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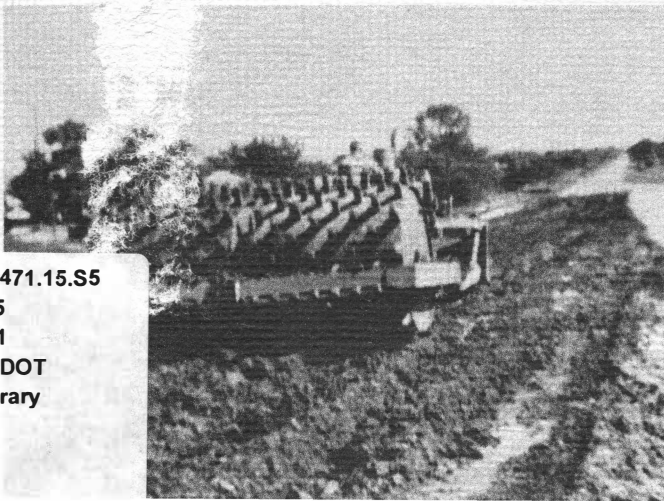


OKLAHOMA DEPARTMENT OF TRANSPORTATION
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STABILIZATION OF OKLAHOMA SHALES

THE UNIVERSITY OF OKLAHOMA
School of Civil Engineering
and Environmental Science

Norman, Oklahoma
December, 1977



**Joakim G. Laguros
Kulanand Jha**

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Final Report

**ODOT Study No. 73-04-2
ORA 158-602**

STABILIZATION OF OKLAHOMA SHALES

Prepared by

**Joakim G. Laguros, Professor
Kulanand Jha, Research Assistant
School of Civil Engineering and Environmental Science**

Submitted to

**OKLAHOMA DEPARTMENT OF TRANSPORTATION
and
DEPARTMENT OF TRANSPORTATION
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From the

**OFFICE OF RESEARCH ADMINISTRATION
UNIVERSITY OF OKLAHOMA
Norman, Oklahoma
December 1977**

SUMMARY

In cooperation with the Oklahoma Department of Transportation and the Federal Highway Administration, U.S. Department of Transportation, a research project was undertaken on June 1, 1973, by the University of Oklahoma to study the stabilization of Oklahoma shales.

Inasmuch as shales had already been studied, identified, classified and characterized in a previous study (1972), the main objective of the present investigation was to determine the type and amounts of stabilizing agents to yield optimum conditions in upgrading the shales for highway construction purposes.

Eight Oklahoma shales representing regional characteristics but differing texturally and mineralogically were selected on the basis of the 1972 study. Of the various stabilizing agents used, it was found that 6 percent hydrated lime, 14 percent portland cement, and 25 percent flyash give optimum conditions by imparting acceptable strength levels to the shales. The degree of strength attained varies inversely with the plasticity of shale. On the basis of compressive strength, cyclic loading, and weatherability criteria, cement is the most effective additive, flyash the least and lime moderately effective. Combinations of the three stabilizing agents show that shales can be upgraded strength-wise when the three main stabilizing agents are used conjunctively; in fact, the data suggest that such uses are more economical and more promising. The addition of small amounts (less than 2 percent) of NaCl, CaCl₂, NaOH, and Na₂CO₃ to the shales which already have been mixed with one of the main stabilizing agents, further enhances the stabilization beneficiation.

With the exception of flyash stabilized shales, delayed compaction appears to produce and overall decrease in the compressive strength properties.

The gain in strength and durability and the resistance to deformation resulting from the stabilization process is explained qualitatively in terms of the bonding and cementation which takes place within the shale mass. To further evaluate the bonding quantitatively, use is made of the aggregation index ratio. Too, electron microscopic studies indicate that the voids of the stabilized shales are reduced substantially compared to the raw shales and there is a direct relationship between void domain characteristics and stabilization effectiveness.

The conclusions of this study also point to avenues which should be further explored. The study of flyash culminating into its wider use is environmentally of high priority. The employment of cyclic loading effects and void domain characteristics as a predictive tool to interpret engineering behavior of raw and stabilized shales is certainly promising. However, more complete studies in these two areas are suggested. Also, it is recommended that a field implementation program be initiated wherein the results of the laboratory studies be applied and any differences be analyzed and studied so that a realistic pavement design method employing stabilized shale can be finalized.

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Explanation of Symbols

LL = Liquid limit (%)

PL = Plastic limit (%)

PI = Plastic Index (%)

OMC = Optimum moisture content (%)

MDD = Maximum dry density (pcf)

DD = Dry density (maximum dry density, pcf)

UCS = Unconfined compressive strength (psi)

C = Cohesion (psi)

ϕ = Friction angle (degree)

σ = Normal stress (psi)

τ = Shear stress (psi)

μ = Micron (10^{-4} cm) .002 mm

AI = Aggregation index

$$= \frac{\% \text{ of nonclay size material of stabilized shale}}{\% \text{ of nonclay size material of raw shale}}$$

SBI = Strength beneficiation index

$$= \frac{(\text{UCS})_{\text{main+secondary stabilizing agent}} - (\text{UCS})_{\text{main}}}{(\text{UCS})_{\text{main}}}$$

RP = Reactive potential

$$= \frac{Y}{100} \left(\frac{X1}{100} (\text{CEC})_1 + \frac{X2}{100} (\text{CEC})_2 + \dots \right)$$

Y = Percent of 2 μ clay

X1, X2 = Percent of 1st, 2nd clay minerals

CEC = Cation exchange capacity of clay minerals (meq/100gram)

V = Void cross sectional area (%)

P = Longest pore intercept (%)

CHAPTER I

INTRODUCTION

Throughout Oklahoma there is a variety of soil materials known as shale. By virtue of its abundant occurrence, shale has been widely used for highway pavement base, sub-base and fill construction. The engineering performance of shale is affected by moisture content, its cementing constituents and the clay minerals present, and the surcharge loads imposed on it throughout its morphological and service life. Variation of these factors with time result in considerable, and often critical, changes in the engineering properties of shale. This situation is described as "weathering". It has been observed that shales with desirable physical properties sometimes weather quickly with adverse results, wherein the properties are altered and assume no longer acceptable levels and thus the shale is not capable of fulfilling its design function.

This limitation was of significance to the Oklahoma Department of Transportation because it curtailed the utilization of shales for pavement construction purposes. Also, the depletion of select material sources further increased the concern of the Department. Consequently, there was justifiable need to attempt to investigate ways and means of increasing the stability and durability of shales. This study aimed at upgrading the substandard soil materials or shales so that they would:

1. Provide lower cost roads on a long-term basis
2. Have a longer service life
3. Give an acceptable or superior performance

The modification in the engineering properties of a soil particulate matter can be brought about in many ways and it arises from:

1. Densification
2. Formation of physico-chemical bonds between particles resulting in stable aggregations, and
3. Waterproofing the soil aggregates

The upgrading of substandard soils and the amelioration of the properties of soil materials have in the past been attained by the addition of stabilizing agents, such as portland cement, hydrated lime, flyash, chlorides of calcium and sodium used alone and/or conjunctively. Ammonia derivatives of organic compounds and salts of large organic cations as well as products of the petrochemical industry have been effectively employed for soil stabilization. Most recently, attention has been focused on using industrial and municipal wastes, too.

In Oklahoma, the current widely-used practice is to add hydrated lime to shales. While some pavements containing lime stabilized shale perform satisfactorily, others are reported to undergo strength losses with time, as manifested by pavement failures.

The purpose of this report is to study the effectiveness and the optimization of the various methods of shale stabilization and the accurate material characterization thereof. This study is divided into the following four important phases:

1. Selection of shales and identification testing
2. Preliminary laboratory stabilization
3. Selective laboratory stabilization and evaluation, and
4. Optimization study.

Also, attempts have been made to establish useful correlations among the significant engineering properties of raw shales and their

stabilized counterparts in a manner that the relationship could be employed as predictive tools.

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CHAPTER II

REVIEW OF LITERATURE

SHALES IN GENERAL

Shale is one of the most commonly occurring soil or rock material on the earth's surface. It is often used as a construction material. The term "shale" has been applied to many classes of materials, which can be generally described as fine-grained, clastic sedimentary rocks. According to Underwood (1967), all definitions of shales contain the same basic terms, "laminated", "sedimented", "clay particles". Shales are likely to include anything indurated, fissible or laminated with uncertain limit. The material tends to break approximately along the bedding plane, although some secondary breakage can occur as a result of vertical overburden pressure. In general, exact application of the term "shale" to any soil mass becomes very difficult when considering many differences of opinion regarding the constitution of the material. Terzaghi (1946) classifies shales as rock-like, that is, able to cause a ringing sound when hit by a hammer. Underwood (1967) considers shale to include a highly indurated and generally fissible equivalent of claystone and/or siltstone. Many engineers are even inclined to discount fissibility as a requirement and so apply the title to heavily compacted laminated clays or silt clays. Ingram (1953) notes three types of fissibility; namely, massive, flaggy, and flaky which includes acceptable of mudrocks of questionable degree of lamination, since massiveness itself implies random particle arrangements. According to Underwood's classification, shales are sedimentary rocks as compaction or "soil-like" and cemented or "rock-like".

A distinguishing test between the two is that the former tends to slake rapidly in water while the latter does not. This implies that the rock-like shale is well cemented.

It would seem appropriate to note that shales in this study, though generally soil-like in nature, do not conveniently fall into either category described, but fall into a category which might be called "partially cemented". Though these shales meet the chemical composition normally proposed by most writers (Troutelot, 1962), they vary in their susceptibility to weathering by their degree of cementation and mineralogy.

CLASSIFICATION OF SHALES

The variability and the wide occurrence of a material such as shale make classification and evaluation of its possible behavior an important stage in the design and construction process. A number of classification systems have been developed to deal with the problem caused by the character of shale. Underwood (1967) presented a summary of the various attempts by many investigators to identify and classify the shales. As evidenced by Figure 1, he favored the geological classification system, which broadly divided the shales into two groups. "Soil-like" shale or the shales which are poorly cemented and are readily disaggregated when slaked with water, and "rock-like" shales or shales which are well cemented and resistant to the action of water. Also, based on the engineering properties of various shales such as density, natural water content, permeability, etc., and their reported performance, he distinguished between desirable and undesirable shales for construction purposes. Gamble (1971) developed a classification system which was based on two basic properties, plasticity characteristics and slaking

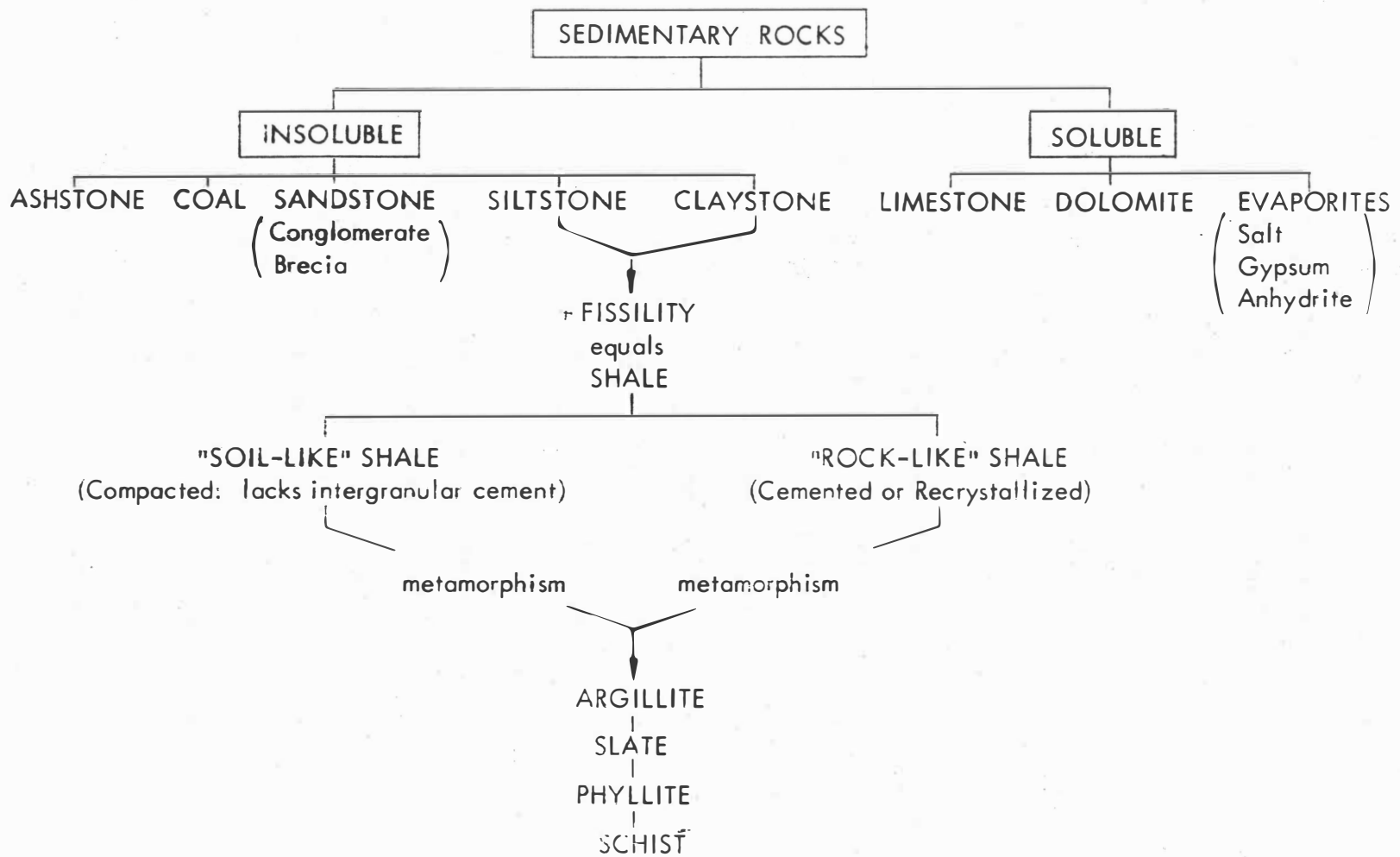


Figure 2.1: Classification of sedimentary rocks (after Underwood, 1967).

durability of the shales. The slaking durability test measures the resistance of the shale to the action of water and readily classifies shales into rock-like and soil-like shales. Plasticity characteristics are indicative of the engineering properties and performance of the shales. Hence, Gamble's method of classification is, in practice, the same as Underwood's.

Shale is intermediate between soil and rock. However, shales in general vary enough that, in many cases, "soil" and "rock" classification systems are inadequate. It is not simple to measure elastic modulus and compressive strength, yet the material can be hard enough to prevent adequate breakdown to prepare it for Atterberg limits determination. These discrepancies have led to the development of several classification systems for shales or argillaceous rocks. Because various researchers had different objectives, there is considerable variation in the types of tests investigated, and also in the types and ages of geologic materials which were tested. Underwood (1967) discussed many pertinent properties of shales whose analysis is depicted in Table 2.1.

Gamble (1971) tested 120 shales from many areas of the United States. He discussed the short-comings of the geological classification systems for shales, and processed the classification and standardized nomenclature shown in Table 2.2 and Figure 2.2. Grain size and breaking characteristics are of primary consideration. Based on extensive laboratory testing including slake durability and Atterberg limits, Gamble also proposed the engineering classification for shales and other argillaceous rocks shown in Figure 2.2.

In a study by Laguros (1972), index property tests were performed. These results were supplemented by X-ray diffraction analysis of the mineral-

Table 2.1:
AN ENGINEERING EVALUATION OF SHALES (UNDERWOOD, 1967).

PHYSICAL PROPERTIES		PROBABLE IN-SITU BEHAVIOR							
LABORATORY TESTS AND IN-SITU OBSERVATIONS	AVERAGE RANGE OF VALUES		High Pore Pressure	Low Bearing Capacity	Tendency to Rebound	Slope Stability Problems	Rapid Slaking	Rapid Erosion	Tunnel Support Problems
	UNFAVORABLE	FAVORABLE							
Compressive Strength	50-300 psi		✓	✓					
		300-5000 psi							
Modulus of Elasticity	20,000-200,000psi			✓					✓
		200,000-2x10 ⁶ psi							
Cohesive Strength	5-100 psi				✓	✓			
		100 psi to >1500psi							
Angle of Internal Friction	10-20°				✓	✓			
		20-65°							
Dry Density	70-110 pcf		✓					✓?	
		110-160 pcf							
Potential Swell	3-15%				✓	✓		✓	✓
		1-3%							
Natural Moisture Content	20-35%		✓			✓			
		5-15%							
Coefficient of Permeability	10 ⁻³ - 10 ⁻⁶ cm/sec		✓			✓	✓		
		>10 ⁻⁵ cm/sec							
Predominant Clay Minerals	Montmorillonite, Illite		✓			✓			
		Kaolinite, Chlorite							
Activity Ratio = $\frac{PI}{\% \text{ Clay}}$	0.75 to >2.0					✓			
		0.35 to 0.75							
Wetting and Drying Cycles	Reduces to grain sizes						✓	✓	
		Reduces to flakes							
Spacing of Rock Defects	Closely Spaced			✓		✓		✓?	✓
		Widely Spaced							
Orientation of Rock Defects	Adversely Oriented			✓		✓			✓
		Favorably Oriented							
State of Stress	>Existing Overburden Load				✓	✓			✓
		≅ Overburden Load							

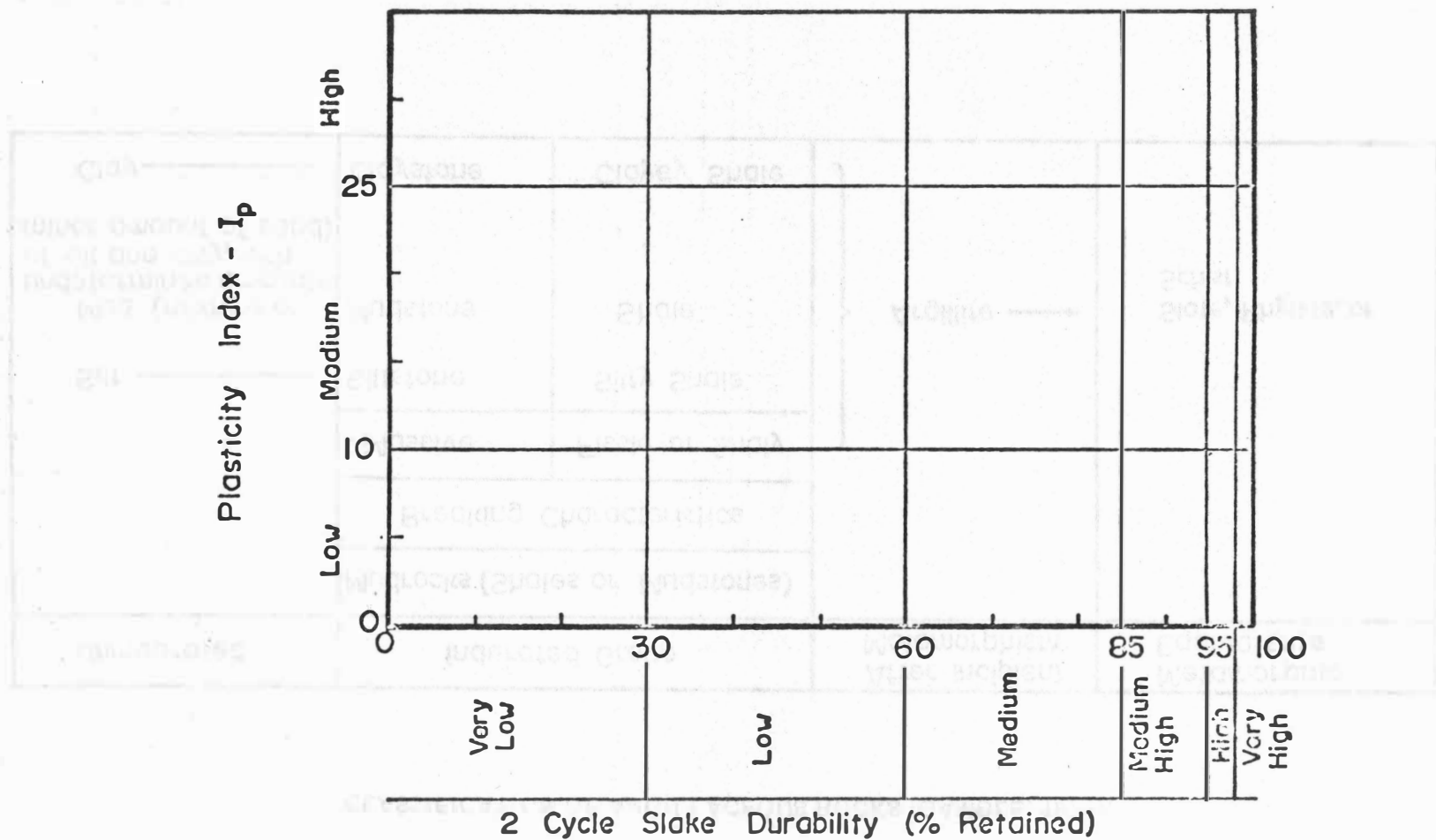


Figure 2.2: Gambles durability-plasticity classification for shales and other argillaceous rocks (after Gamble, 1971).

Table 2.2:

CLASSIFICATION OF ARGILLACEOUS ROCKS (GAMBLE, 1971).

Unindurated	Indurated Group		After Incipient Metamorphism	Metamorphic Equivalents
<p>Silt →</p> <p>Mud (mixture or undetermined amounts of silt and clay, with minor amount of sand)</p> <p>Clay →</p>	Mudrocks (Shales or Mudstones)		<p>Argillite →</p>	<p>Slate, Phyllite, or Schist</p>
	Breaking Characteristics			
	Massive	Fissile or Shaly		
	<p>Siltstone</p> <p>Mudstone</p> <p>Claystone</p>	<p>Silty Shale</p> <p>Shale</p> <p>Clayey Shale</p>		

ogy of the sampled tested. Shales were disaggregated for Atterberg limits and grain size distribution using ultrasonic equipment. This treatment effected changes in grain size distribution, plasticity characteristics, X-ray diffraction patterns, and shale fabric. The duration of the recommended treatment was one hour. The percentage of less than two micron clay was determined to be important in the appraisal of the engineering performance of shales. Based on percentages of silt and clay present, a shale is classified as a "problem" or "no problem" shale. Problem shales, by definition, have more than 40 percent of combined silt and clay. The index properties for problem shales are determined using ultrasonically disaggregated material.

An additional classification scheme, directed specially toward argillaceous materials, was recently reported by Morgernstern and Eigenbrod (1974). The primary objective of the study was to facilitate determination of variations in strength and water deterioration characteristics of shales as a basis for their classification. The method developed is based on the principle that a soil will disintegrate when exposed to water in an unconfined manner while a rock will not.

ENGINEERING PROPERTIES OF SHALES

The literature related to the engineering properties of shales is limited. Since the shales in this study contained large amounts of clays and silts, the literature pertaining to the engineering properties of compacted clays and silt clays becomes relative and pertinent and therefore it is presented herein.

ATTERBERG LIMITS

The liquid limit and plastic limit values characterize the soil water relationship. The values are influenced by chemical, mineralogical, and physical characteristics of soil constituents (Sled, Woodward and Lundgren, 1964). For shales, the limit values can also indicate the state of the shale or the degree of weathering between initial indurated condition and ultimate breakdown. The Atterberg limit values are also influenced by the method of sample preparation. Ultrasonic treatment increases the liquid limit and plasticity index values (Laguros, 1972). Natural weathering of shales also influences the limit values in a similar manner (Laguros, Kumar and Annamalai, 1974).

MOISTURE DENSITY AND VOID RATIO

The natural moisture content and bulk densities of shales vary with the degree of weathering. Underwood (1967) discussed two types of shales; "favorable", having low moisture content (5 to 15%) and high, dry density (110 to 160 pcf), and "unfavorable", having high moisture content (20 to 35%) and low density (70 to 100 pcf). Void ratio, which is inversely related to the dry density, increases with weathering.

STRENGTH

The compressive strength of shales varies over a very wide range, from less than 25 psi for weaker compacted shales to more than 15,000 psi for well cemented shales. It depends on the amount of compaction, type and amount of cementing agents, particle orientation, and moisture content. Weathering would adversely affect the above variables and decrease the strength of soils.

The structure of compacted clays has a decided effect on compressive strengths of soils. Lambe (1958), considering possible arrangements com-

posed of single clay plates, proposed a mechanistic model for the structure of compacted clays, as shown in Figures 3a and 3b. Different visualization of clay structures by various authors are given in Figure 2.3 (Tan, 1958; Aylmore and Quirk, 1960; Olsen, 1962; Young, 1971; Borden and Sides, 1971; Borden, 1972). The strength of clays is also influenced by compaction, thixotrophy, physicochemical properties and test conditions.

The shear strength of compacted clays is influenced by the nature of the compactive effort, amount of water content and amount of clay size fractions of soil particles (Seed and Chan, 1959). Shear strains produced during compaction are responsible for different effects due to the various methods of compaction. Borden and Sides (1970) noted that a flocculated structure results with compaction on the dry side of the optimum and dispersed structure when compaction takes place on the wet side of the optimum. The method of compaction is influenced when soil is compacted dry of optimum. For compaction wet of optimum, the flocculation will occur under all compaction effort but the degree of particle orientation will be dependent upon shear strains produced by compaction (Seed and Chan, 1959).

Elapsed time after compaction can cause significant changes in the properties of compacted clays. Mitchell (1964) suggested that the time between sample preparation and testing should be controlled such that all tests are equally affected by thixotrophy. The bulk of thixotropic stiffening occurs within the first two weeks with the rate of stiffening determined by the type of clay, the molding water content, and the electrolyte concentration (Gray and Kashmeeri, 1971).

Various physicochemical factors, such as double layer osmotic

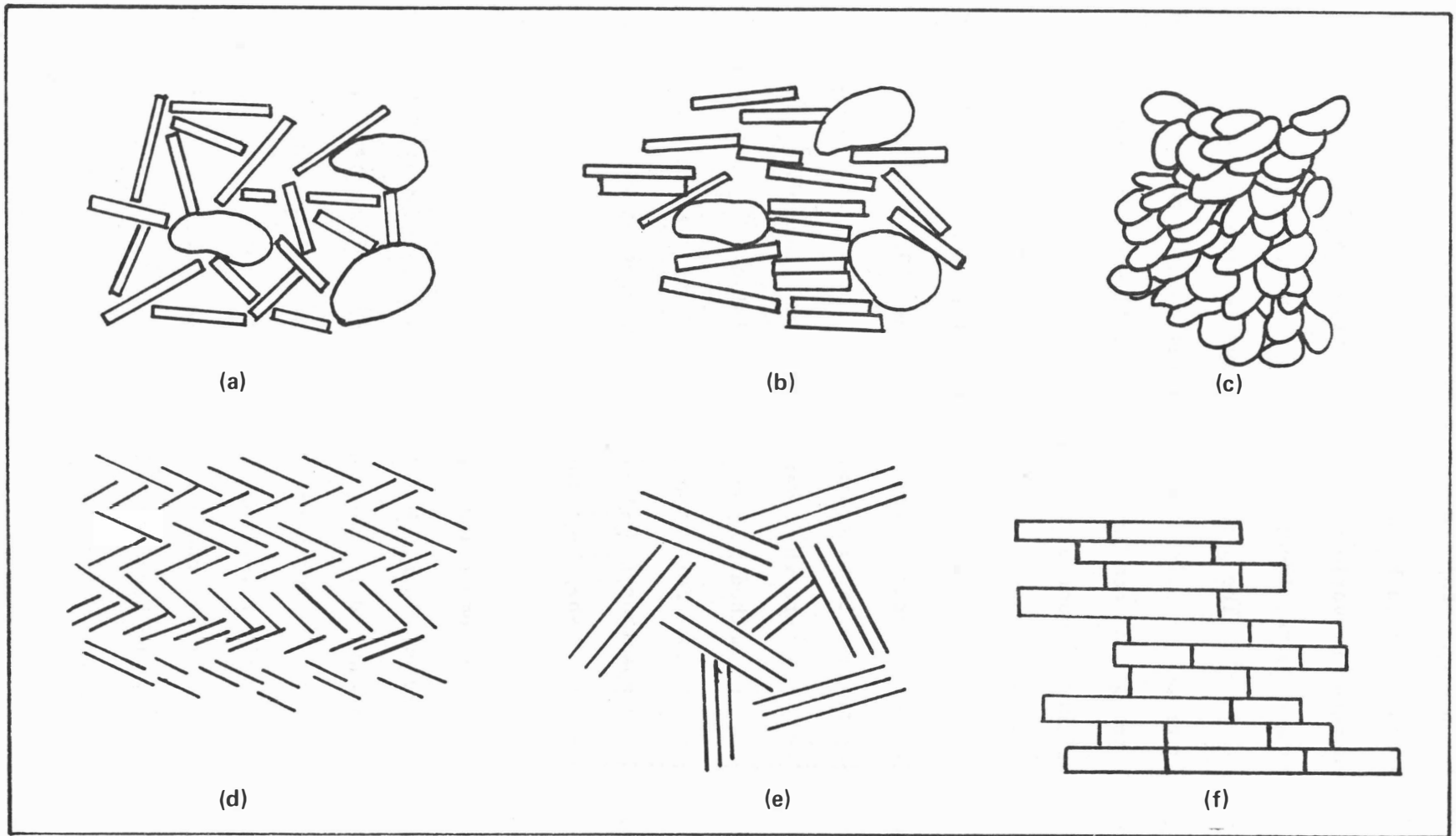


Figure 2.3: Idealized clay structure, (a) card house, and (b) dispersed, after Lambe (1958), (c) clay model, after Tan (1958), (d) turbostratic, after Aylmore and Quirk (1960) and (e) book house, and (f) stack, after Barden and Sides (1971).

expulsive force, Van der Waals attractive force and cementation of some natural clays due to carbonate, iron oxide and possibly organic matter have an influence on the shear strength of clays (Ladd and Kinner, 1967). After studying the shear strength of montmorillonite, Mesre and Olsen (1970) confirmed that the shearing resistance to the applied energy is controlled by a mutual balance of these factors. Shear strength of pure clays is also affected by the nature of the adsorbed cations.

Strength values are also affected by the method of loading, rate of loading, the strain level defining the strength, specimen size, drainage conditions, lateral pressure and base restraints (Lundgren, Mitchell and Wilson, 1968; Row and Borden, 1964; Duncan and Dunlop, 1968; Seed and Chan, 1959; Van Aukan, 1963, and Penloff and Osterberg, 1963).

From the results of conventional triaxial tests and plain strain tests, Lee and Shubeck (1968) concluded that some failure strength was obtained for samples having the same final water content, irrespective of the test methods.

ACTIVITY

Skempton (1953) and Seed, et al (1964) modified a measure of activity of clay soils. This is usually referred as the activity index and is computed according to the following formula.

$$\text{Activity Index} = \frac{\text{plasticity index}}{\% < 2 \text{ micron clay}} \quad (\text{Skempton, 1953})$$

$$\text{Activity Index} = \frac{\text{plasticity index}}{\% < 2 \text{ micron clay} - 9} \quad (\text{Seed, et al, 1964})$$

Several authors, including Underwood (1967), have determined that clay soils containing montmorillonite or illite as their predominant clay

mineral constituents have the highest activity index. Some care must be taken, however, in interpreting the results of activity indices since comparisons between shales depend on other engineering parameters, too. Results seem to vary drastically when computed by using either low PI values or low percentages of 2 micron clay.

STRENGTH MECHANISM AND FAILURE CONCEPTS

Trollope and Chan (1960) observed a stress-strain phenomenon in their measurements data. During shear the soil particles occupied stable and unstable position alternatively. The stress-strain curve consisted of a number of small jumps instead of being smooth. Moving from stable to unstable position required more energy and more stress than moving from unstable to stable positions.

Geuze (1964) assumed that the clay platelets make edge to face contacts with each other oriented at a random fashion. With the progress of shear, the tensile bonds at edge to face contacts are broken continuously. The limit of structured strength is due to the rupture of all the tensile bonds. Musyama and Shibata (1964) also considered the contacts among the clay segments to be only edge to face, bound by thin layers of adsorbed water.

Failure in clay soils may occur either by flow or by localized material rupture. Generally, soils are considered to be frictional material and the Mohr-Coulomb theory indicates that the soil will fail when either the obliquity of resultant stress reaches a maximum value on some plane in the material, or when the maximum tensile normal stress reaches a value characteristic of the material.

Yang and KcKeyes (1971), after studying the yield and failure of

clay under triaxial stresses, concluded that analytical plasticity techniques might be applied successfully to describe the undrained stress-strain behavior of a saturated clay up to a shear stress level of approximately half of the shear strength of the material. Beyond this point, the deformation behavior deviated from that of plastic material and approached that of a frictional medium, the ultimate stresses being describable in terms of the Mohr-Coulomb failure theory.

STABILIZATION OF SOILS

"The alteration of soil properties to meet specific engineering requirements is known as soil stabilization" (Ingles and Metcalf, 1973). In its specific meaning, as commonly understood in highway and airport engineering, soil stabilization describes those methods of construction in which soils are treated to provide sub-bases, base courses, and occasionally surface layers under all normal conditions of moisture and traffic for an economic service life of the paved area. The paved areas may be roads, airport aprons and runways, parking and loading places, feeding courts or other surface structure of comparable stability and durability requirements. The major established uses of soil stabilization are:

1. Lifting a country or region out of mud or out of sand for better economic development
2. Providing bases and surfaces for secondary and farm to market roads, where good primary roads are already in existence
3. Providing bases for high type pavements, where high type rock and crushed gravel normally employed for such bases are not economically available
4. For city and suburban streets where the noise-absorbing and elastic properties of certain stabilized soil systems possess definite advantages over other construction materials

5. For military and other emergencies where an area must be made trafficable within a short period of time.

Soil stabilization involves:

1. Diagnosis of the weather-resistance properties of a given soil and required deficiencies of these properties of soils in terms of physical, physico-chemical and chemical activities to meet certain engineering criteria
2. Replacement of the supplementary requirement of the above materials and processes, and decision to use the specific method
3. Construction consisting of in-place soil preparation, blending, compaction (or densification) and curing, and,
4. Economic consideration relating to the total cost including maintenance for the service life of the structure.

In general, two broad categories of stabilizing agents exist with respect to the stabilization mechanisms. The "active" stabilizing agents, a prime example of which is lime, cause chemical reactions to occur in the soil water-stabilizer system. In this category, the chemical properties of soil are very important. These properties are organic matter content, natural soil pH, predominant type of clay mineral, and, to a certain extent, texture and plasticity. The "inert" stabilizing agents, an example of which is bituminous materials, do not react chemically with the soil or aggregate, but rather provide modification or stabilization to the system by increasing the cohesion of the treated mixture or improving its waterproofing characteristics or both. In this category, the physical characteristics of the soil, such as gradation, texture, and plasticity are the predominant factors which control the extent of modification.

Many active stabilizing agents, such as cement, lime, and flyash,

display both active and inert characteristics and, therefore, both the chemical and physical soil properties should be considered. A comparison of the application techniques and response of major soil components to various types of stabilizing agents are presented in Tables 2.3 and 2.6 (Ingles and Metcalf, 1973). Since shales selected for this study contain significant amount of clay and silt, for which bituminous stabilization is not suitable and commercial petrochemical stabilizers, 'clapak' and 'claset', were not found beneficial, a detailed literature survey of the promising stabilizing agents only as lime, cement, and flyash is presented here. However, a brief description of other stabilizing agents is also included.

LIME STABILIZATION

Various forms of lime have been successfully utilized as a soil stabilizing agent for many years. According to soil lime stabilization committee report (ARMA, 1959) the use of hydrated lime (hereafter referred to as "lime" as a soil stabilizer in the roadway construction has significantly increased during the past three decades. There are only two fundamental types of lime, high calcium and dolomitic. There is little difference in the effectiveness of these two types of lime for stabilization; both have been used successfully. The chemical reaction of lime with soils is twofold. First, it agglomerates the fine clay particles into coarse, friable particles (silt and sand sizes) through a phenomenon called base exchange. Next, it produces a definite cementing of hardening action in which the lime reacts chemically with available

Table 2.3:
GUIDE TO THE UTILIZATION OF SOIL STABILIZERS
(INGLES AND METCALF, 1973).

Stabilizer	Type of soil suitable	Type of Application	Special Advantages	Special Disadvantages	Approx. cost ² per cu.m. soil treated (\$)	
Bitumen	non-cohesive SANDS	Spray; Mix	Availability	Sensitive to moisture	4, (½)	
Cement	Any	Mix	"	Rapid set (2 hours)	2½	
Electro-hardening	saturated CLAYS	Electrode wells	Works below water tables	Very accurate control needed	½	
Freezing	Any	Borehole	Security	Saline ground water bad	100	
GROUTS	Bentonite Bitumen Cement	GRAVELS, SANDS	Borehole injection	- - -	- not durable? for coarse materials only	25
	Chromelignin Silicates TACSS Acrylamide (AM-9)	SANDS, SILTS	Borehole injection	- - Sets on contact with water -	- Syneresis - Toxic	70 120 60 120
	Lime	cohesive soils, CLAYS	Mix	No premature set	-	2
Limepiles	saturated SILTS	Vibrated piles	Works below water tables	-	3½	
Membranes	Any	Lay	Easy placement	Risk of rupture	3	
Meshes	Any	Lay	Easy placement	-	3	
Polymers	SILTS and CLAYS	Mix	-	Even mixing difficult	7	
Road Oil	coarse SILTS, fine SANDS	Spray	Reworkable	Low durability	(½)	
Sandpiles	soft CLAYS	Vibrated piles	Rapid placement	-	1½	
Thermal	CLAYS	Borehole firing	-	unsuited to saturated soil	15	
Vibro-flotation	SANDS	Vibrated piles	Rapid placement	-	3½	

The cost column of this table is based on an assumed soil porosity of 0.3, and no transport costs. It provides an approximate guide only, because local conditions can greatly affect both the prime cost and the relativity. The two values in brackets refer to spray applications, for which the thin film formed can only be costed on a square metre basis.

Table 2.4:
STABILIZATION RESPONSE OF MAJOR SOIL COMPONENTS
(INGLES AND METCALF, 1973).

Dominant Soil Component	Recommended Stabilizers	Reasons
Organic matter	mechanical	other methods ineffective
Sands	clay loam cement bitumens	for mechanical stability for density and cohesion for cohesion
Silts	none known	—
Allophanes	lime*	for pozzolanic strength and densification
Kaolin	sand cement lime	for mechanical stability for early strength for workability and later strength
Illite	cement lime	as for kaolin 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 per cent may be especially favourable.

Table 2.5:

APPLICABILITY OF STABILIZATION METHODS (INGLES AND METCALF, 1973).

Designation	Fine clays	Coarse clays	Fine silts	Coarse silts	Fine sands	Coarse sands
SOIL Particle size (mm)	<.0006	.0006-.002	.002-.01	.01-.06	.06-.4	.4-2.0
SOIL Volume stability	V. poor	Fair	Fair	Good	V. good	V. good
Type of stabilization applicable	LIME	Range of maximum efficiency				
	CEMENT	Range of maximum efficiency			Effective, but quality control may be difficult	
	BITUMENS				Effective, but quality control may be difficult	
	POLYMERIC- ORGANIC		Range of maximum efficiency			
	MECHANICAL ^o		Range of maximum efficiency			
	THERMAL	Range of maximum efficiency				

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

Table 2.6:

**BROAD COMPARISON OF STABILIZATION TECHNIQUES
(INGLES AND METCALF, 1973).**

<i>In situ Material</i>	<i>Pavement Thickness</i>	<i>Mechanical</i>	<i>Cement</i>	<i>Lime</i>	<i>Bitumen</i>
Natural gravel	Min. 10 cm (4 in.)	Fines may be needed to prevent ravelling	Probably not necessary except if plastic. 2-4 per cent	Not necessary except if plastic. 2-4 per cent	Not necessary unless lacking fines. 3 per cent residual bitumen. Use medium or slow curing cut-back or emulsion
Clean sand	Min. 10 cm (4 in.)	Coarse material for strength and fines to prevent ravelling	Unsuitable; produces brittle material	Unsuitable; no reaction	Most suitable 3 per cent residual bitumen. Rapid curing cut-backs may be used. Add 2 per cent lime for wet sand
Clayey sand loam	15-25 cm (6-10 in.)	Coarse material for strength and seal adhesion	4-8 per cent	May be suitable, depending on clay content*	May be suitable. 3-4 per cent.
Sandy clay	15-35 cm (6-14 in.)	Not usually suitable	4-12 per cent	4-8 per cent depending on clay content	May be suitable for light traffic. 3-4 per cent
Heavy clay	25 cm. (10 in.)	Unsuitable	Unsuitable. Mixing may be assisted by pre-treatment with 2 per cent lime then 8-15 per cent cement	Most suitable. 4-8 per cent depending on clay content	Not usually suitable.

