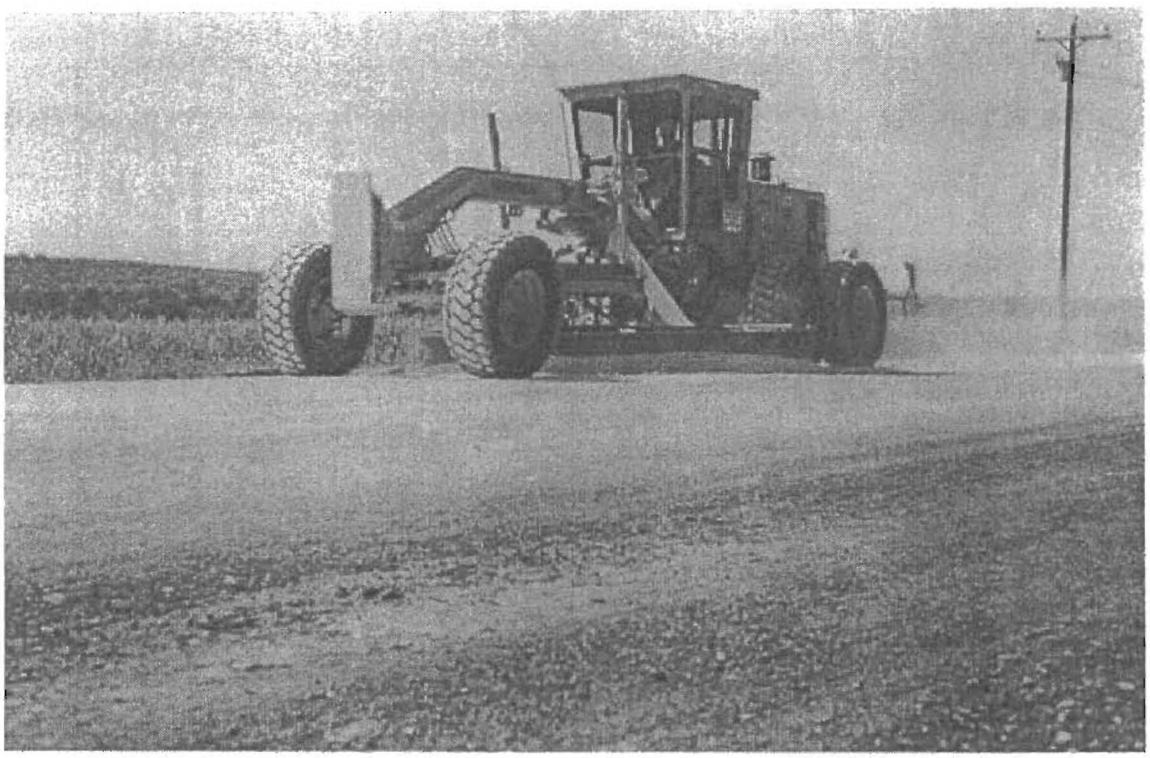


Figure 5.3: Grader Being Used to Distribute Lime Evenly

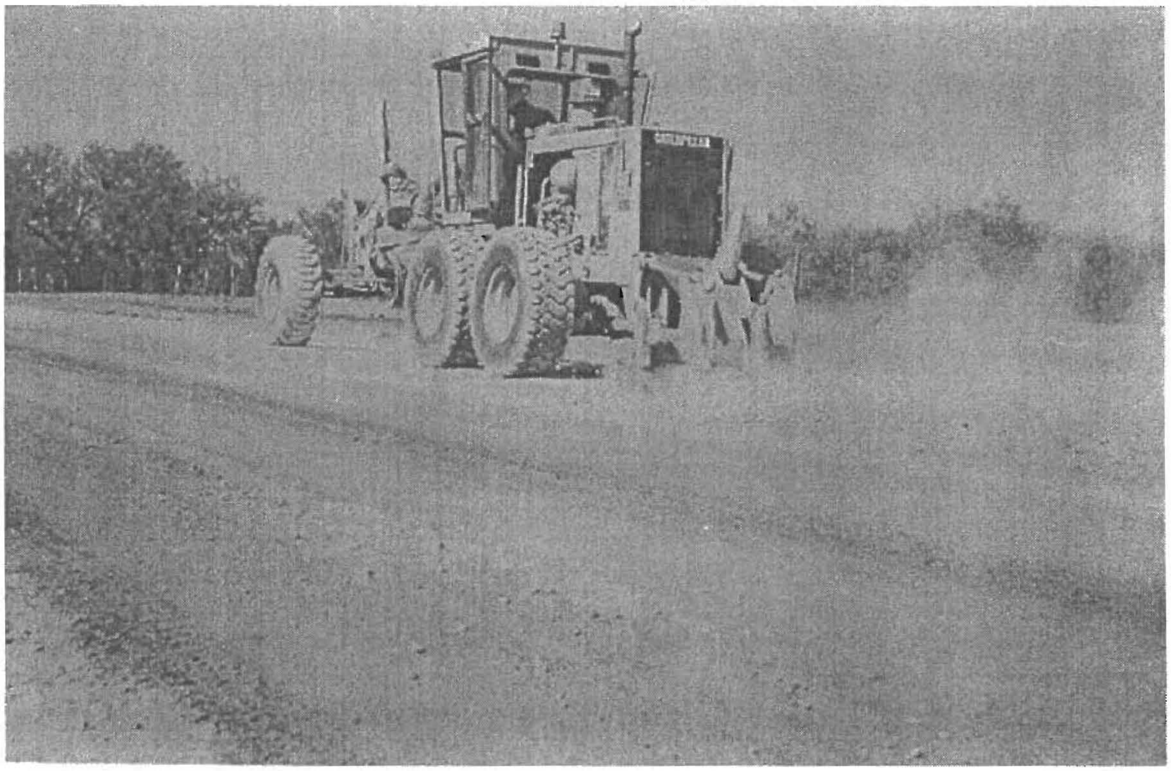


Figure 5.4: Scarification of Lime and Subgrade.

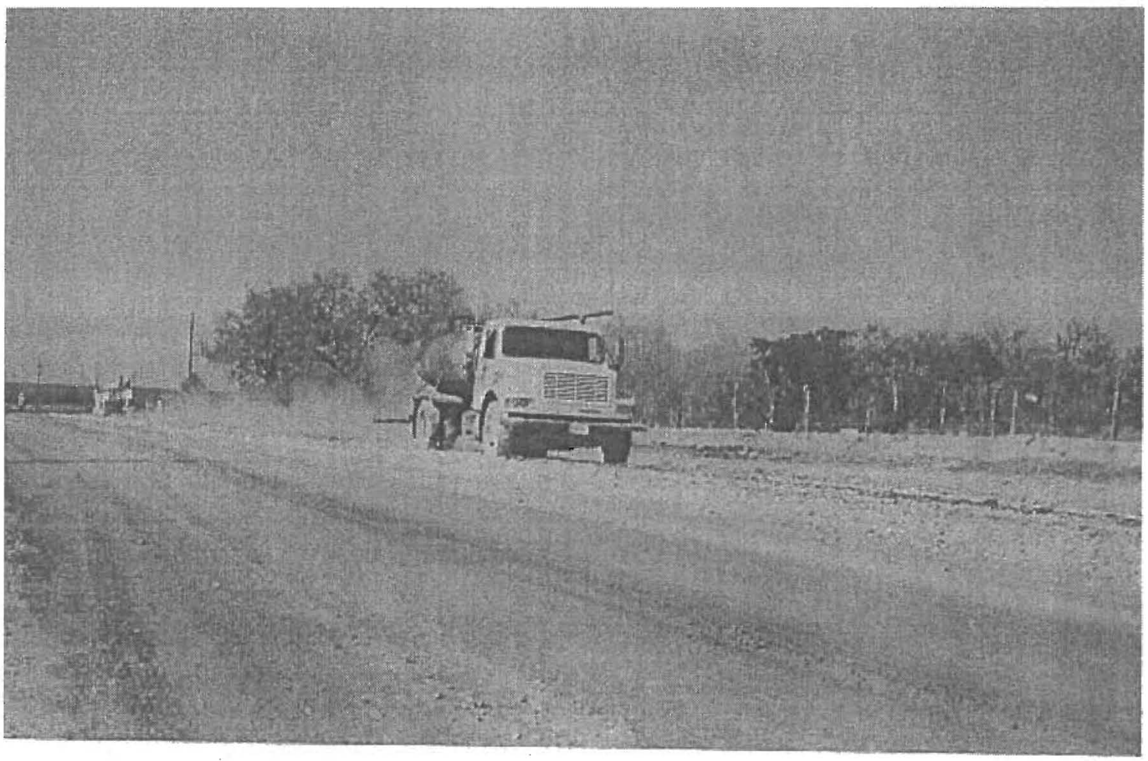


Figure 5.5: Addition of Water using a Water Truck.

Figure 5.6: Use of the Rotary Mixer.

Figure 5.7: Compaction Using Pneumatic Roller.