* Lime may render the material non-plastic if clay content is low. Usually requires less than 4 per cent lime.

silica and some alumina in the raw soil or with pozzolan additives, like flyash, forming calcium silicates and aluminates. In general, lime reacts readily with most plastic soils containing clay, either the fine grained clays or clay gravel types. These soils have plasticity index values ranging from 10 to 50%. In order for lime to react with soils having no or low plasticity, generally a pozzolan (which helps in cementation) is needed. Flyash, a waste material from coal burning boiler plants, is the commonly used pozzolan for this purpose, although others, like volcanic ash and expanded shale fines, have been successfully used. Actually, additional lime changes the physical characteristics of most clay soils. In summary these physical changes are (Wang and Handy, 1966):

1. The plasticity index drops sharply - as much as three or more - fold in some instances
2. The plasticity limit generally increases and the liquid limit decreases
3. The soil binder content decreases substantially
4. The lineal shrinkage and swell decrease markedly
5. The disintegration (break up) of clay clods during pulverization is accelerated and the soils become friable and can be worked easily
6. Unconfined compressive strength increases considerably
7. Load bearing values, as measured by the CBR test, increases substantially
8. In swamps, where soils contain water in excess of the moisture content, the application of lime facilitates drying of the soil
9. A lime stabilized base of subbase forms a water resistant barrier by stopping penetration of gravitational seepage and rapid evaporation of existing moisture of coarse grained soil.

EFFECTS OF LIME STABILIZATION

The effects of lime as a stabilizer are generally well known and may be grouped into the different property modification which it normally is expected to bring about.

1. Short term reduction of plasticity (Figure 2.4)
2. Short term reduction of swelling
3. Short term reduction of permeability
4. Long term increase in strength

Such modifications have made lime an ideal stabilizing agent for highway bases and subbases constructed in clayey soil areas.

SHORT TERM EFFECTS (AMELIORATION)

Short term effects of lime soil mixtures are those expected during the first hour of mixing. These effects are normally produced by the addition of small percentage of lime, usually no more than three percent by dry weight of soil fraction. In the past, the amount of time required to produce all of the amelioration effects noted above without providing for long-term strengthening has been referred to, perhaps mistakenly, as the "lime fixation point" or "lime retention point" (Hilt and Davidson, 1961; Ho and Handy, 1963).

In other cases, it was thought to be that point at which the resulting floc size became maximum (Pinto, Davidson, and Laguros, 1962). It has further been associated with being a pH point just short of the required to cause breakdown of silica and alumina complexes and clay minerals (Eades and Grim, 1966).

Eades and Grim (1966), after considerable review of the lime

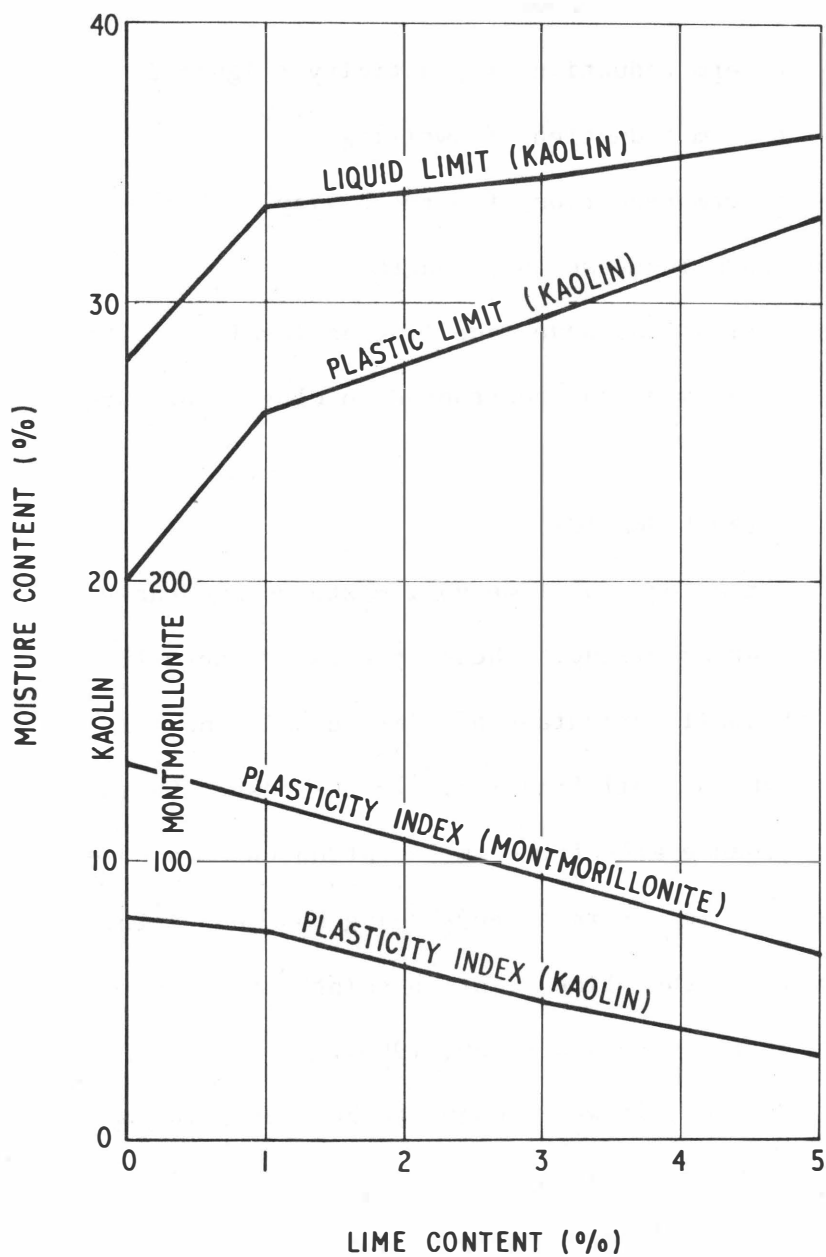


Figure 2.4: Effect of lime content on plasticity. Note, however, that kaolinitic soils often show an increase in plasticity on lime treatment (after Ingles and Metcalf, 1973).

fixation phenomenon, devised a pH dependent test to evaluate the "point" experimentally. They suggested this as a method for determining the lime requirement for "stabilization" of a soil. Because of the considerable difference of opinion as to the definition and significance of this point, the authors elected to define their lime requirement point as that percentage of lime "consumed" by the soil during the first hour of following mixing, that is, the percentage at which a pH versus lime curve for the soil becomes asymptotic (Anessi, 1970). The resulting percentage might have been somewhat larger than that amount which would have resulted through the use of Ho and Handy (1963) "lime retention" approach, since sizeable strength increases were observed by Eades and Grim (1966) after long time curing. This might be attributed, at least partially, to the fact that the pH method was based on an asymptotic point as a pH of 12.2 to 12.4 (Figure 2.5). A soil lime mixture tends to continue increasing in strength until pH values are well below 11. A pH of this value is sufficient to dissolve silica and form calcium silicate hydrate.

Several recent studies (Diamond and Kinter, 1966) clarify the soil-lime mechanism and have resulted in several interesting interpretations.

1. Lime is absorbed physically and at a rapid rate onto most clays from the mix. This absorption removes both calcium and hydroxyl ions from solution concurrently, disagreeing with the calcium crowding concept suggested by the aforementioned Iowa State investigators
2. The total amounts of lime to be absorbed in these tests tends to confirm with values of lime fixation points and to sorption of a little more than a monomolecular layer of calcium hydroxide on the external surface of the clay

Extension of the above discussion led to the proposal that instan-

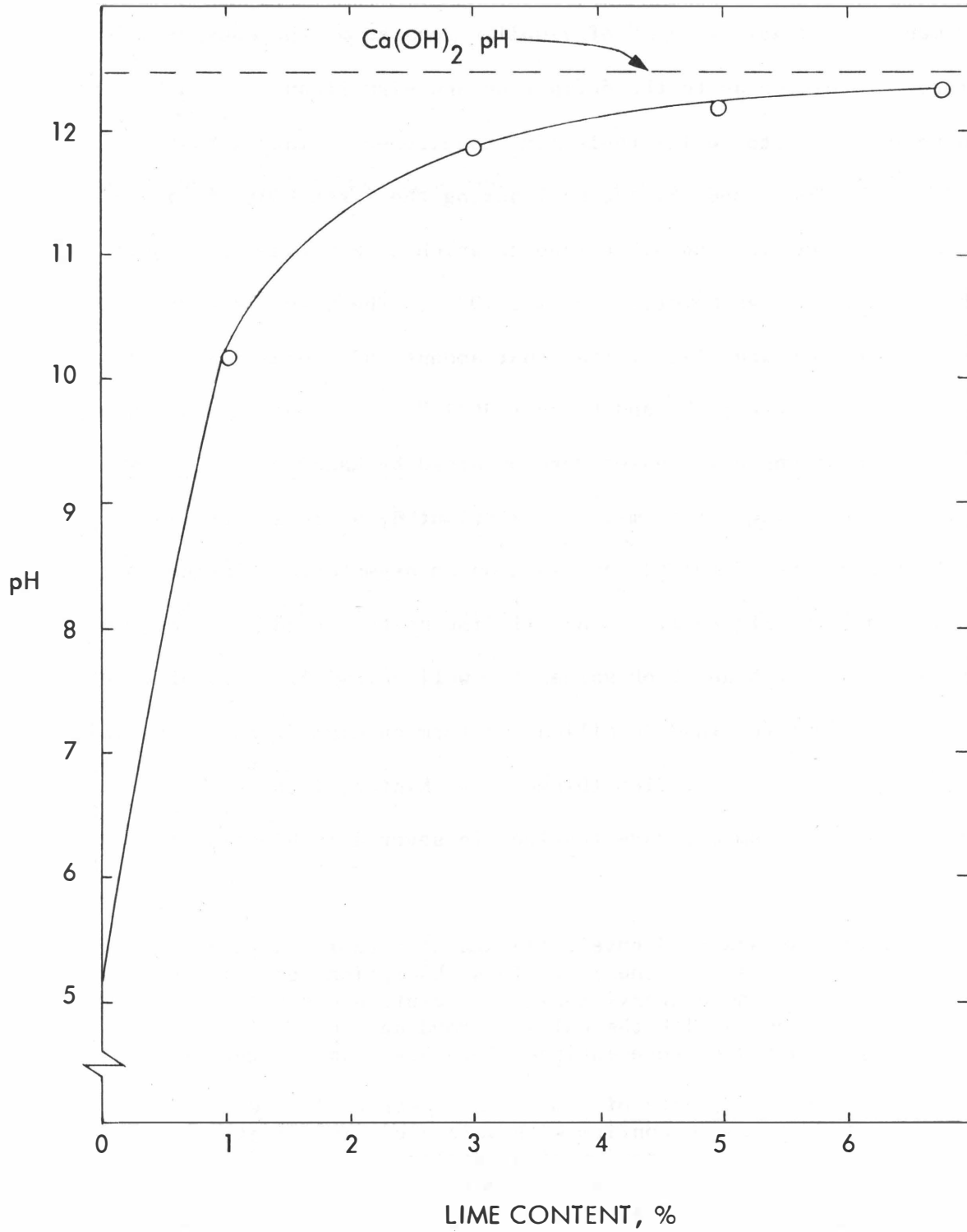


Figure 2.5: Lime determination for soil stabilization using data of Eades and Grim, 1966.

taneous formation of tetracalcium aluminate hydrates (C_4AH_{13}) at edge to face points causes an almost immediate flocculation of particles and thereby increases the effective particle size, as well as establishing a structure which impedes swelling or shrinkage activity which might result from variation of soil moisture content.

LONG TERM EFFECTS OF LIME STABILIZATION

The portion of the reaction between lime and soil which primarily takes place after the first hour or so of curing has been called "long term", "delayed", or "pozzolanic" reaction. It consists, primarily of pozzolanic changes to the mineral although Diamond and Kinter (1966) feel that even this is insufficient as a complete explanation for what occurs during this period. It consists of the slow formation of poorly crystallized hydrated calcium silicates and aluminates which seem to strongly cement constituent particles together. Whether the reaction is between liberated silica and lime or between the lime and clay surface appears to be a bone of contention among various authors. Experimental evidence indicates that the predominant reactions take place between the clay mineral fraction and that little or no reaction takes place with other major constituents of soil such as quartz (Glenn and Handy, 1963).

A list of reaction products compiled from numerous tests is presented in Table 2.7. The types and amounts of product are apparently dependent on the mineralogy of the constituents, the type of lime, the temperature of cure and the amounts of water present. This is the result of investigations conducted on standard clays at high water and lime contents and often at high temperature curing in order to assure adequate

Table 2.7:

SUMMARY OF CRYSTALLINE PRODUCTS OBSERVED FROM X-RAY ANALYSIS^a (ANESSI, 1970)

Curing Temp. °C	1 Day	3 Days	7 Days	14 Days	28 Days	56 Days
5	A?,G	A?G,II	A?,G,II	A?,G,II	A?,G,II	A,G,II
23	A,G	A?G,II	A?,G,II	A?,G,II	A?,G,II	A,G,II
40	A?,G	A?G,II	A?,G,II	A?G,II	A?,G,II	A,G,II
50	A?,G	A?G,II	A,Ga,IIa	A,Ga,IIa	A,Ga,Ia,IIa	N.D.
60	A?,Ga,IIa	A,G,II	A,Ga,IIa	A,Ga,Ia,IIa	N.D.	N.D.

Notes:

A--C₄A'H_n

C--CaO

G--CSH(gel)

A'--Al₂O₃

I--CSH(I)

H--H₂O

II--CSH(II)

S--SiO₂

?--Probable

a--Probable Lattice substitution

N.D.--Not determined

^aAfter Ruff and Ho (1966)

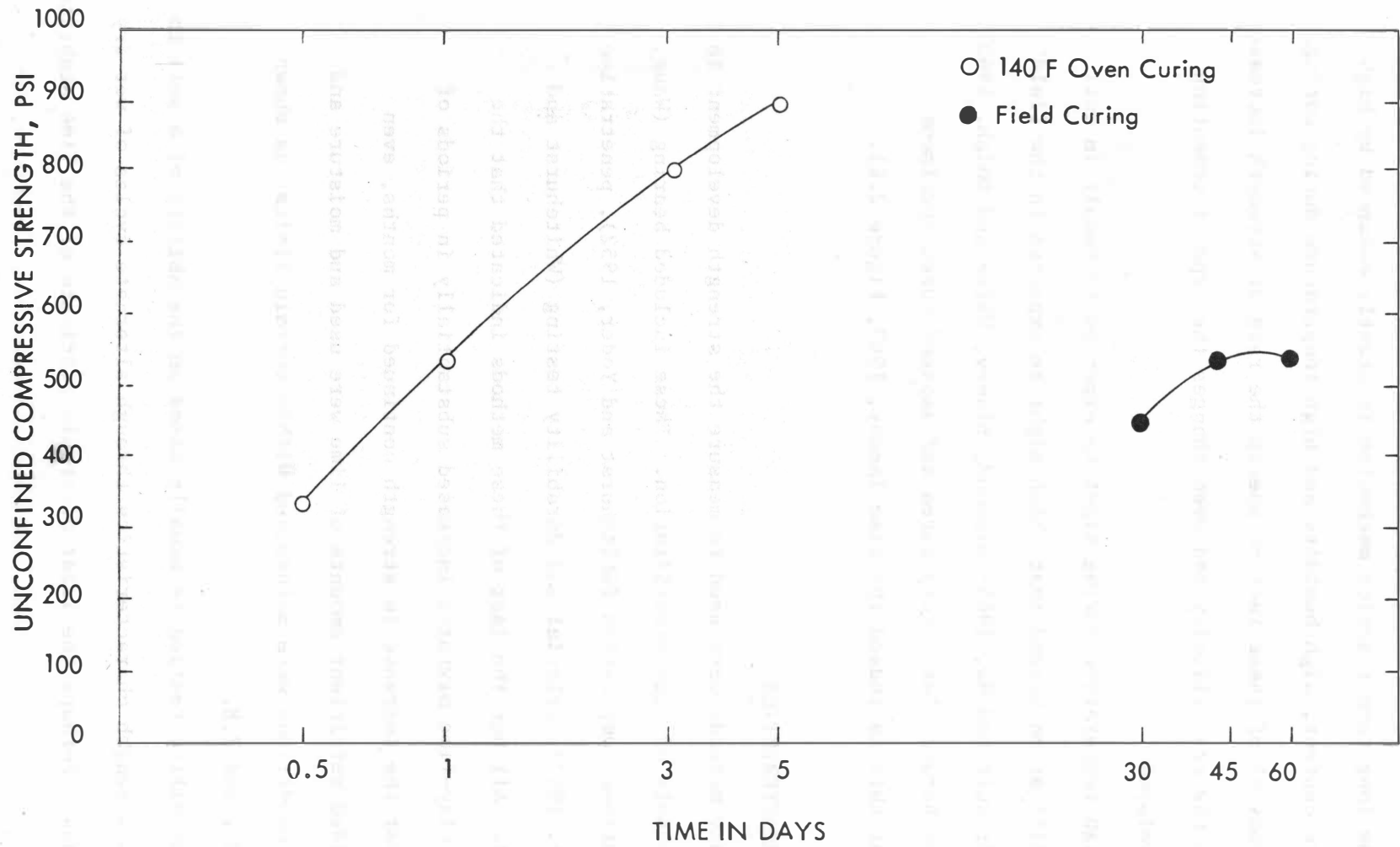


Figure 2.6: Effect of high temperature and field curing on lime stabilized soil, (after Anday, 1963).

amounts reaction product for X-ray diffraction testing (Anessi, 1970).

The long term reaction mechanism is greatly enhanced by high soil moisture content, high humidity and high temperature during curing. Laboratory control of these factors speeds the rate of strength increase and improves the crystallinity and even changes the type of cementing products developed.

High temperature curing might be expected to result in soil strength modification beyond that which might be expected in the field (Anday, 1963; Ruff and Ho, 1965; Diamond, Sidney, White and Dolch, 1963). Relationships between laboratory cured and ambient cured specimens indicate that this is indeed the case (Anday, 1963, Figure 2.6).

STRENGTH CHARACTERISTICS

Many methods were used to measure the strength development in soils as a result of lime stabilization. These included bearing (Wang, 1966), unconfined compression (Whitehurst and Yoder, 1952), penetration (Freeborough, 1947), triaxial and durability testing (Whitehurst and Yoder, 1952). All but the last of these methods indicated that the strength of clay-lime mixtures increased substantially in periods of weeks and that the increase in strength continued for months, even years, provided sufficient amounts of lime were used and moisture and temperature conditions were maintained within certain limits, as shown in Figures 2.7 and 2.8.

Durability testing is usually based on the ability of a soil to maintain its strength characteristics through alternate cycles of wet-dry or freeze-thaw. Perhaps the least desirable attribute of the lime stabilized

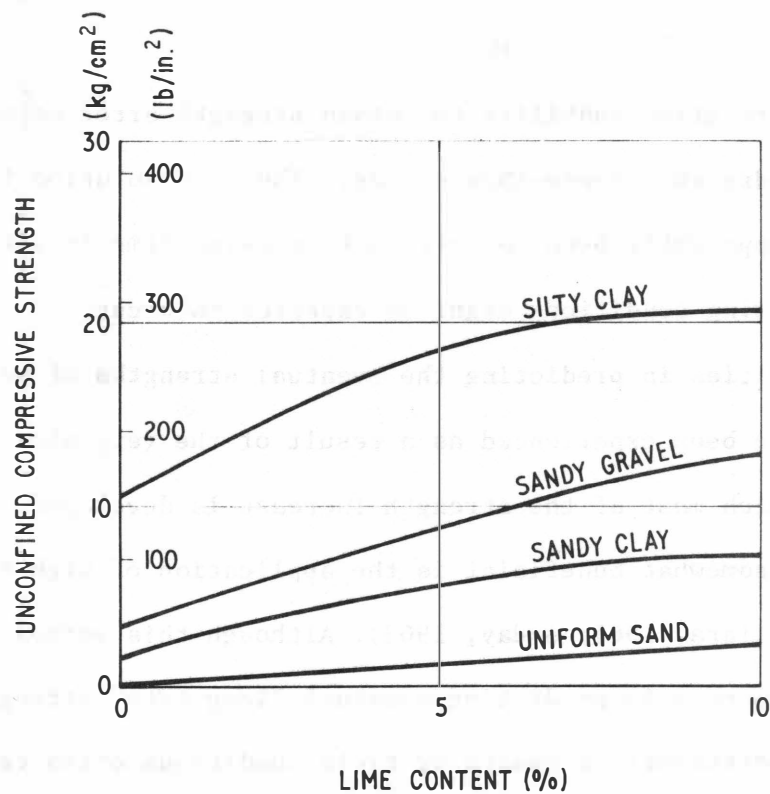


Figure 2.7: Effect of lime content on strength for various soils stabilized with hydrated lime, cured for seven days at 25°C, constant moisture content (after Metcalf, 1973).

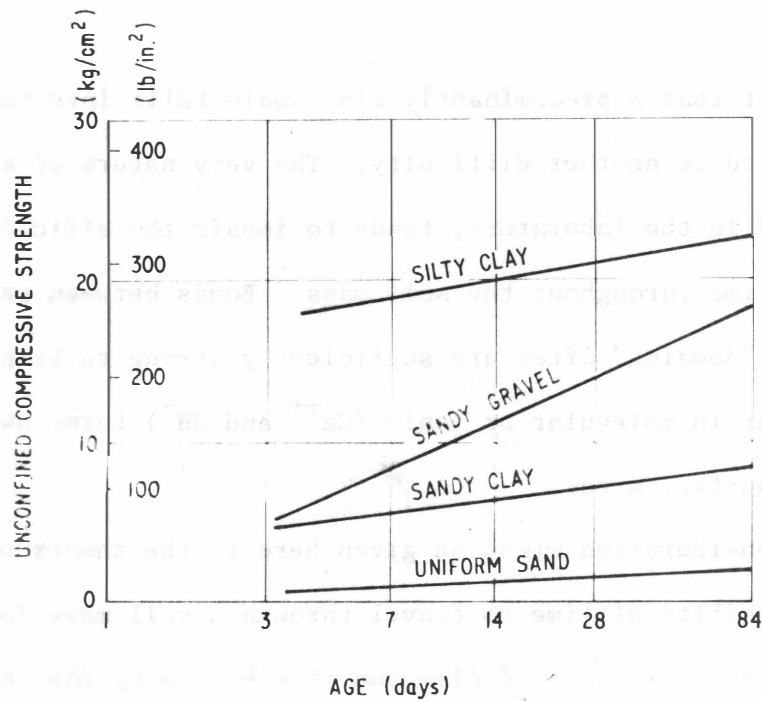


Figure 2.8: Effect of age on strength of various soils stabilized with 5% hydrated lime (after Metcalf, 1973).

clay soil is its relative inability to retain strength after being subjected to wet-dry and freeze-thaw cycles. The best solution for this shortcoming has apparently been to refrain from using lime in situations where such weathering conditions might be expected to occur.

Difficulties in predicting the eventual strengths of soil lime mixtures have been experienced as a result of the very slow curing process during which most of the strength increase is developed. A method which has proven somewhat beneficial is the application of high temperature curing (Ruff and Clara, 1965; Anday, 1963). Although this method was met with reasonable success in predicting eventual "long term" strengths in the laboratory, perturbations caused by field conditions often result in substantial deviations of developed strength from those predicted. Despite these difficulties, laboratory results, when interpreted correctly, provide at least an approximate idea of the eventual strength of the stabilized soil.

The fact that a predominantly clay shale falls into the soil category may introduce another difficulty. The very nature of shales, even when treated in the laboratory, tends to impair the efficient distribution of lime throughout the soil mass. Bonds between particles in "clusters" or "domains" often are sufficiently strong to keep water borne lime, either in molecular or ionic (Ca^{++} and OH^-) form, away from the total available surface area.

Some consideration might be given here to the theory of lime migration, that ability of lime to travel through a soil mass for long distance over extended periods of time and thereby modify that soil significantly (Davidson, Demirel and Handy, 1965). Despite some success

for this theory, lime migration seems significantly dependent on the ability of water to carry the lime through the soil medium, that is, its permeability. Because of the extremely low permeabilities of most shales, lime migration effects are probably minimal, if experienced at all.

CEMENT STABILIZATION

Cement stabilization was started as early as 1915 by an enterprising paving contractor in Sarasota, Florida (Davidson, 1962). During the 1920's several state highway departments, including Iowa, South Dakota, Ohio, California and Texas, experimented with mixture of soil and cement for paving. However, because of lack of knowledge of the application of soil-science to road building the experiments produced unpredictable results. In 1932, the South Carolina State Highway Department began investigations of mixtures of soil and cement under the leadership of the late Dr. C. H. Moorfield, then Chief Highway Commissioner. Several test sections were built in 1933 and 1934. The performance of these test sections showed that soil and cement were compatible materials and that they could be mixed together to form a low cost base material for roads.

The excellent work of the South Carolina State Highway Department stimulated more studies by them and an extensive research program by the Portland Cement Association initiated in January 1953 under the direction of F. T. Sheets, Consulting Engineer and M. D. Catton, Development Department. Cement came with the discovery in 1929 of the moisture-density relationship in soil compaction. To confirm results of the laboratory results, the South Carolina State Highway Department, Bureau of Public Roads and Portland Cement Association cooperated in construction

of one and a half mile section of pavement near Johnsonville in the fall of 1935. This project became known as the first "engineered" soil-cement road.

The success of the Johnsonville, South Carolina test road led to the additional experimental sections in 1936 in South Carolina and in Illinois, Michigan, Missouri and Wisconsin. By 1940 over seven and a half million square yards of soil-cement had been built in the United States, mostly on roads and streets. During the war period, 1941-44, 22 million square yards of soil-cement airports were built, whereas relatively few roads were built.

By 1960 the annual use of soil-cement in the United States and Canada reached 46 million square yards with a total constructed yardage to that date of almost 294 million square yards.

In addition to the widespread use in the United States, soil cement has been used extensively in England, South Africa, the Middle East, South America and Germany. The rapidly expanding applications of mixtures of soil and cement have resulted in the use of several different terminologies such as soil cement, cement treated base, cement modified soil and plastic soil-cement.

CEMENT REQUIREMENT OF SOILS

According to the American Road Builder's Association (1953), the proportion of cement to soil required to produce a satisfactory soil cement mix for each type of soil is determined from the results of tests performed under carefully controlled laboratory conditions. These tests are as follows:

1. Those necessary to classify the soil according to the

system of classifications of the U.S. Bureau of Public Roads

2. The preparation of trial mixtures containing varying proportions of soil to cement and the determination of the "optimum moisture content" and "maximum density" for each mixture
3. Two sets of specimens containing the optimum percentage of water are then molded from material representing each of the trial mixtures. One set of specimens for each trial mixture is subjected to the wetting and drying and other set is subjected to freezing and thawing test (12 cycles, ASTM method)

From the results of these tests, the percentage of cement required to produce a satisfactory mixture is determined by comparing the percentage losses during the test with the following allowable maximum:

Table 2.8:
CEMENT REQUIREMENT FOR SOILS

PRA Soil Group	Revised USBPR Soil Group	Allowable Soil Cement Loss*
A-2	A-1-a	14%
A-3	A-1-b	
	A-2-4	
	A-2-5	
	A-3	
A-4	A-2-6	10%
A-5	A-2-7	
	A-4 A-5	
A-6	A-6	7%
A-7	A-7-5	
	A-7-6	

*Resulting from 12 cycles of either the ASTM wet/dry or freeze-thaw test

Based upon world-wide questionnaire survey on the State-of-art of soil-P.C. stabilization cement requirements by AASHTO, soil groups and other soil types are in Table 2.9.

Table 2.9:

CEMENT REQUIREMENTS FOR SOILS

AASHTO Soil Group or Miscellaneous Soils	Estimated Cement Content and that used in M-D tests, % by weight	Cement Content for W-D and F/T tests, % by weight
A-1 ^a (Gray A horizon, add 4%)	5	3-5-7
A-1 (Black A-horizon, add 6%)	6	4-6-8
A-2	7	5-7-9
A-3	8	7-9-11
A-4	10	8-10-12
A-5	10	8-10-12
A-6	12	10-12-14
A-7	13	10-13-15
Shell Soils	7	5-7-9
Limestone Screenings	5	3-5-7
Red-Dog	8	6-8-10
Shale	10	8-10-12
Caliche	7	5-7-9
Cinders	8	6-8-10
Air-cooled Slag	7	5-7-9
Water-cooled Slag	12	10-12-14

Cement requirements for different types of soil recommended by the Oklahoma Department of Transportation are given in Table 2.10.

DIFFERENT STEPS IN CEMENT STABILIZATION

1. Reduction of Plasticity

The first noticeable property change that occurs when cement is mixed with moist cohesive soils is a marked reduction in plasticity, probably caused by calcium ions released during the initial cement hydration reaction. The mechanism is either a cation exchange or a crowding of additional cations onto the clay, both processes acting to change the electrical charge density around the clay particles. Clay particles then become electrically attracted to one another, causing flocculation and aggregation. The aggregated clay behaves like a silt, which has a low plasticity and cohesion. Aggregation takes place rather quickly, and is caused by the addition of relatively small amounts of cement.

2. Cementation

In compacted cement treated soil the hydration of the different cement constituents occurs at a different rate, providing cementitious, amorphous, and minutely crystalline hydration products responsible for the characteristics of early and long term strength gains. The cementation is mainly chemical in nature and may be visualized as due to the development of chemical bonds or linkages between adjacent cement grain surfaces, and between cement grain surfaces and exposed soil particle surfaces. With cohesive soils, an important part of the mechanism may be the hardening of clay aggregations by lime liberated as a result of the hydration of cement. This would explain both the hardened condition of aggregations

Table 2.10:

ESTIMATED CEMENT REQUIREMENT FOR OKLAHOMA SOILS*

AASHTO Class	% Cement by Dry Weight of Soil							
	% Pass 200 Sieve							
	0	5	10	15	20	25	30	35
A-1-a	7	7	6	-	-	-	-	-
A-1-b	9	8	8	8	7	7	-	-
A-2-4	9	9	9	8	7	7	8	9
A-2-5	9	9	8	8	8	8	8	8
A-2-6	10	10	9	8	8	8	8	9
A-2-7	11	11	10	9	9	9	10	10

SHALES

A-1, A-2, A-3 --Add 2% cement

A-4, A-5, A-6, A-7 --Add 1% cement

AASHTO Class	% Cement by Dry Weight of Soil						
	Group Index						
	0-2	3-5	6-8	9-11	12-14	15-17	18-20
A-4	9	10	11	-	-	-	-
A-5	9	10	11	11	12	-	-
A-6	10	11	12	12	13	14	-
A-7-5	11	11	12	13	13	14	16
A-7-6	11	12	13	14	14	15	17

* Research and Development Division
Oklahoma Department of Highways, 1966

observed where lumps of stabilized soil are removed from a road base sometime after construction and the magnitude of the increase in strength after the hardening of the cement bonds would have been expected to be complete.

3. Fine Grained Soils

The manner in which portland cement stabilizes soils to meet requirements for soil cement differs somewhat for the two principal types of soils. In the fine grained silty and clayey soils, the cement, on hydration, developed strong linkages among and between the mineral aggregates and the soil aggregates to form a matrix that effectively encases the soil aggregates. The matrix forms a honeycomb type of structure on which the strength of the mixture depends, because the clay aggregation within the matrix have little strength and contribute little to the strength of soil cement. The matrix is effective in fixing the particles so they can no longer slide over each other. Thus, the cement not only destroys the plasticity but also provides increased shear strength. The surface chemical effect of the cement reduces the water affinity and thus the water-holding capacity of the clayey soils. The combination of reduced water affinity and water-holding capacity and a strong matrix provide an encasement of the larger unpulverized raw soil aggregates. Because of its strength and reduced water affinity, this encasement serves not only to protect the aggregates but also to prevent them from swelling and softening from absorption of moisture and from suffering detrimental freeze-thaw effects.

4. Granular Soils

In the more granular soils the cementing action approaches that in concrete, except that the cement paste does not fill the voids in the aggregate. In sands, the aggregates become cemented only at points of contact. The more densely graded the soil, the smaller the voids, the more numerous and greater the contact areas, and the stronger the cementing action. Uniformly graded sand (one-size) sand, which has a minimum of contact area between grains, requires a fairly high cement content for stabilization. Because well graded granular soils generally also have a low swell potential and low frost susceptibility, it is possible to stabilize them with lesser cement contents than are needed for uniformly graded sands, the more frost-susceptible silts, and the higher swelling and frost-susceptible clayey soils. For any type of soil, the cementing process is given the maximum opportunity to develop when the mixture is highly compacted at a moisture content that facilitates both the densification of the mix and the hydration of the cement.

DEGREE OF STABILIZATION

Four major variables control the degree of stabilization of soils with cement:

1. the nature of the soil
2. the proportion of cement in the mix
3. the moisture content at the time of compaction
4. the degree of densification attained in compaction

If the moisture content and the density are controlled in accordance with the standard methods and normal mixing and curing procedures are observed, the nature of the soil and the proportion of cement used determine the degree of stabilization. It is possible, simply by varying the cement content,

to provide mixes that after hydration of cement may range from those that result in only a slight modification of the compacted soil (cement) minimum durability and strength requirements. When moisture is increased sufficiently to produce a plastic mix, and the cement content adjusted to meet strength and durability requirements for the plastic condition, the product becomes plastic soil cement. The ability to control the properties of the mix to suit the construction and to control the degree of stabilization to satisfy the strength and durability requirements has resulted in the development of these three principal types of cement treated soil (soil cement, cement modified soil and plastic soil cement).

EFFECT OF CEMENT STABILIZATION

For cement-treated soils the optimum moisture content for maximum density and the optimum moisture content for maximum unconfined compressive strength are not necessarily the same (Davidson, Pitre and Mateos, 1962). Increase in compressive strength of soil-cement with time was observed by Circeo, Davidson and David (1962), which was better than random correlation in both a semi-logarithmic and logarithmic manner:

$$S = A + B \log T$$

where S = Strength

A, B = Constant

T = Time

The best relationship for granular soil cement is semi-logarithmic; silty and clayey soil cement exhibit the best relationship logarithmically. These correlations were found to exist, independently of changes in (a) cement content, (b) time of curing to five years, (c) curing temperature, (d) size of test specimen, (e) type of soil, and (f) immersion

of test specimen before testing. These relationships can be used to predict the compressive strength of soil-cement. The strength age relationships can be used to predict the compressive strength of soil-cement. These strength-age relationship can be determined from the data obtained in standard laboratory tests. The broad properties of compacted cement stabilized soils depend first on cement and secondly on compaction. In the same way as in mechanically stabilized materials, compaction is all important, and not only in degree but in timing. Compaction after cement hydration is, of course, ineffective. The properties naturally change with increasing cement content; strength and bearing capacity increase, durability to wet-dry cycles increases, permeability decreases in general but increases in clayey soils, the tendency to shrink may increase in granular soils, but the tendency of clay soils to swell will be reduced. The most common measures of the effectiveness of cement stabilization are unconfined compressive strength, and/or CBR, and the resistance to cycles of freezing/thawing or wetting/drying conditions. In general, strength increases linearly with current content, but at different rates for different soils (Metcalf, 1959), as shown in Figures 2.9 and 2.10. Of almost equal importance to cement content is density. A reduction in density of 5 pcf may cause a 20 percent reduction in strength (Figure 12). Several workers having demonstrated a relation between strength and density of the form (McClean, 1961; Ingles and Frydenan, 1966; Hutchinson, 1963; Larnach, 1960):

$$S = A e^{bD}$$

where, S is the strength, D is the density, and A,b are constants; the logarithm of strength is linearly related to density. Typical properties of cement stabilized soils are shown in Table 2.11.

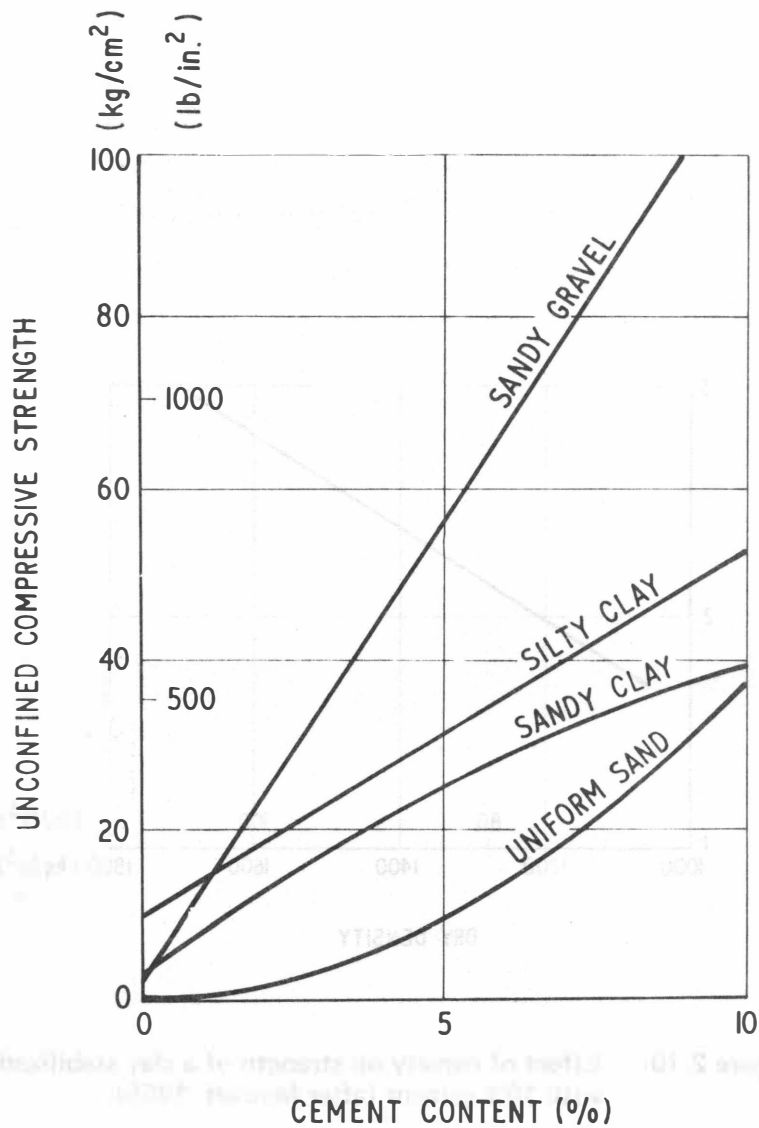


Figure 2.9: Effect of cement content on strength for various soils stabilized with ordinary Portland cement, and cured for seven days at 25°C, constant moisture content (after Metcalf, 1953).

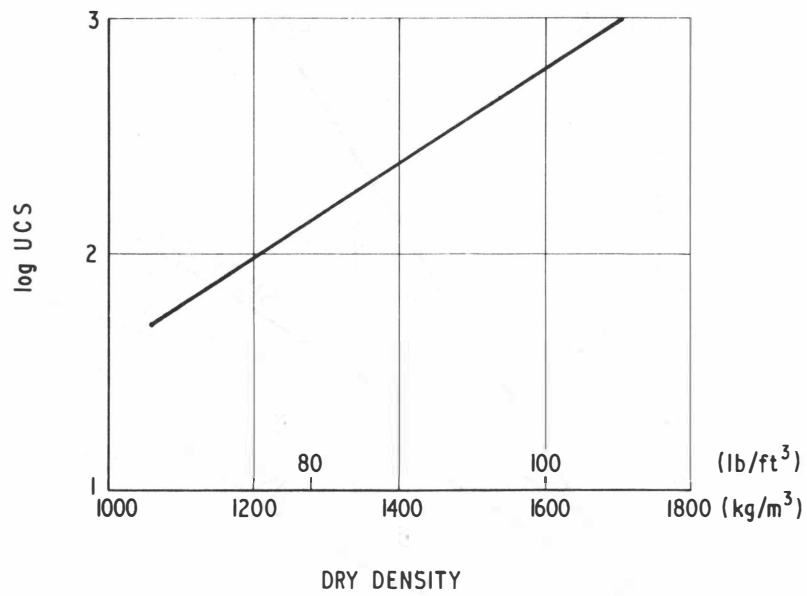


Figure 2.10: Effect of density on strength of a clay stabilized with 10% cement (after Metcalf, 1959).

Table 2.11:
TYPICAL PROPERTIES OF CEMENT-STABILIZED SOILS (INGLES AND METCALF, 1973).

Soil Type	Strength Range kgf/cm ² (lbf/in ²)	'E' Value ² kgf/cm ² (lbf/in ²)	C.B.R. ⁽³⁾	Permeability cm/sec	Thermal Expansion ⁽⁴⁾ in/in °C	Volume Change ⁽⁵⁾	Comments	Use
Well graded gravel-sand- clay sands or gravels	28-105 (400-1500) and more Ratio of wet: dry strength 1:1.5	7.21 x 10 ⁴ (1.3 x 10 ⁶)	More than 600	High Decreased by cement: 15 x 10 ⁻⁵ unstabilized 18 x 10 ⁻⁵ stabilized		Very small less than 1% (Concrete 0.1%)	Too strong Wide- spaced wide cracks Suitable for bituminous stabilization	Base for heavy traffic
Silty sands sandy clays sand and gravel	17-35 (250-500)	7 x 10 ⁴ (1 x 10 ⁶)	600	High Decreased by cement		Small	Good material	Base for heavy traffic
Silty-sandy clays poorly graded sands	7-17 (100-250)	3.5-7 x 10 ⁴ (5 x 10 ⁵) (1 x 10 ⁶)	200	5 x 10 ⁻⁵ unstabilized 0.1 x 10 ⁻⁵ stabilized	10 x 10 ⁻⁵ 7 x 10 ⁻⁵		Compaction difficulties in sands	Sub-base for light traffic
Silts silty clays very poorly graded soils	3.5-10.5 (50-150)	Less than 3.5 x 10 ⁴ (Less than 5 x 10 ⁵)	Up to 100	Low Increased by cement		Moderate		Low-grade sub-base
Heavy clays organic and sulphate rich soils	< 7 (< 100) Ratio of wet: dry strength 1:3	Up to 1.4 x 10 ⁴ (Up to 2 x 10 ⁵)	Up to 50	Very low (10 ⁻⁷) Increased by cement (10 ⁻⁵)	10 x 10 ⁻⁶	High >4% May be increased by cement	Extreme diffi- culty in mixing Use of lime could be benefi- cial. Special treatment for organic and sulphate soils	possibly for upgrading subgrades

NOTES

- (1) Strength given as approximate figure for 7 days cure at constant temperature and moisture content appropriate economic cement content, density and moisture content levels. Ratio of U.C.S. to flexure about 4:1 in sand, 3:1 in kaolin. Ratio of U.C.S. to I.T.S. about 10:1
- (2) From flexure tests. Ratio of static to dynamic values about 1:1. Poissons ratio ranges from 0.1 to 0.3
- (3) Approximate figures for mixes with 7 days U.C.S. of 250 psi (accepted U.C.S. for base construction)
- (4) Coefficient for concrete 3 to 8 x 10⁻⁶; bitumen 6 x 10⁻⁴
- (5) Only very limited data

Delay in compaction, which allows the hydration process to commence and thus build up the strength of clods, is a major cause of loss in strength because the mix becomes more difficult to compact and the final density achieved will, therefore, be lower (West, 1959). Time is also important because strength increases gradually with age of curing (Metcalf, 1959), as shown in Figures 2.11 and 2.12.

Soil cement, however, is not an elastic material (Metcalf, 1966). If a soil cement pavement fails under a wheel load, the failure may be caused by tensile stresses in the underside of the slab under the load or by surface stresses some distance from the load. The tensile strength of soil cement is, therefore, of interest and has been investigated by a number of workers. Generally, the tensile strength at the optimum moisture content and maximum density will be about 10% of the compressive strength at this condition (as shown in Figure 2.13). At low cement contents it is, of course, possible to measure a reduction in plasticity, but this becomes meaningless as cementation is more complete. It is also possible to measure shrinkage and swell and it has been shown (George, 1971) that in granular soils 20% of the shrinkage movement is due to cement hydration as opposed to drying out of the material.

COMPARISON OF LIME STABILIZATION AND CEMENT STABILIZATION

The unconfined compressive strength of soil lime mixture increases with increasing lime content to a certain level, usually about 8% for clay soils. The rate of increase then diminishes until no further strength gain occurs with increasing lime content. This is in contrast to cement stabilization where the increase in strength continues to quite high

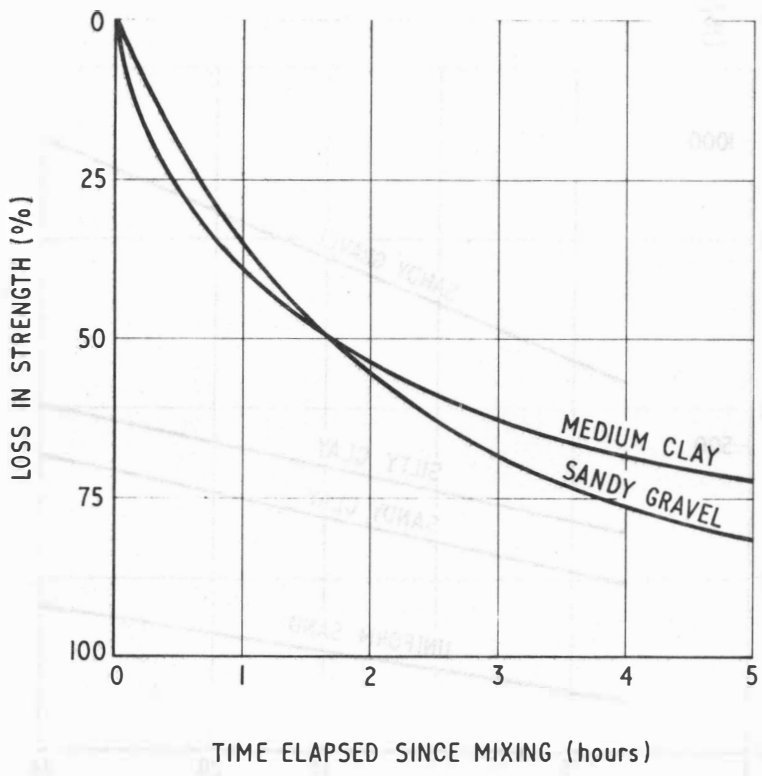


Figure 2.11: Loss in strength due to delay in compaction for two soils stabilized with 10% cement; standard compaction (after West, 1953).

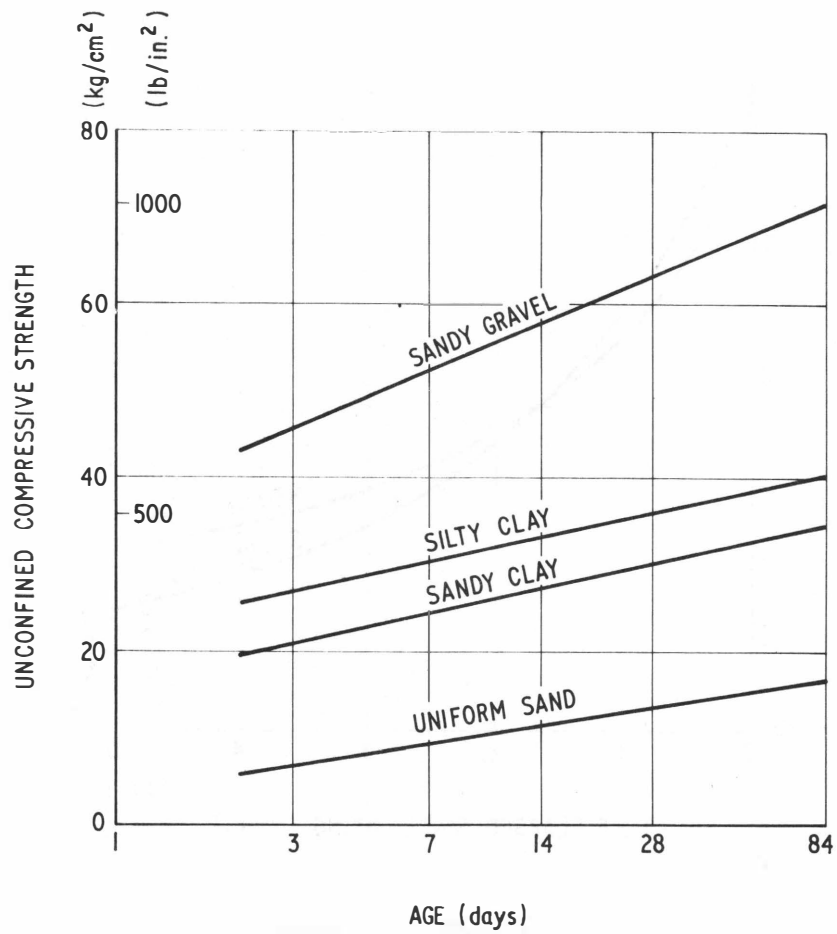


Figure 2.12: Effect of age on strength of various soils stabilized with 5% cement (after Metcalf, 1973).

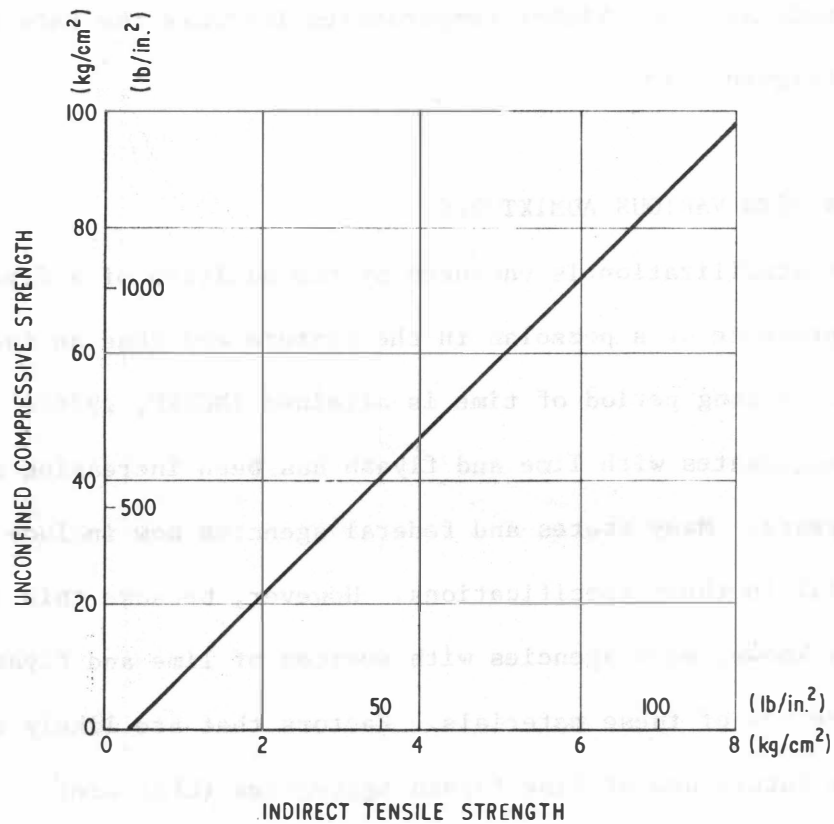


Figure 2.13: Relation between compressive and tensile strength for cement stabilized soils (after Ingles and Metcalf, 1973).

cement content (20%). Rapid increase in optimum moisture content occurs as a result of delayed compaction (Figure 2.14). It may be more economical to compact quickly than to add extra water. Time is also important because strength increases gradually with age of curing. The various ways of curing also effect strength, but the only generalization that may be made are that higher temperatures increase the rate of gain of strength (Figure 2.15).

STABILIZATION WITH VARIOUS ADMIXTURES

Lime stabilization is enhanced by the addition of a flyash, which insures the presence of a pozzolan in the mixture and thus an increase in strength after a long period of time is attained (NCHRP, 1976). Stabilizing aggregates with lime and flyash has been increasing during the past 20 years. Many states and federal agencies now include this paving material in their specifications. However, because this technology is not widely known, many agencies with sources of lime and flyash do not make extensive use of these materials. Factors that are likely to influence the future use of lime flyash aggregates (LFA) are:

1. Increase in use of coal for fuel
2. Low energy requirements for producing LFA mixtures
3. New technology for LFA use
4. Widespread availability of lime and flyash

Flyash is the fine residue that results from the combustion of coal. It has been estimated that the flyash production in the United States will approach 40 million tons annually by 1980 (NCHRP, 1976).

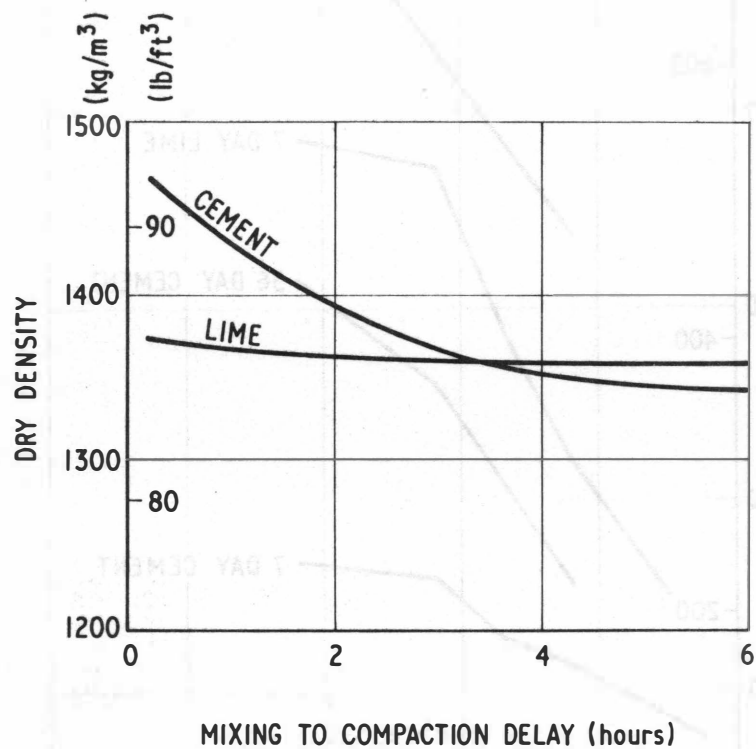


Figure 2.14: Effect of delay on the compacted density of a heavy clay, stabilizer content 10 per cent (after Dumbleton, 1962).

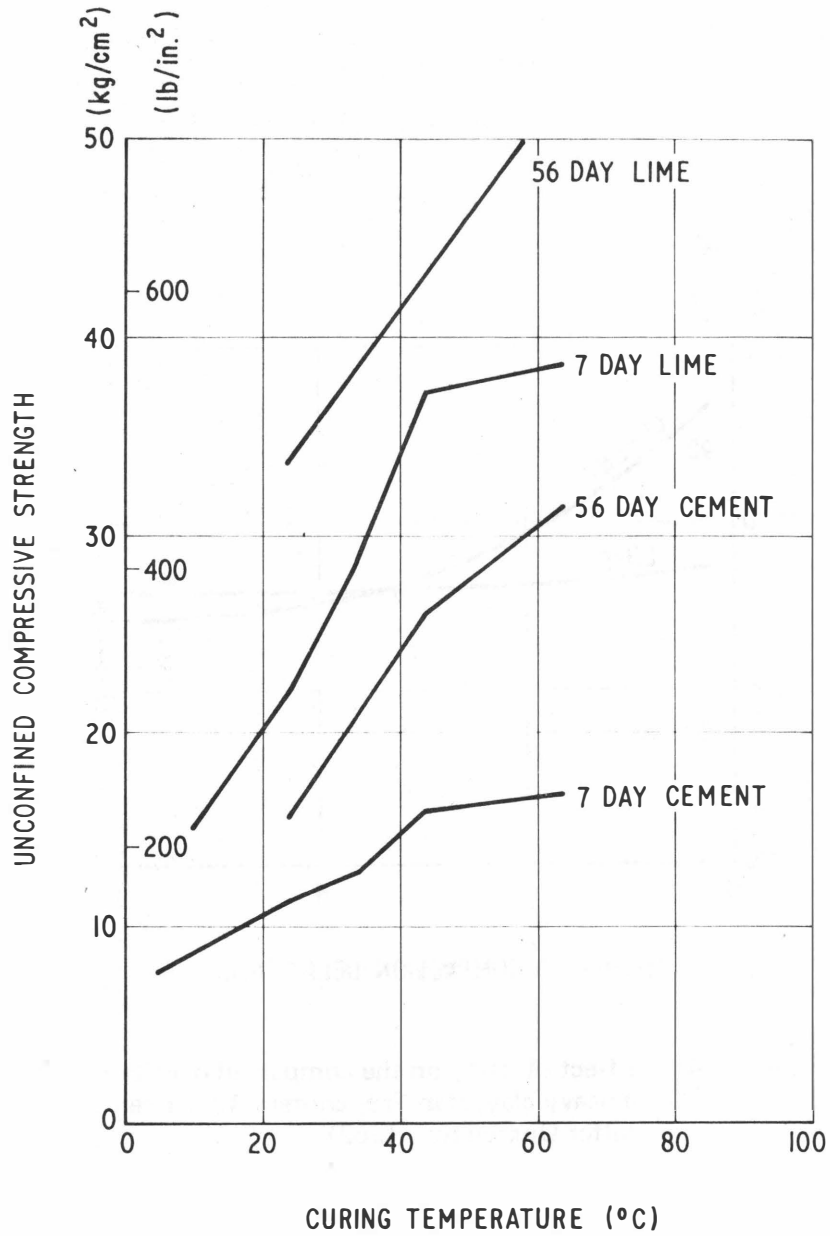


Figure 2.15: Effect of curing temperature on strength of stabilized heavy clay (after Ingles and Metcalf, 1973).

FACTORS INFLUENCING LFA PROPERTIES

The characteristics of flyash, lime, and aggregate, their relative mixing proportions, and curing periods influence the properties. This can be presented in a simple diagram as shown in Figure 2.16. Unconfined compressive strength is frequently used to evaluate the quality of cured LFA mixtures. A general range of typical strengths for various LFA mixture is given in Table 2.12. Barenberg (1974) states that standard ASTM C593 curing (7 days at 100° F.) develops mixture compressive strength ranging from about 500 to 1200 psi. The shear strength of LFA mixtures has not been extensively considered. Unconfined compressive strength data for typical mixtures indicate that shear strength failures are not likely for normal pavement applications. Typical shear strength (Mollan and Marles, 1962) for lime flyash gravel mixtures indicate that angles of shearing resistance varied from 49° to 53° and cohesion ranged from 55 to 128 psi. Mixture curing was equivalent to 28 day moist-sand curing at approximately 70 to 75° F.

STABILIZATION WITH BITUMINOUS MATERIALS

Bituminous stabilization is limited to sandy soil and poor quality base material. Two broad concepts are in existence regarding bituminous stabilization.

1. Using a design criterion based on cementation and strength
2. Attempt only to waterproof the soil in order to maintain the inherent strength of the stabilized material under all conditions of weathering

The advantage of the first concept is that it obtains maximum stability for continued traffic. One distinct disadvantage is the relatively high cost for stabilization. Also, it requires a good pavement material to

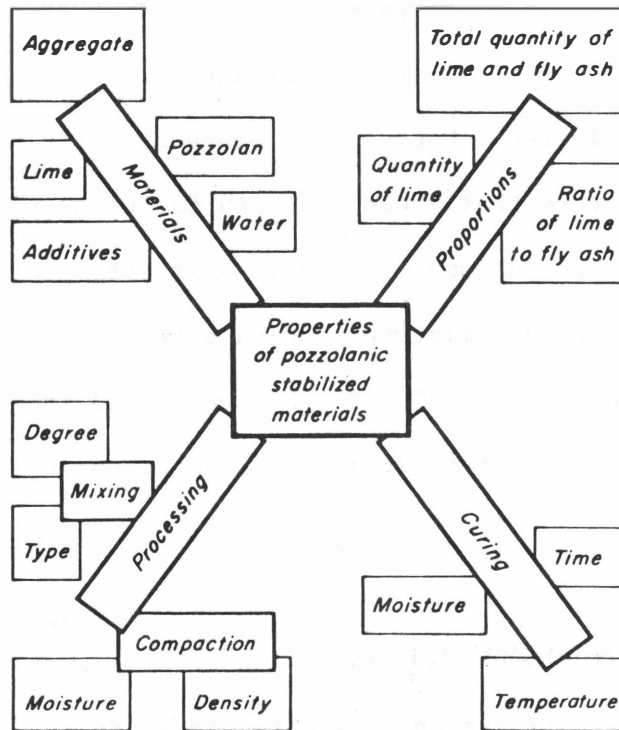


Figure 2.16: Factors influencing the properties of lime-fly ash-stabilized materials (after George, 1961).

Table 2.12:

RANGE OF COMPRESSIVE STRENGTH FOR THE
LIME- FLY ASH- STABILIZED SHALE

Material	28 Day Immersed Compressive Strength	
	psi	(kPa)
Gravels	400-1300	(2800-9000)
Sands	300- 700	(2100-4800)
Silts	300- 700	(2100-4800)
Clays	200- 500	(1400-3400)
Crushed Stones and Slag	1400-2000	(10,000-14,000)

7 K

2 kPa / 1 Psi

even attempt this type of construction. For example, an extremely dirty gravel which has considerable quantities of fines and some plasticity may be worsened by stabilization. This results from increasing the plasticity of the material by addition of too much bituminous material.

The principal advantage of the second concept is low cost of stabilization; low quantities of bituminous materials used to waterproof the fines. It is most important to provide proper compaction to high densities at closely controlled moisture contents during construction.

STABILIZATION WITH CALCIUM CHLORIDE AND SODIUM CHLORIDE

Calcium chloride has been used as a stabilizing agent, as well as a dust palliative, particularly in the case of a soil-aggregate mixture with low soil binder content. The benefit derived from chlorides is retarding the rate of evaporation of moisture from a mechanically stabilized soil during construction. Since moisture content is critical for proper compaction of granular materials, the moisture holding tendencies of calcium chloride are of great benefit. Performance surveys have pointed favorably to calcium over sodium chloride, in that gravel roads treated with calcium chloride require relatively little maintenance. When properly integrated with the correct proportions of soil and aggregate and compacted to unit weight required in standard practice, the salt will help the mass to harden and continue to compact under traffic, primarily because of water retention over long periods of time. Under equal drying conditions, a calcium chloride or sodium chloride stabilized road will not lose as much moisture as one without the admixture; in fact, when the relative humidity is 75% or more, moisture will be absorbed from the air. A property of the chlorides that

helps reduce the rate of evaporation is that of increasing the surface tension of water.

Calcium chloride and sodium chloride may slightly decrease the stability of a stabilized soil aggregate mixture just after construction, because the chlorides increase the plasticity of the fine fraction to a small extent. However, since the chlorides act as a water retaining agent, they will result in increased strength by retaining water in the mix, which aids in the compaction under traffic. Maintenance costs of gravel surfaces can be greatly reduced by the use of calcium chloride and sodium chloride because of a reduction in aggregate loss during dry seasons of the year.

STABILIZATION WITH WATER RETARDING AGENTS

Many miscellaneous chemicals are available for stabilization, which impart water-retarding characteristics to the soil so that it becomes hydrophobic. They are generally used in very small quantities (often as little as 1/2% by weight) and, as a result, mixing during construction is very important. Use of these chemicals is generally restricted to isolated cases and special conditions. The cost of the chemical itself is often so great that some other means of stabilization are best suited.

WEATHERABILITY STUDIES OF STABILIZED SHALES

During grading operation, soil is excavated from its original location and transported to the fill site. The soil is then spread in thin layers and compacted to maximum dry density and optimum moisture content. After completion of grading operation, the road is left unpaved for a certain period of time. During this period, climatic and environmental factors

act on the subgrade soil. High temperatures and low relative humidity conditions cause drying of the compacted soil. On the other hand, precipitation and frost action tend to increase subgrade moisture content. The effects of cycles of wet-dry and freeze-thaw on compressive strength of soils is used as a durability criteria. However, in Oklahoma, freezing of subgrades is not a major problem except in the northwest section; thus, increases in moisture content are due primarily to precipitation and high relative humidity conditions.

After the placement of the pavement structure, the system is subjected to the action of the traffic loads. The intensity and frequency of such load applications are dictated by traffic conditions but their effect on pavements is difficult to evaluate accurately. From their experiments with sand-asphalt beams, Moavenzadeh and Carnaghi (1966) and on soils, Seed, et al. (1967) concluded that as the number of load applications increases, the deflection per load cycle decreased.

Cyclic loading provides a characteristic method of stimulating the gradual changes due to traffic induced stresses after construction of highways. Evaluation of the effects of cyclic loading are discussed in detail in Chapter IV.

EFFECT OF DELAYED COMPACTION

Laboratory investigations traditionally have followed the procedure wherein the stabilizing agent is mixed with the soil and is immediately compacted. In the field, however, compaction takes place within a time range of one-half to three hours after mixing. It appears that delayed compaction has an adverse effect on strength gain depending on the type of soil material. Mitchell and Hooper (1961) studied the effect of

delay by using an organic expansive clay to which 4% dolomitic hydrated lime was added and found that a delay of 24 hours caused up to eight pcf loss in the maximum dry unit weight and 30% loss in strength as measured by triaxial compression tests. Expansion characteristics were also adversely affected by elapsed time before compaction.

Mateos and Davidson (1961) made a selected study of the effect of delay upon mixtures involving either a lime sand or a "gumbotil" (heavy montmorillonite clay) stabilized with three flyashes and a high calcium hydrated lime. Samples of the various mixtures were prepared immediately, and four and 24 hours after wet mixing. Decreases in the dry unit weights of the sand-flyash-lime mixtures were almost negligible after the 24 hours delay period and strength losses were correspondingly small. However, with the "gumbotil" increases in the delay time brought about considerable lowering in the dry unit weights and unconfined compressive strengths of the samples; in fact, with a 24 hour delay one mixture gave losses varying between 32 and 49% depending on the curing period.

Anderson and Shields (1963) found with a "Falha" clay (a heavy illite clay) to which added 20% high calcium hydrated lime that a 24 hour delay before compaction lowered the dry unit weights of samples appreciably.

Pinto et. al. (1962) undertook an evaluation of the effect of elapsed time on mixtures of cement and lime on two loess soils. Samples were compacted at various moisture contents (to account for variations in optimum moisture content values) immediately after mixing and also after two hour and four hour delay periods. Results showed that the dry unit weights of the samples decreased only slightly as a result of the delays, irrespective of whether lime was present in the soil-cement mixtures or not. Strength values were also lower after delaying the compaction,

but here the addition of lime minimized these decreases. The low strength of some samples which were molded "wet", i.e., at moisture contents above the optimum value for maximum dry unit weight, were improved by a two hour delay before compaction due to the lime cement-soil reactions causing an apparent drying out of the wet mixture and allowing crack-free samples to be compacted at the same energy level.

Dumbleton (1962) showed that a heavy London clay and 10% high calcium hydrated lime mixture gave less than 1.0 pcf decrease in dry unit weight when compacted six hours after mixing, while mixtures of the same clay with 2 to 5% and 10% cement showed decreases of over 7.0 pcf in a similar period. Initially, the lime clay samples had dry unit weights which were 5.5 pcf below those of the cement-clay specimens, but after eight days delay before compaction, the positions were reversed and cement clay specimens had lower values by 2.0 pcf. In practice, it is probable that the action of lime on a heavy clay can result in such an improvement in handling and mixing that it may outweigh the extra cost of compaction to maintain a required dry unit weight. Compared with cement soil, lime soil, because of its relatively slow rate of strength gain, also has the advantage that if it cannot be compacted immediately because of heavy rain then it can be left to dry for some time at least with less detrimental effect. According to Herrin and Mitchell (1961), the time limit for reworking lime-soil mixtures without the addition of more lime depends on the field conditions and the quantity of lime originally used; if extra lime must be added, the amount required to attain the original design criteria will usually be small. The effect of delayed compaction is shown in Figure 2.16 for cement stabilized and lime stabilized soils.

MINERALOGY AND VOID DOMAIN CHARACTERISTICS

Among the lime clay reaction products, the tricalcium silicate hydrates, C_3SH or the tobermorites, are the most important (Glenn and Handy, 1963; Ruff and Ho, 1966; and Taylor, 1966). Tobermorites are formed as immediate products in almost all reactions in which calcium ions and silicate ions come together. The other important lime clay reaction products are CAH , C_3AH , C_4AH_n , C_2S and C_3S (Noble, 1967; Ruff and Ho, 1966; and Taylor, 1966).

Herzog and Mitchell (1963) indicated the following on the basis of X-ray diffraction of clay-cement mixtures:

- 1) Portland cement forms calcium hydroxide in hydrating clay cement
- 2) The hydration of C_3S is modified in the presence of clay
- 3) After one week curing no crystalline calcium hydroxide was detectable in clay cement specimens
- 4) Small decreases in the intensity of kaolinite reflections and large decrease in the intensity of montorillonite reflections is observed under 12 weeks curing of clay cement
- 5) New lines of low intensity were observed in some clay cement patterns after long curing time, suggesting the formation of new crystalline reaction products

Apparently studies have not been performed on pore domain characteristics of stabilized soils. Some electron microscopic studies of stabilized soils have been done in this study and are presented in Chapter V.

CHAPTER III

SELECTION AND PREPARATION OF MATERIALS

On the basis of a previous study (Laguros, 1972) eight shales were selected for this investigation. They represent typical characteristics of shales in Oklahoma and they cover a range of geologic, physiographic and geographic as well as engineering properties. Routine engineering and geologic information related to these shales were obtained at the Soils Laboratory University of Norman, Oklahoma. The shales obtained from the field were air dried and then ground to pass U.S. Standard Sieve No. 10. The material passing the No. 10 sieve was used for the investigation of this study. Sampling locations of these shales are shown in Figure 3.1, and their relevant geologic and engineering data are depicted in Tables 3.3 to 3.7. For studying the effects of portland cement stabilization, Type I cement was used. The composition of Type I cement is as follows:

Table 3.1:
COMPOSITION OF TYPE I CEMENT

C_3S (tricalcium silicate)	49%
C_2S (dicalcium silicate)	25%
C_3A (tricalcium aluminate)	12%
C_4AF (tetracalcium aluminoferrite)	8%
$CaSO_4$ (calcium sulphate)	2.9%
CaO (calcium oxide)	0.8%
MgO (magnesium oxide)	2.4%

The hydrated lime used for lime stabilization was U.S.P. (Powder)

Ca (OH)₂, manufactured by J. T. Baker Chemical Company, Phillipsburg, New Jersey 08865. The maximum limits for impurities for this are given as:

Table 3.2:
IMPURITIES IN LIME

Chloride	0.003%
Iron (Fe)	0.05%
Sulphate (SO ₄)	0.09%
Other heavy metals (Pb)	0.002%
Aluminum Hydroxide	0.39%
Mg and Alkali salt	0.32%
Insoluble in HCl	0.02%
the amount of lime - 95 to 100%	

In this study, flyash is also included as a possible low cost stabilizing agent for Oklahoma shales in view of its abundant supply (112,500 tons/year). The particular sample of flyash used in this study was supplied by the Oklahoma Gas and Electric Company, Oklahoma City. The flyash, derived from use of Wyoming coal is their Muskogee plant, will be available in future. The chemical analysis of flyash indicated the amount of CaO to be approximately 24% ranging mostly within 15-34%. Although flyash is generally used as an admixture in soil lime or soil cement stabilization, the large amount of CaO in this flyash encouraged its use as a main stabilizing agent. Elemental and mineral analysis of flyash is presented in Tables 3.8 and 3.9.

Under this investigation, a commercial chemical stabilizer "clapak" was tried. Manufacturers of this product recommended the use of chemical admixture termed "claset" along with "clapak" for soils with plasticity index more than 10. Since all shales except shale 35 had plasticity indices more than 10, both clapak and claset were tried. The concentrations of both chemicals were chosen according to recommendations of the manufacturer. The solutions were made up at desired concentrations and used in place of water with the amount corresponding to optimum moisture

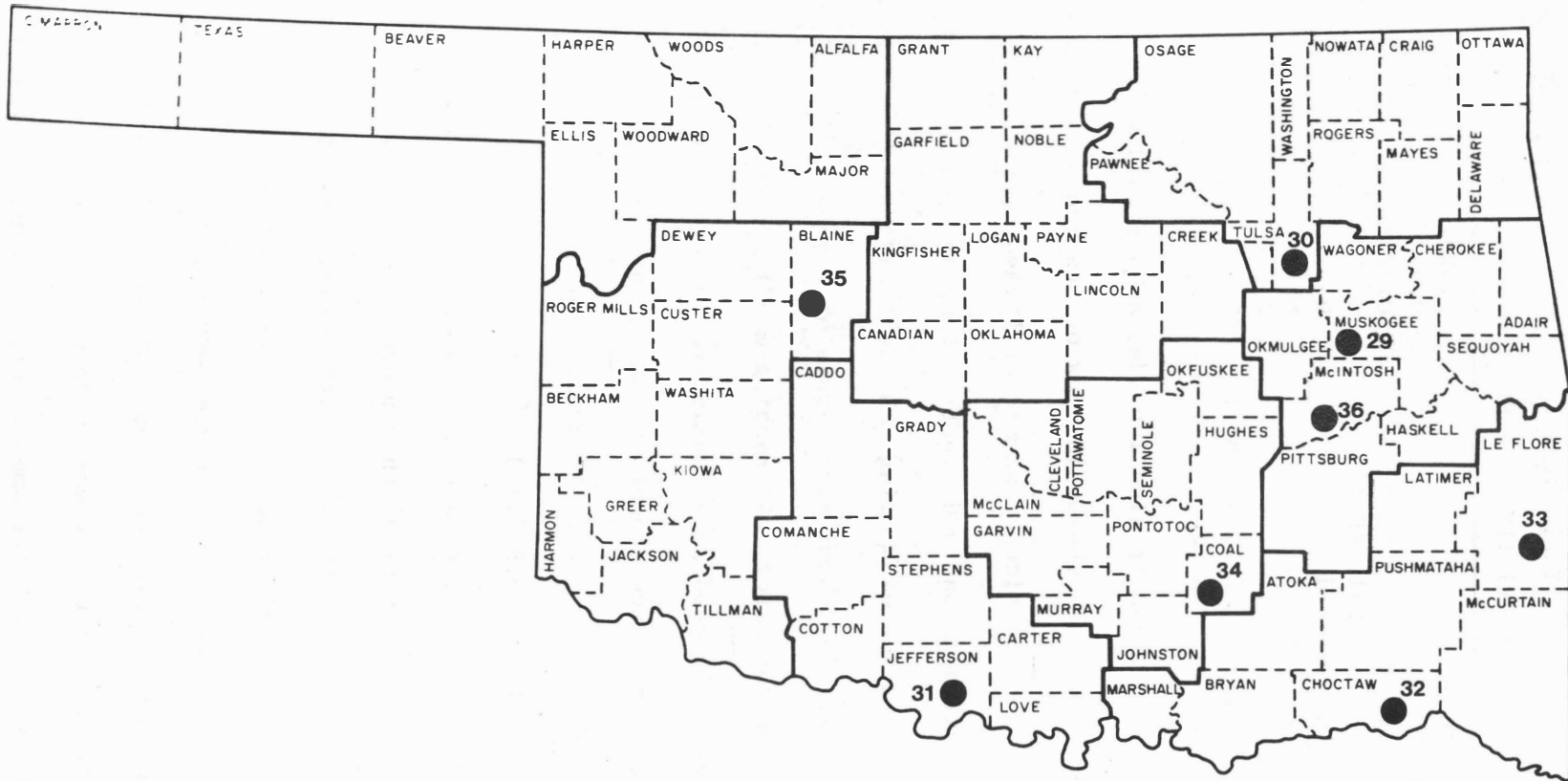


Figure 3.1: Locations of sampling sites in Oklahoma.

Table 3.3:

DESCRIPTION OF SHALES

Shale Number	Geologic Unit	County	Sampling Location	Field Description
29	Atoka	Muskogee	SE $\frac{1}{4}$ SE $\frac{1}{4}$ SE $\frac{1}{4}$ Sec. 5 T11N R19E roadway cutbank on NW corner of intersection of county roads. Sample from 4-5 feet below base on Blackjack Sch. sandstone.	Gray, fissile fine sandy shale
30	Labette	Tulsa	Center W $\frac{1}{2}$ SE $\frac{1}{4}$ Sec. 18 T26N R17E northeast cutbank. Sample from 18-20 feet below base of Oologah limestone.	Gray, blocky shale
31	Claypool	Jefferson	Shale-2000' West of the SW corner of Sec. 34 T3S R7W on north side of road. Sample from zone 12-14 feet below base of Asphaltum sandstone.	Red, blocky shale
32	Fredericksburg	Choctaw	Center NE $\frac{1}{4}$ Sec. 31 T6S R20E. Sample from zone 10-12 feet below the Duck Creek limestone.	Olive, soft, fissile to blocky shale
33	John's Valley	LeFlore	$\frac{1}{2}$ mile north of the SE corner of Sec. 22 T3N R25E along creek bank just off south side of Mineral Springs road.	Black, hard, fissile shale
34	Boggy	Coal	Center W $\frac{1}{2}$ Sec. 16 T3N R10E. Sample from deep cut along creek bank just off south side of Mineral Springs road.	Gray, blocky shale
35	Flowerpot	Blaine	NE $\frac{1}{4}$ NE $\frac{1}{4}$ NW $\frac{1}{4}$ Sec. 9 T17N R11W in side of gully. Sample from about 120' below base of Blaine gypsum.	Red, blocky shale
36	Senora	McIntosh	NW $\frac{1}{4}$ SW $\frac{1}{4}$ Sec. 12 T11N R14E north backslope of west bound lane. FAP I-40-6(45) (46), Sta. 277+00. Sample from shally zone between thick sandstones.	Gray and brown, fissile to blocky shale

Table 3.4:

TEST DATA AND CLASSIFICATION OF SHALES

Shale Number	Grain Size Analysis			Liquid Limit, %	Plasticity Index, %	Soil Classification		
	Silt, % < 5 μ	Clay*, % < 2 μ	Clay, %			AASHTO	Unified	USDA
<i>Atoka</i> 29	31	19	① <i>lowest</i> 11	36	② 11	③ A-6(4)	ML	Loam
30	40	33	21	33	13	A-6(9)	CL	Silt loam
31	27	33	23	57	25	A-7-5(13)	MH	Silt loam
32	11	88	75	78	30	A-7-5(20)	MH	Clay
<i>JV</i> → 33	18	27	17	38	12	<i>2</i> A-6(3)	ML	Loam
34	43	55	37	37	12	A-6(9)	ML	Silty clay loam
35	19	29	20	31	<i>1</i> 5	<i>/</i> A-4(2)	CL	Loam
36	12	55	35	41	18	A-7-6(11)	CL	Silty clay loam

* Amount less than 5 μ size and includes 2 μ size particles

Table 3.5:

STRENGTH RELATED DATA OF SHALES

Shale Number	Specific Gravity	Dry Density pcf	Optimum Moisture Content, %	Unconfined Compressive Strength, psi	Strength Parameters*	
					Cohesion, psi	Angle of Friction, degrees
29	2.71	112.3	16.5	1 ^{1/2} 35.4	9 12.3	5 21.3
30	2.73	119.1	14.3	27.3	9.3	9 22.0
31	2.71	106.5	20.0	2 36.9	2 14.6	13.3
32	2.73	95.9	26.4	1 40.2	1 15.9	14.4
33	2.67	113.3	16.0	32.8	3 12.6	19.0
34	2.67	113.0	16.9	3 36.3	10.3	2 23.8
35	2.77	107.2	20.2	24.7	7.4	1 25.1
36	2.68	112.0	16.8	29.5	10.4	3 22.6

* Determined from unconsolidated-undrained triaxial shear tests.

Atoka

Table 3.6:

EFFECT OF ULTRASONIC TREATMENT ON
INDEX PROPERTIES OF SHALES

Shale Number	Grain Size Analysis						Liquid Plasticity			
	Silt, %		Smaller than 5 μ clay, %		Smaller than 2 μ clay, %		Limit %		Index %	
	*R	*U	R	U	R	U	R	U	R	U
<i>Atoka</i> 29	31	30	19	69	11	48	36	41	11	12
30	40	44	33	53	21	31	33	35	13	15
31	27	13	33	84	23	54	57	56	25	29
32	11	14	88	86	75	76	78	85	40	48
33	18	18	27	81	17	54	38	51	12	23
34	43	43	55	71	37	55	37	40	12	18
35	19	19	29	77	20	59	31	38	5	20
36	32	32	55	68	35	48	41	41	18	17

* R-raw shales

* U-after ultrasonic treatment

Table 3.7:

CLAY MINERAL COMPOSITION OF SHALES *

Shale Number	Montmorillonite (%)	Chlorite (%)	Illite (%)	Kaolinite (%)	Mixed Layer (M-1) (%)
29	--	7	55	38	--
30	--	16	41	43	--
31	75	--	--	3	22
32	67	--	2	11	20
33	39	--	58	3	--
34	5	--	73	22	--
35	--	6	94	--	--
36	--	--	41	--	59

*Approximate percentages determined from x-ray diffractograms of sedimented slides.

Table 3.8:
 ANTICIPATED MINERAL ANALYSIS OF ASH FROM OKLAHOMA GAS AND
 ELECTRIC COMPANY, MUSKOGEE, OKLAHOMA
 OPERATING UNIT NO. 4 AND 5

	Typical	Range
Phos Pentoxide	0.94	0.07 - 2.80
Silica	29.51	19.64 - 43.31
Ferric Oxide	4.95	2.41 - 6.78
Alumina	15.49	10.57 - 19.73
Titania	1.19	0.11 - 1.59
Lime	23.93	14.70 - 34.40
Magnesia	4.85	3.22 - 7.43
Sulfur Trioxide	15.68	7.64 - 24.50
Potassium Oxide	0.33	0.05 - 1.67
Sodium Oxide	1.55	0.00 - 2.45
Undetermined	2.41	0.00 - 12.78
	100.00%	
Alkalies as Na ₂ O, DCB =	0.10	0.02 - 0.21
Silica Value	48.33	34.11 - 56.99

Fusion Temperature of Ash

	Reducing		Oxidizing	
	Typical	Range	Typical	Range
Initial Deformation	2160	1950-2610	2210	2020-2700
Softening (H = W)	2210	2045-2700	2258	2120-2700
Softening (H = 1/2W)	2238	2080-2700	2289	2130-2700
Fluid	2284	2090-2700	2339	2140-2700

This analysis is based on laboratory analysis of approximately 100 full seam coal cores and is not an analysis of ash from a coal fired electric generating station.