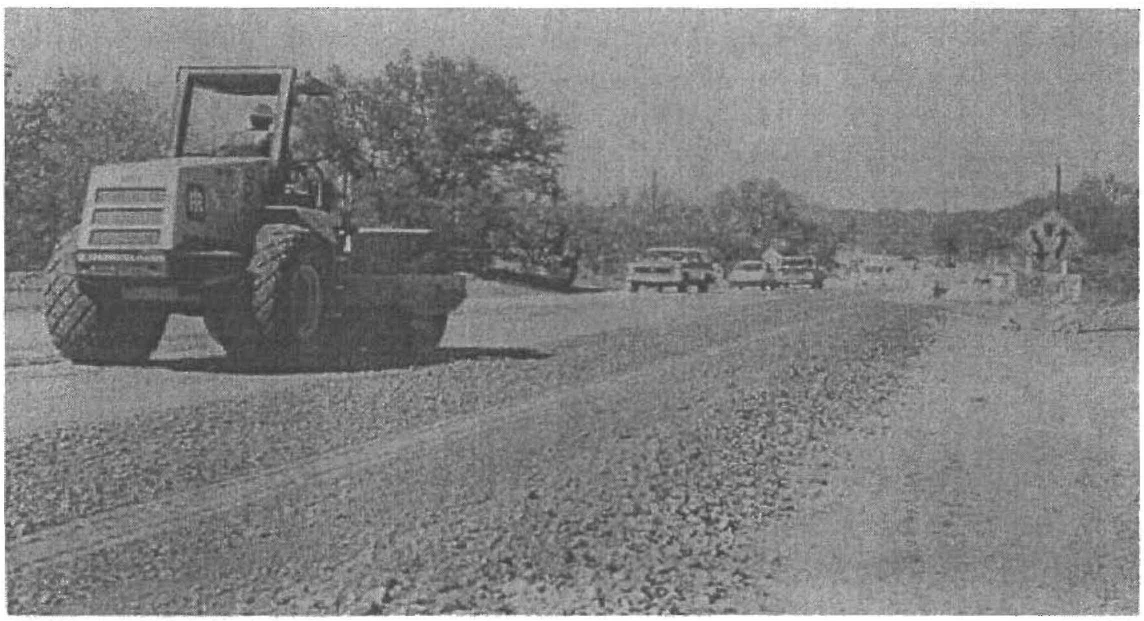


Figure 5.8: Final Compaction Using Vibrating Steel Roller

5.3 CONSTRUCTION PROCEDURES: CEMENT STABILIZATION

Fortunately, standard construction equipment can be used with cement stabilization. Basic equipment includes a motor grader with teeth (or plow) to scarify the surface (Figure 5.9), a cement transport truck to evenly distribute dry cement

(Figure 5.10) or a modified water truck if a slurry is used (Figure 5.11), a roto-mill or pulverizer to thoroughly mix the cement with the soil (Figure 5.12), and a water truck to add water to bring the mixture to optimum moisture content (Figure 5.13). Standard compaction equipment includes a steel-wheel roller (Figure 5.14) and a pneumatic roller (Figure 5.15). A motor grader is used for final grading.

Guidelines for this type of construction are available through Portland Cement Association (i.e. PCA 1992b) as well as cement manufacturers (e.g. Capitol Cement). For a cement-modified soil there are two methods currently in use, slurry application and dry application. In this project, dry cement was used. The following steps describe the process of constructing the cement test pavement:

Step 1: Cement was added to the soil. The plans for this project indicated that three percent cement was to be added to the subgrade at the test site. It was the intent, however, that this percentage be modified pending recommendations from the research project. This intent was not communicated to the contractor, who added

three percent cement to the soil. The mistake was realized only after construction operations were well underway, and there was no way to correct the mistake without delaying the project and incurring significant expenses. As shown in Figure 5.16, the addition of three percent cement will still result in a significant reduction in the plasticity index, to about 13.

Step2: The project site was scarified. It is possible that adding cement before scarifying could make incorporation of the cement into the soil less efficient. However, on this project a roto-mill, which is very effective in breaking up clods, was used.

Step 3: The cement was mixed with the soil. Because the width of the construction area was narrow and the roto-mill was efficient as a pulverizer, making passes at various angles to break up clods was not practical or necessary.

Step 4: A motor grader then made numerous passes mixing the treated soil with water applications from the water truck. After several hours, the water content was close to optimum for maximum density and compactive efforts began.

Step 5: A pneumatic roller was initially used to gain density in the treated subgrade. A vibratory-steel-wheel was then used to achieve the desired density.

Step 6: After the cement had been adequately mixed with the soil, a sample was obtained by TxDOT personnel. From laboratory testing on this sample, by TxDOT personnel, the maximum dry density was determined to be 95.0 lb/ft3 with an optimum moisture content of 11.4

percent. The results of this test procedure was made known to the contractor within several hours, thus preventing any delay on the project. The contractor was required to obtain no less than 98 percent of the maximum laboratory dry density on the subgrade. Two nuclear density readings were taken in the test section. Both readings indicated that the required density had been achieved. One reading indicated a dry density of 98.6 lb/ft3 with a moisture content of 11.6 percent, the other, a dry density of 96.7 lb/ft3 with a moisture content of 13.2 percent. Attempts were made to obtain in-situ samples after density in the subgrade had been achieved, but the material proved to be too brittle. Total width of the treated subgrade is 35 ft.

Step 7: A motor grader was then used for final grade followed by a pneumatic roller to seal the surface.

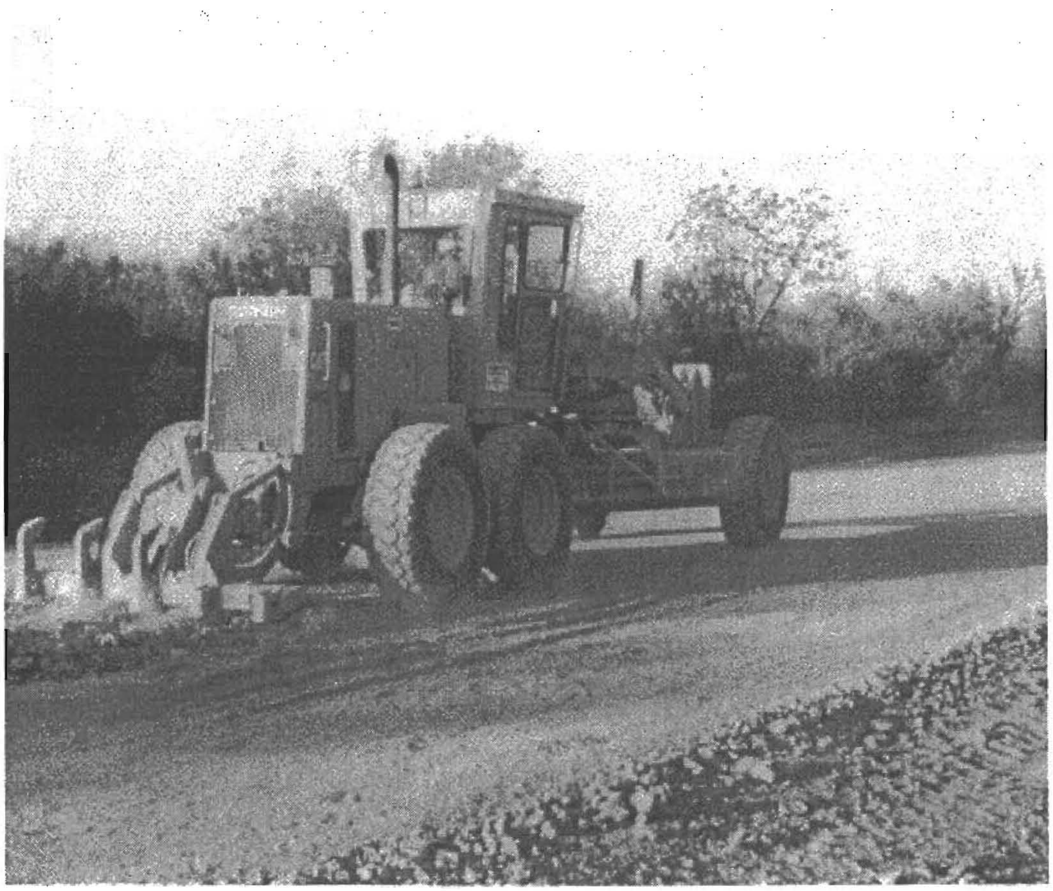


Figure 5.9: Use of Motor Grader.