Table 3.9:

ELEMENTAL ANALYSIS OF FLYASH FROM AMEX
MINE COAL AT GILLETTE WYOMING

Elemental Analysis of Ash	Assay %
SiO_2	31.50%
Al_2O_3	18.50
TiO_2	0.84
Fe_2O_3	7.00
CaO	28.00
MgO	4.82
Na_2O	1.47
K_2O	0.34
P_2O_5	0.41
SO_3	5.47
Ash Viscosity Calculations	
Base Content (PCT)	45.03%
Acid Content (PCT)	54.98
Dolomite Content (PCT)	78.84
Base to Acid Ratio	0.82
Silica to Alumina Ratio	1.70
T250 (Temp in $^{\circ}\text{F}$ for 250 Poise)	2199 $^{\circ}\text{F}$

This is an analysis of a sample of fly ash from a modern electric power generating station burning Amex coal.

content requirements. The results are presented in Chapter V. On the basis of dry and immersed strengths, it was concluded that chemical stabilization either with clapak alone or with clapak-claset, is not suitable for shales considered in this study.

CHAPTER IV

EXPERIMENTAL PROCEDURES

This research primarily envisions the determination of the type and amounts of stabilizing agents to be added in order to upgrade the shales used in highway construction and the development of methods of stabilization best suited for Oklahoma. All the tests employed in this study follow standard specifications and they are those generally used for evaluating engineering properties of raw and stabilized highway soil materials for design purposes. An outline of the research procedures followed in this study is presented in Figure 4.1.

GRAIN SIZE ANALYSIS

Grain size distribution was determined in accordance with the ASTM Designation D422-63 (AASHTO Designation T88-57). Calgon was used as the dispersing agent. The Iowa jet dispersion apparatus was used to disperse the soil particle under an air-pressure of 10 psi for about five minutes. All the samples were ultrasonically treated for one hour (Laguros, 1972) to provide better design criteria. The ultrasonic apparatus is shown in Figure 4.2

SPECIFIC GRAVITY

The specific gravity of shales samples was determined in accordance with the ASTM Designation D854-65 (AASHTO Designation T100-60).

ATTERBERG LIMITS

The liquid limit tests were run in accordance with the ASTM

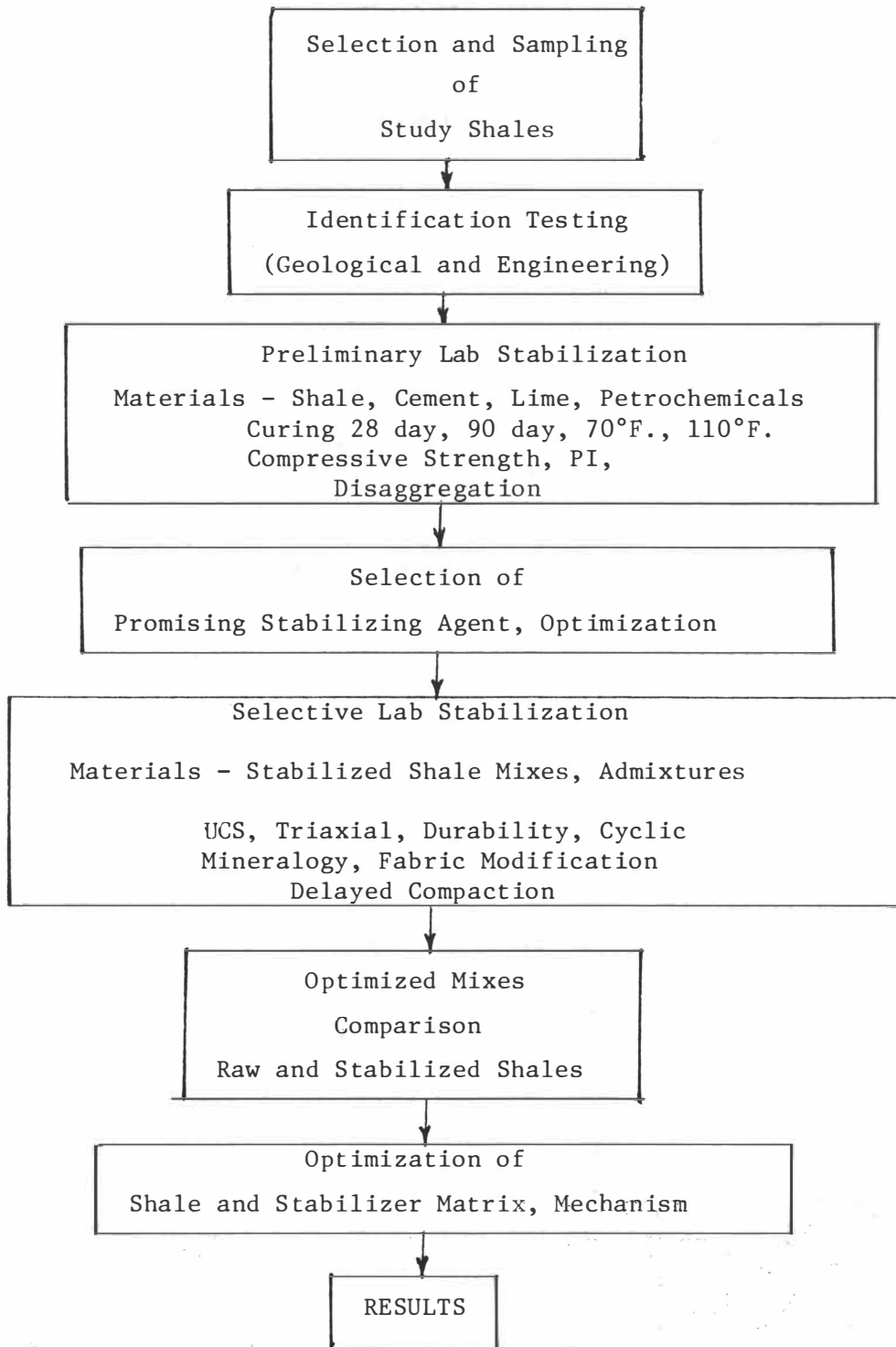


Figure 4.1: Outline of Procedures.

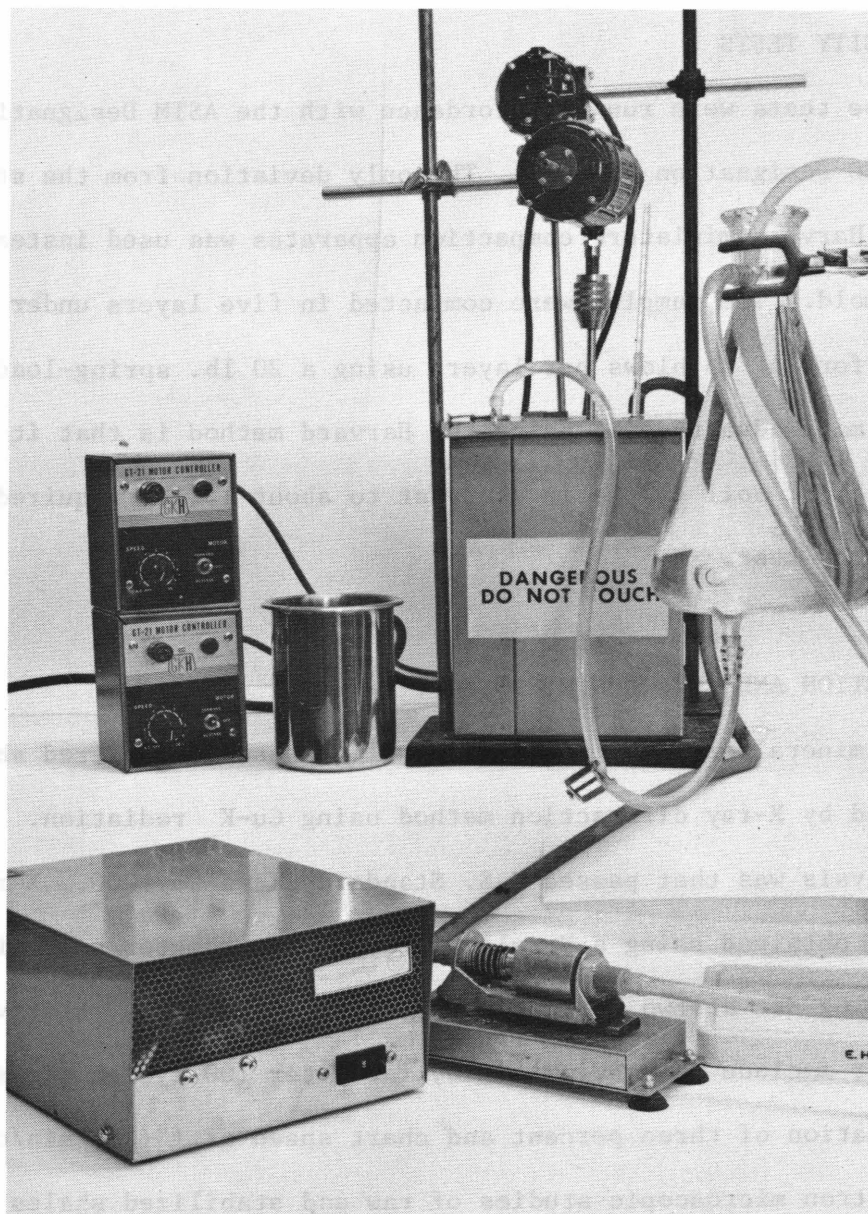


Figure 4.2: General view of ultrasonic equipment and water circulation system.

Designation D423-66 (AASHTO Designation T89-60). The plastic limit tests were run in accordance with the ASTM Designation D424-65 (AASHTO Designation T90-61).

MOISTURE DENSITY TESTS

These tests were run in accordance with the ASTM Designation D98-66 (AASHTO Designation T99-61). The only deviation from the standards was that the Harvard miniature compaction apparatus was used instead of the Proctor mold. The samples were compacted in five layers under a compactive effort of 25 blows per layer, using a 20 lb. spring-loaded hammer. The main advantage in using the Harvard method is that it requires only 1 to 5 lb. of soil sample in contrast to about 15 lb. required for standard proctor test.

X-RAY DIFFRACTION AND ELECTRON MICROSCOPY

The mineralogical composition of all raw and stabilized shales was determined by X-ray diffraction method using Cu-K_α radiation. Material used for analysis was that passed U.S. Standard Sieve No. 200. X-ray patterns were obtained using a Norelco X-ray diffractometer unit with the rate of scanning set at two degrees per minute. Other data relative to diffractometer include KV = 40, MA - 20, ratemeter 100 cycles per sec., standard deviation of three percent and chart speed of 1°(2θ)/min/cm.

Electron microscopic studies of raw and stabilized shales are also presented in this study in Chapter V. The sample was dried by evaporation under 7×10^{-5} mm Hg vacuum. The magnification was 3000 times.

MOLDING AND CURING OF SAMPLES

Samples were statistically compacted in the apparatus shown in

detail in Figure 4.3. The dimensions of a sample prepared by this method are: diameter 1.35 inch and height 2.95 inches. The total volume of the sample prepared by this apparatus is ten percent greater than that of the sample obtained with the Harvard miniature compaction apparatus. The calculated amount of mix was poured into the molding tube which was then placed under a compression machine along with the two plungers and compressed until the flanges of plungers were in complete contact with the ends of the tube. The assembly was left under the machine for five minutes after which the load was removed and the plungers withdrawn. Using the extraction plunger, the sample was extracted from the molding tube under the pressure applied through the hydraulic jack.

The extracted sample was wrapped in aluminum foil, labeled and stored in a humidity cabinet. For the preliminary laboratory stabilization phase the samples were cured at two temperatures, 70° F. and 110° F., and for 28 days and 90 days at 90-100% relative humidity. All shales were treated with 3%, 6%, and 9% lime respectively by the weight of soil at above curing conditions.

In the case of portland cement, as suggested by the Oklahoma Department of Transportation (described in Chapter II), all shales were treated with 10%, 14%, and 18% cement respectively at same curing conditions and temperatures as described above for preliminary laboratory stabilization.

On the basis of the results of preliminary laboratory stabilization, it was decided to use lime, cement and flyash (6%, 14%, and 25% by the weight of soil, respectively) as main stabilizing agents. For better environmental control for the 90 day curing period, a curing temperature of 110° F. at 90-100% relative humidity was decided for use in selective laboratory stabilization.

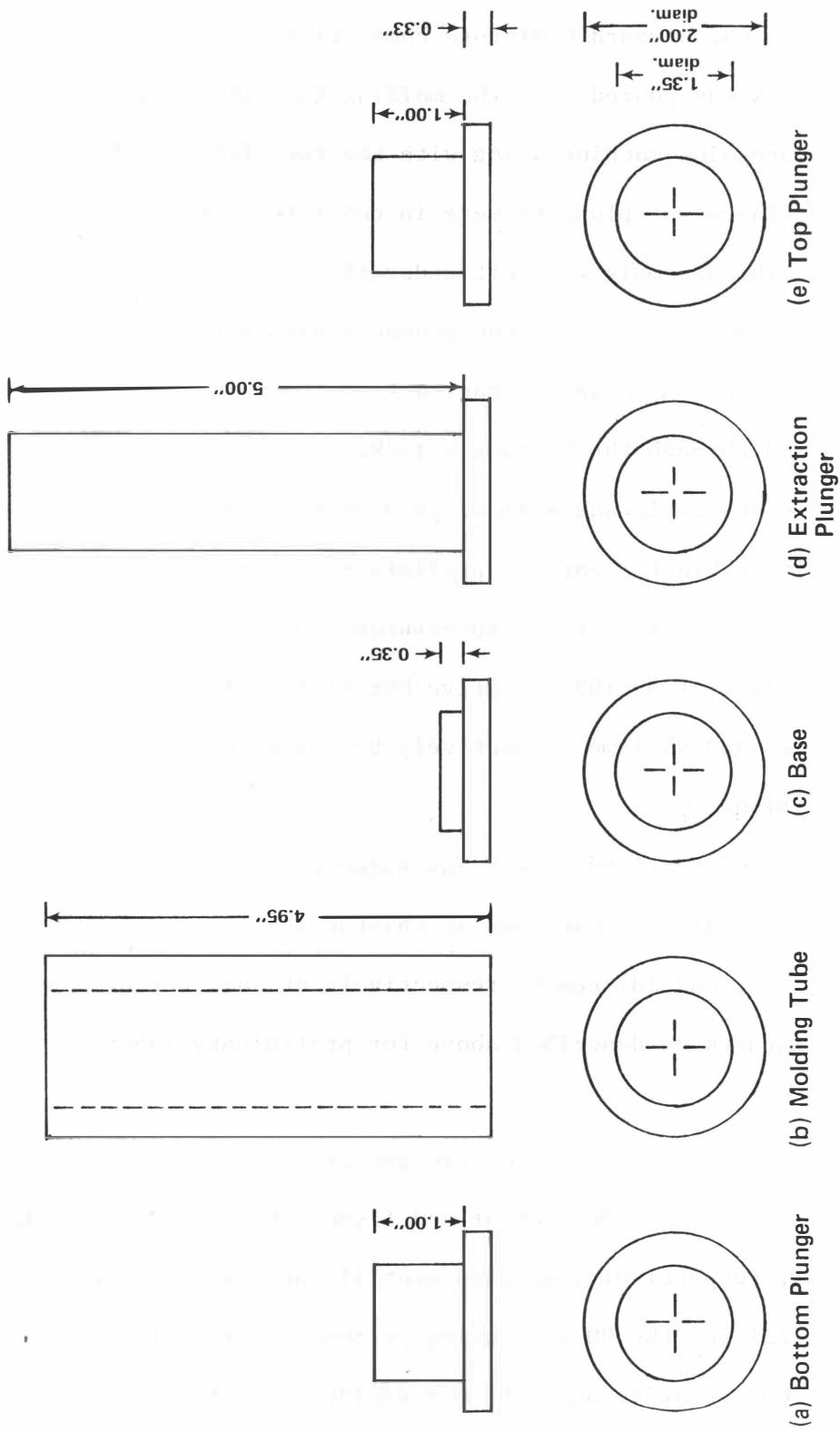


Figure 4.3: Details of sample molding apparatus.

UNCONFINED COMPRESSIVE STRENGTH

For the 28-day and 90-day cured samples at 110° F. and 90-100% relative humidity the unconfined compressive strength was determined in a compression testing machine, Soiltest Model AP-170B, with a load capacity of 10,000 pounds (see Figure 4.4). Dial readings were recorded and later converted to strength terms and three-specimen averages determined. Various charts and graphs for unconfined compressive strength vs. curing time and percentage of stabilizing agents, are presented in Chapter V.

To find the effects of different stabilizing agents in addition to compressive strength, plasticity properties and gradation analysis were also conducted. To reflect the change in particle sizes of clay due to different stabilizing agents, a parameter aggregation index (AI) was defined. Mathematically represented as:

$$\text{Aggregation Index (AI)} = \frac{\% \text{ of nonclay size material of stabilized shale}}{\% \text{ of nonclay size material of raw shale}}$$

The AI values for lime, cement and flyash stabilization are given in Chapter V.

To enhance the effect of the main stabilizing agents a secondary (or admixture) additive was used. Small addition of admixtures or its combination was found very effective. The parameter strength beneficitation index (SBI) was defined to reflect the effectiveness of admixture.

$$\text{Strength Beneficitation Index (SBI)} = \frac{\text{UCS (main + secondary)} - \text{UCS (main)}}{\text{UCS (main)}}$$

TRIAXIAL COMPRESSIVE STRENGTH

The advantage of the triaxial compressive test is that the field conditions prior to and during construction can be duplicated in the

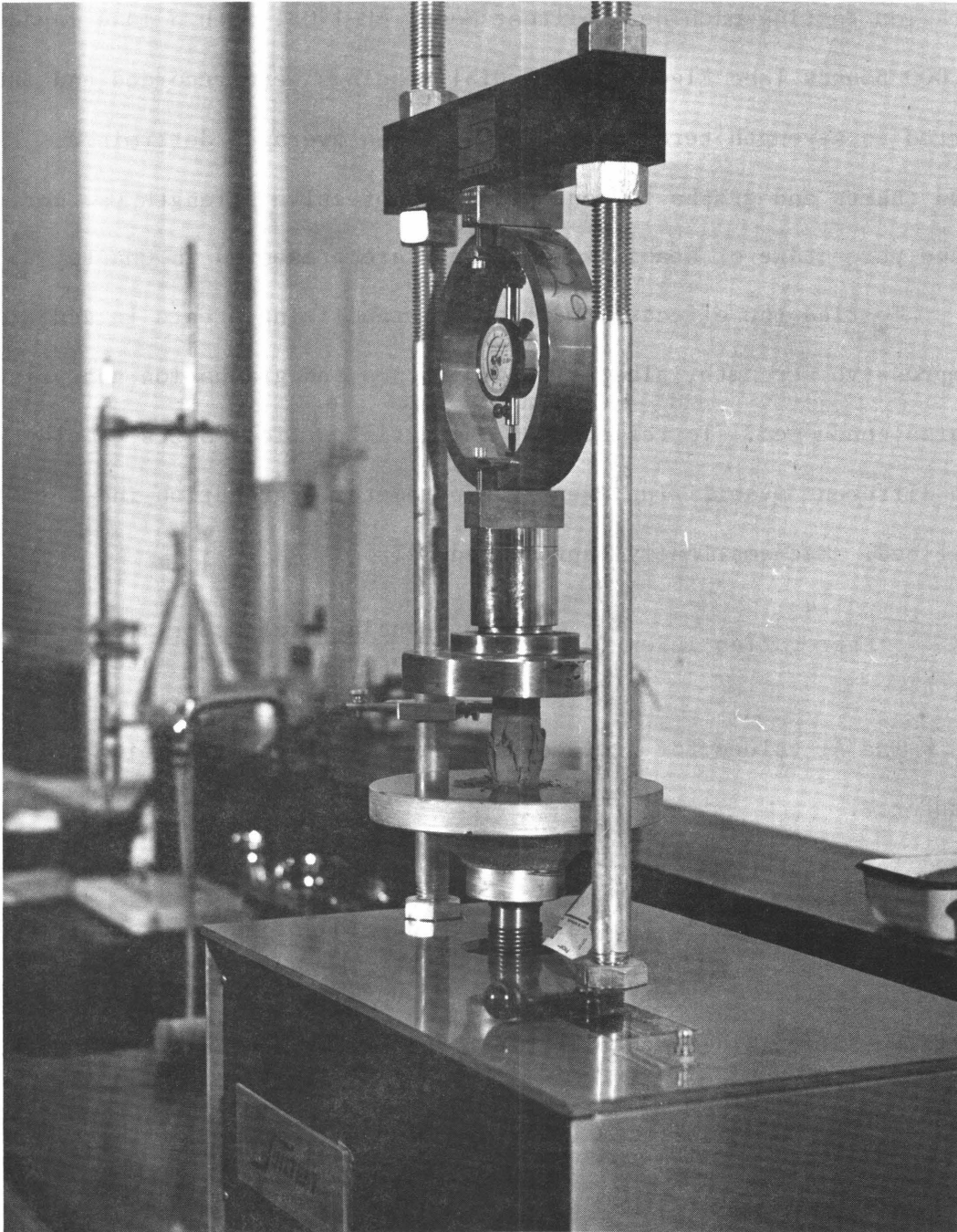


Figure 4.4: Compression strength testing device.

laboratory to study the behavior of the soil. The general Mohr-Coulomb failure law is expressed by the formula:

$$\zeta = C + \sigma \tan \phi$$

ζ = shear stress, psi

C = cohesion, psi

σ = normal total stress, psi

ϕ = angle of friction, degree

The normal total stress, σ , includes a number of parameters or terms and for a generalized soil-air-water system, it may be expressed as

$$\sigma = \bar{\sigma} \cdot A_m + U_a \cdot A_a + U_w + A - R$$

A_m = (area of mineral-mineral contact)/total area

U_a = pore air pressure

A_a = (area of air-mineral contact)/total area

U_w = pore water pressure

A_w = (area of water-mineral contact)/total area

A = net attractive forces existing between clay platelets

R = net repulsive forces between clay particles

$\bar{\sigma}$ = contact stress at contact point of mineral-mineral

However, A and R cannot be isolated and measured experimentally but considered to be predominant in dispersed plastic clays. For other textured soils, A and R are ignored. Also, it is assumed that $\bar{\sigma}$ is very large, $\bar{\sigma} \cdot A_m$ is finite and equal to $\bar{\sigma}$ (effective stress), and $A_a + A_w \approx 1$ for a partially saturated soil, the same equation is written as:

$$\sigma = \bar{\sigma} + U_a + A_w (U_w - U_a)$$

and for a saturated soil

$$\sigma = \bar{\sigma} + U_w$$

In terms of the effective stress, the Mohr-Coulomb equation is given by

$$\zeta = \bar{c} + \bar{\sigma} \tan \phi$$

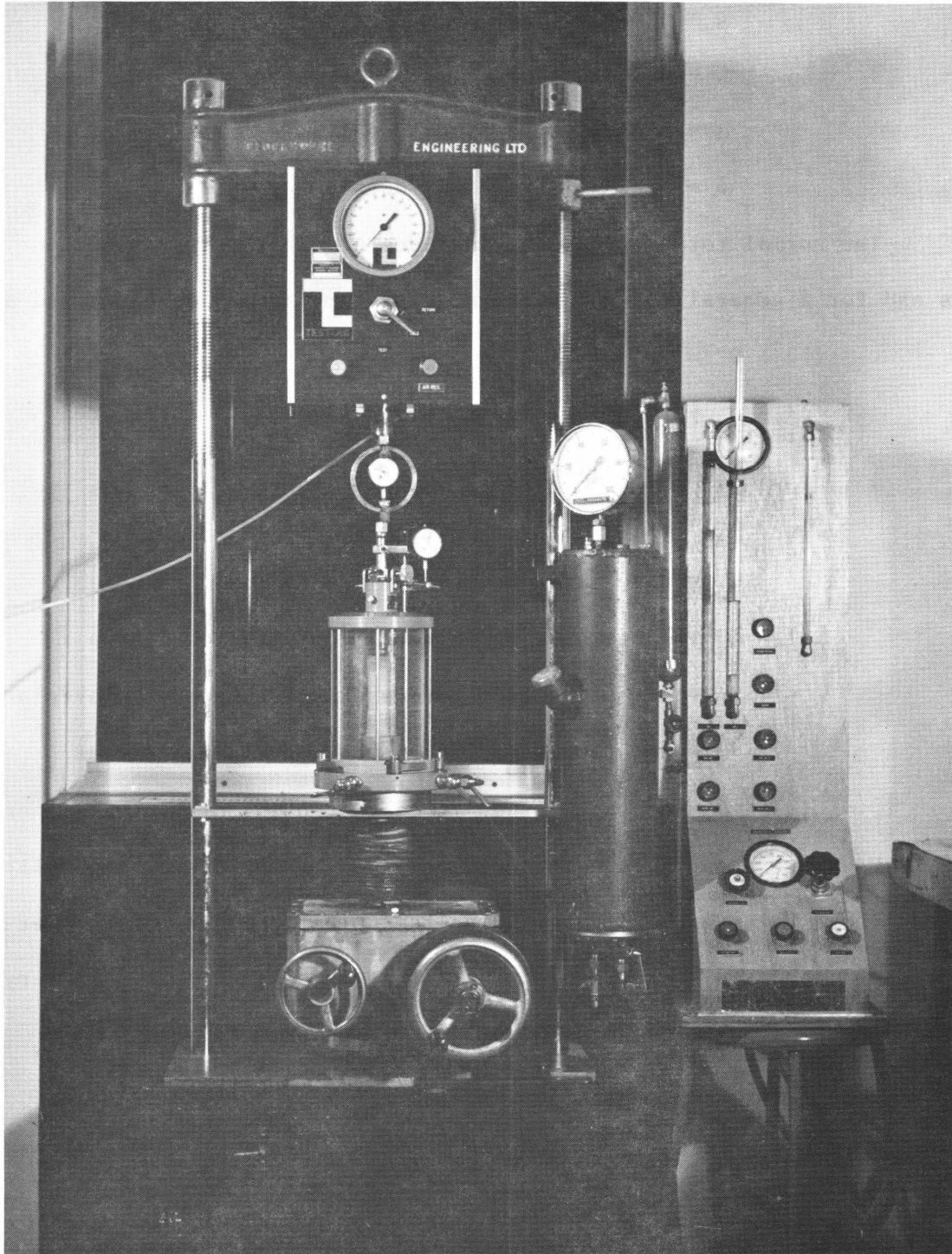


Figure 4.5: Triaxial compression test set up.

where:

\bar{c} = true cohesion, psi

$\bar{\sigma}$ = effective normal stress, psi

ϕ = true angle of friction, degrees

Both c , ϕ , and \bar{c} , $\bar{\phi}$ are frequently employed in the design, depending on the actual field conditions. In this investigation, all triaxial compressive tests were run as unconsolidated-undrained tests without pore water pressure measurement for the following reasons:

1. In bases and subbases, the thickness involved being small, the changes in pore water pressures are not critical in strength determinations. Strength parameters determined from undrained tests are used for the design of pavements
2. In the construction of an embankment or excavation of a slope, the stability of slope is often governed by undrained shear strength of the soil as the soil is stressed quickly and no time is allowed for a drained condition to be established and for the dissipation of pore water pressure

All raw and stabilized shale samples were tested for three different lateral pressures (10 psi, 20 psi, and 30 psi). The rate of shear was .05 inch/minute. The strength apparatus and various sample failure patterns are shown in Figure 4.5 and 4.6, respectively.

WEATHERING CYCLES

In addition to the data on shales, the data for a number of weather cycles for various parts of Oklahoma also were obtained. To determine these cycles, data for ten years (Laguros, 1972) were collected from 16 locations in Oklahoma (Table I.1 in Appendix 1) and then evaluated. The data for the freeze-thaw cycles are shown in Table I.2 and those for wet-dry cycles in Table I.3 of Appendix A. A freeze-thaw cycle is considered as any period in which the average temperature goes from 32° F. or higher to 31° F. or lower and back to 32° F. or higher. A methodology similar to

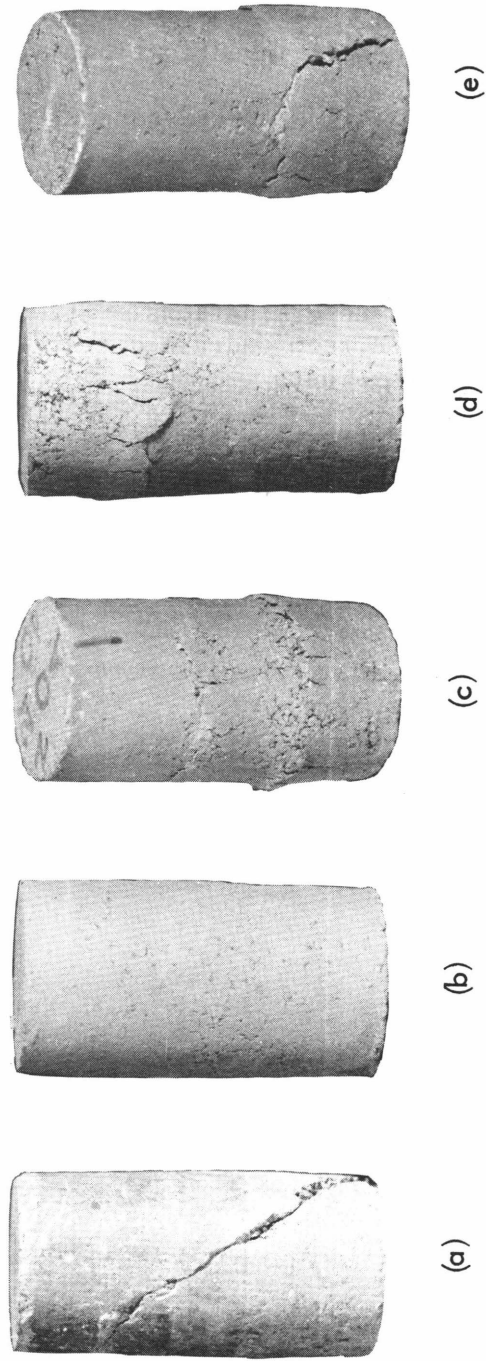


Figure 4.6: Failure patterns of specimens.

freeze-thaw cycle has been adopted for wet-dry cycles. A cycle is defined as a dry period in a 24 hour interval; any rainfall less than 0.10 inch is disaggregated unless this rainfall links two 24 hour periods with at least a total of 0.10 inch rainfall. The third highest number of freeze-thaw or wet-dry cycles is chosen as the number of weather cycles. The number of such wet-dry cycles is 300 to 45 and that of freeze-thaw is 6 to 12. The application of wet-dry and freeze-thaw cycles is expected to simulate the effect of weather on shale samples. A higher number (more than 5) of freeze-thaw cycles is unlikely to occur in Oklahoma weather. Since the time lapse between construction of subgrade and placement of pavement on it is usually in the range of six months, the number of wet-dry cycles and freeze-thaw cycles to which the subgrade material is expected to be subjected is about 15 and 5, respectively. It was therefore decided to find the effect of applying 0, 5 and 15 cycles of wet-dry and freeze-thaw on the samples.

CYCLIC LOADING

Cyclic loading provides a characteristic method of simulating the gradual changes in the pavement system due to traffic induced stresses after construction of highways. The arrangements of equipment is shown in Figures 4.7 and 4.8. For this study it was decided to undertake the investigation of repetitive load testing on stabilized shales for 40, 80, 120 and 160 lb. loads at 24 and 40 applications per minute (APM). Since at 160 lb. application of 40 APM was beyond the capacity of this machine, the testing had to be extended only to 160 lb. load and 24 APM frequency. Twenty-four APM and 40 APM are equivalent to 35,000 ADT and 60,000 ADT on two way, four-lane highways and urban freeways, respectively (HRB Special Bulletin No. 87, 1965). This testing was done for 40, 80, 120 and 160 lb. loading at two APM

frequencies (24 APM and 40 APM). The sequence of loading is shown in Table 4.1

In each case, the APM frequency was constant (either 24 APM or 40 APM) and at each loading was applied 50 times (cycles). The loading was increased starting 40 to maximum 160 lb. at an interval of 40 lbs. until failure occurred. The maximum load to cause failure for all shales are given in Chapter V. For most of the stabilized shales, the failure was brittle (or crushed at higher loads without giving significant strain). However, for lime stabilized shales and flyash stabilized shales failure was detected after obtaining significant strains.

Table 4.1:

SEQUENCE OF LOADING FOR CYCLIC LOADING

Sample	Order	Loads Applied (lb.)	Frequency (APM)
1	1	40	24
	2	80	24
	3	120	24
	4	160	24
2	5	40	40
	6	80	40
	7	120	40

DELAYED COMPACTION

Laboratory investigations traditionally have followed the procedure where after the addition of the stabilizing agent, the mixture is immediately compacted. In the field, compaction takes place within a time range of one-half to three hours after mixing. To assess the influence of delayed compaction on strength, a selective study was initiated on stabilized shales. The results are presented in detail in Chapter V. The shales and

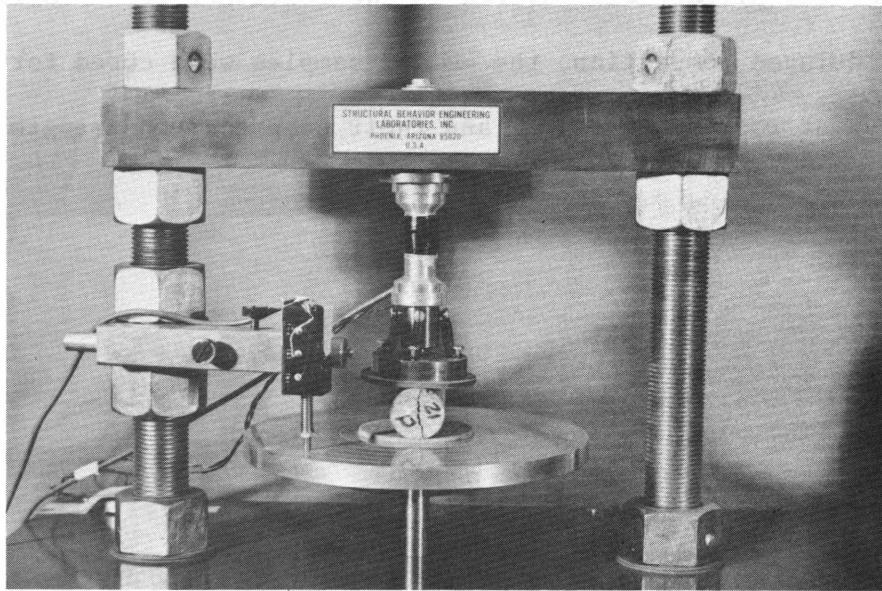


Figure 4.7: Arrangement of load cell and displacement strain gage during testing.

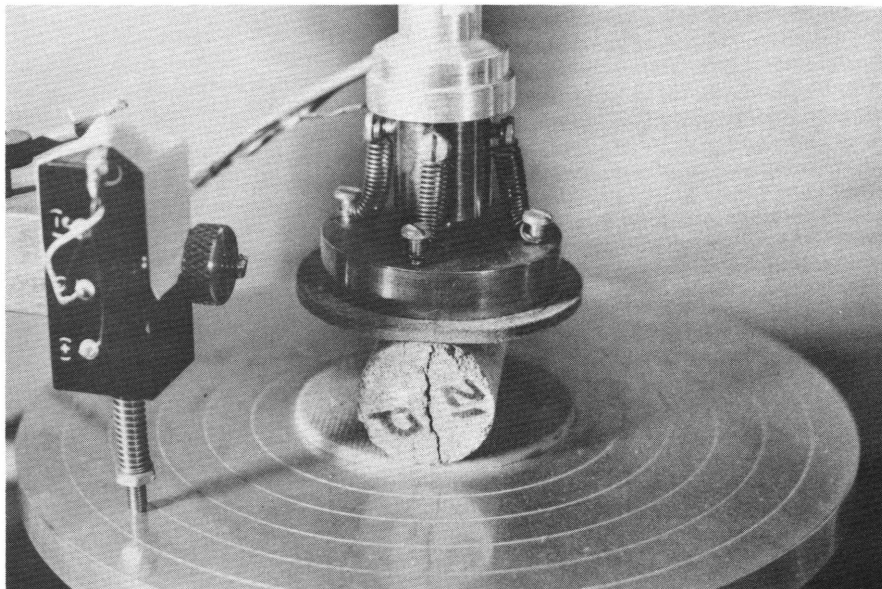


Figure 4.8: Detailed view of sample under test.

the various stabilizing agents (lime, cement, flyash, NaCl, CaCl₂, etc. were mixed to meet requirements of optimum moisture content and left for 1, 2, and 4 hours before molding or compaction. To prevent moisture loss by evaporation the mixture was covered in the container with aluminum foil. After delayed compaction, the molded samples were cured for 28 and 90 days at room temperature, and their compressive strength determined.

CHAPTER V

PRESENTATION AND DISCUSSION OF TEST DATA

The purpose of this research is to determine the type and amounts of stabilizing agents to be added to shales in order to upgrade them for highway construction purposes. This study was conducted in two phases:

1. Preliminary Laboratory Stabilization:

The shales were mixed with various amounts of stabilizing agents and at 70° and 100° F. temperatures at 90%-100% relative humidity. On the basis of the improvement of engineering properties of stabilized shales and pattern characteristics of data obtained, it was found that the most suitable quantity of stabilizing agent were 6% lime, 14% cement, and 25% flyash. These quantities were used for further investigation at 110° F. (for better environmental control) curing temperature and 90 day curing period.

2. Selective Laboratory Stabilization:

This phase included the evaluation of compressive strength, resistance to weatherability (wet-dry and freeze-thaw), response to cyclic loading, delayed compaction, mineralogy and electron microscopic studies of stabilized shales.

The engineering properties determined for stabilized shales are presented here.

GRAIN SIZE ANALYSIS

The grain size analysis was performed on the stabilizing shales cured for 28 days. To determine the effect of stabilizing agents on the clay content of shales, the numerical parameter "Aggregation Index," as mentioned in Chapter IV was used. Typical grain size analysis curves for raw and stabilized shale 32 and the relationship between AI and clay content

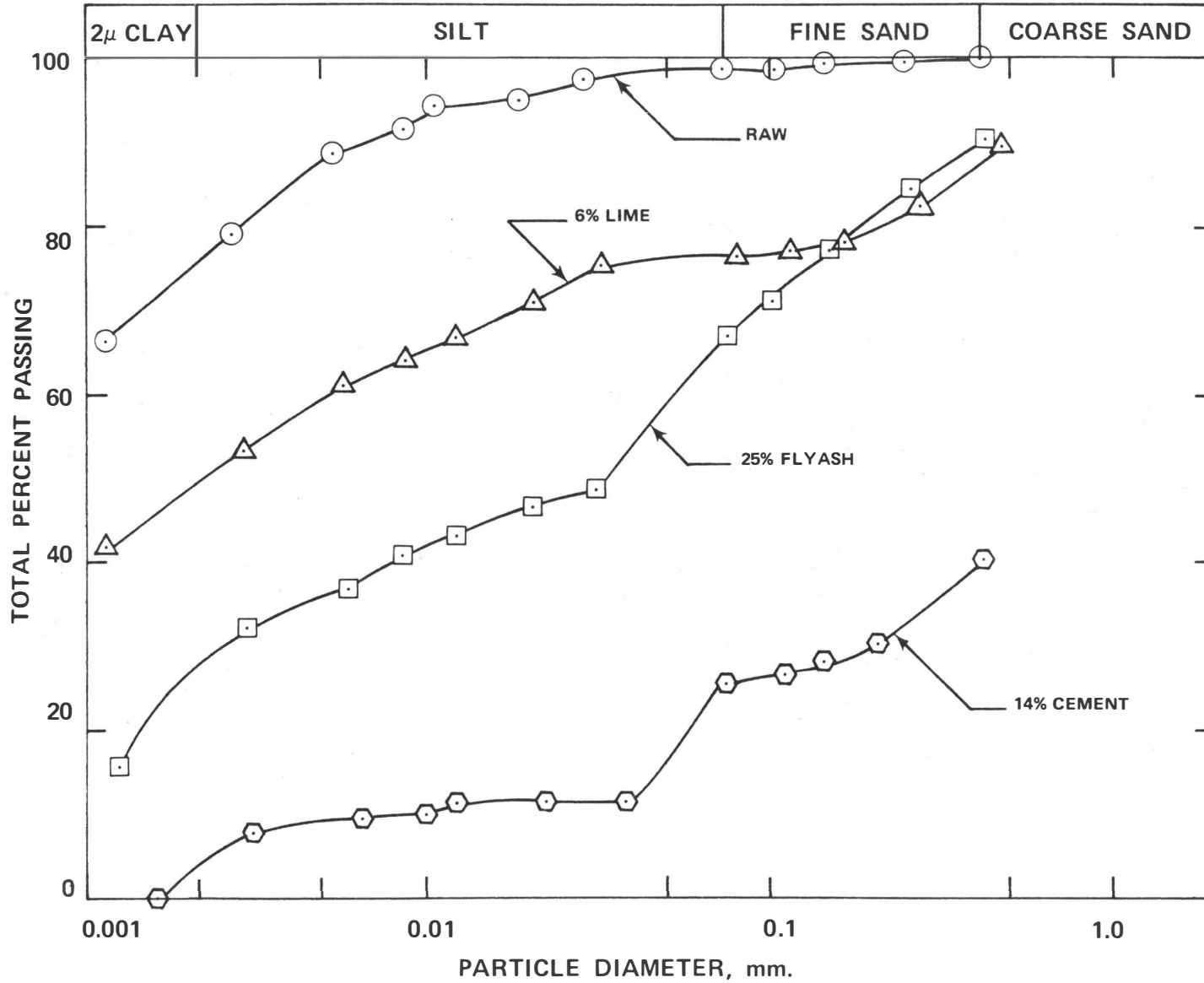


Figure 5.1: Grain size distribution curve for shale 32.