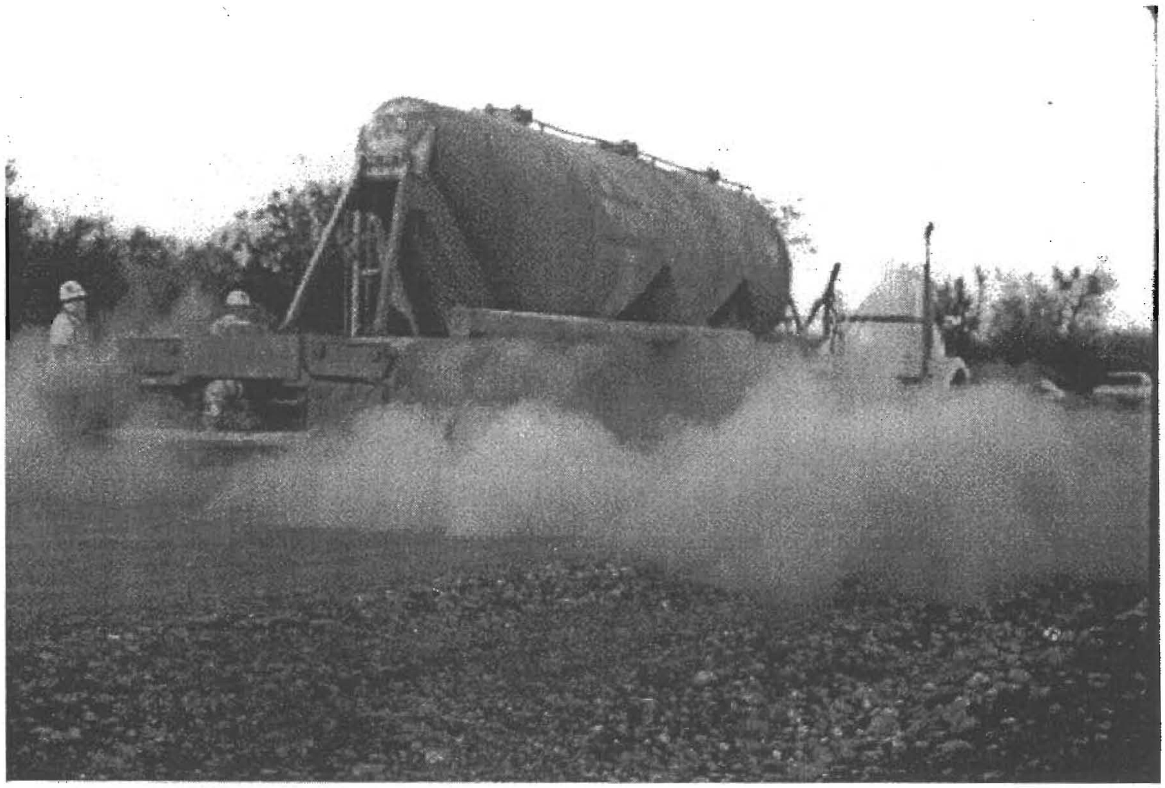


Figure 5.10: Distribution of Dry Cement Powder with a Cement Transport Truck.

Figure 5.11: Modified Water Truck for Slurry Application of Cement.

Figure 5.12: Use of Roto-Mill to Mix Cement with Soil.

Figure 5.13: Addition of Water Prior to Compaction.

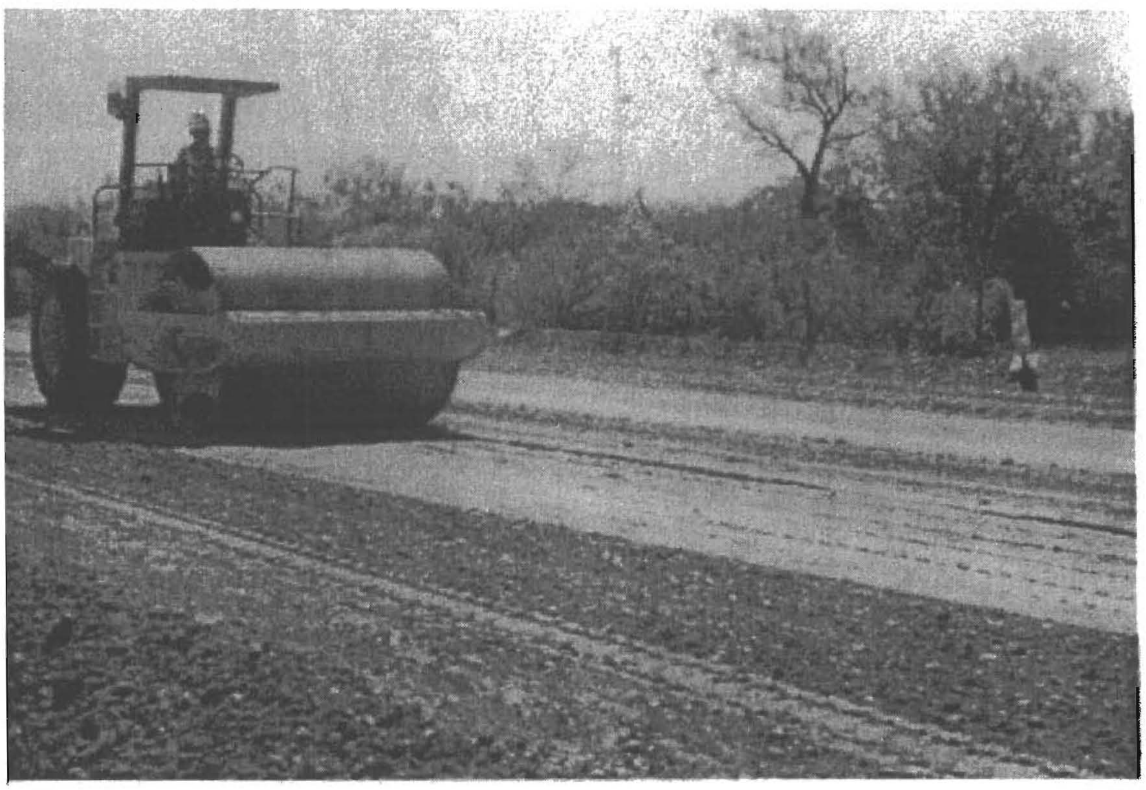


Figure 5.14: Use of Steel-Wheel Roller for Compaction of Cement-Soil Mix.

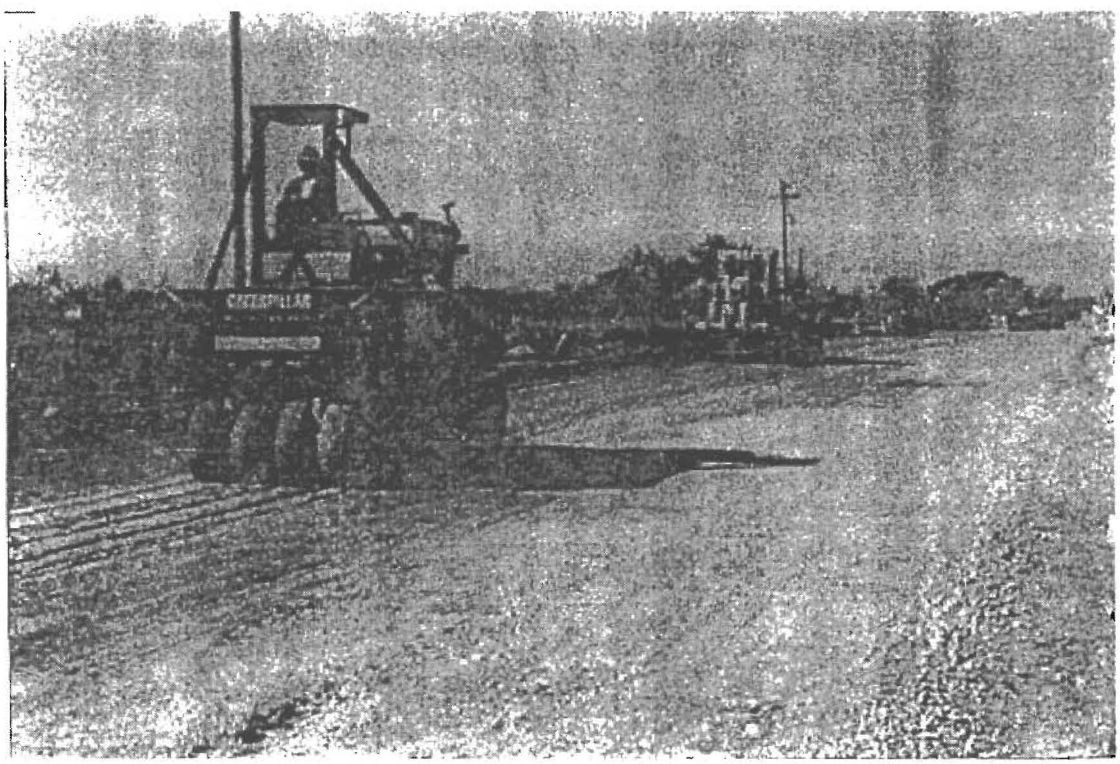


Figure 5.15: Use of Pneumatic Roller for Compaction of Cement-Soil Mix.

VI PERFORMANCE MONITORING

6.1 INTRODUCTION

Lime and cement are two chemical agents that are commonly used to stabilize pavement subgrade soils. These two chemical agents help reduce moisture susceptibility, i.e. shrink/swell behavior, that may otherwise occur in subgrade soils that have not been stabilized. The primary objective of this research was to compare soils stabilized with cement to soils stabilized with lime in order to determine which additive was most effective under similar conditions.

While short-term performance results are important, it is also important to study the long-term effectiveness and durability of subgrade soils stabilized with cement and lime. Therefore, as part of this research, a program for monitoring the long-term performance of the pavement was established. The terms of the research were that researchers from Texas Tech University would conduct the initial monitoring, but at the conclusion of the study, the TxDOT engineers would take over and continue the monitoring activities.