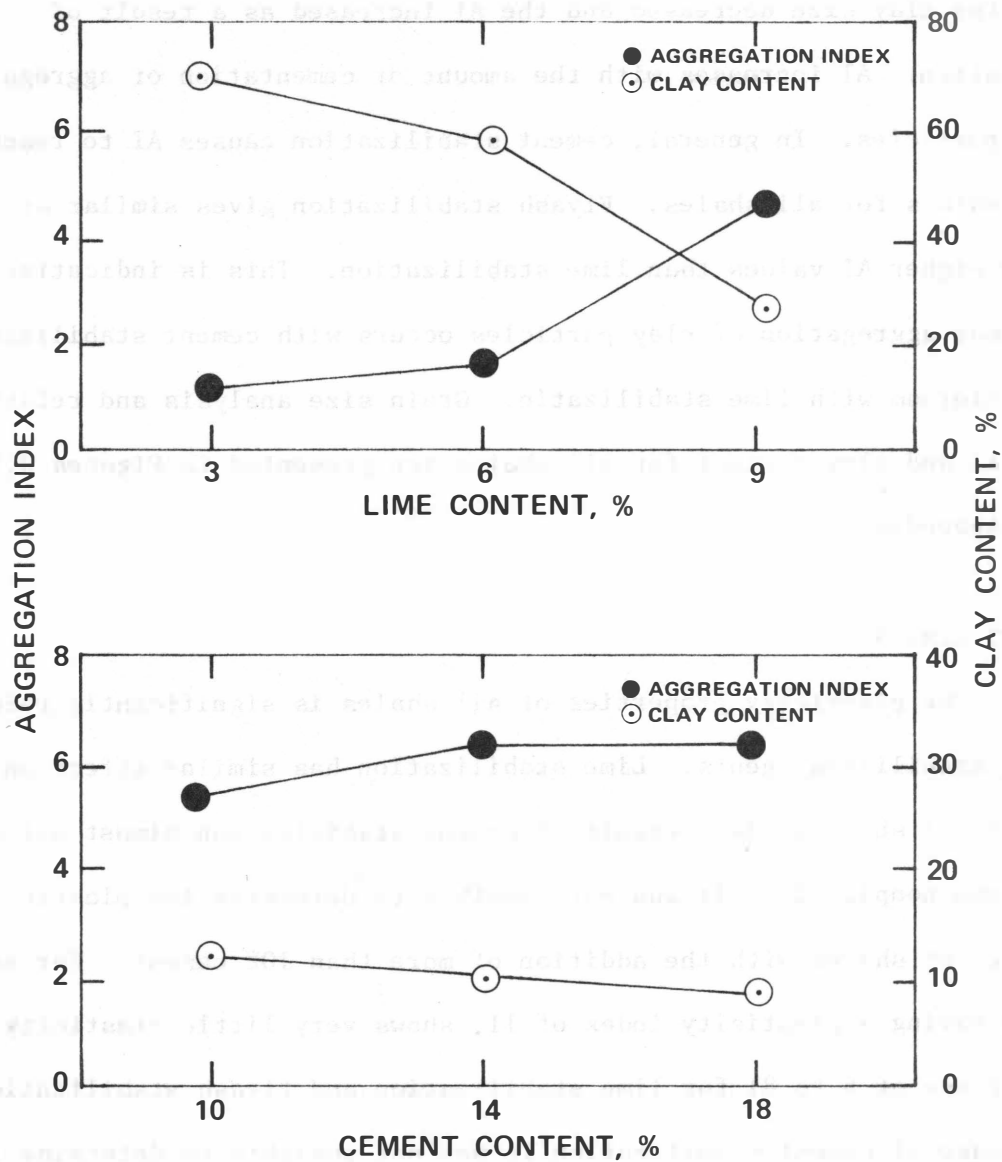


Figure 5.2: Aggregation index and clay content relationship for shale 32 stabilized with lime and cement at 110°F and cured for 28 days.

are shown in Figures 5.1 and 5.2, respectively.

Tables A.1 and A.7 (Appendix A) present the AI values for lime stabilized, cement stabilized, and flyash stabilized shales. For all shales, the clay size decreased and the AI increased as a result of stabilization. AI increases with the amount of cementation or aggregation of clay particles. In general, cement stabilization causes AI to reach the highest values for all shales. Flyash stabilization gives similar or slightly higher AI values than lime stabilization. This is indicative that the maximum aggregation of clay particles occurs with cement stabilization and the minimum with lime stabilization. Grain size analysis and relationship between AI and clay content for all shales are presented in Figures J.1 to J.13 in Appendix J.

ATTERBERG LIMITS

The plasticity properties of all shales is significantly reduced with all stabilizing agents. Lime stabilization has similar effect on the plasticity of shales. As a result of cement stabilization almost all shales have become nonplastic. It was not possible to determine the plastic properties of shales with the addition of more than 10% cement. For example, shale 29 having a plasticity index of 11, shows very little plasticity index (in the range of 5 to 8) for lime stabilization and flyash stabilization and for the case of cement stabilization it was not possible to determine its plastic properties at all.

The result of liquid limit, plastic limit and plasticity index of lime stabilized, cement stabilized flyash stabilized shales cured for 28 days at 70^o F and 100^o F at 90-100% relative humidity are presented in Tables B.1 to B.4 in Appendix B.

MOISTURE - DENSITY RELATIONSHIPS

For lime stabilized shales (6%), cement stabilized shale (14%),

and flyash stabilized shale (25%) the following relationships which predict the different engineering parameters have been established:

- (a) For lime stabilized shale (6%), the maximum dry density equation gives the prediction value in the range of -3% to +6% of actual density value:

$$\text{MDD (pcf)} = 137.66 - 1.69 (\text{OMC})$$

- (b) For cement stabilized shales, the following equations predict the maximum dry density in the range of -6% to +6% of actual dry density values:

$$\text{MDD (pcf)} = 142.91 + 1.79 (\text{OMC})$$

and in terms of plasticity index and leatham 2μ clay content of raw shale, the equation is:

$$\text{MDD (pcf)} = 118.35 - 0.129 (2\mu) - 0.28 (\text{PI})$$

- (c) For flyash stabilized shales, the following equations predict the maximum dry density values in the range of -8% to +10% of actual dry density values:

$$\text{MDD (pcf)} = 123.62 - 1.45 (\text{OMC})$$

including the 2μ clay content parameter, the equation becomes:

$$\text{MDD (pcf)} = 121.92 - 0.06 (2\mu) - 1.27 (\text{OMC})$$

The graphs of optimum moisture content versus maximum dry density of lime stabilized (6%), cement stabilized (14%), and flyash stabilized (25%) shales are presented in Figures 5.3 to 5.5. The optimum moisture content and maximum dry density for various amounts of stabilizing agents for all shales are given in Tables C.1 to C.3, in Appendix C. In comparing the density and moisture properties of raw shales and their corresponding stabilized forms the following observation can be made. The raw shales have the highest maximum dry densities and the flyash stabilized shales the lowest maximum dry densities. On the other hand, lime stabilizes shales have the highest optimum moisture content values and the raw shales have the lowest optimum moisture content values.

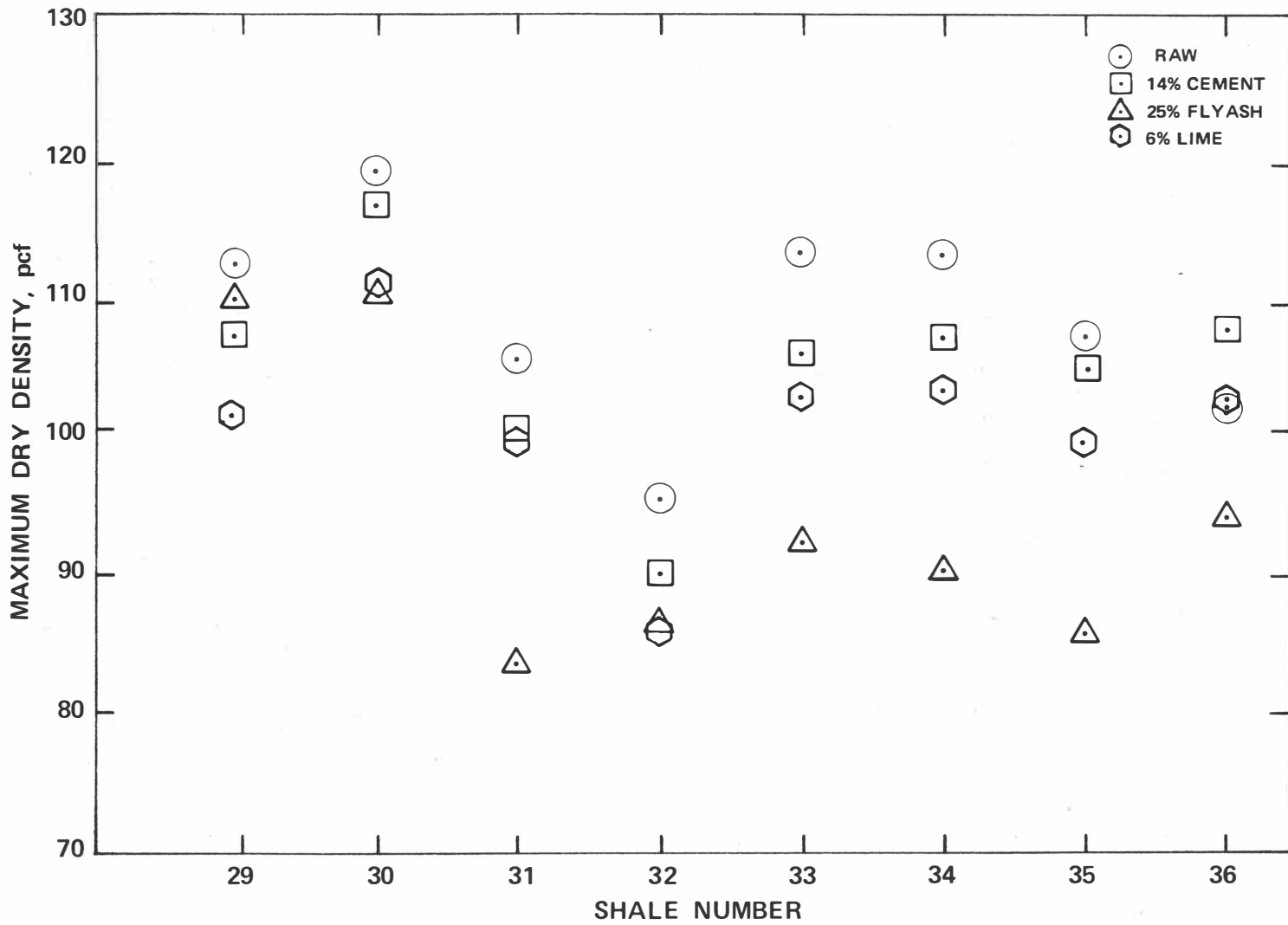


Figure 5.3: Maximum dry density of raw and stabilized shales.

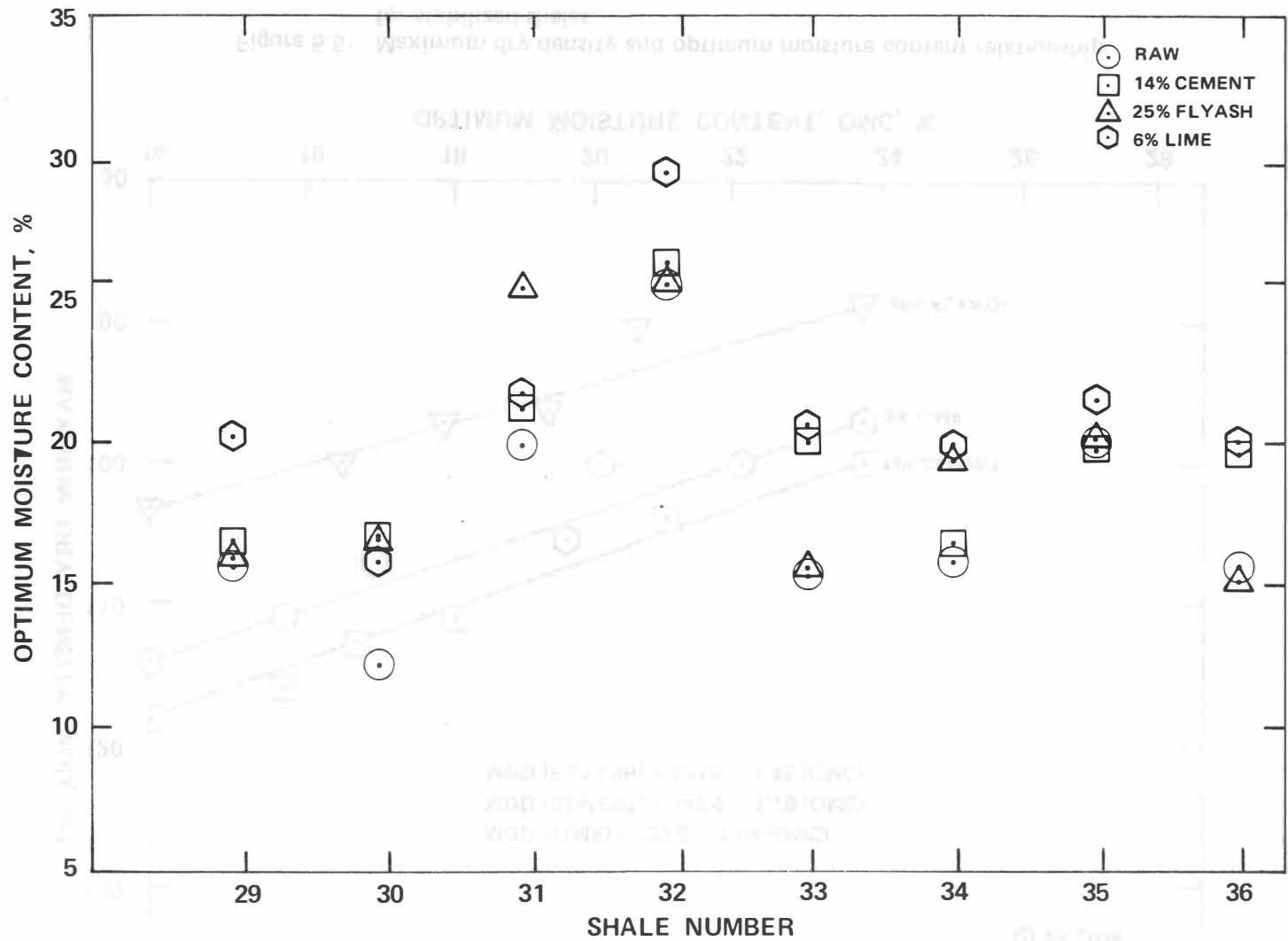


Figure 5.4: Optimum moisture content of stabilized shales.

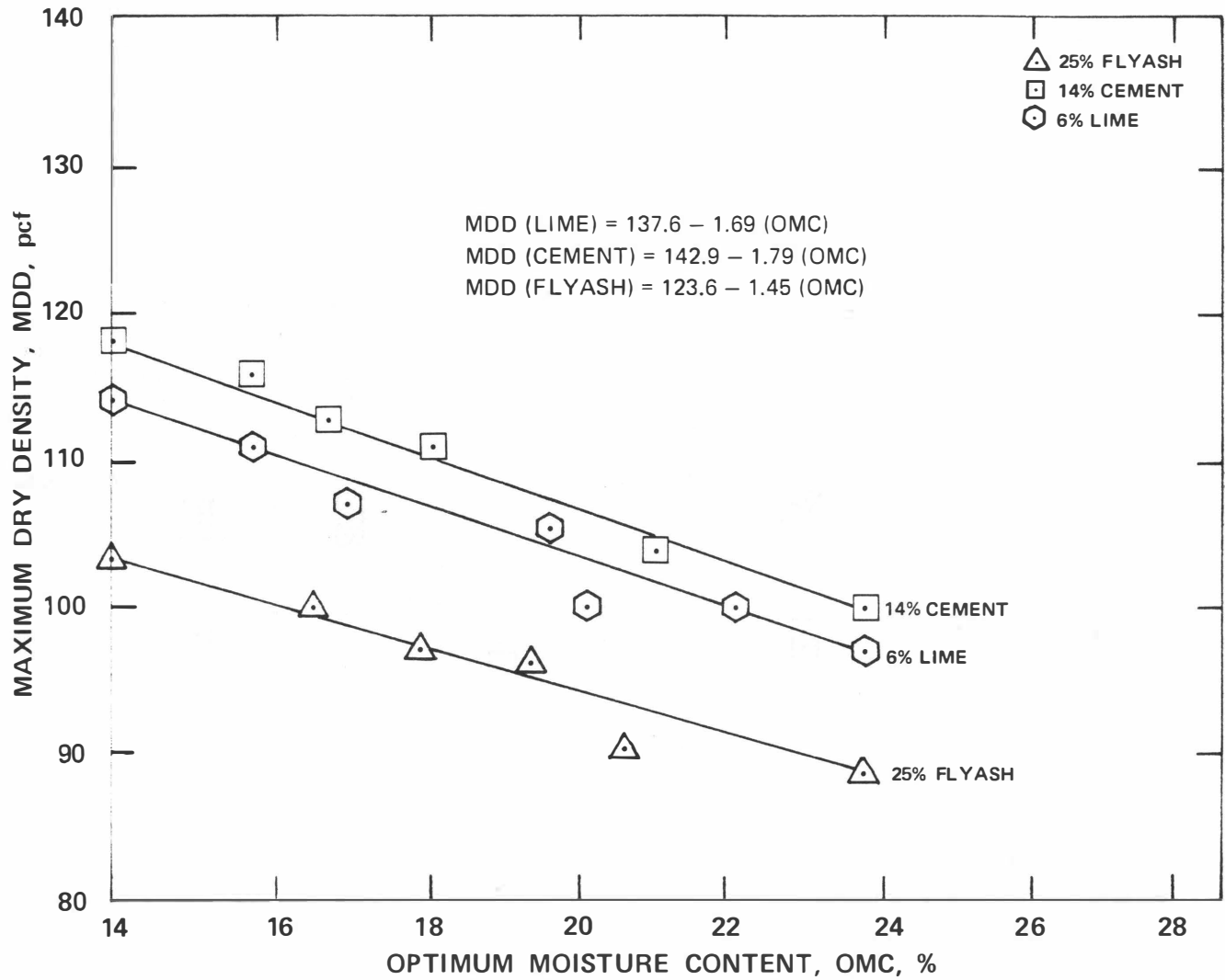


Figure 5.5: Maximum dry density and optimum moisture content relationship for stabilized shales.

DRY AND IMMERSSED STRENGTHS

Dry and immersed strengths of lime stabilized, cement stabilized, flyash stabilized and petrochemical stabilized shales cured for 28 days and 90 days at 70° F. and 110° F. and 90-100% relative humidity are presented in Tables D.1 to D.15 in Appendix D.

LIME TREATMENT

Compressive strengths of lime stabilized shales have significantly increased compared to the compressive strength of raw shales. Compressive strengths of the 90-day cured samples are higher than those of the 28-day cured samples for both curing temperatures of 70° F. and 110° F., except for shales 31 and 32. The latter two shales, which are predominantly montmorillonitic, show higher strength at 70° F. only for 90-day curing period suggesting that a longer curing period at low curing temperature is better for the montmorillonitic soil.

Immersed strength is included here as an additional test for evaluating strength and durability criteria. After 24 hours immersion in water, compressive strengths of 28-day cured samples are significantly decreased for both curing temperatures. Shales 29, 30, and 36 having comparatively lower amounts of clay, dispersed after immersion. The strength of shale 35, which is predominantly illitic and is most benefited by lime treatment, is very slightly reduced by immersion.

For the 90-day cured samples there was slight strength gain upon immersion. All strength values exhibit inconsistent pattern with increase in lime content.

STATISTICAL ANALYSIS

The statistical analysis conducted on raw and lime stabilized (6% lime, 28-day cure) shales yielded the following relationships:

Table 5.1:

CORRELATION MATRIX FOR ENGINEERING
PROPERTIES OF RAW SHALES +

<u><2μ Clay Content, %</u>	<u>PI, %</u>	<u>OMC, %</u>	<u>DD, pcf</u>	<u>C, pcf</u>	<u>φ, degree</u>	<u>RP</u>	<u>UCS, Psi</u>	
1	2	3	4	5	6	7	8	
1.00	<u>0.83</u>	<u>0.78</u>	-0.74	0.49	-0.40	<u>0.92</u>	0.52	1
	1.00	<u>0.76</u>	-0.76	<u>0.83</u>	-0.32	<u>0.95</u>	<u>0.70</u>	2
		1.00	-0.99	<u>0.55</u>	-0.54	<u>0.88</u>	<u>0.49</u>	3
			1.00	-0.61	0.57	-0.87	-0.53	4
				1.00	-0.94	<u>0.73</u>	<u>0.88</u>	5
					1.00	-0.69	-0.72	6
						1.00	<u>0.63</u>	7
							1.00	8

+ Based on IBM/360 Scientific Subroutine Package (Multiple Linear Regression)

Table 5.2:

CORRELATION MATRIX FOR VARIOUS ENGINEERING PROPERTIES OF LIME
STABILIZED SHALES AND CURED FOR 28 DAYS +

2 μ *, %	PI, %	OMC, %	MDD, pcf	LC, %	RP	UCS, Psi	Temp, F ^o	2 μ **, %	2 μ ***, %	
1	2	3	4	5	6	7	8	9	10	
1.00	0.56	0.38	-0.29	-0.74	0.46	-0.07	0.09	0.68	0.41	1
	1.00	0.49	-0.51	-0.13	0.82	-0.13	0.00	0.26	0.26	2
		1.00	-0.96	0.00	0.84	0.44	0.00	0.37	0.74	3
			1.00	-0.16	-0.85	-0.38	0.00	-0.22	-0.76	4
				1.00	0.00	0.14	0.00	-0.70	0.00	5
					1.00	0.17	0.00	0.31	0.92	6
						1.00	0.42	0.01	0.12	7
							1.00	0.00	0.00	8
								1.00	0.32	9
									1.00	10

* 2 μ clay content (%) for lime treated shales for 28 days curing period

** 2 μ clay content (%) for lime treated shales for zero day curing period

*** 2 μ clay content (%) for raw shales

+ Based on IBM/360 Scientific Subroutine Package (Multiple Linear Regression)

1. Raw Shale:

The two equations given below predict the unconfined compressive strength of raw shale in the range of $\pm 10\%$ of actual strength value:

$$\text{UCS (psi)} = -32.62 + 3.11C + 0.99\phi \dots\dots\dots (1)$$

$$\begin{aligned} \text{UCS (psi)} = & -158.36 + 0.11 (2\mu) - 0.36 (\text{PI}) . \\ & + 1.71 (\text{OMC}) + 0.97 (\text{DD}) + \\ & 0.36 (\text{C}) + 0.66 (\phi) \dots\dots\dots (2) \end{aligned}$$

2. For lime treated shales it was not possible to establish strength related equations within the proper accuracy limit. However, the following MDD and OMC relation was obtained:

$$\text{MDD (pcf)} = 137.66 - 1.69 (\text{OMC}),$$

which has already been described in the previous section of this chapter. The correlation matrix for raw and lime stabilized shales are presented in Tables 5.1 and 5.2.

The reaction potential of a shale may be evaluated as follows:

$$\begin{aligned} & [\text{RP}]_i \\ & = \frac{Y_i}{100} \left[\frac{X_1}{100} (\text{CEC})_1 + \frac{X_2}{100} (\text{CEC})_2 + \dots + \frac{X_n}{100} (\text{CEC})_n \right]_i \end{aligned}$$

$i = \text{ith shale}$

$[X_{1,2} \dots]_i = \text{Percent of 1st, 2nd, etc. clay mineral in ith shale}$

$[\text{CEC}]_i = \text{Cation exchange capacity of 1st, 2nd, etc. clay mineral of the ith shale, meq/100 gram}$

$Y_i = \% \text{ of } <2\mu \text{ clay in ith shale}$

CEMENT STABILIZATION

Compressive strengths of cement stabilized shales are higher than those of lime stabilized shales for both curing temperatures (70° F. and 110° F.) and curing periods (28 days and 90 days). Shales 31 and 32, which contain predominantly montmorillonite, indicate comparatively less strength gain than other shales for cement stabilized. The compressive

Table 5.3:

CORRELATION MATRIX FOR ENGINEERING PROPERTIES
OF CEMENT STABILIZED SHALES CURED
FOR 28 DAYS AT 110°F +

2 μ *, %	PI*, %	C.C., %	OMC, %	MDD, pcf	UCS, Psi	
1	2	3	4	5	6	
1.0	<u>0.83</u>	0.03	0.20	<u>-0.75</u>	-0.26	1
	1.0	0.02	0.16	<u>-0.76</u>	-0.54	2
		1.0	<u>0.66</u>	-0.01	-0.07	3
			1.0	-0.18	-0.52	4
				1.0	0.26	5
					1.0	6

* Engineering property of raw shales

+ Based on IBM/360 scientific subroutine package (Multiple Linear Regression).

strength predictive equation for cement (14%) stabilized 28-day cured samples at 110° F has been established. It depicts the strength value in the range of $\pm 25\%$ of actual strength value.

$$\text{UCS (psi)} = 2873 + 11.77 (2\mu) - 40.13 (\text{PI}) - 9.0 (\text{OMC} - 13.98 (\text{MDD}))$$

28 day

where $<2\mu$ clay content and PI are the values of raw shale.

IMMERSED STRENGTH

Immersed strengths of cement stabilized shales are higher when compared to lime stabilized shales. There is no significant difference between 28-day and 90-day immersed strengths for cement stabilized. For cement stabilized (14%) shales cured for 28-days at 110° F., the immersed strength predictive equation may be expressed as:

$$y = 17.60 - 3.47 (x) + 0.002 x^2$$

where,

y = immersed strength in psi

x = dry strength in psi

PETROCHEMICAL STABILIZATION

Clapak and claset which are commercial stabilizers, are sulphorated petroleum products. As the exact composition of these chemicals is not known, it is difficult to explain the mechanism of stabilization, if any. The concentration at which these chemicals are used is recommended by the manufacturer and it is based on the plasticity index values of shales. The solutions were made at specified concentrations and used in place of water with the amount corresponding to optimum moisture content requirements. Clapak was used alone and also conjunctively with the admixture 'claset' as recommended by the manufacturer for the shales with

plasticity index greater than 10. Since all shales, except shale 35, had plasticity indices greater than 10, both clapak and claset were tried. Even for the 28-day curing period, strengths were lower than those for raw shale for both curing temperatures of 70° F and 110° F as shown in Tables D.10 to D.12 in Appendix D.

It was not possible to determine the immersed strength of either 28-day or 90-day cured samples for 70° F. and 110° F. Dry strength of shales treated with clapak and claset and clapak alone and cured at 70° F. are lower than those of raw shale.

In general, it is concluded that chemical stabilization either with clapak alone, or conjunctively with claset, it is not suitable for shales considered in this study.

FLYASH STABILIZATION

As already discussed in Chapter IV, flyash has been used in this study as one of the main stabilizing agents. Compressive strengths of shales stabilized with 25% flyash are significantly increased compared to raw shales. On the basis of 28-day strengths, shales 29, 30, 33 and 36 stabilized with flyash have higher compressive strength than their lime stabilized counterpart. For all curing temperatures and all curing conditions, lime stabilized and flyash stabilized shales show lower strengths than cement stabilized shales as shown in Figure 5.6 for shale 35.

For flyash stabilized shales, longer curing periods (longer than 28 days) do not appear to further increase the compressive strength substantially. The correlation matrix for flyash (25%) stabilized shales is presented in Table 5.4.

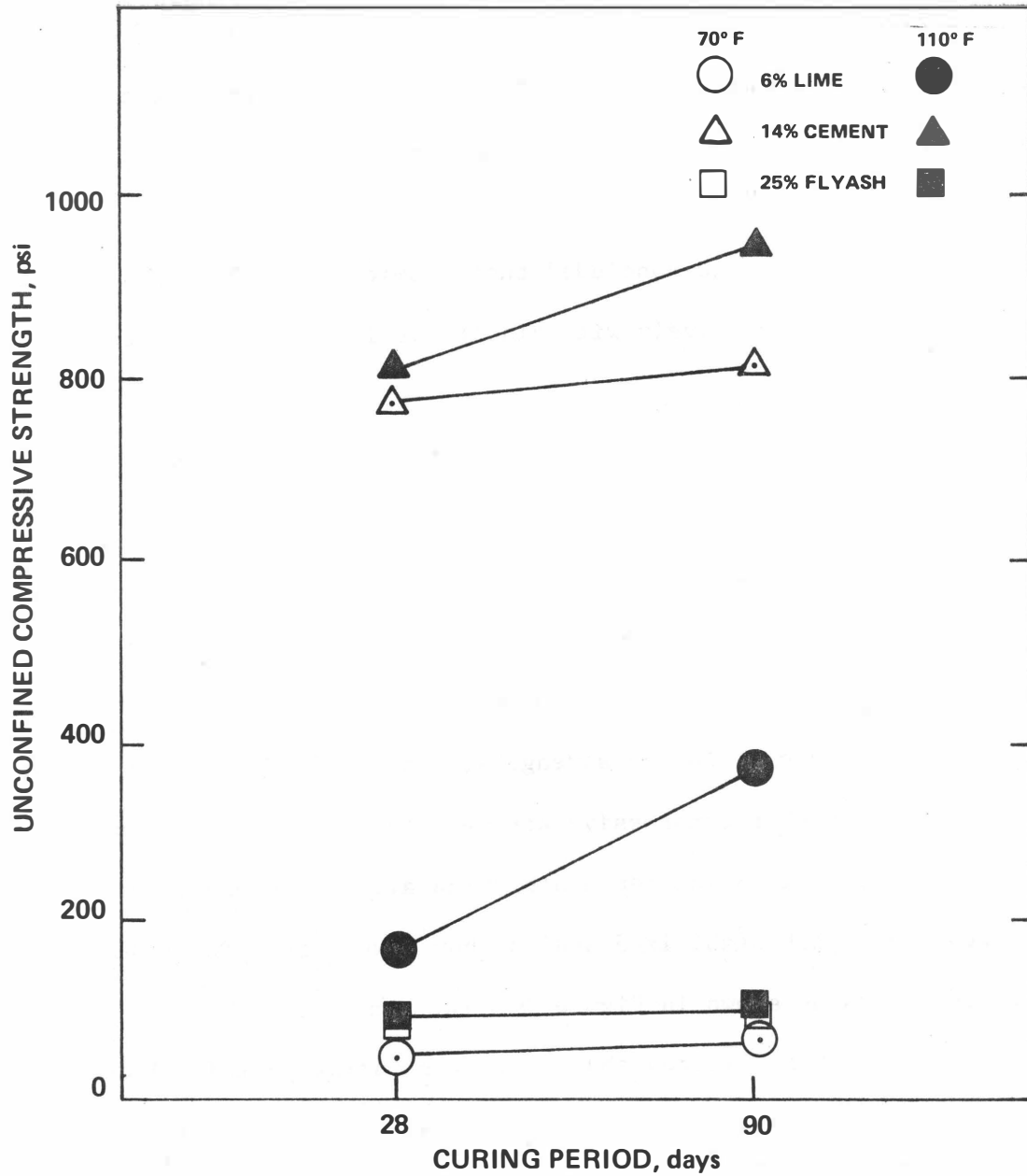


Figure 5.6: Compressive strengths of stabilized shale 34.

Table 5.4:

CORRELATION MATRIX FOR VARIOUS ENGINEERING
PROPERTIES OF FLY-ASH (25%) STABILIZED
SHALES CURED FOR 28 DAYS +

2 μ *, %	PI*, %	CMC, %	MDD, pcf	Temp, f ^o	UCS, Psi	
1	2	3	4	5	6	
1.0	<u>0.83</u>	<u>0.56</u>	-0.42	0.0	-0.33	1
	1.0	<u>0.72</u>	-0.34	0.0	-0.25	2
		1.0	<u>-0.59</u>	-0.1	-0.27	3
			1.0	0.0	0.75	4
				1.0	0.29	5
					1.0	6

* Engineering property of raw shales.

+ Based on IBM/360 scientific subroutine package
(Multiple Linear Regression)

Table 5.5:

OPTIMUM STABILIZATION CONDITIONS *

Test Condition	29	30	31	32	33	34	35	36
L-3-70-28...			X			X	X	
L-3-110-28...			X	X		X	X	X
L-6-70-28...			X		X	X	X	
L-6-110-28...	X		X	X		X	X	X
L-9-70-28...					X		X	
L-9-110-28...	X		X	X		X	X	X
C-10-70-28...		X			X	X	X	X
C-10-110-28.	X	X	X	X	X	X	X	X
C-14-70-28..	X	X			X	X	X	X
C-14-110-28.	X	X	X	X	X	X	X	X
C-18-70-28..	X	X			X	X	X	X
C-18-110-28.	X	X	X	X	X	X	X	X
FA-25-70-28.	X	X			X		X	X
FA-25-110-78	X	X			X		X	X
CP-70-28....								
CP-110-28...								
CPCS-70-28..								
CPCS-110-28.								

*Optimum stabilization conditions, indicated by X, are based on strength criteria using 27 day immersed compressive strength (for cement 250 Psi minimum and for other stabilizers, 40 psi, considering 18 kips standard minimum axle load).

Table 5.6:
COMPARISON OF STRENGTH GAIN BY THE ADDITION OF STABILIZING
AGENTS TO SHALES ON THE BASIS OF 28-DAY AND 90-DAY CURING

Shale No.	Stabilizing Agents			Lime versus Flyash Stabilization
	Lime	Cement	Flyash	Remark
29	Low	High	Medium	Flyash is better than lime
30	Low	High	Medium	
31	Medium	High	Low	Lime is better than flyash
32	Medium	High	Low	
33	Low	High	Medium	Flyash is better than lime
34	Medium	High	Low	Lime is better than flyash
35	Medium	High	Low	Lime is better than flyash. Maximum strength beneficia- tion for both stabilizing agents
36	Low	High	Medium	Flyash is better than lime

Akoka

COMPARATIVE EVALUATION OF STABILIZING AGENTS

On the basis of dry and immersed compressive strengths of stabilized shales cured for 28 days and 90 days at 70° F and 110° F, the various stabilization conditions are compared for all shales in Tables 5.5 and 5.6

Test conditions are designated as L-3-7-28, C-14,110-28, etc., which indicate stabilizer type-amount-curing temperature-curing period, respectively. For example, L-3-70-28 means stabilizer: lime, amount: 3%, curing temperature, 70° F and curing period: 28 days. On the other hand, with petrochemical stabilizers the amount varies for each shale, and therefore, the amount is not indicated. The symbols used for stabilizers are L for lime, C for cement, FA for flyash, CP for clapak, and CPCS for clapak-claset. Optimum stabilization conditions for each shale, indicated in Table 5.5, are selected on the basis of immersed compressive strength. For cement stabilization, a minimum immersed compressive strength of 250 psi is chosen as recommended by the Portland Cement Association (250 psi for light traffic and 400 psi for heavy traffic). For other stabilizing agents (considering 18 kips standard axle load), the minimum is set at 40 psi. Comparison of strength characteristics for various stabilization conditions for all shales are shown in Figures J.14 to J.21 in Appendix J.

It appears that no stabilizer is effective for shale 32 at a curing temperature of 70° F. Shale 35 responds very well for all stabilizers used except the commercial petrochemical stabilizers 'clapak' and 'claset'. The data lead to the following conclusions which are also summarized in Table 5.6:

1. Cement stabilization imparts the highest strength gain
2. All stabilizing agents impart the least strength to shale 32 (having the maximum PI and significant amount of montmorillonite) at the curing temperature of 70° F.

3. Shale 35, having the lowest PI and being predominantly illitic, responds well to all stabilizers
4. Lime and flyash stabilization contribute to moderate strength gain. For shales 29, 30, 33, and 36, having comparatively lower optimum moisture content than other shales, flyash was found more beneficial than lime
5. Chemical stabilization is not effective for this study

Based on the results of dry and immersed strengths, the selective laboratory stabilization included the following:

1. The use of stabilizing agents lime 6%, cement 14%, flyash 25% alone and conjunctively, and the latter with amounts reduced. Also, chemicals such as NaCl, NaOH, CaCl₂ and Na₂CO₃ were used as secondary additive
2. The use of 110° F. for better environmental control

WET-DRY CYCLES

The lime stabilized, cement stabilized, and flyash stabilized shales were cured for 28 days and 90 days at 70° F. and 90-100% relative humidity and subsequently were subjected to 5, 10 and 15 wet-dry cycles. The data are presented in Tables E.1 to E.3 in Appendix E and in Figure 5.7.

1. Lime Stabilization:

As a result of 5 and 15 wet-dry cycles, compressive strengths of all shales, except shales 30, 33, 34 and 36 were reduced. For plastic shales 31 and 32, it was not possible to measure their strength due to the complete breakdown of the sample when subjected to wet-dry cycles. Shales 30, 34 and 36, containing comparatively higher amounts of silt, show increase in strength as a result of wet-dry cycles. Shale 33, having the maximum aggregation index (cementation as a result of lime treatment), shows increase in compressive strength only for the 28-day curing period.

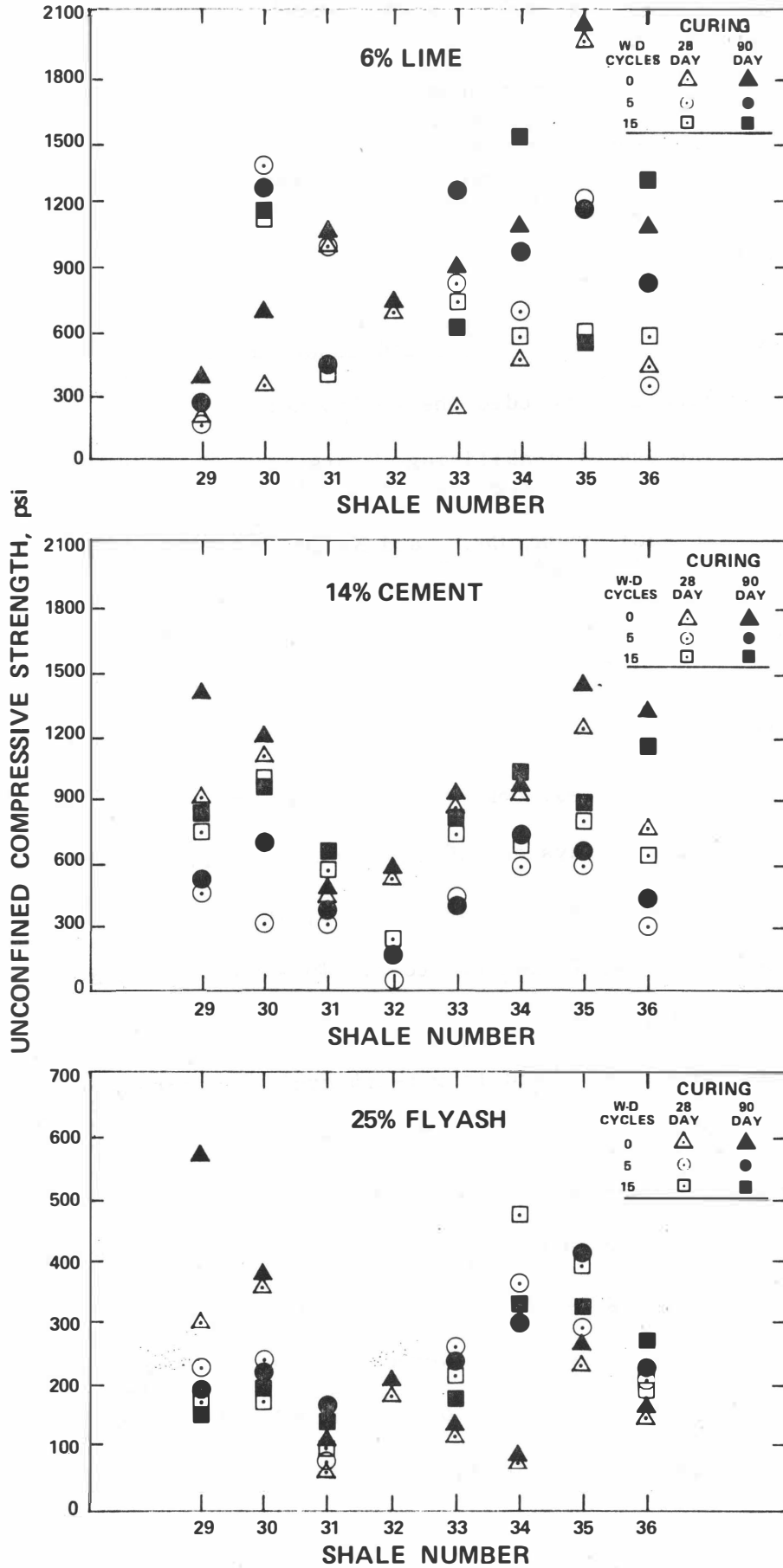


Figure 5.7: Compressive strengths of stabilized shales after being subjected to wet-dry cycles.