The specific monitoring activities included: (a) moisture condition underneath the pavement by using TCPs (thermocouple psychrometers), (b) visual condition survey to record surface distresses on the pavement, (c) Profilometer measurements to monitor pavement roughness and (d) Falling Weight Deflectometer Testing to determine the structural capacity. This chapter will explain these monitoring activities as they pertain to the lime and cement test sections.

6.2 MOISTURE CONDITION UNDERNEATH THE PAVEMENT

Being able to measure the changes in moisture levels underneath the paved sections allows researchers to evaluate the effectiveness of subgrade stabilization post-construction. This monitoring can be accomplished by examining the soil suction level (the negative pressure in the pore water of the soil).

There are two different ways to measure soil suction: (a) bring samples to the lab and then use the filter paper method or (b) install thermocouple psychrometers (TCP's) in the soil. The first method can be used to determine the initial soil moisture conditions, but cannot be used to track changes in moisture level in the already completed pavement.

6.2.1 Filter Paper Test. The filter paper method was used to gauge the initial soil suction of the undisturbed samples. The results of the filter paper tests are found in Chapter IV, Section 4.3.2.5.

6.2.2 Thermocouple Psychrometers. Thermocouple psychrometers (TCP's) were installed in the soil in order to measure the long-term soil suction levels underneath the test pavement sections. In the following excerpt, AASHTO (1986) explains how TCP's work to measure soil suction:

> The thermocouple psychrometer measures the relative humidity in the soil by a technique called Peltier cooling. By causing a small direct current of approximately 4 to 8 milliamperes to flow through the thermocouple junction for approximately 15 seconds in the correct direction, this junction will cool and water will condense on it when the dewpoint temperature is reached. Condensation of this water inhibits further cooling of the junction and the voltage difference developed between the thermocouple and the reference junctions can be measured using a microvoltmeter. With proper calibration, the thermocouple psychrometer output in microvolts can be converted directly to soil suction in convenient units of pressure. Typical thermocouple psychrometer output voltages vary from less than one microvolt for relative humidities close to 100 percent or total soil suction less than 1 tsf to about 25 microvolts for relative humidities of about 95 percent or total soil suctions of about 60 tsf. (p.1190).

An example of the relationship between soil suction and the physical behavior of soils is shown in Table 6.1. The suction level is shown as a positive value in units of pF (and kPa). The pF is the logarithm of (tension or suction) head in centimeters of water. While pF is a common unit of measure, kPa is the preferred unit of measure. However, in the technical literature, soil suction is also expressed in other units of pressure. Other units and conversion factors are shown in Table 6.2.

6.2.2.1 Installation of Thermocouple Psychrometer Instrumentation

Prior to installation, the pyschrometers were calibrated at 25° Celsius in accordance with ASTM Designation D5298. Calibration consisted of taking microvolt readings with the readout device at various NaCl solutions of known water potential. The microvolts were then plotted against the corresponding water potential to develop the curve. A typical calibration chart is shown in Figure 6.1. This curve allows the microvolt reading from the field to be converted into a pF of kPa value. To correct readings at temperatures other than 25° Celsius, the following equation may be used:

$$E_{25} = \frac{Et}{0.325 + 0.027t} \tag{6.2}$$

where, E_{25} = electromotive force at 25° C, microvolts,

E_t = electromotive force at t °C, microvolts,

t = test temperature, °C. 1992.

Table 6.1: Suction Correlation with Physical Behavior of Soils.

Behavior	Suction Level, pF (kPa)
Saturation	0.00
Liquid Limit	1.0 (0.98)
Field Capacity	2 - 2.5 (9.8 - 31)
Plastic Limit	3.2 -3.5 (155 - 310)
Plant Wilting Point	4.2 - 4.5 (1,554 - 2,101)
Tensile Strength of Water	5.3 (19,566)
Shrinkage Limit / Air Dry	5.5 (31,010)
Oven Dry	7 (980,638)

Source: McKeen, A Model for Predicting Expansive Soil Behavior, 1992.

Table 6.2: Conversion Table for Various Units of Soil Suction.

Cm of Water	Units			
	pF	Bars	psi	kPa
10	1	0.00981	0.142	0.981
100	2	0.0981	1.42	9.81
1,000	3	0.981	14.2	98.1
10,000	4	9.81	142.2	981
100,000	5	98.1	1422	9810
1,000,000	6	981	14220	98100

Source: Austin, Estimating Shrink/Swell in Expansive Soils Using Soil Suction, 1987.

Two sets of thermocouple pychrometers were installed in the cement stabilized test section at approximate Station 638+50 (Figures 6.2 and 6.3). One set was installed under the pavement structure approximately four feet east of the west edge of the roadway and the other set installed approximately three feet west of the pavement structure. Two pyschrometers were installed at each depth for each location. Installation depths, from the surface, are one, two, four, six, ten and fifteen feet. Figure 6.4 shows the installation test site for the lime section. Figures 6.5 shows the actual installation at the test sites. The agreement between TxDOT and the researchers was that the researchers would take the first three monthly measurements. The researchers would then teach the TxDOT personnel how to take the measurements. The first readings were taken August 21, 1996, approximately one month after installation. Results taken from the edge of the cement test section of roadway for the months of August and September are shown in Figure 6.6. Readings from the lime test section of roadway for the same time frame are found in Figures 6.7 and 6.8. Subsequent readings will be made on a monthly basis. The purpose of these readings are to develop a range of seasonal soil suction values with which to predict any potential heave,

6.3 Visual Condition Survey. A visual survey of the test site was made in August 1996, approximately two weeks after the flexible base course was constructed. The purpose was to identify any cracking or surface aberrations. Cracks could allow water into the foundation of the roadway, eventually causing the pavement to fail. Other defects on the surface could indicate a problem in the foundation. However, there were no visible signs of cracking or distress. Another visual inspection was made approximately one month later, in September, 1996, after the first course of a proposed two-course surface treatment was completed (Figure 6.9). There had been several inches of rainfall during this period. Again, there was no visible evidence of cracking or distress of any kind. After completion of the second course of the surface treatment, it is anticipated that visual inspections will be made at six month intervals by TxDOT personnel, along with other testing, as part of the long-term monitoring established for this project.

6.4 PROFILOMETER MEASUREMENTS

The profilometer is a device used for measuring the initial profile, or roughness, of the roadway. This device consists of an eight-foot wide bar mounted to the front bumper of a mini-van, called the profiler/rutbar vehicle (Figure 6.10). There are five ultrasonic sensors mounted on this bar which send out sound waves and measure the time it takes for the waves to return. Gathered data is sent to a processing computer inside the profiler/rutbar vehicle. A graphical display shows the roughness of the pavement.

TxDOT has recently developed this device, but it has not yet been used on the project. It is intended that TxDOT personnel will make a survey of the test site on a monthly basis to record any changes in the roughness of the roadway. This test is especially important, since one of the goals of TxDOT is to provide a smooth riding surface on its roadways. A typical graphical display of the data is shown in Figure 6.11.

6.5 Falling Weight Deflectometer Tests

The Falling Weight Deflectometer (FWD) helps determine the structural integrity of the pavement structure. The FWD drops a weight on the pavement and the resulting deflection is measured using six sensors. The deflection data is reported using a parameter called the Structural Strength Index Score. Scores from 0 to 100 represent strengths from very weak to very strong. It is intended that TxDOT personnel will perform this test on a monthly basis to determine any strength changes in the pavement. Figure 6.12 depicts the use of this device at the test site.

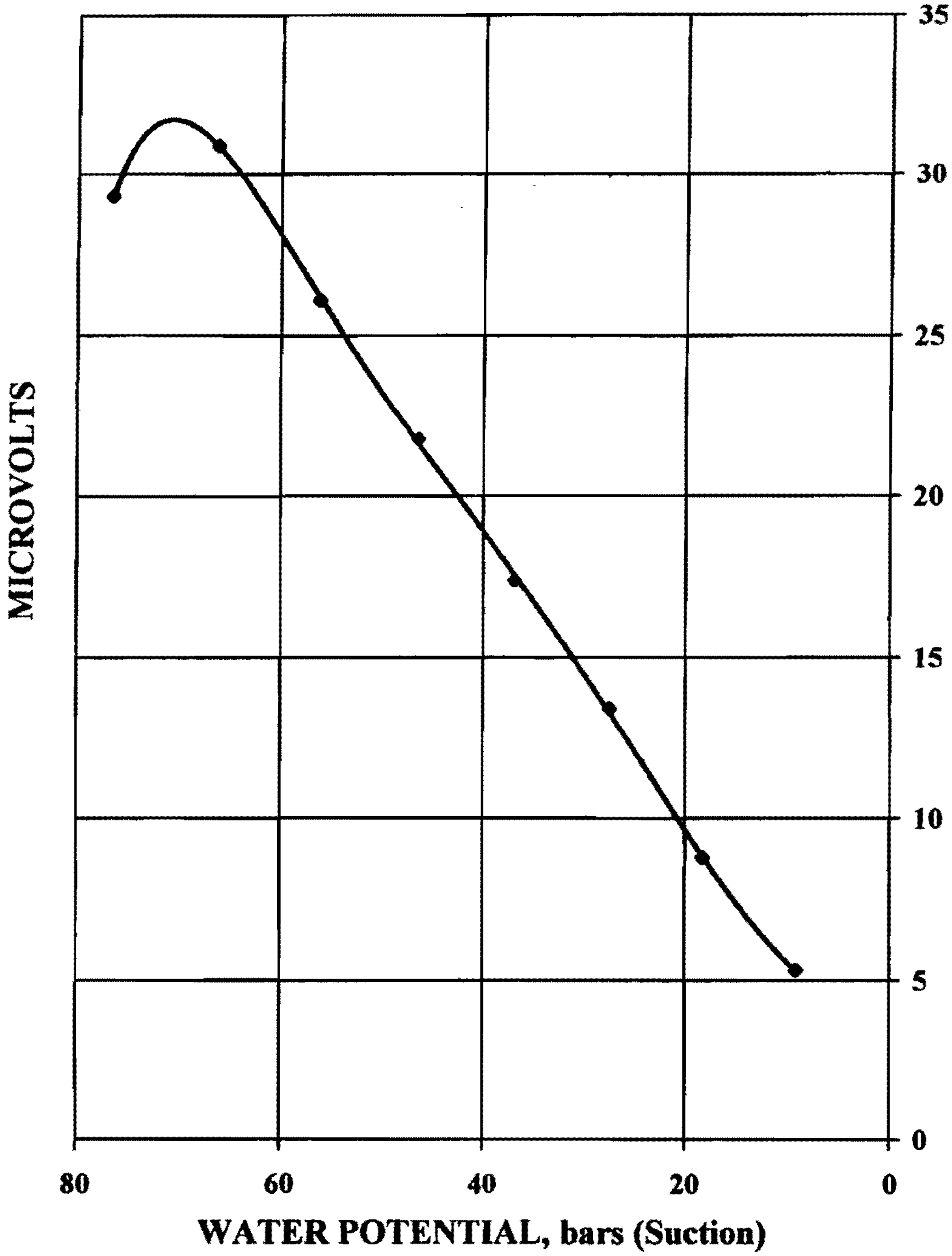


Figure 6.1: Typical Calibration Curve for a Thermocouple Psychrometer.

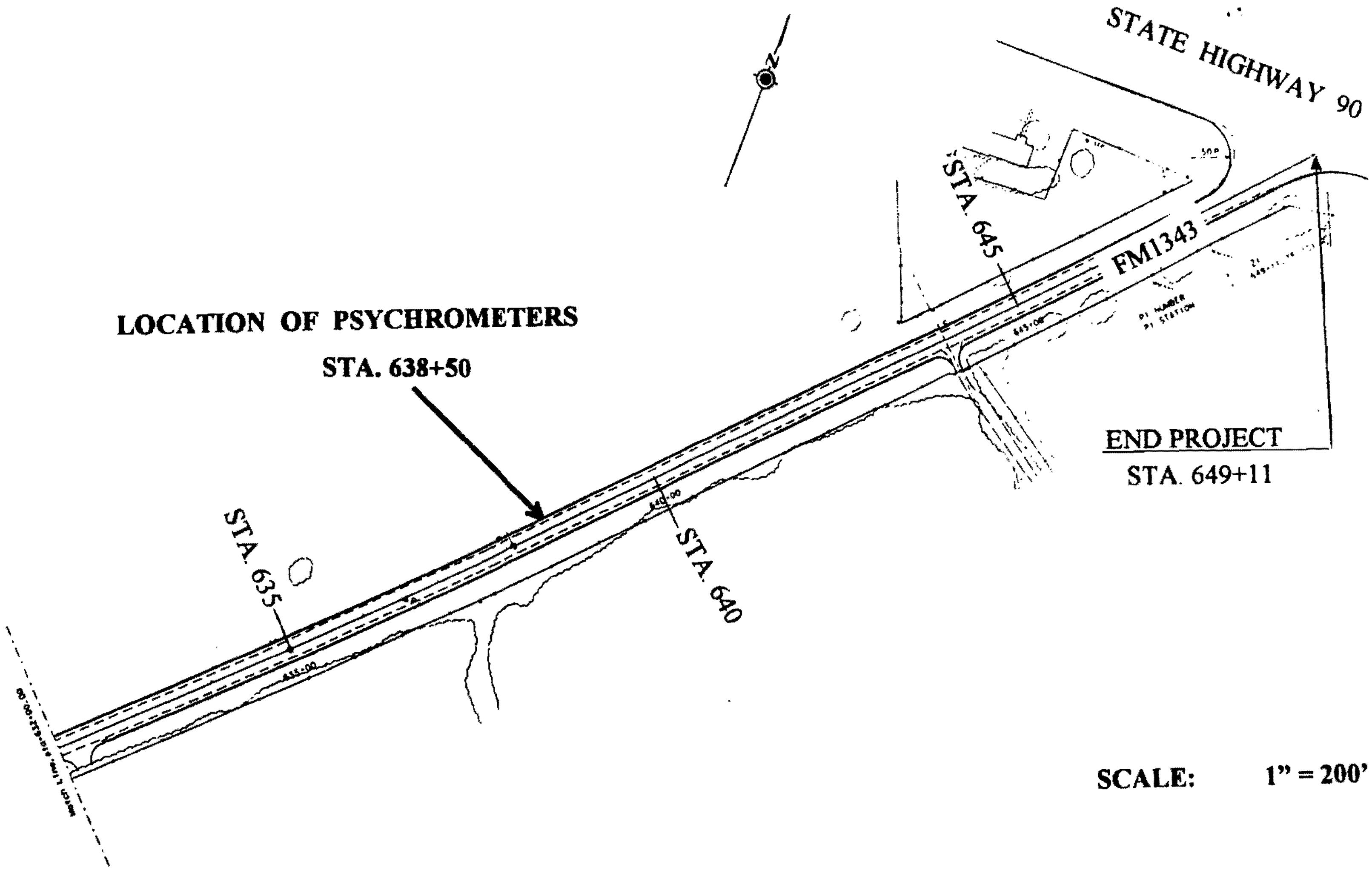


Figure 6.2: Location of the Psychrometers on the Project, Plan View.

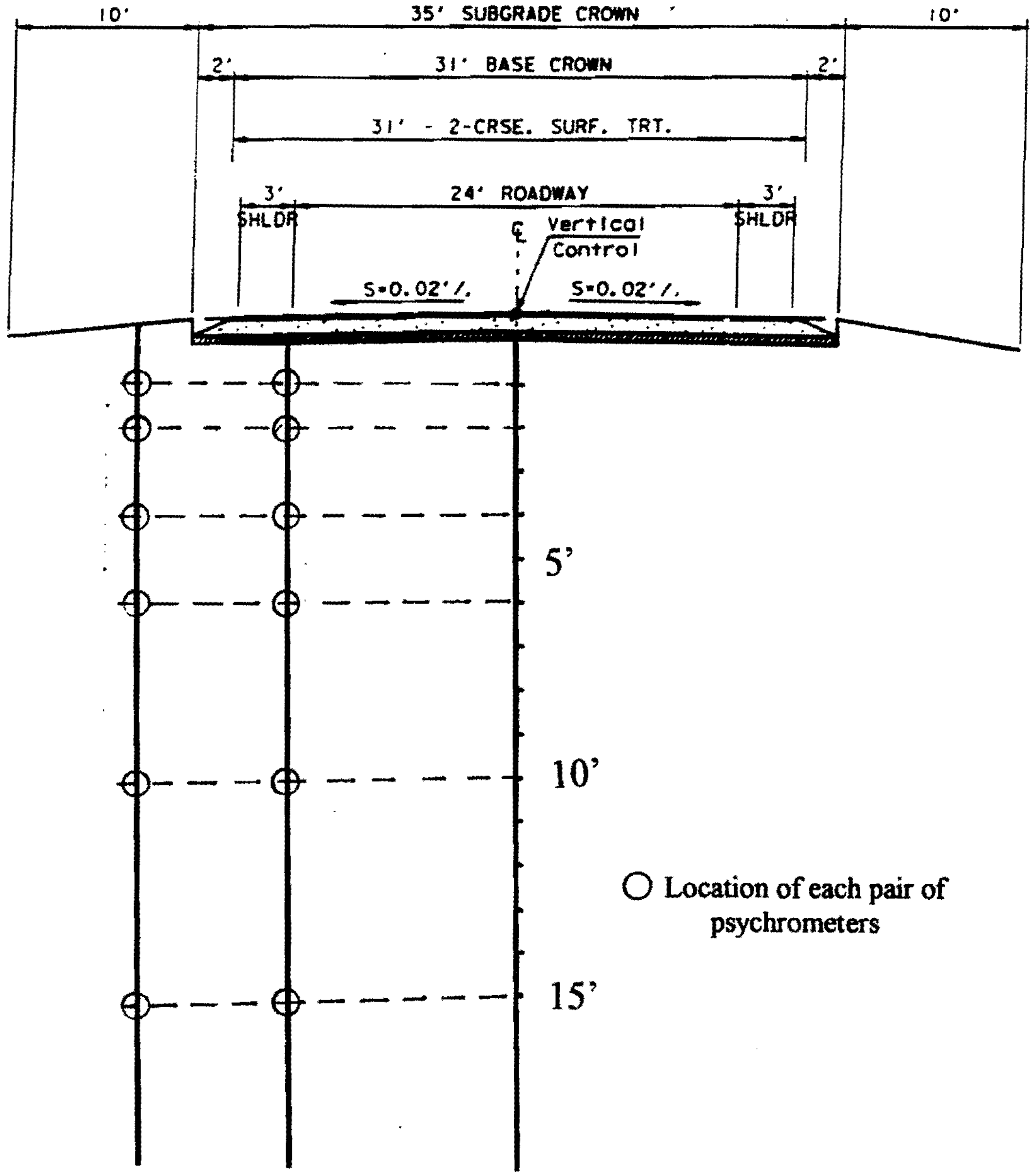


Figure 6.3: Location of the Psychrometers on the Cement Project Site, Elevation View.