2. Cement Stabilization:

As a result of wet-dry cycles, the compressive strengths of all cement stabilized shales, except shale 31, are nearly equal. Shale 31, which is predominantly montmorillonitic, indicates slight increase in strength as a result of wet-dry cycles for both 28-day and 90-day curing periods after being subjected to 5, 10 and 15 wet-dry cycles. The compressive strength of cement stabilized shales are higher than lime stabilized and flyash stabilized shales.

3. Flyash Stabilization:

Compressive strengths of all flyash stabilized shales, except shales 33, 34, 35 and 36, are reduced as a result of wet-dry cycles. For shale 32, having the highest PI, it was not possible to determine the compressive strength due to disintegration. Shales 33, 34, 35 and 36, containing significant amount of illite, show significant increase in compressive strength as a result of wet-dry cycles.

FREEZE-THAW CYCLES

The compressive strengths of lime stabilized shales cured for 28 days and 90 days at 110° F and 90-100% relative humidity and subsequently subjected to 5, 10 and 15 freeze-thaw cycles are presented in Tables E.4 to E.6 in Appendix E and in Figure 5.8.

1. Lime Stabilization:

The compressive strengths of all shales are reduced as a result of freeze-thaw cycles. For shales 29, 36 and 32, containing very low and very high amounts of less than two micron (2μ) clay, respectively, it was not possible to measure the compressive strength due to the breakdown of the samples when subjected to freeze-thaw cycles for both 28-day and 90-day curing periods. For 90-day cured samples, the reduction due to 5 freeze-thaw cycles is not significant.

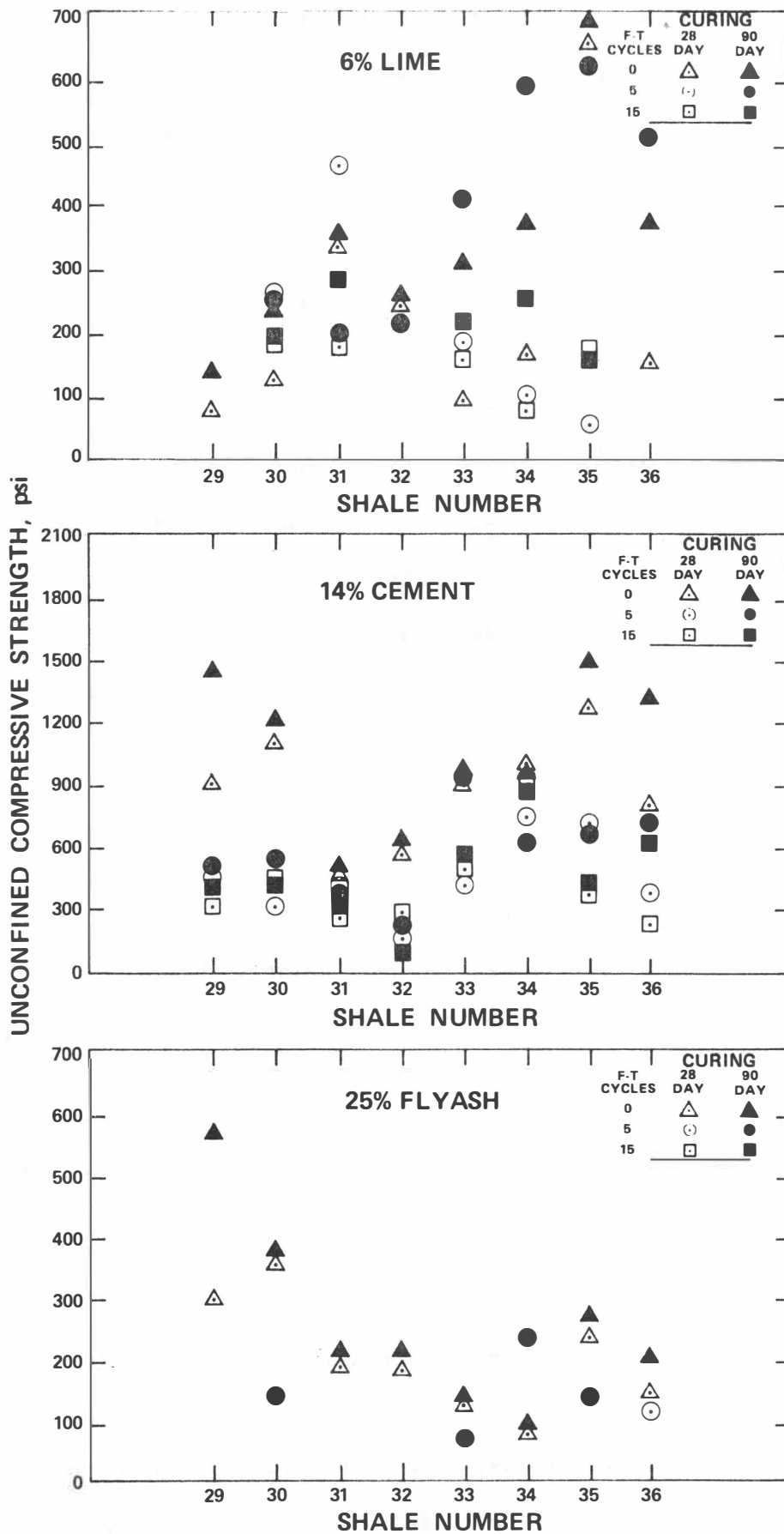


Figure 5.8: Compressive strengths of stabilized shales after being subjected to freeze-thaw cycles.

2. Cement Stabilization:

For all shales, the compressive strength is reduced by significant amounts as a result of freeze-thaw cycles. However, the reduced strengths of cement stabilized shales are higher than for lime stabilization for all shales.

3. Flyash Stabilization:

For flyash stabilization it was possible to measure the strength of nonplastic shales (33, 34, and 35) only which were cured for 90 days and were subjected to 5 freeze-thaw cycles (Figure 5.7); however, it was not possible to measure compressive strengths of these shales after 15 freeze-thaw cycles. Admittedly, such a high number of freeze-thaw cycles is unlikely to occur in Oklahoma.

CYCLIC LOADING

The repetitive load testing on lime stabilized, cement stabilized, and flyash stabilized shales were conducted for 40, 80, 120 and 160 lb. loads at 20 and 40 APM. Typical test results are presented in Figures 5.9 and 5.10 for all stabilizing agents and all shales. As previously stated each load and frequency were applied 50 times until failure was detected.

1. Lime Stabilization:

There is no consistent pattern for failure loads of cyclic loading for dry samples of lime stabilized shales. In general, almost all shales, except 30, 34 and 35, cured for 90 days show failure loads of 80 lb. There is a tendency of the failure loads to decrease at higher rates of load application. Shales 30 and 34, containing higher amounts of silt, and the predominantly illitic shale 35, did withstand the failure load of 120 lb.

2. Cement Stabilization:

Almost all cement stabilized shales, except shale 32,

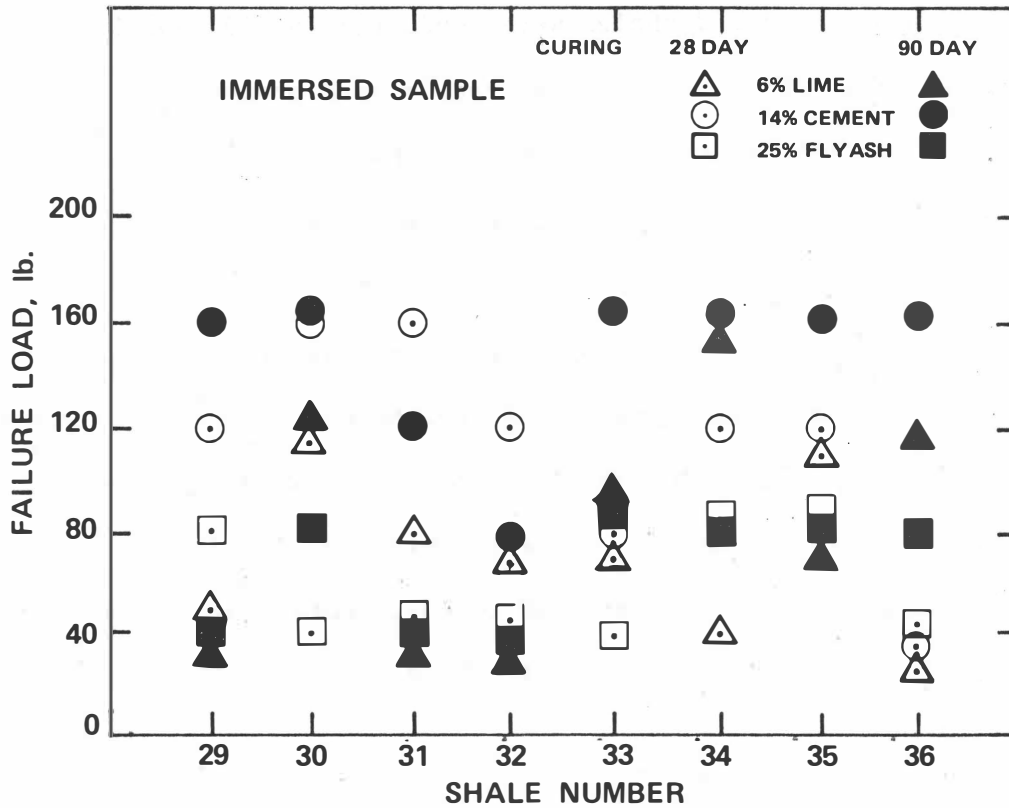
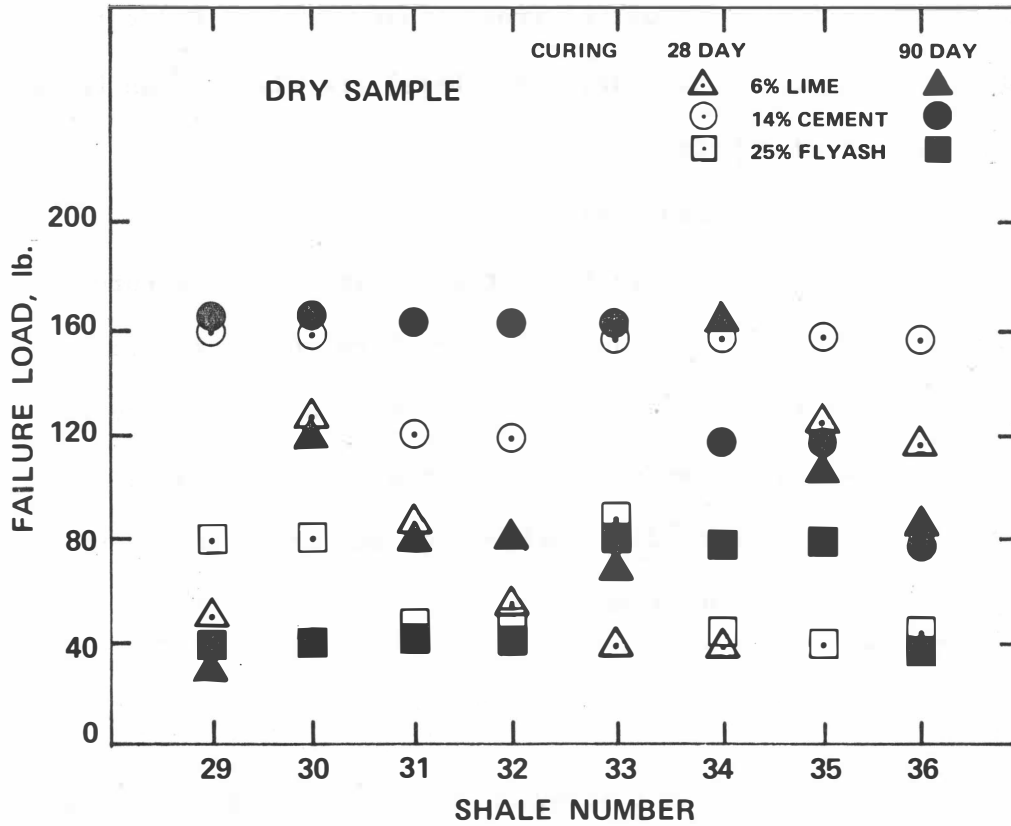


Figure 5.9: Failure loads of stabilized shales for cyclic loading at 24 applications per minute frequency.

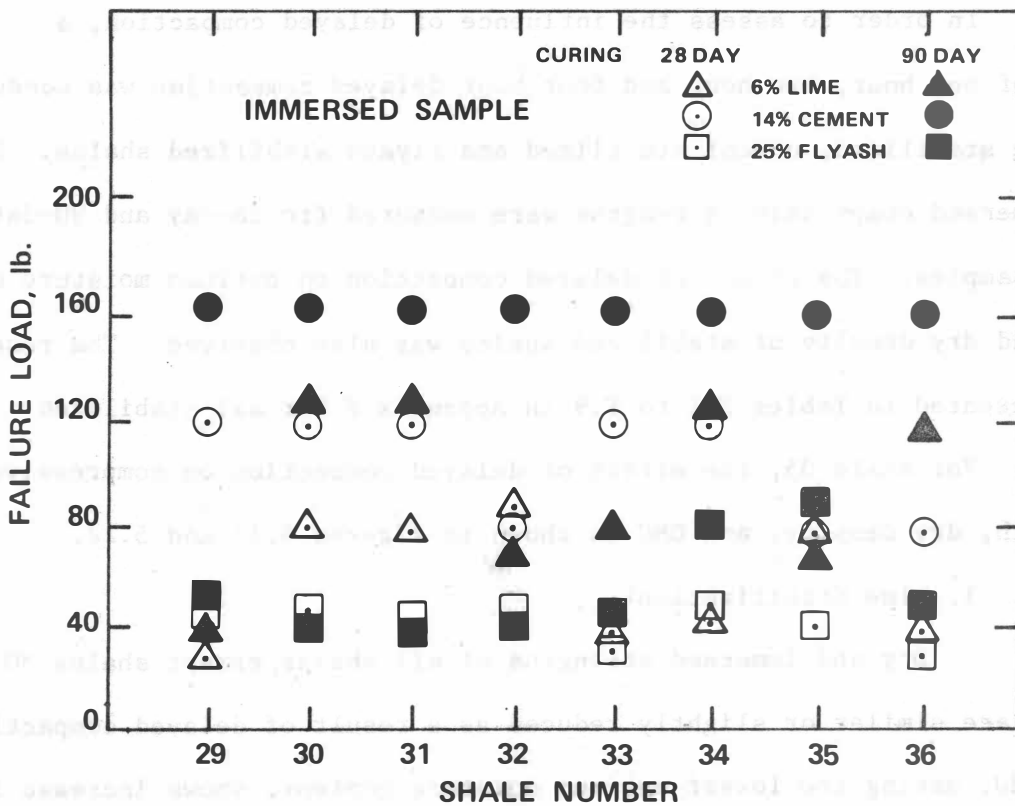
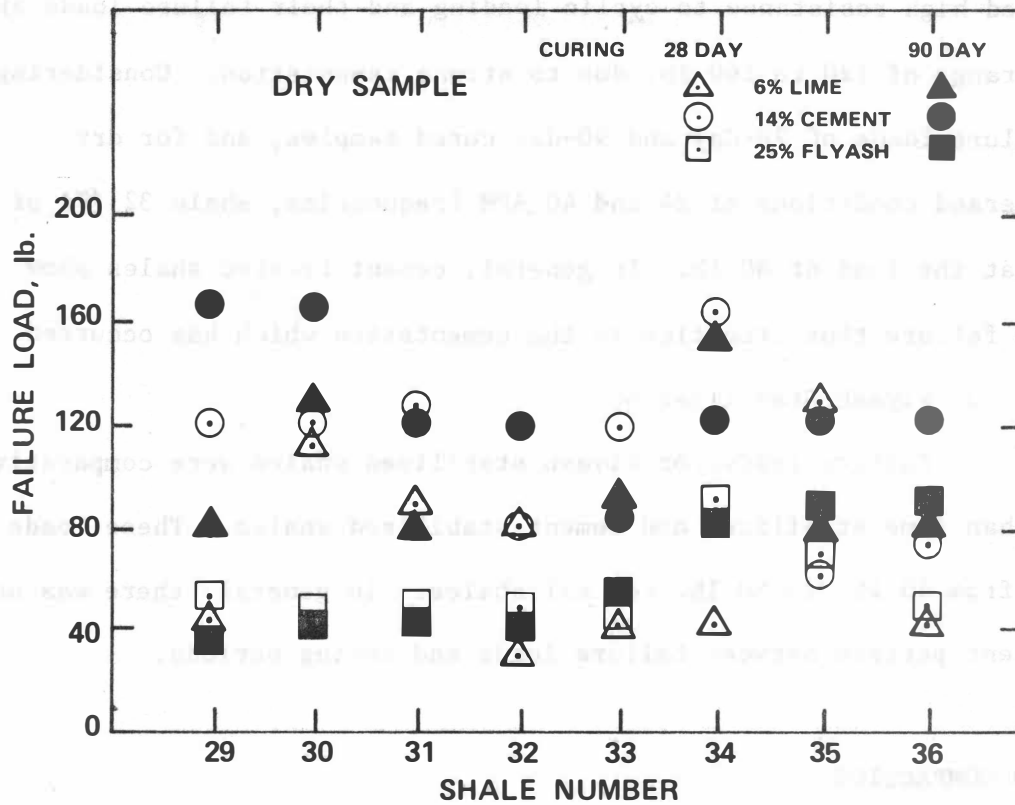


Figure 5.10: Failure loads of stabilized shales for cyclic loading at 40 applications per minute frequency.

exhibited high resistance to cyclic loading and their failure loads are in the range of 120 to 160 lb. due to strong cementation. Considering all failure loads of 28-day and 90-day cured samples, and for dry and immersed conditions at 24 and 40 APM frequencies, shale 32 (Pi of 40), failed at the load of 80 lb. In general, cement treated shales show brittle failure thus attesting to the cementation which has occurred.

3. Flyash Stabilization:

Failure loads for flyash stabilized shales were comparatively lower than lime stabilized and cement stabilized shales. These loads varied from 40 lb. to 80 lb. for all shales. In general, there was no consistent pattern between failure loads and curing periods.

DELAYED COMPACTION

In order to assess the influence of delayed compaction, a study of one hour, two hour and four hour delayed compaction was conducted on lime stabilized, cement stabilized and flyash stabilized shales. Dry and immersed compressive strengths were measured for 28-day and 90-day cured samples. The effect of delayed compaction on optimum moisture content and dry density of stabilized shales was also observed. The results are presented in Tables F.1 to F.9 in Appendix F for all stabilized shales. For shale 35, the effect of delayed compaction on compressive strength, dry density, and OMC is shown in Figures 5.11 and 5.12.

1. Lime Stabilization:

Dry and immersed strengths of all shales, except shales 30 and 31, are similar or slightly reduced as a result of delayed compaction. Shale 30, having the lowest optimum moisture content, shows increase in dry strength. Shale 31, predominantly montmorillonitic, shows significant gain for dry and immersed condition as a result of delayed compac-

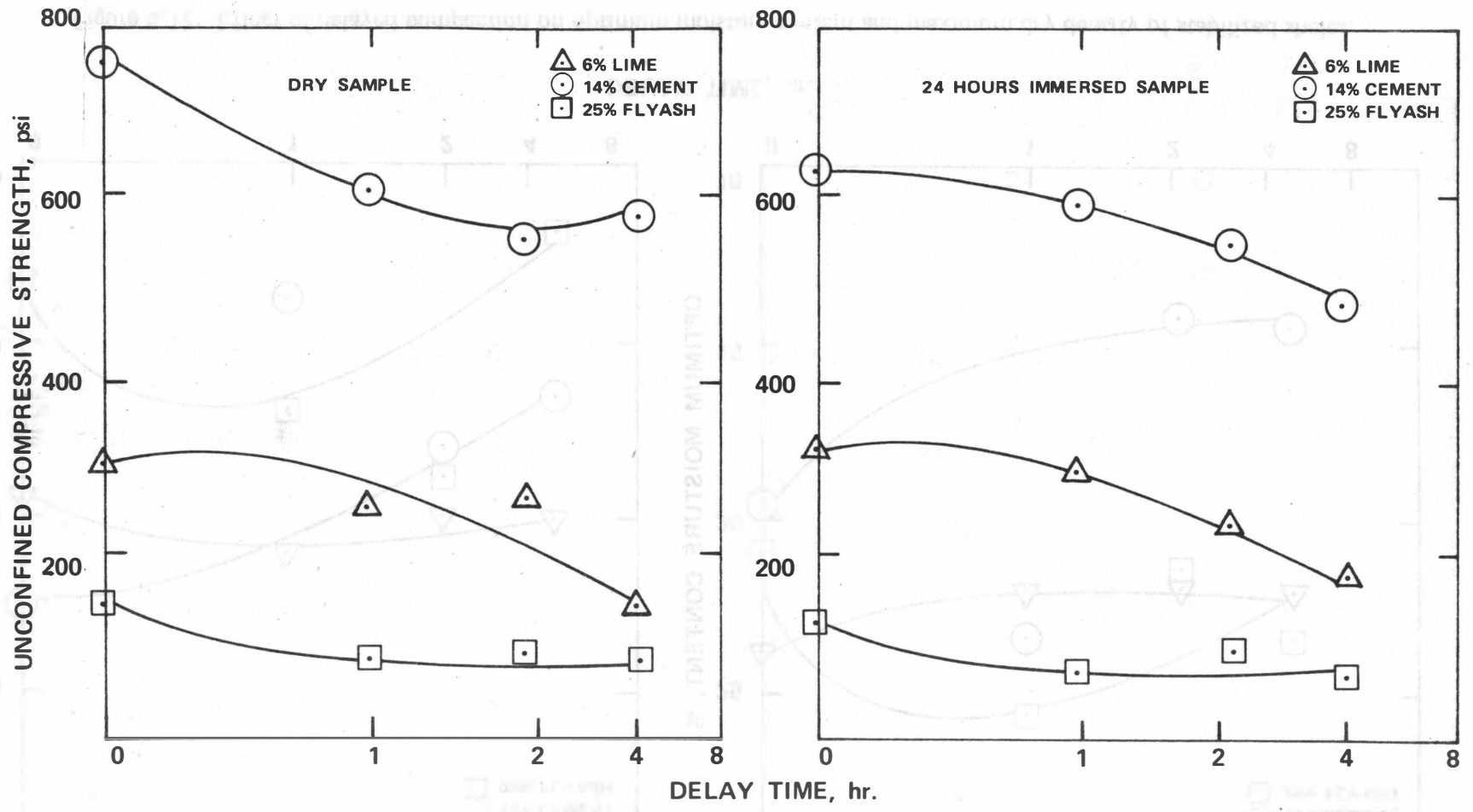
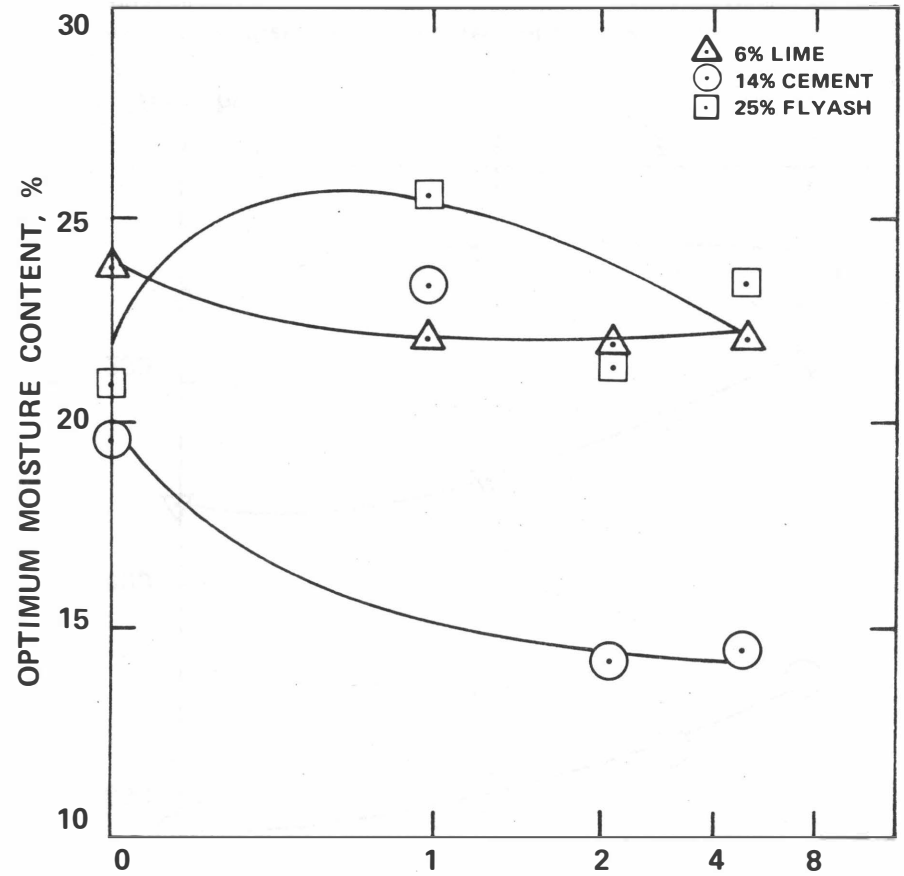
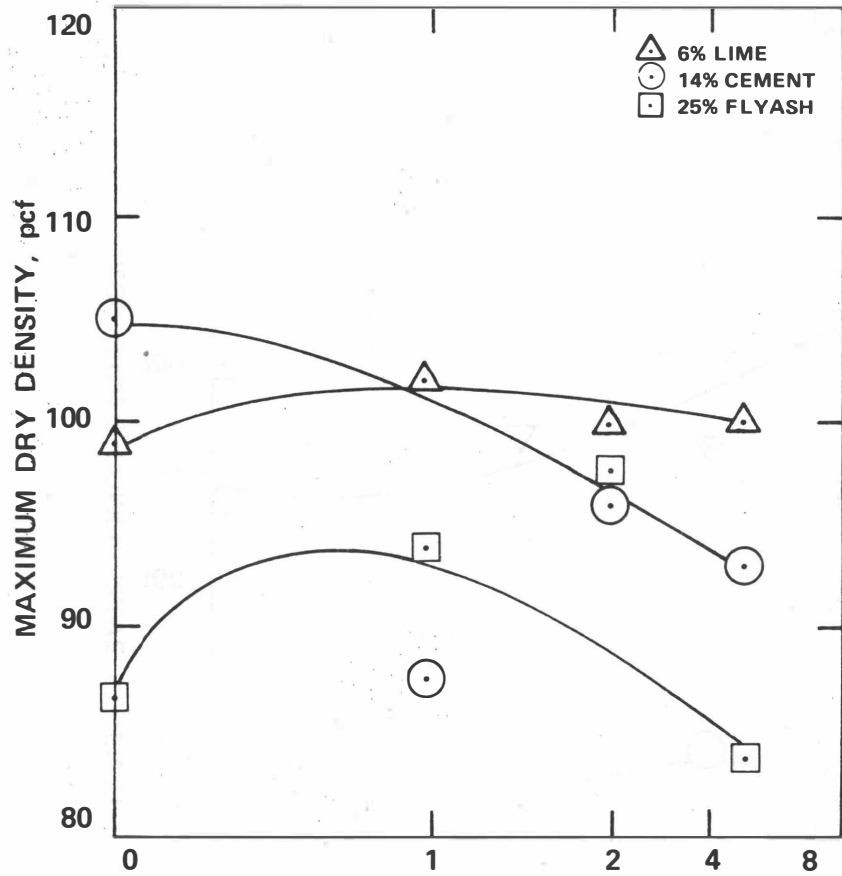


Figure 5.11: Effect of delayed compaction on stabilized shale 35 cured for 28 days.



DELAY TIME, hr.

Figure 5.12: Effect of delayed compaction on optimum moisture content and maximum dry density of stabilized shales.

tion. The reduced strength of shale 35 (as a result of delayed compaction), which is most benefited by lime stabilization, is higher than the reduced strength of other shales. The optimum moisture content and dry density of lime shales are reduced as a result of delayed compaction.

2. Cement Stabilization:

Dry and immersed strengths of all cement stabilized shales, except shale 31, cured for 28 and 90 days are reduced as a result of delayed compaction. The optimum moisture content and dry density of all shales are also reduced as a result of delayed compaction. Shale 31, which contains the highest amount of montmorillonite and which was least benefited by cement addition, shows significant strength gain as a result of delayed compaction.

3. Flyash Stabilization:

Delayed compaction for flyash stabilized shales appears to be slightly beneficial in that the dry and immersed strengths of all shales, except shale 35, did not decrease; in fact, in some cases the strengths increased as a result of delayed compaction. Shale 35, with the lowest PI and the maximum amount of illite, shows slight decrease in strength as a result of delayed compaction. There is no consistent pattern for optimum moisture content and dry density. In general, OMC and dry density for all shales, except shale 34, are reduced. Shale 34, containing the highest amount of silt, shows significant increase in dry density as a result of delayed compaction.

TRIAXIAL COMPRESSIVE STRENGTH

The triaxial compressive strength test was used as a part of the selective laboratory stabilization. The results of these tests on 90-day, 110° F. cured samples for lime stabilized (6%), cement stabilized

Table 5.7:

TRIAXIAL TEST RESULTS ON LIME STABILIZED
SHALES CURED FOR 90 DAYS AT 110°F

Shale Number	Maximum Deviatoric Stress at				Cohesion (c) psi	Angle of Friction (ϕ), degrees
	$\sigma_3 = 0$, psi	$\sigma_3 = 10$, psi	$\sigma_3 = 20$, psi	$\sigma_3 = 30$ psi		
29	131.4	346.0	524.2	556.0	23.2	64.1
30	238.5	758.3	666.4	903.2	78.4	59.3
31	360.7	651.3	702.4	758.6	73.1	65.5
32	258.8	629.6	612.9	552.7	34.8	64.1
33	809.6	901.2	591.4	928.4	36.5	65.5
34	370.2	1387.2	1470.1	782.2	74.6	73.7
35	698.0	1473.2	1805.4	1532.2	62.5	68.4
36	369.6	1083.2	1211.2	690.5	93.0	64.1

Table 5.8:

TRIAxIAL TEST RESULTS ON CEMENT STABILIZED
SHALES CURED FOR 90 DAYS AT 110°F

Shale Number	Maximum Deviatoric Stress at				Cohesion (c) psi	Angle of Friction (ϕ), degrees
	$\sigma_3 = 0$, psi	$\sigma_3 = 10$, psi	$\sigma_3 = 20$, psi	$\sigma_3 = 30$ psi		
29	1438.1	1298.2	1176.1	1336.2	74.0	73.7
30	1204.1	1821.2	2154.2	1864.2	88.2	70.6
31	477.5	986.2	1113.2	1442.2	35.7	45.2
32	611.9	557.2	921.4	845.3	52.2	64.1
33	938.8	1809.0	1696.2	1728.2	111.1	73.7
34	941.3	1943.2	582.1	1329.1	95.3	78.2
35	1238.7	1543.2	1482.0	2102.2	125.0	68.5
36	1286.7	2305.0	1853.2	1691.2	111.1	73.7

Table 5.9:

TRIAXIAL TEST RESULTS ON FLYASH STABILIZED
SHALE CURED FOR 90 DAYS AT 110°F

Shale Number	Maximum Deviatoric Stress at				Cohesion (c), psi	Angle of Friction (ϕ), degrees
	$\sigma_3 = 0$, psi	$\sigma_3 = 10$, psi	$\sigma_3 = 20$, psi	$\sigma_3 = 30$ psi		
29	569.0	330.2	326.3	349.2	64.6	42.1
30	381.6	390.4	414.1	588.4	27.2	51.1
31	117.9	305.2	228.5	390.6	26.7	68.2
32	216.5	223.3	262.4	323.2	27.4	64.6
33	130.2	288.1	230.2	281.3	21.5	56.2
34	94.8	629.4	461.5	716.6	38.8	59.0
35	273.7	714.2	611.4	656.3	46.6	59.1
36	204.7	453.2	440.5	517.6	39.9	53.4

(14%), and flyash stabilized (25%) shales are presented in Tables 5.7 to 5.9.

1. Lime Stabilization:

As a result of lime stabilization, the cohesion and the internal friction angles of all shales are significantly increased. In general, shales displaying higher cohesion values have higher compressive strengths. Shale 29, containing the lowest amount of less than 2 μ clay and the lowest cohesion value of 23.2 psi after lime stabilization, shows the lowest compressive strength of 131 psi.

2. Cement Stabilization:

The internal friction of cement stabilized shales are similar to lime stabilized shales, but the cohesion of cement stabilized shales is higher than that of lime stabilized shales. In general, shales having high cohesion value, show high compressive strengths. Shale 31, predominantly montmorillonitic and having the lowest cohesion and internal friction angle (35 psi and 45°, respectively) displays the lowest compressive strength of 477 psi after cement stabilization.

3. Flyash Stabilization:

The cohesion and internal friction angles of flyash stabilized shales are lower than those for cement stabilized and lime stabilized shales for almost all shales. Shale 29, containing the lowest amount of less than 2 μ clay, shows the highest cohesion of 64.6 psi after flyash stabilization and the lowest cohesion of 23.2 psi after lime stabilization. Also the compressive strength of flyash stabilized shale 29 is higher than its lime stabilized counterpart.

STABILIZATION WITH ADMIXTURES

To enhance the effectiveness of main stabilizing agents, the

Table 5.10:

TEST OF EFFECTIVENESS FOR CEMENT AND
CEMENT + LIME TREATED SHALES

F* DISTRIBUTION LEVEL OF
SIGNIFICANCE 0.05

Shale Number	Compressive Strength, Psi		F _{α*} =F.05	Hypothesis	Remark
	Cement (14%)	Cement + Lime (10% + 4%)		(Ho) $\sigma_1^2 = \sigma_2^2$	
29	515.5	502.0	32.47	Accepted	$S_c = S_c + C$
30	895.4	469.5	.084	Rejected	$S_c > S_c + L$
31	88.7	359.0	.037	Rejected	$S_c < S_c + L$
32	386.6	295.5	0.295	Rejected	$S_c > S_c + L$
33	568.3	402.6	0.314	Rejected	$S_c > S_c + L$
34	768.5	386.0	.051	Rejected	$S_c > S_c + L$
35	772.2	885.3	4.70	Accepted	$S_c \approx S_c + L$
36	507.2	416.2	207.42	Rejected	$S_c > S_c + L$

* F.05 (1,2) = 161

Ho: $\sigma_1^2 = \sigma_2^2$ (σ_1^2 and σ_2^2 are variance of strength of cement
and cement and cement + lime treated shales)

S = Strength

* Statistics in Research-OSTLE, Iowa State College Press

following two approaches were used:

1. Conjunctive use of main stabilizing agents (lime, cement and flyash). In this approach the amount of each stabilizing agent in the combination was less than the amount used when the shale was stabilized with one agent. For example, as a main stabilizing agent the amounts of cement, lime and flyash were 14%, 6%, and 25%, respectively, by the dry weight of shale. But when cement and lime were used conjunctively, the amounts of cement and lime in this combination were 10% and 4%, respectively, instead of 14% and 6%. The conjunctive use serves two purposes simultaneously. First, it reduces the amount of costly stabilizing agents and second, it gives equivalent desirable strength. For example, as shown in Table 5.10, the combination of cement and lime (10% and 4%) for montmorillonitic shale 31 is better than 14% cement stabilization alone. In this case, we find that shale 31, which could not be stabilized adequately with 14% cement, was stabilized by reducing cement to 10% but with the addition of 4% lime. Similarly, for other shales (shale 29 and 35) we find the same pattern of behavior. To determine the effectiveness of the combination of stabilizing agents, a unitless parameter Strength Beneficiation Index (SBI), as mentioned in Chapter IV, is employed. The SBI values for all possible combinations of main stabilizing agents are given in Appendix G.

2. The second approach to enhance the effectiveness of main stabilizing agent was to use very small amounts (varying 0.5% to 2%) of NaCl, CaCl₂, Na₂CO₃, or NaOH as a secondary additive with the various main stabilizing agents (or its combinations). To determine the effectiveness of the secondary additives, the Strength Beneficiation Index (SBI) was used and the results are given in Appendix G.

For lime stabilization, the addition of small amount of NaCl to

silty shales seems beneficial. Shale 33, which contains the least amount of clay, is most beneficiated by the addition of 2% NaCl in (6%) lime stabilization. A possible reason might be the moisture retention property of sodium chloride, which reduces the rate of moisture evaporation from soil. In other words, the addition of NaCl to silty soils partially fills the voids with the required moisture and finally increases the compressive strength.

COMPARATIVE EVALUATION OF STABILIZING AGENTS

To stabilize the selected shales, three approaches were used:

1. Used main stabilizing agents separately (cement, lime and flyash)
2. Conjunctive use of main stabilizing agents
3. Small amount of secondary additives with a main stabilizing agent or with a combination of main stabilizing agents

All possible sets of main stabilizing agents, the conjunctive use of main stabilizing agents, and combinations with secondary additives are given in Table 5.11. To decide on the suitability of stabilizing agents for shales, the criterion used was the 28-day immersed compressive strength. For cement stabilized shales, the immersed compressive strength of 250 psi was used (as recommended by Portland Cement Association, 250 psi for light traffic and 400 psi for heavy traffic). For other stabilizing agents, there are no guidelines available; hence, the immersed compressive strength of 100 psi was used (considering 18 kips of single axle wheel load and 2 to 2.5 factor of safety).

On the basis of 28-day immersed compressive strengths (Table 5.12) recommendations concerning suitable stabilizing agents for all shales are summarized in Table 5.13.