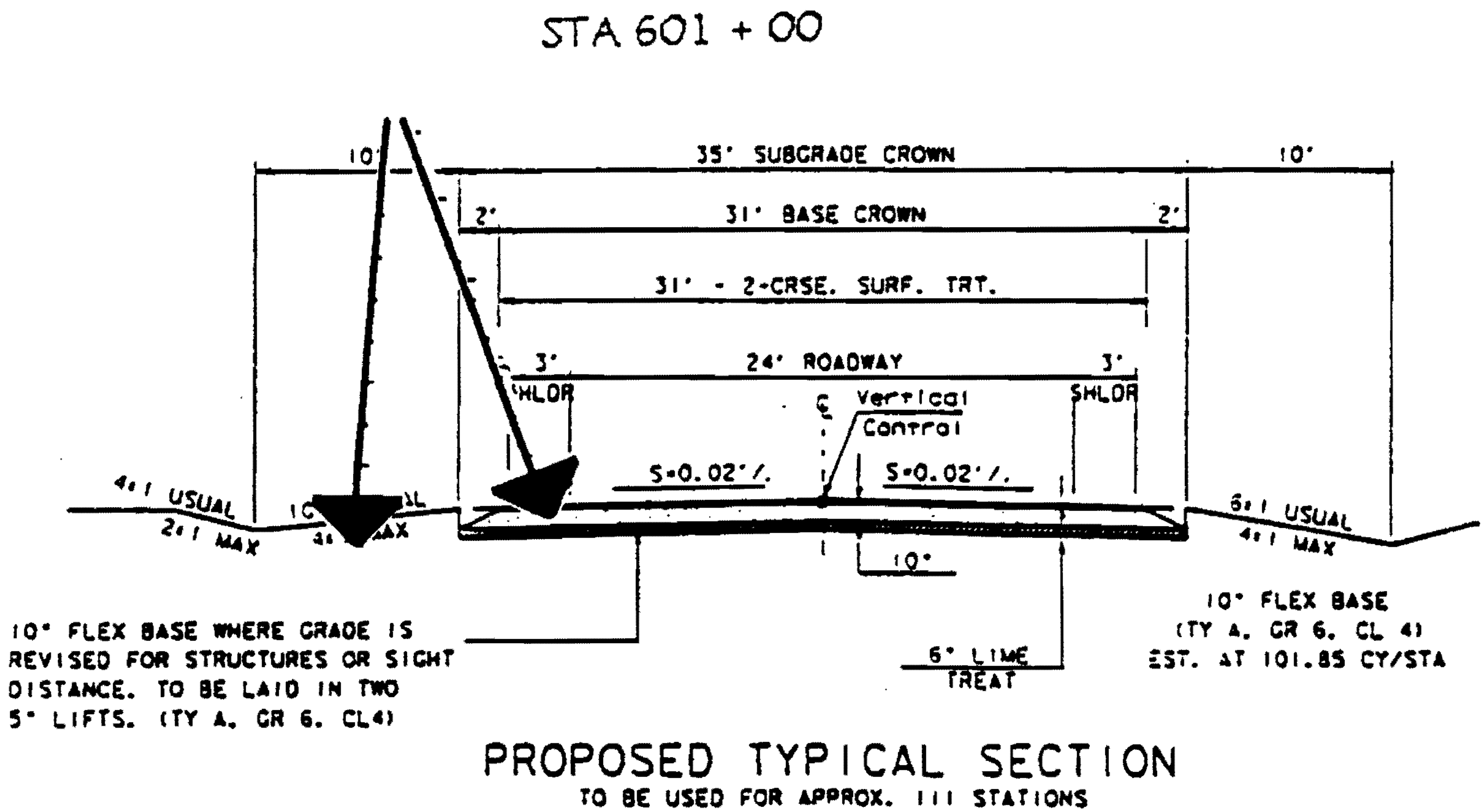


Figure 6.4: Location of the Psychrometers on the Lime Project Site

Figure 6.5: Drill Truck Using a 6 inch Core Barrel for Installing the Thermocouple Psychrometers at the Test Site.

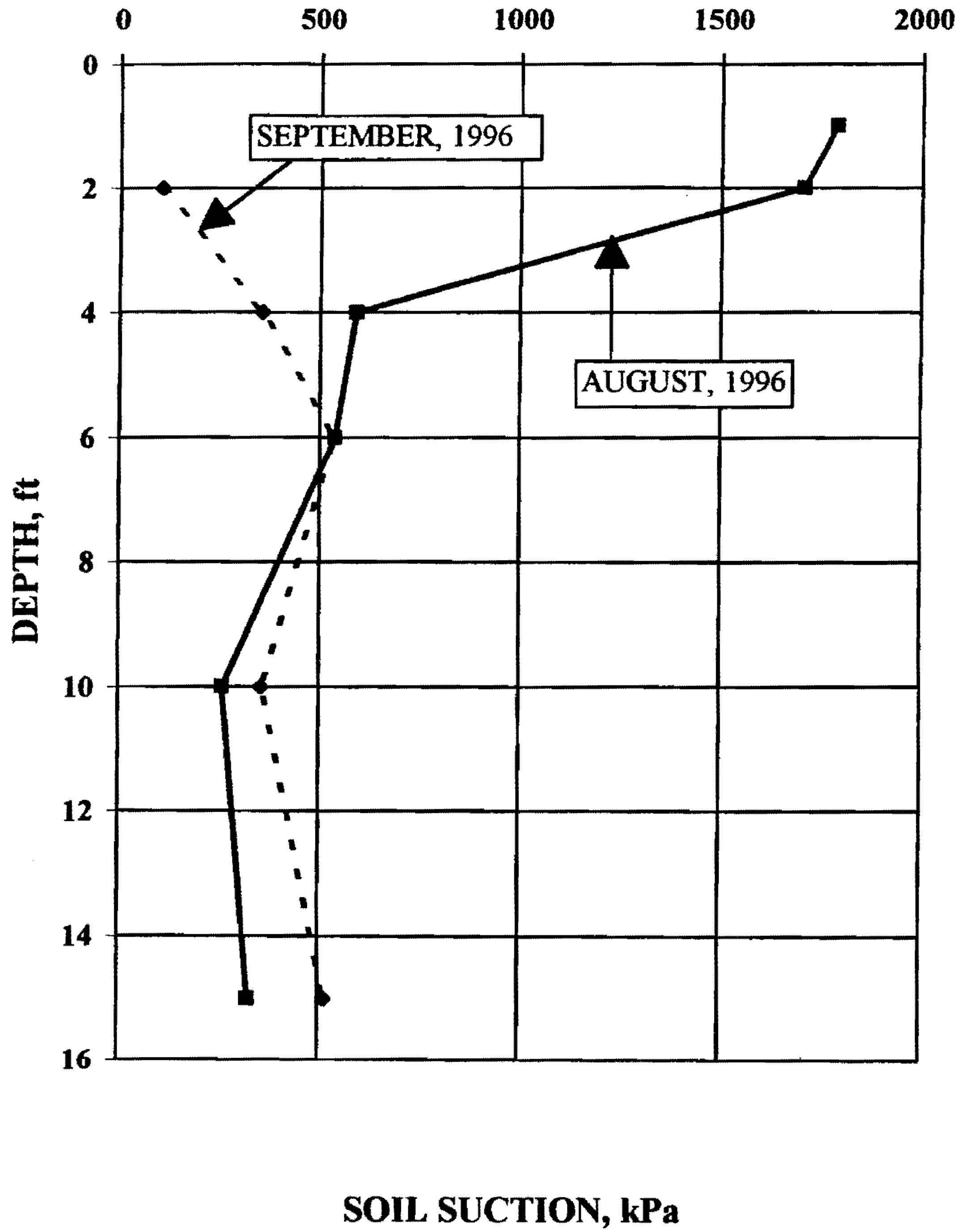


Figure 6.6: Soil Suction Values Taken from the Edge of the Roadway

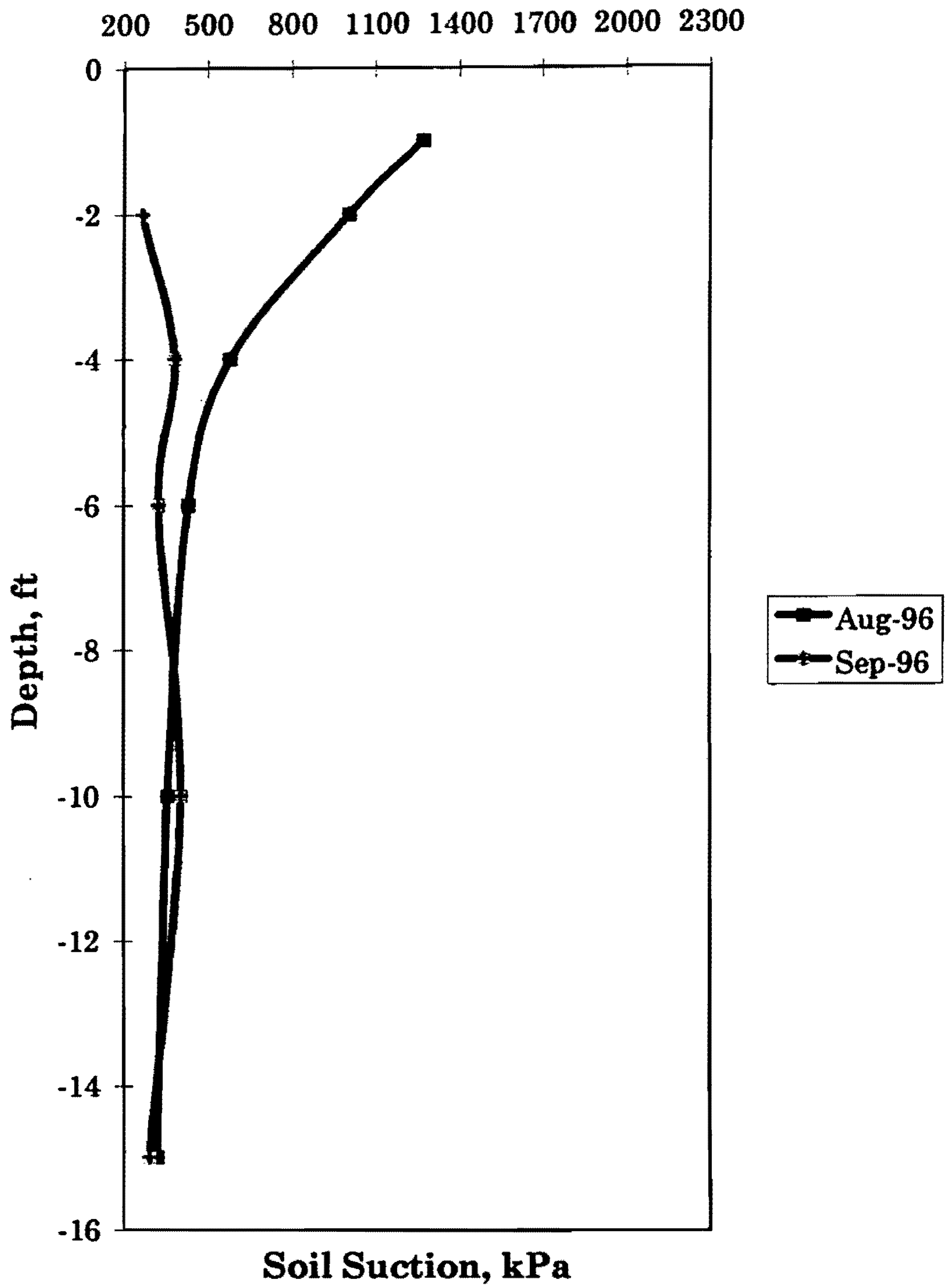


Figure 6.7: Soil Suction Profile under the Influence of the Pavement in the Lime Test Section

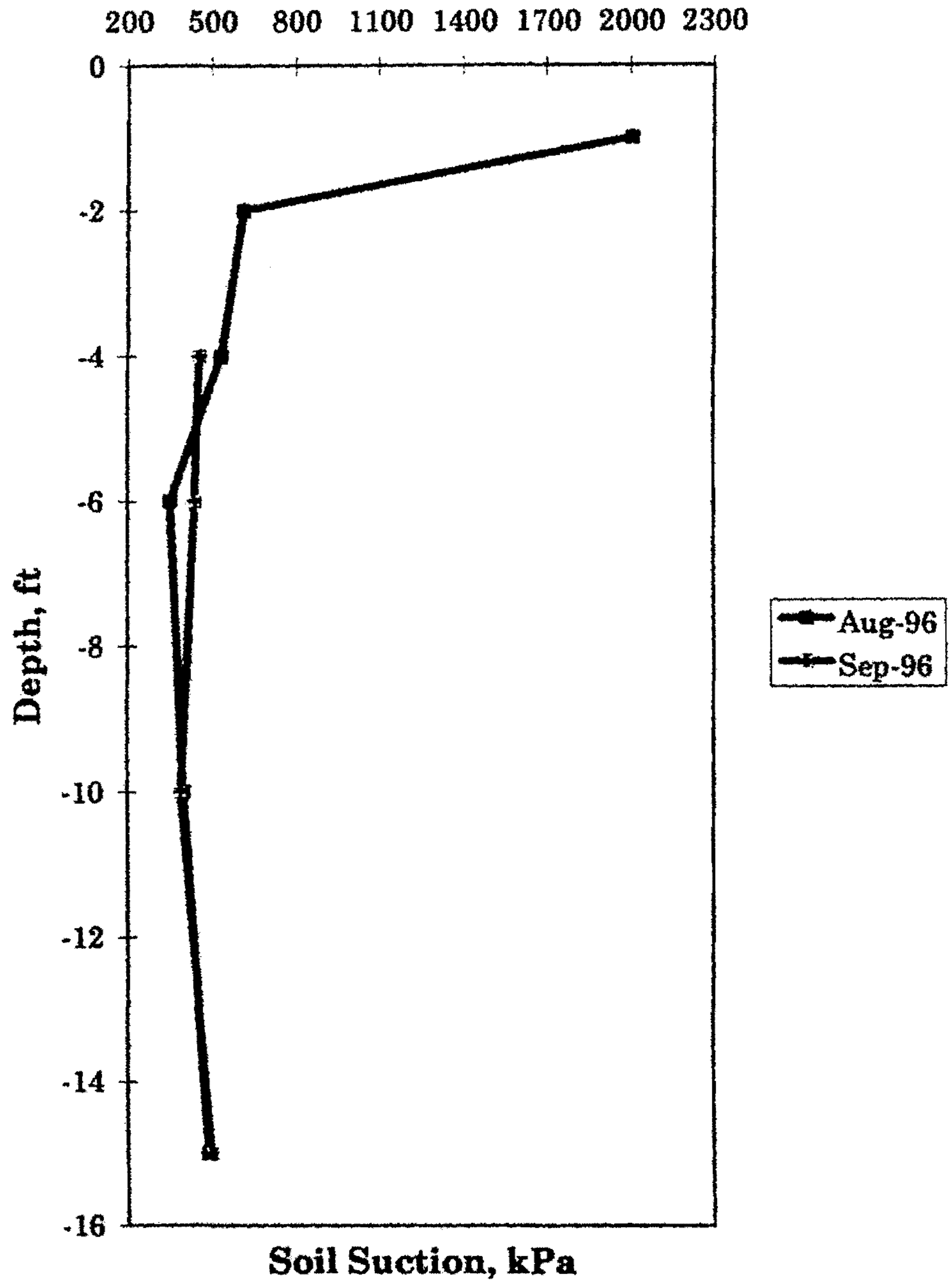


Figure 6.8: Soil Suction Profile out of the Influence of the Pavement in the Lime Test Section.

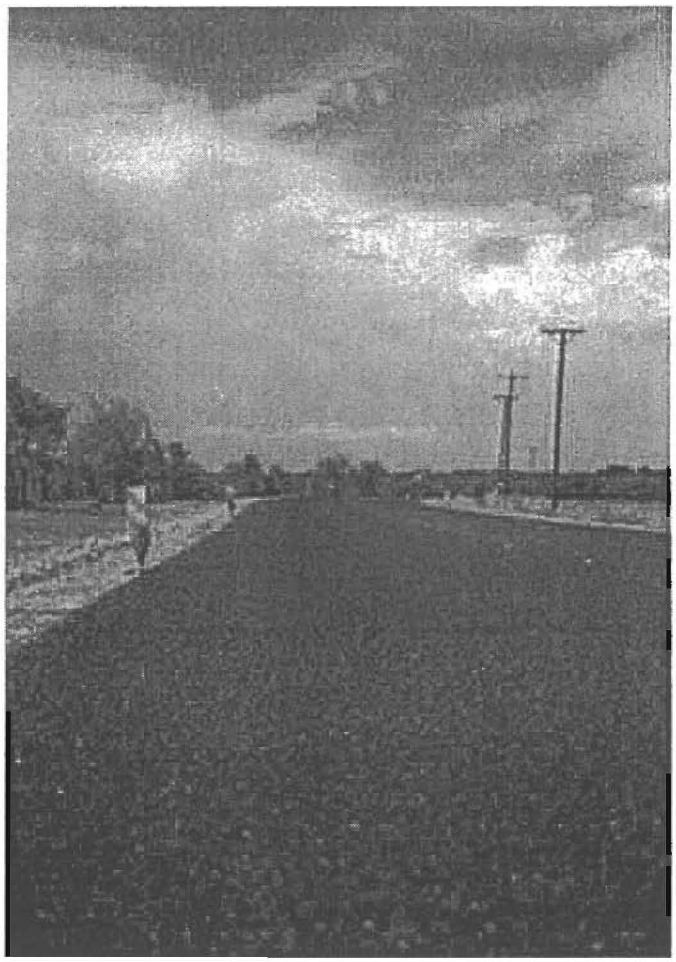


Figure 6.9: View of the Roadway after Completion of the
First Course of Surface Treatment.