Table 5.11:

LIST OF STABILIZING SETS USED

Approach	Set of Stabilizing Agents
1. Main Stabilizing Agent	1. Cement 14% 2. Lime 6% 3. Flyash 25%
2. Conjunctive Use of Main Stabilizing Agents	4. Cement 10% + Lime 4% 5. Cement 10% + Flyash 4% 6. Lime 5% + Flyash 10% 7. Cement 8% + Lime 2% + Flyash 4%
3. Use of Small Amounts of Secondary Additives	8. Lime 6% + NaCl 2% 9. Cement 10% + Flyash 4% + Na_2CO_3 0.5% 10. Lime 5% + Flyash 10% + NaCO_3 0.5% 11. Lime 5% + Flyash 10% + CaCl_2 0.5% 12. Lime 5% + Flyash 10% + NaOH 0.5%

Table 5.12:
28 DAY IMMERSSED COMPRESSIVE STRENGTH, PSI

Shale Number	C ^a (14%)	L (6%)	FA (25%)	C+L (10%+4%)	C+FA (10%+4%)	L+FA (5%+10%)	C+L+FA (8%+2%+4%)	L+NaCl (6%+2%)	C+FA+Na ₂ CO ₃ (10%+4% +0.5%)	L+FA+Na ₂ CO ₃ (5%+10% +0.5%)	L+FA+CaCl ₂ (5%+10% +0.5%)	L+FA+NaOH (5%+10% +0.5%)
29	278	--	159	517	178	215	167	115	214	228	169	130
30	526	--	174	329	545	191	362	34	390	295	202	172
31	167	86	51	344	502	233	413	233	345	208	201	164
32	91	20	--	130	77	86	133	59	73	46	12	40
33	693	48	94	362	306	132	281	103	291	120	127	55
34	309	41	52	292	360	131	309	106	201	221	212	185
35	646	332	121	660	521	285	494	287	345	410	321	302
36	438	--	97	306	150	45	127	31	315	149	140	90

^aC = Cement, L = Lime, FA = Flyash

-- = Sample dispersed during immersion

Table 5.13:
SUITABLE STABILIZING AGENTS FOR SELECTED SHALES

Shale Number	Stabilizing Agent for Maximum Strength	Recommendation of Suitable Stabilizing Agents
29	Cement 10% + Lime 4%	<ol style="list-style-type: none"> 1. Cement 14% for secondary roads only 2. Flyash 25% 3. Cement 10% + Lime 4% 4. Cement 10% + Flyash 4% + Na_2CO_3 0.5% 5. Lime 5% + Nacl 2% 6. Lime 5% + Flyash 10%
30	Cement 14%	<ol style="list-style-type: none"> 1. Cement 14% 2. Flyash 25% 3. Cement 10% + Flyash 4% 4. Lime 5% + Flyash 10% + Na_2CO_3 0.5%
31	Cement 8% + Flyash 4% + Lime 2%	<ol style="list-style-type: none"> 1. Cement 10% + Lime 4% 2. Cement 10% + Flyash 4% 3. Cement 8% + Flyash 4% + Lime 2% 4. Lime 5% + Flyash 10% 5. Lime 6% + Nacl 2%
32	Cement 8% + Flyash 4% + Lime 2%	<ol style="list-style-type: none"> 1. Cement 10% + Lime 4% 2. Cement 10% + Flyash 4% 3. Cement 8% + Lime 2% + Flyash 4% 4. Lime 5% + Flyash 10%
33	Cement 14%	<ol style="list-style-type: none"> 1. Cement 14% 2. Flyash 25% 3. Cement 10% + Flyash 4% 4. Lime 6% + Nacl 2% 5. Lime 4% + Cement 10% 6. Lime 5% + Flyash 10%
34	Cement 10% + Flyash 4%	<ol style="list-style-type: none"> 1. Cement 14% for secondary roads only 2. Cement 10% + Flyash 4% 3. Lime 6% + Nacl 2% 4. Lime 5% + Flyash 10% + Na_2CO_3 0.5%
35	Cement 10% + Lime 4%	<ol style="list-style-type: none"> 1. Cement 14% 2. Lime 6% 3. Flyash 25% 4. Cement 10% + Lime 4% 5. Cement 10% + Flyash 4% 6. Lime 5% + Flyash 10% + Na_2CO_3 0.5%
36	Cement 14%	<ol style="list-style-type: none"> 1. Cement 14% 2. Flyash 25% 3. Lime 5% + Flyash 10% + CaCl_2 0.5% 4. Lime 5% + Flyash 10% + Na_2CO_3 0.5%

GEOLOGICAL TESTS ON RAW AND STABILIZED SHALES

X-RAY DIFFRACTION

The X-ray diffraction patterns do not show any change in the location of the identified clay minerals of stabilized shales considered in this study. However, lime stabilization clearly indicates the apparent modification in the mineralogical property of clay fraction, while for other stabilizing agents the clay mineral peaks are reduced significantly or become almost undefined. For cement stabilization, all diffraction peaks are least defined, as shown in Figure 5.13. One possible explanation is that an aggregation of silica and alumina present in cement takes places on the crystalline surface of clay minerals.

When comparing X-ray diffraction peaks for lime stabilization and flyash stabilization, the latter are well defined. The possible reason might be the presence of a significant amount of CaO (in the range of 29% to 30%) in the flyash used in this study. Though the presence of unreacted lime, calcite, tricalcium silicate (C_3S), dicalcium silicate (C_2S), calcium aluminate hydrates (C_4AH) etc., was detected through X-ray diffraction analysis for lime stabilized shales, the patterns failed to provide patterns which could lend themselves to measurable amounts of reaction products. This may be accounted for by two observations:

1. Stabilization reaction is a surface phenomenon and thus the major part of clay minerals is not modified
2. Crystallinity of reaction products is very low

In general, X-ray diffraction analysis techniques could not be used as a predictive tool for determining the quantity or rate of strength development which took place during curing and the mobilization of the stabilization process primarily because of the poor crystallinity of the reaction products.

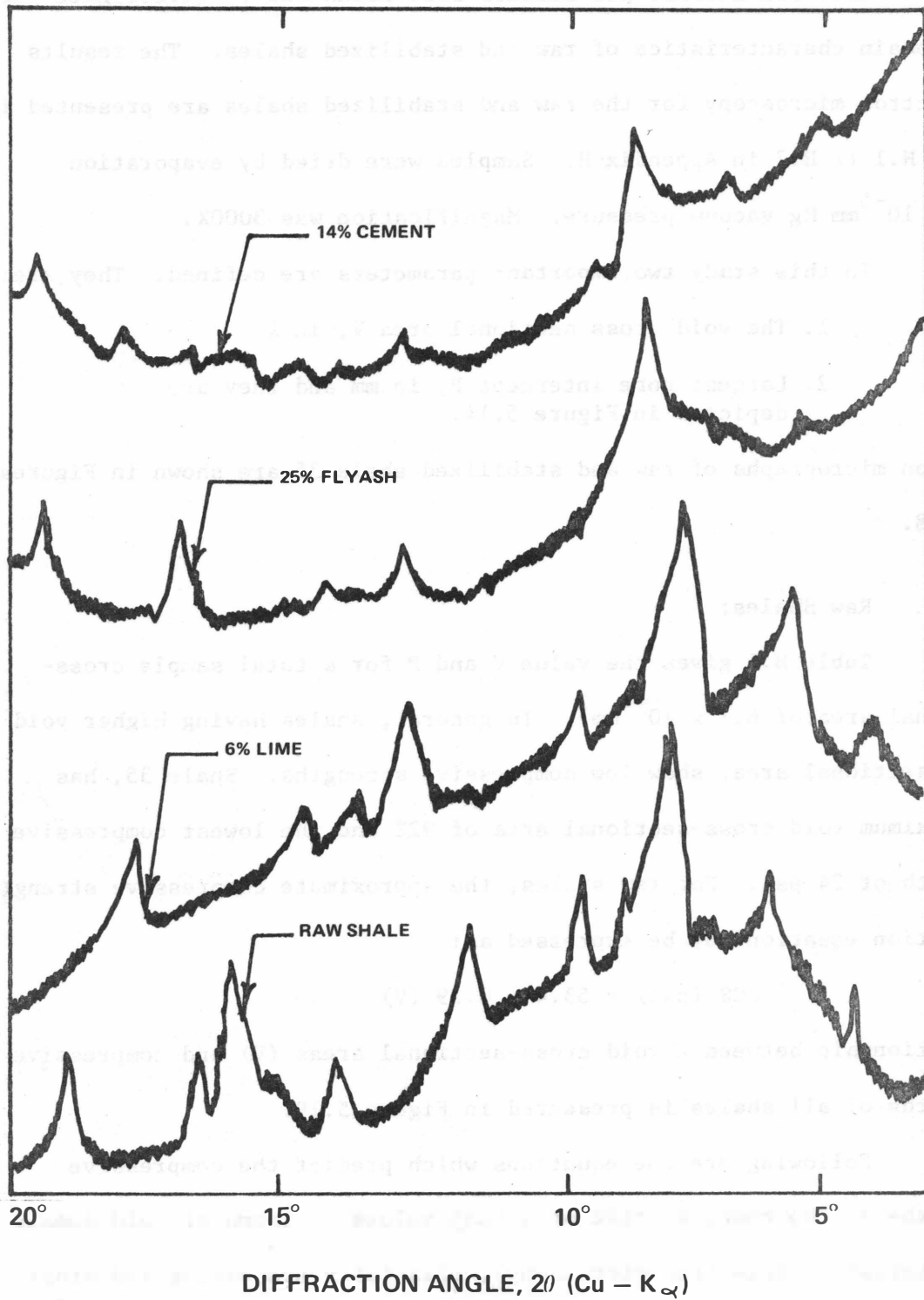


Figure 5.13: X-ray diffractogram for shale 35.

ELECTRON MICROSCOPY

Electron microscopic studies were conducted to investigate the void domain characteristics of raw and stabilized shales. The results of electron microscopy for the raw and stabilized shales are presented in Tables H.1 to H.7 in Appendix H. Samples were dried by evaporation at 7×10^{-5} mm Hg vacuum pressure. Magnification was 3000X.

In this study two important parameters are defined. They are:

1. The void cross sectional area V, in %
2. Largest pore intercept P, in mm and they are depicted in Figure 5.14.

Electron micrographs of raw and stabilized shale 35 are shown in Figures 5.15 to 5.18.

Raw Shales:

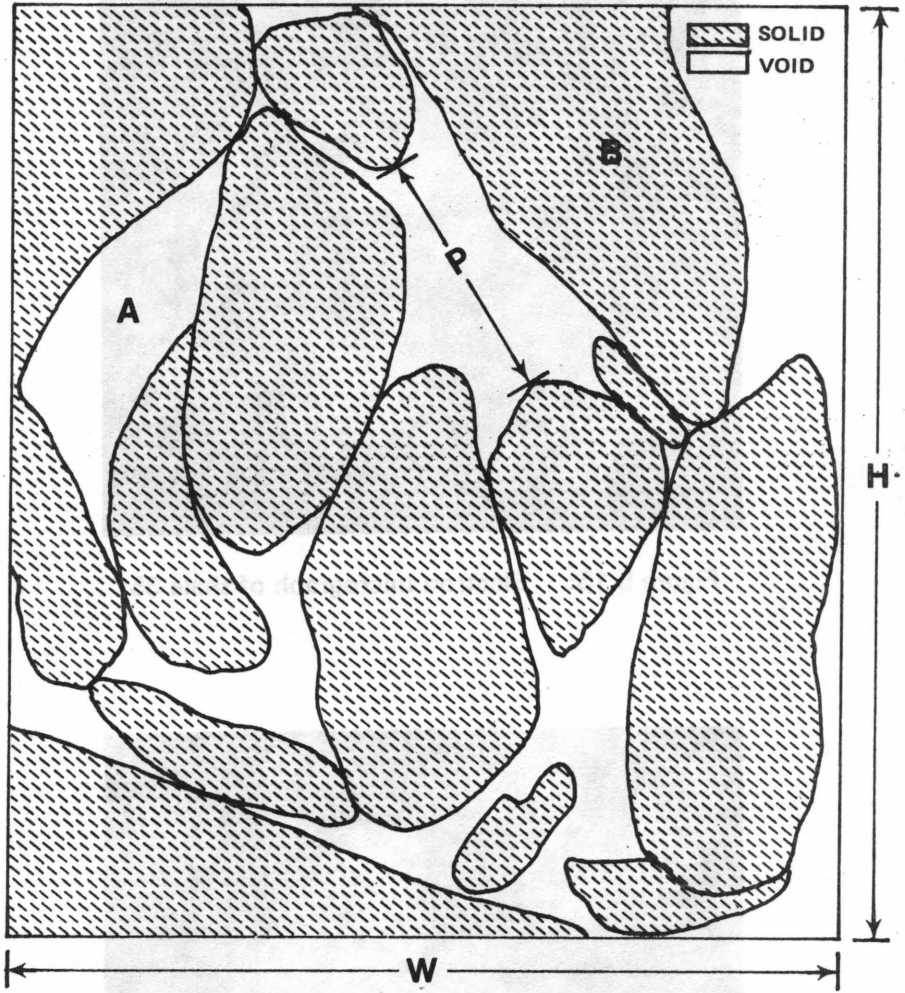
Table H.1 gives the value V and P for a total sample cross-sectional area of $6.7 \times 10^{-4} \text{ mm}^2$. In general, shales having higher void cross-sectional area, show low compressive strengths. Shale 35, has the maximum void cross-sectional area of 92% and the lowest compressive strength of 24 psi. For raw shales, the approximate compressive strength prediction equation may be expressed as:

$$\text{UCS (psi)} = 53.4 - 0.39 (V)$$

A relationship between % void cross-sectional areas (V) and compressive strengths of all shales is presented in Figure 5.19.

Following are the equations which predict the compressive strengths in the range of $\pm 10\%$ of actual values in terms of void domain characteristics (electron microscopy), plasticity properties and other strength parameters of raw shales.

1. $\text{UCS (psi)} = 3.60 - 0.15(V) + 0.03(\text{PI})$
2. $\text{UCS (psi)} = 31.5 - 0.15(V) + 0.22(\text{LL})$
3. $\text{UCS (psi)} = 31.3 - 185.9(P) - 0.12(V) + 0.27(\text{LL})$
4. $\text{UCS (psi)} = -13.1 - 499(P) + 0.01(V) + 0.87(\phi) + 3.0 (C) + 0.12(\text{PI})$



$W \times H = 6.7 \times 10^{-4} \text{ mm}^2$

$V = \left[\frac{\sum A}{W \times H} \right] \times 100$

P = Longest pore (void) intercept, mm.

Figure 5.14: Electron micrograph of soil mass.



Figure 5.15: Electron micrograph of shale 35.

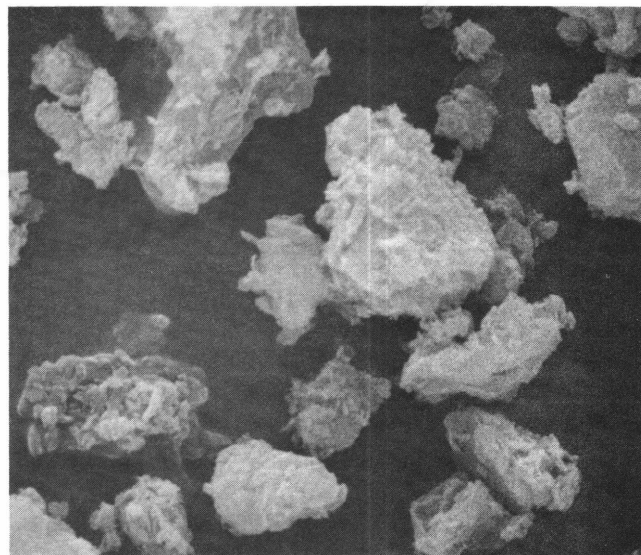


Figure 5.16: Electron micrograph of shale 35 stabilized with lime (6%) and cured for 28 days at 110° F.

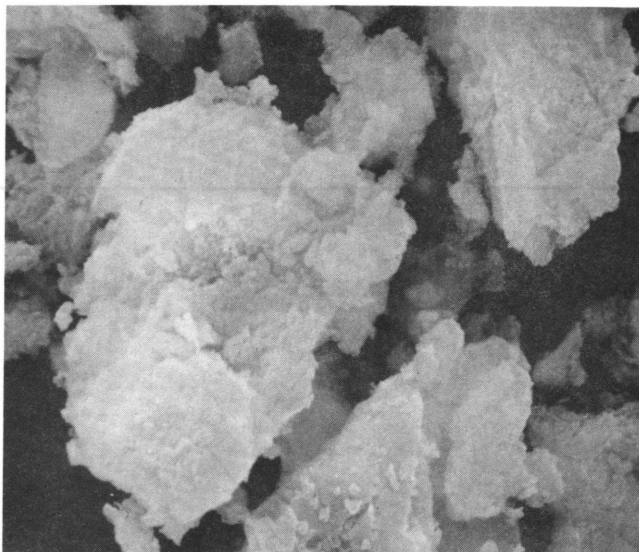


Figure 5.17: Electron micrograph of shale 35 stabilized with cement (14%) and cured for 28 days at 110° F.

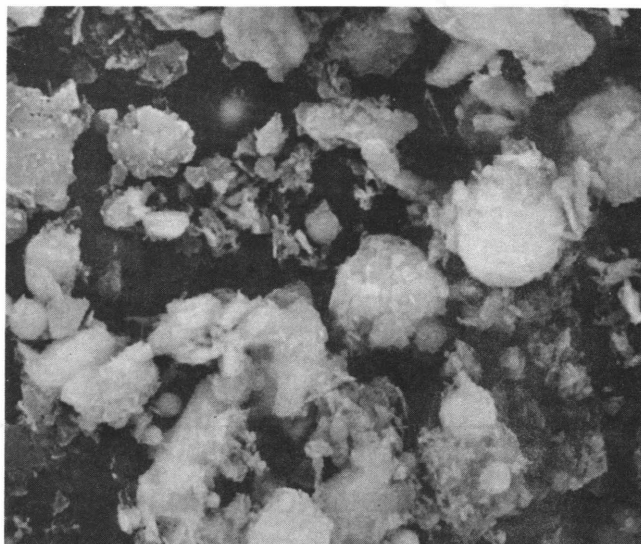


Figure 5.18: Electron micrograph of shale 35 stabilized with flyash (25%) and cured for 28 days at 110° F.

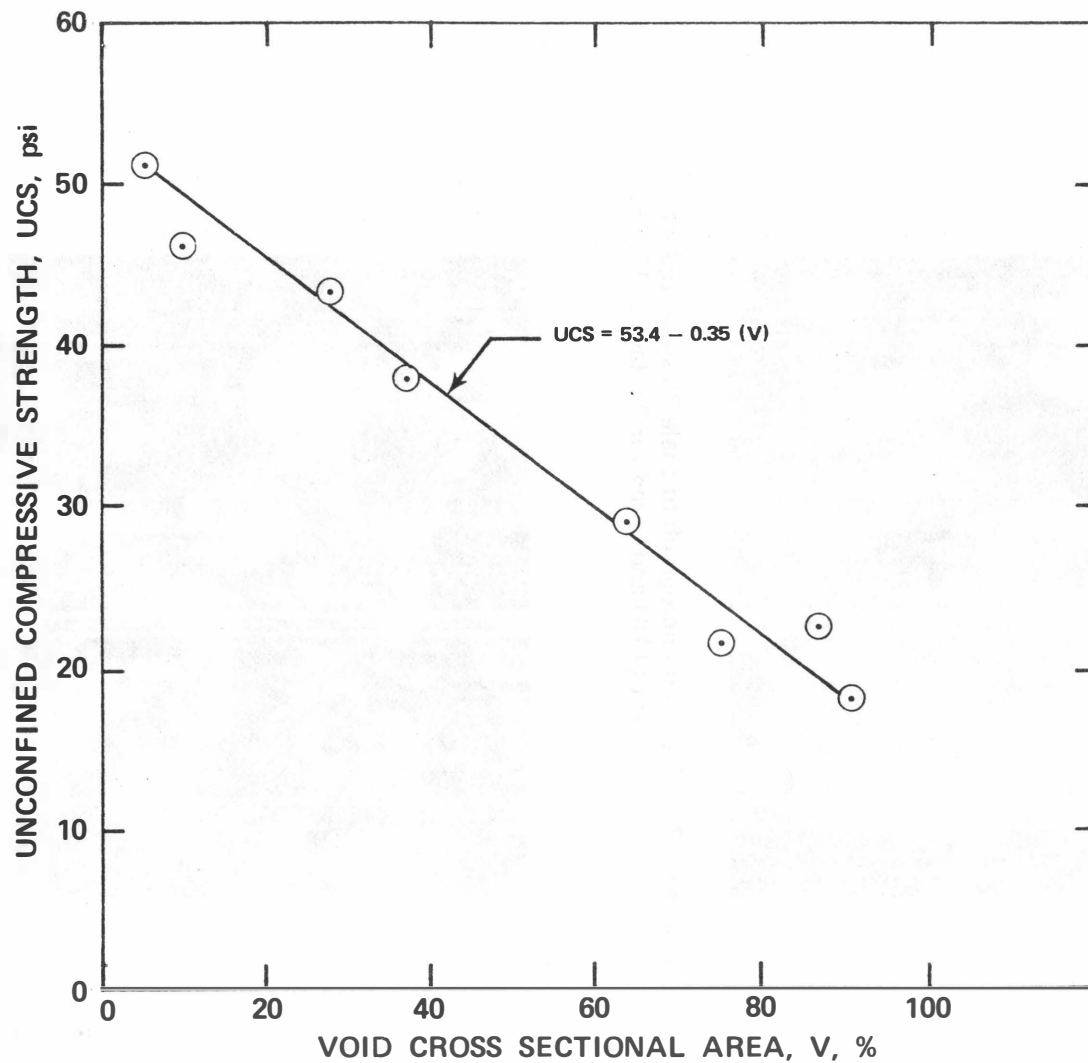


Figure 5.19: Electron microscopic study of raw shales.

Lime Stabilization: For lime stabilization, the percentage void cross-sectional area and the largest pore intercept of all 28-day cured samples are reduced significantly as a result of longer curing period (90 days), as shown in Tables H.2 and H.3 in Appendix H. However, the compressive strengths for 90-day cured shales are higher than the 28-day cured shales. This indicates that the longer curing period reduces the amount of voids and increases the compressive strengths for lime stabilization. For lime stabilization, the approximate compressive strength prediction equation (for 90-day cured samples) in terms of percentage void cross-sectional area (excluding shales 29 and 36) may be expressed as:

$$\text{UCS (psi)} = -45.75 + 12.53(V)$$

where V is greater than zero

Cement Stabilization: The percentage void cross-sectional area of cement stabilized shales are lower than lime stabilization for all shales, as shown in Tables H.4 to H.5 in Appendix H. There is no significant difference of void cross-sectional area and largest pore intercept between the 28-day and 90-day cured samples for cement stabilization. This indicates that aggregation which occurred after 28 days does not change significantly for longer curing periods (90 days).

The approximate relationship (as shown in Figure 5.20) between the largest pore intercept and the compressive strength for 28-day cured cement stabilized shales, may be expressed as:

$$\text{UCS (psi)} = 1950 - 50,000(P)$$

where P is greater than zero

Flyash Stabilization: The void cross-sectional area of flyash stabilized shales is higher than that of the cement stabilized shales but

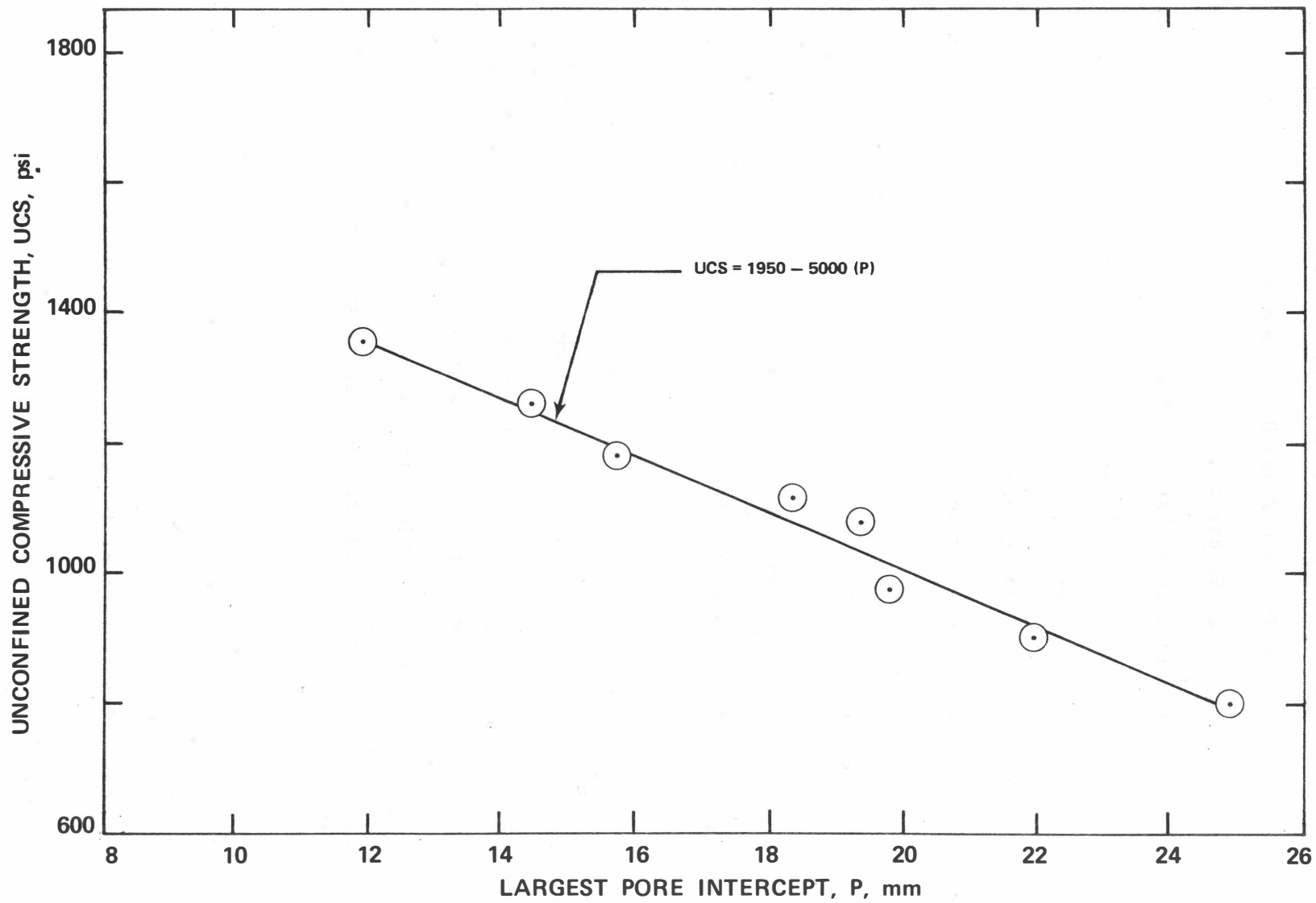


Figure 5.20: Electron microscopic study of cement (14%) stabilized shales.

similar to lime stabilized shales. However, for flyash stabilization there is no consistent pattern for compressive strength, void cross-sectional area, and largest pore intercept of shales. Results of electron microscopy of flyash stabilized shales are presented in Tables H.6 and H.7 in Appendix 7.

CHAPTER VI

MECHANISM OF STABILIZATION

MECHANISM OF LIME STABILIZATION

Addition of lime to soil increases the electrolyte concentration and, consequently, the pH of the pore water of the soil. Ion exchange between lime and soil particles produces Ca-clay and causes clay particles to flocculate (Herzog and Mitchell, 1963). Increased pH, also, makes alumina and silica more soluble and the calcium ions react with hydrous alumina to form hydrated calcium aluminate (Diamond and Kinter, 1966; Eades and Grim, 1966). This rather fast reaction is supplemented by a comparatively slower reaction of silica with lime to generate hydrated calcium silicate or "tobermorite gel" ($3\text{CaO} \cdot 2\text{SiO}_2 \cdot 3\text{H}_2\text{O}$). The gel hardens gradually and imparts strength to the soil lime mix. The reaction products of calcium, alumina and silica spall, liberating fresh clay surfaces for further adsorption and reaction (Diamond and Kinter, 1966). X-ray diffraction patterns indicate a destruction of the clay mineral structure by lime (Diamond and Kinter, 1966; Eades and Grim, 1966; Huang and Roderick, 1969; Anessi, 1970; Kumar, 1974), and the soil lime mix continues to exhibit strength gains for a long period of time. The hydration process ceases, like in cement concrete, only when moisture content in soil mass reaches a state of equilibrium with the relative humidity of surrounding air (Pihlajarara, 1967).

During the initial stage, the formation of very small quantities of cementing products at the points of contact between the edges of one particle and the faces of adjacent particles is believed to

be sufficient to stabilize the flocs so that the index properties are modified; however, the bonding among the flocs is not strong enough to provide sufficient strength to the clay mass and thus the clay seems to have been ameliorated (short term effect; first hour effect) but not stabilized. To stabilize the mix, compaction to minimum void ratio is essential. From the strength point of view the production of hydrated calcium aluminates and silicates during long term reactions is responsible for the stabilization of soils (Diamond and Kinter, 1966). Strength increases as the reaction proceeds. At any time, there may be more than one phase of reaction present depending on the curing time and temperature (Ruff and Ho, 1966). Hill and Davidson (1960) have shown that some of the time is utilized in the early amelioration of clay and thus it does not enter into pozzolanic reaction. Ho and Handy (1963, 1964) indicate that during the initial stages of reaction, lime does not show up on differential thermal analysis curve, suggesting its adsorption on clay surface. At higher temperatures and after longer curing periods this effect may be reduced due to the dissolution of clay and the accompanying release of adsorbed calcium.

The mechanism of lime stabilization of clay soils is presented in Figure 6.1 (after Ingles, 1970).

Practically, all fine-grained soils display cation exchange and flocculation agglomeration reactions when treated with lime. The reactions occur quite rapidly when soil and lime are intimately mixed (committee on lime and lime-flyash stabilization, 1976). The general order of replaceability of the common cations associated with soils is given by lyotropic series, $\text{Na}^+ < \text{K}^+ < \text{Ca}^{++} < \text{Mg}^{++}$. Cations tend to replace cations to the left in the series and monovalent cations are usually replaceable by multivalent cations (NCHRP, 1976). The addition of lime to a soil in a

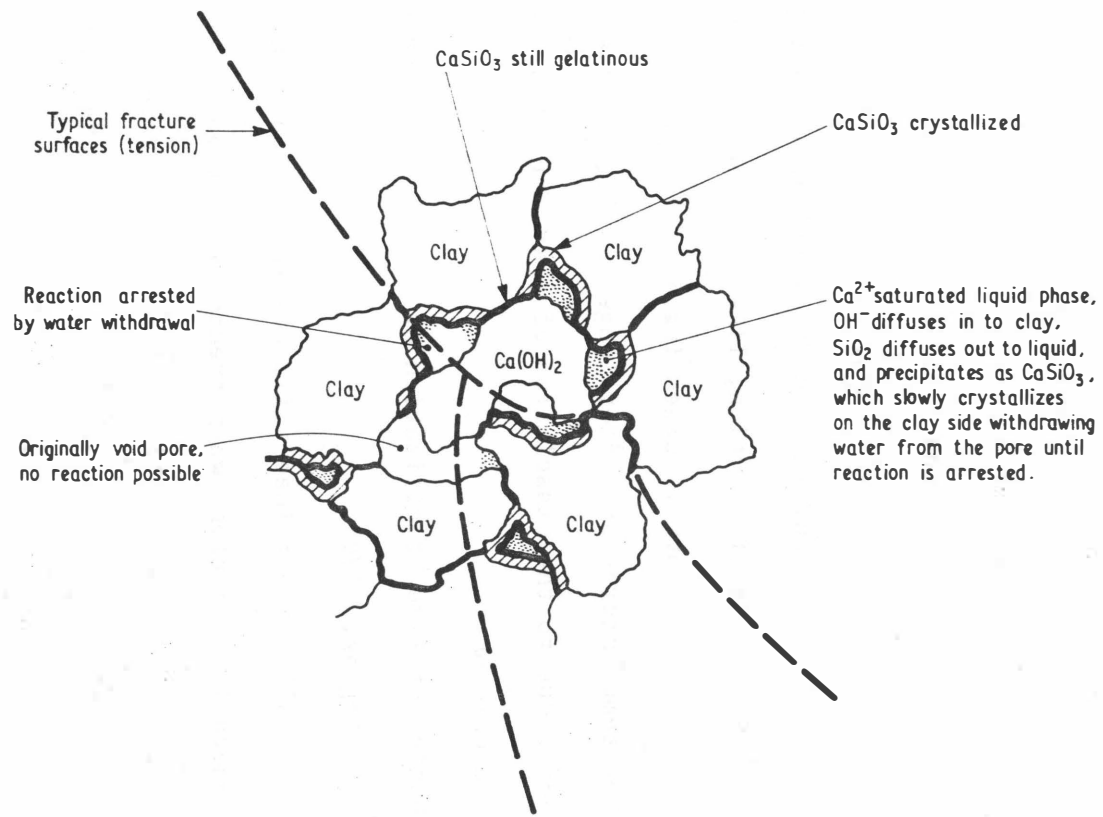


Figure 6.1: Mechanism of lime stabilization of clay soils (after Ingles, 1970).

sufficient quantity supplies an excess of Ca^{++} and cation exchange will occur, with Ca^{++} replacing dissimilar cations from the exchange complex of the soil. In some cases the exchange complex may be Ca^{++} saturated before the lime addition and cation exchange does not take place, or is minimized.

Flocculation and agglomeration produce an apparent change in texture with the clay particles "clumping" together into larger sized "aggregates." According to Herzog and Mitchell (1963), the flocculation and agglomeration is caused by the increased electrolyte content of the pore water and as a result of ion exchange by the clay to the calcium form. Calcium aluminate hydrate cementing materials are significant in the development of flocculation-agglomeration tendencies in soil lime mixtures (NCHRP, 1976).

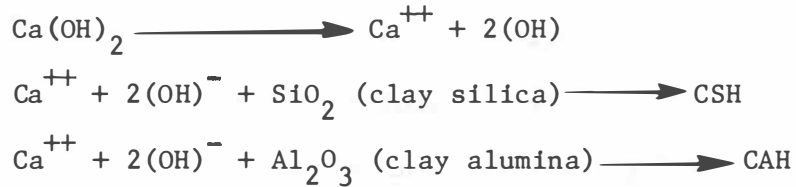
SOIL LIME POZZOLANIC REACTION

As mentioned earlier, the reactions between lime, water, and various sources of soil silica and alumina to form cementing type materials are referred to as a soil lime pozzolanic reactions. Possible sources of silica and alumina in typical soils include clay minerals, quartz, feldspars, micas and other similar silicate or alumino-silicate minerals either crystalline or amorphous in nature.

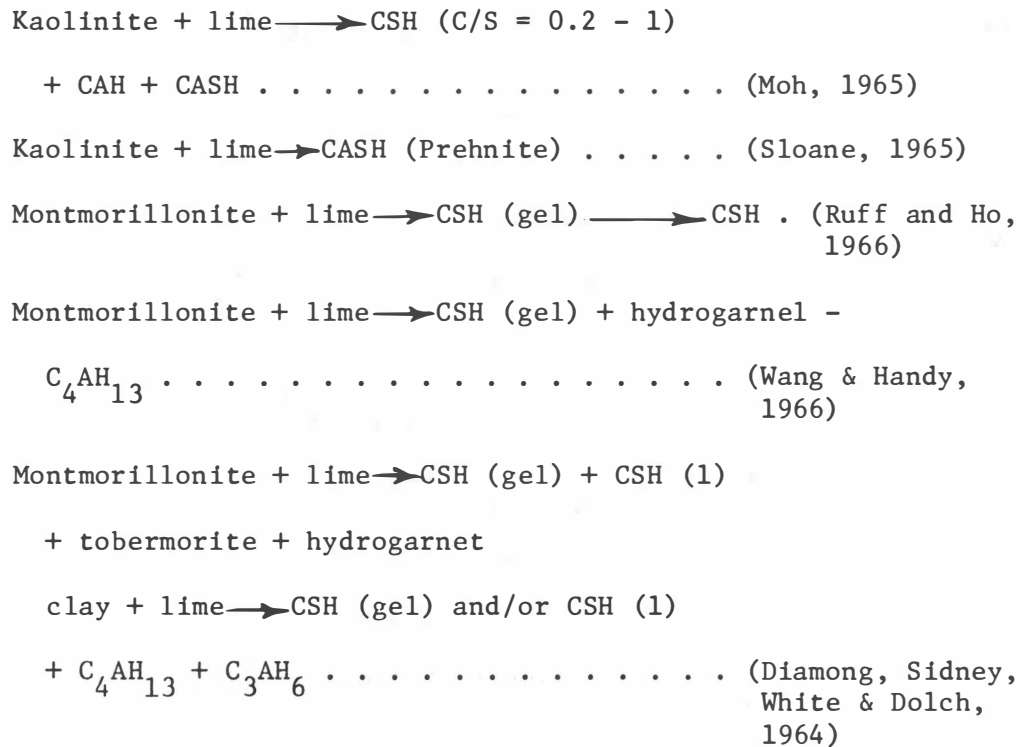
When a significant amount of lime is added to a soil, the pH of the soil lime mixture is elevated to approximately 12.4, the pH of saturated lime water. This is a substantial pH increase compared to the pH of natural soils. The solubilities of silica and alumina are greatly increased at elevated pH levels. In an early study of soil lime reactions (Eades, 1962) suggested that high pH causes silica to be solved out of the structure of the clay minerals, thereby causing available to combine

with Ca^{++} to form calcium silicates and that this reaction will continue as long as $\text{Ca}(\text{OH})_2$ exists in the soil and there is available silica.

An oversimplified qualitative view of some typical soil lime reactions is summarized below.



A wide variety of hydrated forms can be obtained, depending on reaction conditions, quantity and type of lime, soil characteristics, curing time, and temperature. Typical soil lime reactions are:



where:

C = CaO

S = SiO_2

A = Al_2O_3 , and

H = H_2O

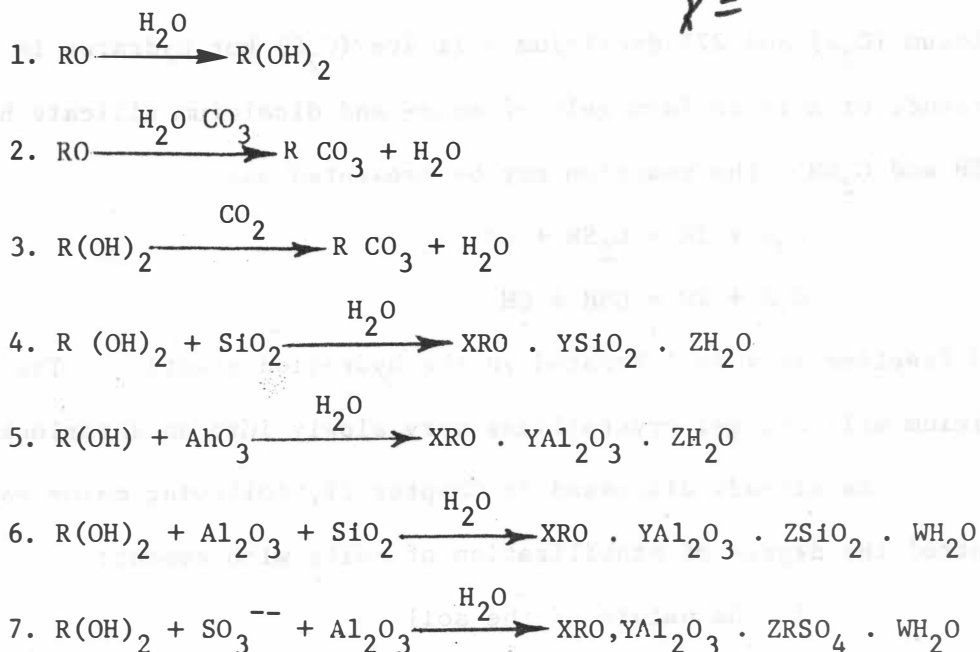
Some of the major soil properties and characteristics which influence the lime reactivity of a soil or ability of soil to react with lime to

produce cementitious materials, are soil pH, organic carbon content, natural drainage, presence of excessive quantities of exchangeable sodium, clay mineralogy, degree of weathering, presence of carbonates, extractable iron, silica-sesquioxide ratio and silica-alumina ratio (TRB Circular, No. 180, September 1976).

Flyash is generally used as an admixture in soil lime or soil cement stabilization. However, as discussed earlier, due to significant amount of CaO present in flyash, it has been used as a main stabilizing agent in this study.

LIME FLYASH REACTIONS

The reactions that occur in the lime flyash water system to form cementitious materials are complex. However, several studies provide basic information pertaining to the reactions. Minnick (1967) presents an illustrative list of reactions, as follows, and acknowledges that other reactions are also possible. If R_2Ca^{++} or Mg^{++} or combination of these ions:



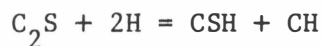
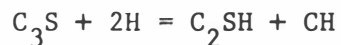
In addition to the primary reaction between lime and flyash, the lime may

also react with "fines" in the material being stabilized. Soil lime reaction that may occur are cation-exchange, flocculation/agglomeration, and a soil lime pozzolanic reaction.

MECHANISM OF CEMENT STABILIZATION

The mechanism by which a small proportion of cement can change the properties of a large mass of soil is still to be completely defined. It has been suggested that the cement forms either strong nuclei distributed throughout the voids in such a manner as to restrain the unaffected soil (Davidson, 1962). Herzog (1963) has shown that in a montmorillonite clay, the stress strain behavior supports a nucleated structure at low cement contents, which changes to a skeleton structure as a cement contents increases and the nuclei grow into each other as a result of secondary cementation process. This continuous skeleton was formed in his experiments at a cement content of only 2.5%.