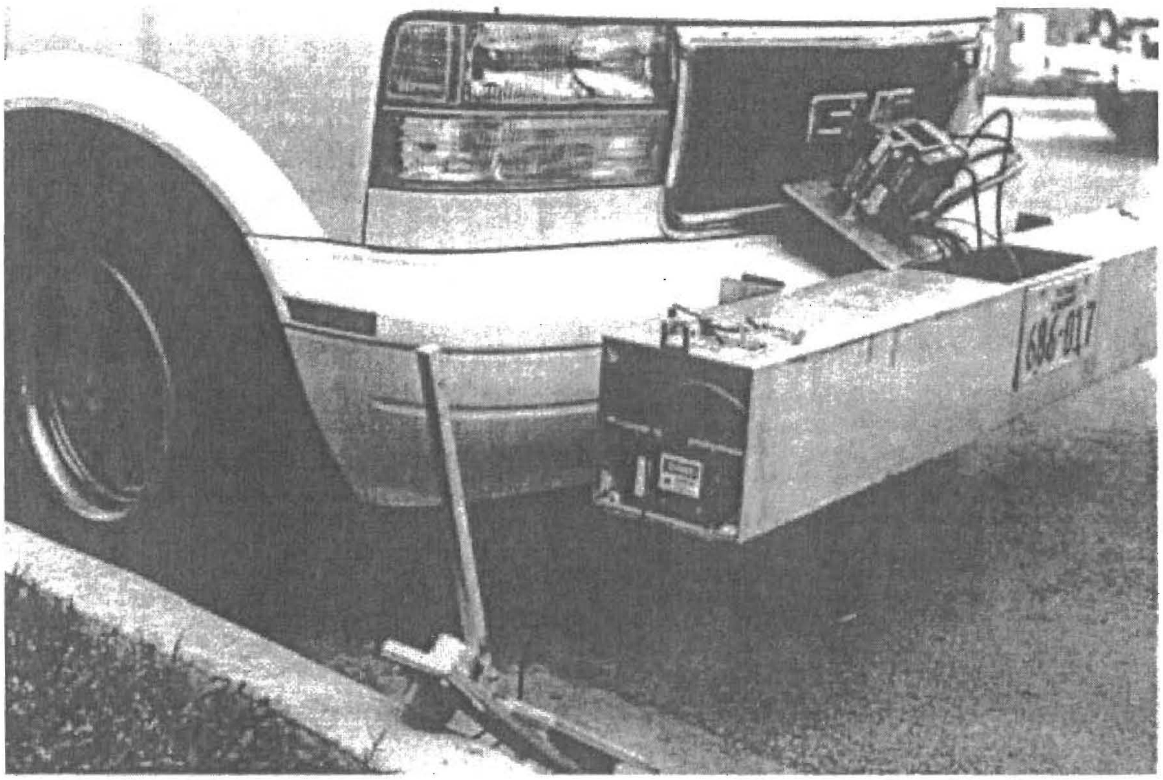


Figure 6.10: Sensor Bar Mounted to the Profiler/Rubar Vechicle.

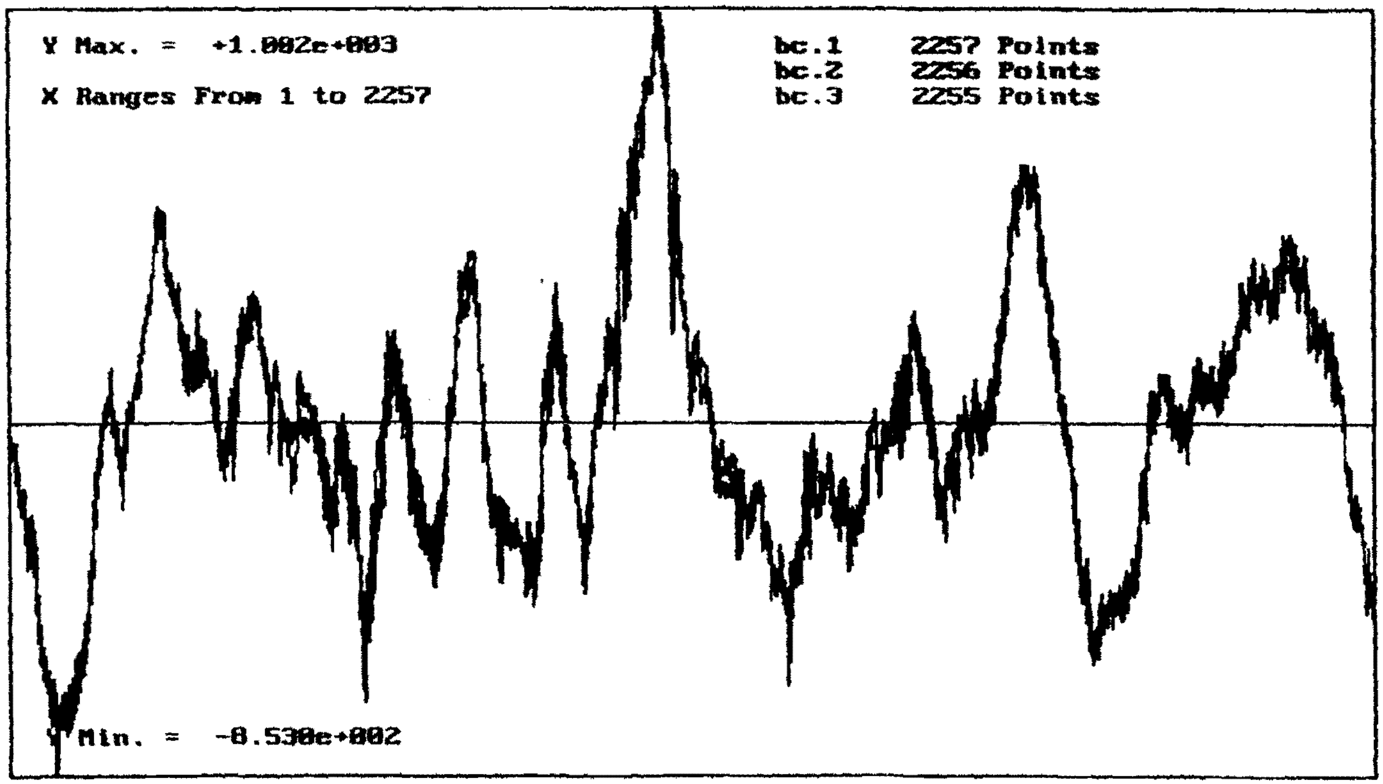


Figure 6.11: A Typical Graphical Display of Data Measured by the Profilometer.

Figure 6.12: Falling Weight Deflectometer Collecting Data from the Test Sites.

VII CONCLUSIONS AND RECOMMENDATIONS

The primary objective of this one-year research study was to complete the necessary preliminary tasks that would enable a comparison of the long-term effectiveness of cement and lime as stabilization agents for moisture susceptible pavement subgrade soils. These tasks included: selection of a suitable construction site for the experimental projects, characterization of soil conditions at the site, construction monitoring, and installation of instrumentation for long-term monitoring. According to this research plan, the two experimental projects, one with cement stabilized subgrade and the other with lime stabilized subgrade, will be monitored over the next several years. Any conclusion with respect to the effectiveness of cement versus lime can only be made at the end of this period after the performance monitoring data collected have been reviewed and analyzed. Nevertheless, a number of useful observations were made during the course of this research study. These observations are documented in the preceding sections—a summary of these follows.

Cement performed as an effective stabilizer for the low plastic clayey subsoil found at the site. With the addition of 4 percent cement, the PI decreased to an acceptable level and the strength and durability significantly increased. The optimum additive percentage to be used in the case of lime stabilization was also determined to be 4% by both pH and PI tests. At 4% lime content, the soil PI was significantly lower and the strength and durability was higher. However, the improvements in strength and durability were not as pronounced as in the case of cement stabilized soil. Lime had virtually no impact on the permeability of the stabilized material, whereas cement caused the permeability to decrease dramatically.

Long term monitoring was established with the successful installation of the thermocouple psychrometers, and initial testing with the FWD. Two visual inspections were completed and findings recorded. The profilometer is operational and scheduled for initial testing.

It is recommended that the long term monitoring continue for a period of five years. Data from the psychrometers should be collected on a monthly basis. It is also recommended that the FWD and the profilometer be performed on a monthly basis and the data recorded. The visual inspection should be performed every six months, but could easily be performed more frequently when performing the other tests. All data should be entered into the same database and updated monthly. At the end of the five years, the change in moisture as indicated by the psychrometer readings should be compared with the other test data. This information will be important in determining how the cement treated subgrade has performed, as the moisture in the soil below it has fluctuated, and its subsequent effect on the pavement structure.

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