Since ordinary portland cement consists of about 45% the calcium (C_3S) and 27% dicalcium silicate (C_2S) but hydrates in the presence of soil to form gels of mono- and dicalcium silicate hydrate (CSH and C_2SH), the reaction may be presented as:



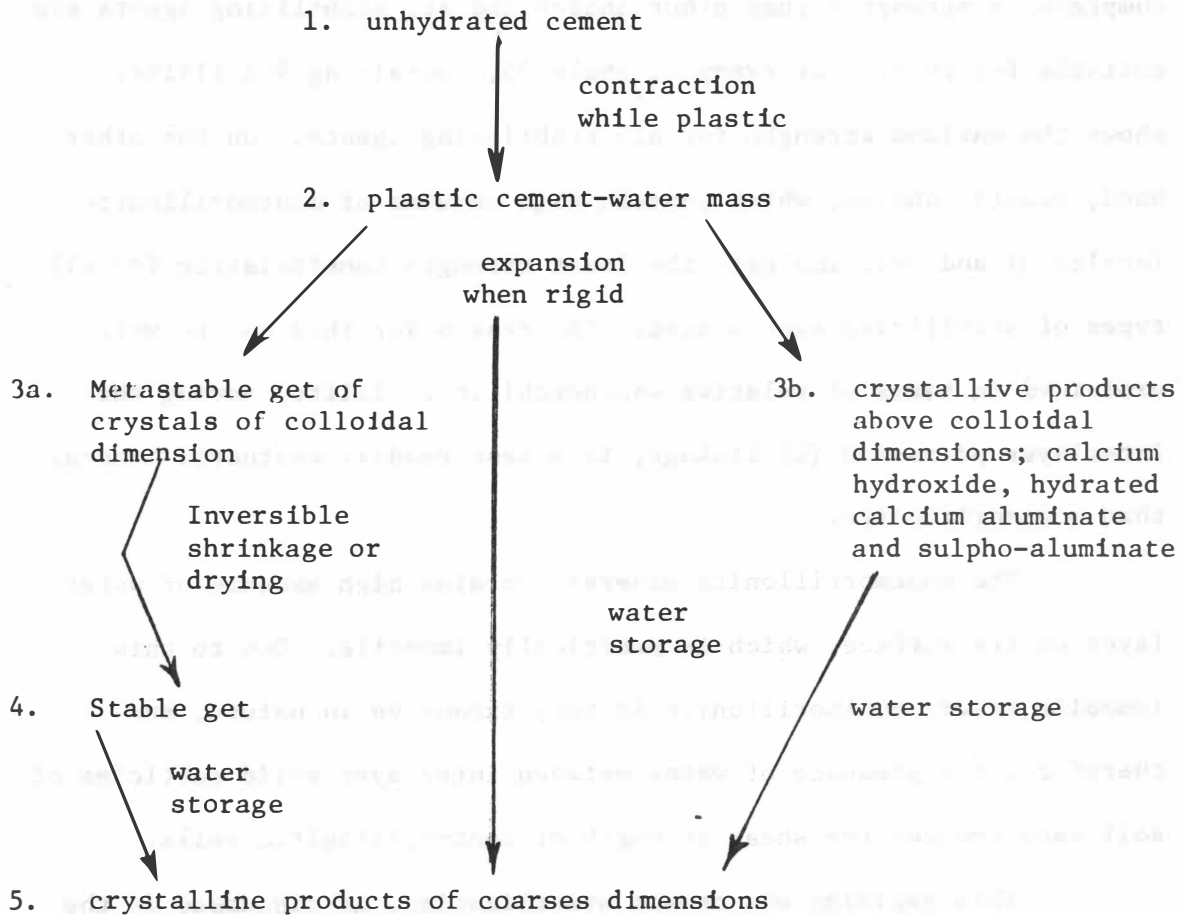
and frelime (CH) is liberated in the hydration reaction. The insoluble calcium silicate gel crystallizes very slowly into an interlocking matrix.

As already discussed in Chapter II, following major variables control the degree of stabilization of soils with cement:

1. The nature of the soil
2. The proportion of cement in the mix
3. The moisture content at the time of compaction
4. The degree of desiccation attained in compaction

Table 6.1:

PROCESS DIAGRAM FOR CEMENT SETTING, HARDENING
AND AGING (LAGUROS, 1962)



DISCUSSION

The various compressive strength test results indicate that shales containing high or significant amounts of illite show higher compressive strengths than other shales and all stabilizing agents are suitable for them. For example, shale 35, containing 94% illite, shows the maximum strength for all stabilizing agents. On the other hand, plastic shales, which contain high amounts of montmorillonite (shales 31 and 32), indicate the least strength beneficiation for all types of stabilizing agents used. The reason for this may be well explained in terms of relative weatherability. Illite, having the interlayer potassium (K) linkage, is a less readily weathered mineral than montmorillonite.

The montmorillonite mineral contains high amounts of water layer on its surface, which is practically immobile. Due to this immobile layer, montmorillonite is very expansive in nature, and therefore, the presence of water between interlayer solid particles of soil mass reduces the shear strength of montmorillonitic soils.

This explains why cement stabilization, as discussed in the previous chapter, is the least beneficial for plastic or montmorillonitic shales (shales 31 and 32). The possible reason for this is that cement, which acts very quickly or faster than (in short time) other stabilizing agents (lime and flyash) and causes cementation of clay particles, acts at a slower rate now because of the excess amount of water present in pore spaces of montmorillonitic soils. For example, shales 31 and 32, which are plastic in nature and predominantly montmorillonitic, show the least strength gain with cement stabilization. When comparing cement stabilization (14%), lime stabilization (6%) and flyash stabilization (25%), maximum dry densities of all flyash (25%) stabilized shales are the lowest

and those of cement (14%) stabilized shales are the highest. This indicates that maximum dry densities of stabilized shales are dependent on the unit weight of stabilizing agents. Considering all stabilizing agents used in this study, cement has the maximum unit weight of 185 pcf and flyash has the minimum weight of 80 pcf.

As the result of stabilization of shales, cohesion (C) and frictional angles (ϕ) of all shales are increased and the clay size portion is decreased due to aggregation or cementation of fine fractions. Generally, cohesion (C), frictional angle (ϕ) and aggregation of fine fraction for cement stabilization (14%) are higher than for lime stabilization (6%) and flyash stabilization (25%). This helps explain why cement stabilized shales resisted and sustained higher failure loads than the shales stabilized with lime and flyash. This phenomenon was observed when measuring compressive strength as well as evaluating load response of shales. However, plastic shales show comparatively lower failure loads for cyclic loading. The repetitive nature of cyclic loading weakens the bond between the soiled particles of the soil mass and the attendant effect is reduction in cohesion and eventually the lowering of the shear strength. This is why at higher rate of load application the failure loads are lower.

Lime, which helps reduce plasticity by increasing the plastic limit of soils, is more beneficial for plastic soils. This explains why shale 31, which contains the highest amount of montmorillonite (75%), shows higher compressive strength for the combination of 10% cement + 4% lime treatment than 14% cement treatment alone.

When comparing lime stabilization (6%) and flyash stabilization (25%), the latter appears to be more beneficial for the shales having comparatively lower amounts of clay (shales 29, 30, 33, and 36). The

possible reason is that the flyash used contains high amounts of fine fraction (about 70% 2μ size particles) which imparts moderate cohesion to the soil-flyash mixture and helps retain moisture. Addition to lime to the soil-flyash mixture seems beneficial for plastic soils. For example, plastic shales 31 and 32, for which (due to high amount of fine fraction) flyash (25%) stabilization is not suitable. The combination of flyash and lime (10% + 4%) shows significant strength gain.

Addition of moisture retention agents to soil containing low amounts of less than 2μ clay is very beneficial for lime stabilization. For shale 33, for which 6% lime stabilization was not suitable, an addition of 2% NaCl imparts very high strength. Delayed compaction is beneficial for only flyash stabilization. There is a significant amount of fines in the flyash. These fines slowly get saturated and fill the voids in the soil flyash mixture thus increasing the cohesion as well as the shear strength of mixture. The quick hardening feature of cement stabilization puts delayed compaction in conflict with the attainment of strength. As for lime it contains comparatively less amount of fines than flyash and, therefore, delayed compaction is not considered beneficial in lime stabilization.

In studying the durability of stabilized shales, it becomes apparent that the strength behavior differs significantly after wet-dry and freeze-thaw cycles. As a result of wet-dry cycles, only shales 34 and 36, containing significant amount of silt, show slight increase in strength for lime stabilization and cement stabilization. The reason is that silty shales are less expansive than plastic shales. Because of this, the amount of water on the surface of solid particles is less and it does not affect the cementation or cohesion forces at the point of contact of solid particles.

But as a result of various wet-dry cycles, air present in the voids escapes, resulting in the reduction of voids or void areas available. This ultimately reduces the total void cross-sectional area in the soil mass and increases the compressive strength. The expansive plastic shale 32 in its lime or flyash stabilized form disperses due to swelling resulting for moisture absorption. It is reasoned that dispersion occurs because the hardening process, in the case of lime stabilization and flyash stabilization, is not as fast as cement stabilization.

As a result of freeze-thaw cycles, compressive strengths of all cement stabilized shales are reduced. This may be explained in terms of the volumetric expansion of pore water that freezes during freeze-thaw cycles. During freezing, pore water expands and weakens the cohesive forces between two solid particles of soils, resulting in reduction of compressive strength of cement stabilized shales. In the case of lime and flyash stabilized shales, disintegration was observed because the hardening process in this case was not as quick as that of cement.

As indicated earlier, void domain characteristic studies appear to be a useful tool in explaining stabilization mechanism. The void cross-sectional areas are reduced after stabilization, which indicates the cementation of clay particles or increase in the size of the colony of clay particles that eventually reduces the voids or partially fills the voids present in soil mass. Reduction in voids with curing periods for lime stabilized shales supports the fact that lime clay reaction continues for a long period of time. On the other hand, similar void cross-sectional areas for both 28 and 90-day curing periods for cement stabilization indicate the quick hardening or cementation process for cement stabilization.

There seems to be a direct relationship between void domain

characteristics and compressive strength of shales. Shales having high void cross-sectional areas show low compressive strengths. For example, all cement stabilized (14%) shales having the minimum void cross-sectional areas show the maximum compressive strengths.

CHAPTER VII

CONCLUSIONS

The present laboratory investigation was conducted to determine the type and amount of stabilizing agents (cement, lime, flyash) to be added to upgrade Oklahoma shales used in highway construction. On the basis of data obtained from property and evaluation tests conducted in the laboratory on eight shales selected from different parts of Oklahoma, the following principal conclusions are drawn:

1. Predominantly illitic shales are comparatively better shales from an engineering performance standpoint and they are effectively stabilized with cementitious agents.
2. For shales containing significant amounts of montmorillonite, cement stabilization is the least effective.
3. Clayey shales exhibit the least strength beneficiation (gain) with any of the stabilizing agent used.
4. The commercial petrochemical stabilizers, 'clapak' and 'claset', were found not effective. Also, due to significant amounts of clay in the shales, bituminous stabilization was found unsuitable.
5. Used separately the most suitable and economically optimum stabilizing agents are cement (14%), lime (6%), and flyash (25%), by dry weight of shales.
6. For all shales, cement stabilization imparted maximum strength gain but lime and flyash addition resulted in moderate strength gain.
7. Grain size analysis of raw and stabilized shales showed

Cement
Flyash

that through stabilization aggregation of clay particles takes place. With cement, the aggregation is higher than it is with lime or flyash. To define the degree of aggregation, a unitless parameter 'Aggregation Index' (AI) is used and it is expressed as:

$$AI = \frac{\text{Percent of nonclay size material of stabilized shale}}{\text{Percent of nonclay size material of raw shale}}$$

8. As a result of clay particle cementation, the plasticity of shales is reduced significantly after stabilization. For cement stabilization, beyond the addition of more than 10% cement, it was not possible to determine the plastic properties at all. With lime stabilization and flyash stabilization, the plasticity indices of all shales become very low, ranging from 2 to 10.

9. Stabilization decreases MDD but increases OMC. Flyash stabilized shales have the lowest MDD and cement stabilized shales have the highest MDD. On the other lime stabilized shales have the highest OMC and cement stabilized shales have the lowest OMC. The relationships presented below show the statistically significant parameters influencing density.

a) Lime Stabilization:

$$MDD \text{ (pcf)} = 137.6 - 1.69 \text{ (OMC)}$$

b) Cement Stabilization:

$$MDD \text{ (pcf)} = 142.91 - 1.79 \text{ (OMC)}$$

$$MDD \text{ (pcf)} = 118.3 - 0.129 \text{ (} 2\mu \text{)} - 0.28 \text{ (PI)}$$

c) Flyash Stabilization:

$$MDD \text{ (pcf)} = 123.6 - 1.45 \text{ (OMC)}$$

$$MDD \text{ (pcf)} = 121.9 - 0.06 \text{ (} 2\mu \text{)} - 1.27 \text{ (OMC)}$$

10. Cement stabilization (14%) imparted maximum dry and immersed compressive strengths to all shales. Lime stabilization (6%) and flyash stabilization (25%) showed moderate strength gain. For raw and cement stabilized shales, the following relationships have been established:

a) Raw Shale:

$$\text{UCS (psi)} = -32.6 + 3.11C + 0.99\phi$$

$$\begin{aligned} \text{UCS (psi)} &= -158.3 + 0.11 (2\mu) - 0.36 (\text{PI}) \\ &+ 1.71 (\text{OMC}) + 0.97 (\text{MDD}) + 3.6 (C) \\ &+ 0.66 (\phi) \end{aligned}$$

b) For cement stabilized (14%) shales, the 28-day compressive strength may be expressed as:

$$\begin{aligned} \text{UCS (psi)} &= 2873 + 11.77 (2) - 40.13 (\text{PI}) \\ &- 9.0 (\text{OMC}) - 13.98 (\text{MDD}) \end{aligned}$$

and the 28-day immersed strength, in psi, may be expressed as:

$$(\text{UCS})_{\text{Imm}} = 17.6 - 3.47(\text{UCS})_{\text{dry}} + 0.002(\text{UCS})_{\text{dry}}^2$$

11. As a result of 5 and 15 wet-dry cycles, compressive strengths of all cement stabilized (14%), lime stabilized (6%), and flyash stabilized (25%) shales were reduced. Plastic shales stabilized with lime (6%), and flyash (25%) dispersed when subjected to 5 wet-dry cycles.

12. Although more than 5 freeze-thaw cycles are unlikely to occur in Oklahoma, a study was conducted to find the effect of 5 and 15 freeze-thaw cycles on compressive strengths of stabilized shales.

Compressive strengths of all cement stabilized (14%) shales are significantly reduced as a result of 5 and 15 freeze-thaw cycles.

Lime stabilized (6%) shales, containing either very low or very high 2μ clay, dispersed when subjected to 5 freeze-thaw cycles. However, for lime stabilized and flyash stabilized shales, containing significant amounts of non-expansive illite reduction in strength was comparatively less for longer curing period (90 days).

13. Triaxial shear tests on 90-day cured samples of cement stabilized (14%), lime stabilized (6%), and flyash stabilized (25%) shales yielded high values of cohesion and friction angle compared to raw shales. These values ranged from 21 to 111 psi and 45° to 73° , respectively, for both lime stabilization (6%) and cement stabilization (14%). In the case

of flyash stabilization (25%), these values were comparatively lower.

In general, shales having higher cohesion values showed higher compressive strengths.

14. To simulate the effect of traffic induced stresses on pavements, repetitive load testing on stabilized shales was conducted. Cement stabilization (14%) showed higher resistance to failure loads than lime stabilization (6%) and flyash (25%) stabilization. Failure loads varied from 120 to 160 lbs. for cement stabilization (14%), 40 to 80 lb. for lime stabilization (6%), and mostly 40 lb. for flyash stabilization (25%). In general, at higher rate of load application (APM) the failure load was lower.

15. On the basis of a numbered strength criteria for shale 29, A-6(4), the following main stabilizing agents and secondary additives are recommended:

Atoka

- 1) Cement 14% for secondary roads or light traffic only
- 2) Flyash 25%
- 3) Cement 10% + Lime 4%
- 4) Cement 10% + Flyash 4% + Na_2CO_3 0.5%
- 5) Lime 6% + NaCl 2%
- 6) Flyash 10% + Lime 5%

16. For shale 30, which is an A-6(9) soil, the following main stabilizing agents and secondary additives are recommended:

Lafayette

- 1) Cement 14% for secondary roads or light traffic only
- 2) Flyash 25%
- 3) Cement 10% + Flyash 4%
- 4) Flyash 10% + Lime 5% + Na_2CO_3 0.5%

17. For shale 31, predominantly montmorillonitic, A-7-5(13), the following stabilizing agents and secondary additives are recommended:

Claypool

- 1) Cement 14%
- 2) Cement 10% + Flyash 4%
- 3) Cement 8% + Lime 2% + Flyash 4%
- 4) Lime 6% + NaCl 2%
- 5) Flyash 10% + Lime 5%

18. For clayey shale 32, A-7-5(20), the following stabilizing agents and secondary additives are recommended:

Fredricksburg

- 1) Cement 10% + Lime 4%
- 2) Cement 10% + Flyash 4%
- 3) Cement 8% + Flyash 4% + Lime 2%
- 4) Flyash 10% + Lime 5%

19. For shale 33, A-6(3), the following stabilizing agents and secondary additives are recommended:

John's Valley

- 1) Cement 14%
- 2) Flyash 25%
- 3) Cement 10% + Lime 4%
- 4) Lime 6% + NaCl 2%
- 5) Flyash 4% + Cement 10%
- 6) Flyash 10% + Lime 5%

20. Shale 34, A-6(9), the following main stabilizing agents and secondary additives are recommended:

Boyer

- 1) Cement 14%
- 2) Cement 10% + Lime 4%
- 3) Lime 6% + NaCl 2%
- 4) Lime 5% + Flyash 10% + Na₂CO₃ 0.5%

21. For shale 35, A-4(2), predominantly illitic, the following main stabilizing agents and secondary additives are recommended:

Fluor pt

- 1) Cement 14%
- 2) Lime 6%
- 3) Flyash 25%
- 4) Cement 10% + Flyash 4%
- 5) Cement 10% + Lime 4%
- 6) Flyash 10% + Lime 5% + Na₂CO₃ 0.5%

22. For shale 36, A-7-6(11), the following stabilizing agents and secondary additives are recommended:

Senor @

- 1) Cement 14%
- 2) Flyash 25%
- 3) Flyash 10% + Lime 5% + CaCl₂ 0.5%
- 4) Flyash 10% + Lime 5% + Na₂CO₃ 0.5%

23. In order to assess the influence of delayed compaction on density properties a study of 1-hour, 2-hour, and 4-hour delayed compaction was undertaken. As a result of 1-hour, 2-hour, and 4-hour delayed compaction, the maximum dry density and optimum moisture contents

of lime stabilized (6%) and cement stabilized (14%) shales are significantly reduced. For flyash stabilization (25%), the maximum dry density and optimum moisture content do not show any consistent pattern.

24. The 1-, 2-, and 4-hour delayed compaction studies show that they are beneficial for flyash stabilization only. Dry and immersed compressive strengths of flyash stabilized (25%) shales are slightly increased with delayed compaction.

25. X-ray diffraction analysis shows well-defined peaks of identifiable clay minerals for lime stabilization and least defined peaks for cement stabilization. While it serves as an interpretative tool, X-ray diffraction cannot be used as a predictive method of determining the quantity or rate of strength development which takes place during the curing phase of stabilization. The poor crystallinity of the reaction products apparently impedes it.

26. The void cross-sectional area is inversely proportional to the compressive strength. There the stabilized shales which had higher compressive strengths than raw shales displayed smaller road cross-sectional areas. An approximate relationship between unconfined compressive strength and void cross-sectional area of raw shales may be expressed as:

$$\text{UCS (psi)} = 53.4 - 0.29 V (\%)$$

27. While curing periods longer than 28 days increase the compressive strength of cement stabilized (14%) shales, the measurement of the void cross-sectional area fails to predict or reflect this increase primarily because strength increase is due to the effect of the cement soil mix. The 28-day strength of cement stabilized shales may be better expressed in terms of the largest pore intercept, P, for values of P greater

than zero. The equation is:

$$UCS \text{ (psi)} = 1950 - 5 \times 10^4 P \text{ (mm)}$$

28. For flyash stabilization (25%) void cross-sectional areas are decreased but it was not possible to observe any pattern conducive to a mathematical relationship.

29. Lime stabilized (6%) shales showed lower void cross-sectional areas for curing periods longer than 28 days. This indicated the slow rate of strength gain associated with lime stabilization reaction. An approximate relationship between compressive strength and void cross-sectional areas, for 90-day cured samples may be expressed as:

$$UCS \text{ (psi)} = -45.7 + 12.5 V \text{ (\%)}$$

where V is in excess of 16%.

CHAPTER VIII
RECOMMENDATIONS

The extensive laboratory investigations pursued and the data collected in this study of stabilization of Oklahoma shales lead to recommendations which fall into two broad categories. The first is further studies and the second is field implementation; they are presented herein.

1. Response of shales to the cyclic loading tests and the relationships established between their void domain characteristics and strength are promising as predictive tools with regard to the engineering behavior of shales. However, more detailed studies in these two areas will be helpful in providing these tests with greater dependability.

2. The information on shale stabilization has reached a level that permits the initiation of field implementation; in fact, it is deemed necessary. Lime stabilization is already in use in Oklahoma; and, within the limits of the conclusions of this study, cement and flyash could be used, too. Especially, the utilization of flyash becomes mandatory because of the energy and environmental concerns expressed recently.

3. Field implementation is expected to yield observations that, in all likelihood, will deviate from laboratory behavior. The deviations will accrue, for example, when No. 4 material is used instead of No. 10 or when delayed compaction in the field assumes dimensions different than those in the laboratory. The assessment of these deviations is essential for purposes of formulating design procedures and specifications.

4. Laboratory studies are performed under well controlled conditions. Differences in stabilization effectiveness may also result, primarily in

terms of strength and durability, from less strict requirements employed in the field. Accordingly, a program of assessing the degree of field quality control and assurance should be initiated so that such statistical parameters as variability tolerances could be evaluated and service related to the performance of stabilized shale pavements.

5. Associated with field implementation there should be improvised a program leading to the development of a pavement design. Accordingly, the design should have the elements of a time continuous method wherein changing properties of the stabilized material below the highway pavement could be taken into account. Starting from the time of opening the highway to traffic, samples from the highway construction projects should be obtained at periodic intervals to study the effects of weathering and traffic stresses on the durability and other predictive characteristics of the stabilized shale materials. The data obtained from the present study could then be correlated with and/or modified according to the field data.

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APPENDIX A

GRAIN SIZE ANALYSIS OF STABILIZED SHALES

Table A.1:

AGGREGATION INDEX (AI) FOR SHALES STABILIZED
WITH LIME (3%) AND CURED FOR 28 DAYS

Shale Number	Curing Temperature	
	70 ^o F	110 ^o F
29	0.87	0.56
30	0.04	0.10
31	0.89	1.72
32	0.73	1.06
33	1.52	1.73
34	0.72	0.72
35	1.60	1.78
36	0.73	0.73

Table A.2:

AGGREGATION INDEX (AI) FOR SHALES STABILIZED
WITH LIME (0%) AND CURED FOR 28 DAYS

Shale Number	Curing Temperature	
	70 ^o F	110 ^o F
29	1.06	1.06
30	0.36	0.56
31	1.65	1.82
32	2.53	1.60
33	2.94	3.46
34	1.44	1.86
35	2.39	2.30
36	1.66	1.46

Table A.3:

AGGREGATE INDEX (AI) FOR SHALES STABILIZED
WITH LIME (9%) AND CURED FOR 28 DAYS

Shale Number	Curing Temperature	
	70°F	110°F
29	1.15	1.09
30	0.91	0.52
31	1.75	1.82
32	4.73	4.86
33	4.00	3.68
34	1.82	1.68
35	2.82	2.65
36	2.00	1.83

Table A.4:

AGGREGATE INDEX (AI) FOR SHALES STABILIZED
WITH 10% CEMENT AND CURED FOR 28 DAYS

Shale Number	Curing Temperature	
	70°F	110°F
29	2.96	2.90
30	1.97	1.91
31	5.31	5.87
32	6.25	5.50
33	4.15	4.52
34	3.27	3.10
35	4.13	3.10
36	3.25	3.25

Table A.5:

AGGREGATE INDEX (AI) FOR SHALES STABILIZED
WITH 14% CEMENT AND CURED FOR 28 DAYS

Shale Number	Curing Temperature	
	70°F	110°F
29	2.96	2.92
30	1.95	1.87
31	6.25	6.25
32	5.92	6.42
33	5.10	5.0
34	3.27	3.13
35	4.34	4.34
36	2.85	2.95

Table A.6:

AGGREGATE INDEX (AI)* FOR SHALES STABILIZED
WITH CEMENT (15%) AND CURED FOR 28 DAYS

Shale Number	Curing Temperature	
	70 ^o F	110 ^o F
29	2.80	3.22
30	1.70	2.06
31	5.81	6.25
32	6.57	6.50
33	5.21	4.57
34	3.24	3.48
35	4.34	4.21
36	2.85	2.95

Table A.7:

AGGREGATE INDEX (AI) FOR FLY-ASH (25%)
STABILIZED SHALES CURED FOR 28 DAYS

Shale Number	AI*	
	70°F	110°F
29	2.35	2.54
30	2.12	1.71
31	3.68	3.68
32	4.57	4.57
33	3.21	5.0
34	2.75	2.79
35	2.86	2.43
36	2.31	2.68

* All results are based on gradation characteristics of one hour ultrasonic treated samples.

$$AI = \frac{\% \text{ of nonclay material of stabilized shale}}{\% \text{ of nonclay material of raw shale}}$$

APPENDIX B

PLASTIC PROPERTIES OF STABILIZED SHALES

Table B.1:

PLASTIC PROPERTIES OF LIME STABILIZED
SHALES CURED FOR 28 DAYS AT 70°F

Shale Number	Lime Content, %	Liquid Limit	Plastic Limit	Plasticity Index
29	3	32	27	5
	6	36	30	6
	9	37	32	5
30	3	33	27	6
	6	36	31	5
	9	35	30	5
31	3	38	35	3
	6	36	NP	NP
	9	NP	NP	NP
32	3	62	39	23
	6	53	50	3
	9	49	45	4
33	3	34	25	9
	6	38	NP	NP
	9	39	36	2
34	3	33	29	4
	6	36	33	3
	9	38	NP	NP
35	3	35	31	4
	6	NP	NP	NP
	9	NP	NP	NP
36	3	36	30	6
	6	35	31	4
	9	39	35	5

Table B.2:

PLASTIC PROPERTIES OF LIME STABILIZED
SHALES CURED FOR 28 DAYS AT 118°F

Shale Number	Lime Content %	Liquid Limit	Plastic Limit	Plasticity Index
29	3	32	27	5
	6	38	33	5
	9	39	35	4
30	3	34	27	7
	6	35	26	9
	9	36	32	4
31	3	38	33	5
	6	35	NP	NP
	9	NP	NP	NP
32	3	57	41	16
	6	51	51	0
	9	53	51	2
33	3	37	30	7
	6	38	36	2
	9	40	36	4
34	3	35	28	7
	6	36	NP	NP
	9	40	NP	NP
35	3	33	NP	NP
	6	NP	NP	NP
	9	NP	NP	NP
36	3	34	30	4
	6	40	36	4
	9	40	NP	NP

Table B.3:
INDEX PROPERTIES OF CEMENT STABILIZED SHALES CURED FOR 28 DAYS

Shale Number	Cement Content, %	Curing Temperature					
		70°F			110°F		
		Liquid Limit	Plastic Limit	Plasticity Limit	Liquid Limit	Plastic Limit	Plasticity Limit
29	10	36	33	3	36	35	1
	14	34	-	-	33	-	-
	18	31	-	-	-	-	-
30	10	31	-	-	32	28	4
	14	32	-	-	33	32	1
	18	-	-	-	-	-	-
31	10	37	-	-	38	-	-
	14	36	-	-	37	-	-
	18	36	-	-	-	-	-
32	10	48	33	15	44	-	-
	14	-	-	-	41	-	-
	18	41	-	-	40	-	-
33	10	39	34	5	35	-	-
	14	-	-	-	34	-	-
	18	-	-	-	-	-	-
34	10	-	-	-	32	31	1
	14	-	-	-	-	-	-
	18	34	31	3	-	-	-
35	10	-	-	-	-	-	-
	14	-	-	-	-	-	-
	18	-	-	-	-	-	-
36	10	-	-	-	-	-	-
	14	32	32	1	-	-	-
	18	-	-	-	-	-	-

- Not possible.

Table B.4:

INDEX PROPERTIES OF FLYASH (25%) STABILIZED
SHALES CURED FOR 28 DAYS

Shale Number	Curing Temperature					
	70 ^o F			110 ^o F		
	LL	PL	PI	LL	PL	PI
29	37	--	--	33	25	8
30	31	27	4	32	26	6
31	39	36	3	36	32	4
32	52	41	11	50	40	10
33	33	27	6	--	--	--
34	36	31	5	33	29	4
35	31	--	--	35	33	2
36	35	39	6	36	32	4

TABLE 1

OPTIMUM MOISTURE CONTENT AND DRY DENSITY OF STABILIZED SOILS

Optimum Moisture Content (%)	Dry Density (pcf)	Compaction Method	Soil No.
10.0	115.0	0	10
10.5	116.0	1	
11.0	117.0	0	
11.5	118.0	4	
12.0	119.0	0	20
12.5	120.0	1	
13.0	121.0	0	
13.5	122.0	0	
14.0	123.0	0	30
14.5	124.0	2	
15.0	125.0	0	
15.5	126.0	0	
16.0	127.0	0	40
16.5	128.0	0	
17.0	129.0	0	
17.5	130.0	0	50
18.0	131.0	0	
18.5	132.0	0	
19.0	133.0	0	60
19.5	134.0	0	
20.0	135.0	0	
20.5	136.0	0	70
21.0	137.0	0	
21.5	138.0	0	
22.0	139.0	0	80
22.5	140.0	0	
23.0	141.0	0	
23.5	142.0	0	90
24.0	143.0	0	
24.5	144.0	0	
25.0	145.0	0	100
25.5	146.0	0	
26.0	147.0	0	
26.5	148.0	0	
27.0	149.0	0	
27.5	150.0	0	
28.0	151.0	0	
28.5	152.0	0	
29.0	153.0	0	
29.5	154.0	0	
30.0	155.0	0	
30.5	156.0	0	
31.0	157.0	0	
31.5	158.0	0	
32.0	159.0	0	
32.5	160.0	0	
33.0	161.0	0	
33.5	162.0	0	
34.0	163.0	0	
34.5	164.0	0	
35.0	165.0	0	
35.5	166.0	0	
36.0	167.0	0	
36.5	168.0	0	
37.0	169.0	0	
37.5	170.0	0	
38.0	171.0	0	
38.5	172.0	0	
39.0	173.0	0	
39.5	174.0	0	
40.0	175.0	0	
40.5	176.0	0	
41.0	177.0	0	
41.5	178.0	0	
42.0	179.0	0	
42.5	180.0	0	
43.0	181.0	0	
43.5	182.0	0	
44.0	183.0	0	
44.5	184.0	0	
45.0	185.0	0	
45.5	186.0	0	
46.0	187.0	0	
46.5	188.0	0	
47.0	189.0	0	
47.5	190.0	0	
48.0	191.0	0	
48.5	192.0	0	
49.0	193.0	0	
49.5	194.0	0	
50.0	195.0	0	
50.5	196.0	0	
51.0	197.0	0	
51.5	198.0	0	
52.0	199.0	0	
52.5	200.0	0	
53.0	201.0	0	
53.5	202.0	0	
54.0	203.0	0	
54.5	204.0	0	
55.0	205.0	0	
55.5	206.0	0	
56.0	207.0	0	
56.5	208.0	0	
57.0	209.0	0	
57.5	210.0	0	
58.0	211.0	0	
58.5	212.0	0	
59.0	213.0	0	
59.5	214.0	0	
60.0	215.0	0	
60.5	216.0	0	
61.0	217.0	0	
61.5	218.0	0	
62.0	219.0	0	
62.5	220.0	0	
63.0	221.0	0	
63.5	222.0	0	
64.0	223.0	0	
64.5	224.0	0	
65.0	225.0	0	
65.5	226.0	0	
66.0	227.0	0	
66.5	228.0	0	
67.0	229.0	0	
67.5	230.0	0	
68.0	231.0	0	
68.5	232.0	0	
69.0	233.0	0	
69.5	234.0	0	
70.0	235.0	0	
70.5	236.0	0	
71.0	237.0	0	
71.5	238.0	0	
72.0	239.0	0	
72.5	240.0	0	
73.0	241.0	0	
73.5	242.0	0	
74.0	243.0	0	
74.5	244.0	0	
75.0	245.0	0	
75.5	246.0	0	
76.0	247.0	0	
76.5	248.0	0	
77.0	249.0	0	
77.5	250.0	0	
78.0	251.0	0	
78.5	252.0	0	
79.0	253.0	0	
79.5	254.0	0	
80.0	255.0	0	
80.5	256.0	0	
81.0	257.0	0	
81.5	258.0	0	
82.0	259.0	0	
82.5	260.0	0	
83.0	261.0	0	
83.5	262.0	0	
84.0	263.0	0	
84.5	264.0	0	
85.0	265.0	0	
85.5	266.0	0	
86.0	267.0	0	
86.5	268.0	0	
87.0	269.0	0	
87.5	270.0	0	
88.0	271.0	0	
88.5	272.0	0	
89.0	273.0	0	
89.5	274.0	0	
90.0	275.0	0	
90.5	276.0	0	
91.0	277.0	0	
91.5	278.0	0	
92.0	279.0	0	
92.5	280.0	0	
93.0	281.0	0	
93.5	282.0	0	
94.0	283.0	0	
94.5	284.0	0	
95.0	285.0	0	
95.5	286.0	0	
96.0	287.0	0	
96.5	288.0	0	
97.0	289.0	0	
97.5	290.0	0	
98.0	291.0	0	
98.5	292.0	0	
99.0	293.0	0	
99.5	294.0	0	
100.0	295.0	0	

APPENDIX C

OPTIMUM MOISTURE CONTENT AND DRY DENSITY FOR RAW AND STABILIZED SHALES

Table C.1:

EFFECT OF LIME ADDITION ON MOISTURE-DENSITY RELATIONSHIPS

Shale Number	Lime Content, %	Dry Density, pcf	Optimum Moisture Content, %
29	0	112.3	16.5
	3	103.5	20.0
	6	101.9	20.6
	9	100.6	18.4
30	0	119.1	14.3
	3	112.4	16.1
	6	111.3	16.9
	9	109.2	16.2
31	0	106.5	20.0
	3	100.8	23.3
	6	99.0	23.2
	9	98.5	22.8
32	0	95.9	26.4
	3	86.2	30.9
	6	86.0	29.5
	9	81.4	31.6
33	0	113.3	16.0
	3	104.2	20.5
	6	102.9	21.0
	9	101.8	21.1
34	0	113.0	16.9
	3	104.0	20.5
	6	103.4	21.0
	9	100.7	21.1
35	0	107.2	20.2
	3	97.4	25.1
	6	99.6	23.5
	9	97.0	24.6
36	0	112.0	16.8
	3	104.2	20.0
	6	102.3	20.3
	9	101.3	20.2

Table C.2:

EFFECT OF CEMENT ADDITION ON MOISTURE
DENSITY RELATIONSHIP

Shale Number	Cement Content, %	Dry Density, pcf	Optimum Moisture Content, %
29	0	112.3	16.5
	10	107.0	17.8
	14	107.9	17.5
	18	109.1	18.5
30	0	119.1	14.3
	10	115.6	17.2
	14	116.0	16.2
	18	116.2	15.7
31	0	106.5	20.0
	10	99.2	21.0
	14	100.0	22.2
	18	99.6	20.2
32	0	95.9	26.4
	10	91.0	29.8
	14	90.3	27.0
	18	93.5	27.6
33	0	113.3	16.0
	10	106.8	18.2
	14	106.4	19.8
	18	107.8	18.4
34	0	113.0	16.9
	10	106.5	18.5
	14	107.1	17.8
	18	107.8	17.8
35	0	107.2	20.2
	10	104.50	20.0
	14	105.1	19.4
	18	104.7	19.8
36	0	112.0	16.8
	10	108.5	19.0
	14	107.8	19.0
	18	109.0	18.3

Table C.3:

MOISTURE CONTENT AND DRY DENSITY RELATIONSHIP
OF FLY-ASH STABILIZED SHALES

Shale Number	Optimum Moisture Content, %	Maximum Dry Density, pcf
29	17.2	110.0
30	18.0	110.0
31	26.1	84.85
32	26.20	87.64
33	16.7	92.6
34	19.4	90.71
35	20.6	86.40
36	16.0	94.1

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TABLE 1

MECHANICAL PROPERTIES OF SHALES TREATED WITH VARIOUS STABILIZING AGENTS

Sample No.	Moisture Content (%)	Unconfined Compressive Strength (psi)	Soil Description
1	10.5	100	Clay shale
2	11.2	120	Clay shale
3	12.1	150	Clay shale
4	13.0	180	Clay shale
5	14.0	220	Clay shale
6	15.0	280	Clay shale
7	16.0	350	Clay shale
8	17.0	450	Clay shale
9	18.0	550	Clay shale
10	19.0	700	Clay shale
11	20.0	850	Clay shale
12	21.0	1000	Clay shale
13	22.0	1200	Clay shale
14	23.0	1500	Clay shale
15	24.0	1800	Clay shale
16	25.0	2200	Clay shale
17	26.0	2800	Clay shale
18	27.0	3500	Clay shale
19	28.0	4500	Clay shale
20	29.0	5500	Clay shale

APPENDIX D

DRY AND IMMERSED STRENGTHS OF SHALES TREATED WITH VARIOUS STABILIZING AGENTS

Table D.1:

UNCONFINED COMPRESSIVE STRENGTH, PSI, OF LIME STABILIZED
SHALES CURED FOR 28 DAYS

Shale Number	Lime Content, %	Curing Temperature	
		70°F	110°F
29	3	36.8	53.8
	6	51.7	73.2
	9	79.5	97.9
30	3	47.9	96.4
	6	62.6	120.3
	9	59.6	72.6
31	3	158.7	207.3
	6	63.0	337.8
	9	93.0	515.5
32	3	155.5	143.7
	6	113.8	248.4
	9	154.6	403.5
33	3	16.1	72.7
	6	29.0	87.8
	9	20.3	88.9
34	3	17.0	177.7
	6	46.0	161.2
	9	44.8	148.6
35	3	234.0	325.1
	6	311.4	659.0
	9	223.0	477.7
36	3	61.7	120.9
	6	43.9	145.1
	9	46.5	237.5

Table D.2:

UNCONFINED COMPRESSIVE STRENGTH, PSI, OF LIME STABILIZED
SHALES CURED FOR 90 DAYS

Shale Number	Lime Content, %	Curing Temperature	
		70°F	110°F
29	3	51.4	85.1
	6	58.4	131.4
	9	68.4	204.8
30	3	66.8	123.5
	6	96.3	238.5
	9	58.4	159.6
31	3	208.5	197.9
	6	142.2	360.7
	9	181.3	633.5
32	3	137.0	119.9
	6	228.8	258.8
	9	250.3	404.4
33	3	42.5	193.3
	6	47.8	309.6
	9	30.0	301.6
34	3	54.0	341.6
	6	71.7	370.2
	9	68.0	422.0
35	3	285.7	316.6
	6	455.9	698.0
	9	334.2	666.2
36	3	66.0	293.4
	6	63.8	369.6
	9	73.6	366.7

Table D.3:
 IMMERSED STRENGTH, PSI, OF LIME STABILIZED
 SHALES CURED FOR 28 DAYS

Shale Number	Lime Content, %	Curing Temperature	
		70°F	110°F
29	3	-	8.1
	6	-	47.5
	9	35.9	92.0
30	3	-	-
	6	-	-
	9	-	21.4
31	3	120.7	215.9
	6	85.6	210.1
	9	27.7	152.0
32	3	29.7	66.6
	6	19.8	57.6
	9	-	97.3
33	3	12.1	17.4
	6	48.0	13.8
	9	87.2	19.0
34	3	47.0	178.9
	6	41.2	144.9
	9	21.1	152.0
35	3	256.1	234.0
	6	332.3	427.4
	9	215.7	569.8
36	3	20.8	84.2
	6	-	55.4
	9	-	48.5

- Not possible.

Table D.4:

IMMERSED STRENGTH, PSI, OF LIME STABILIZED
SHALES CURED FOR 90 DAYS

Shale Number	Lime Content, %	Curing Temperature	
		70°F	110°F
29	3	13.9	6.9
	6	14.0	164.7
	9	13.6	97.2
30	3	-	116.7
	6	34.4	41.2
	9	29.1	58.7
31	3	136.8	79.1
	6	131.5	164.5
	9	278.0	409.5
32	3	3.4	27.5
	6	23.1	144.5
	9	11.3	137.7
33	3	6.94	13.8
	6	85.9	66.5
	9	6.93	41.8
34	3	121.7	245.2
	6	59.5	349.7
	9	110.3	178.6
35	3	270.7	123.01
	6	157.4	260.0
	9	222.8	357.5
36	3	14.0	77.9
	6	14.0	149.6
	9	34.7	115.6

- Sample broke on immersion in water.

Table D.5:

UNCONFINED COMPRESSIVE STRENGTH, PSI, OF CEMENT
STABILIZED SHALES CURED FOR 28 DAYS

Shale Number	Cement Content %	Curing Temperature	
		70°F	110°F
29	10	332.8	500.8
	14	515.5	902.5
	18	693.7	1156.6
30	10	758.5	919.0
	14	895.4	1094.7
	18	1122.0	1194.0
31	10	150.3	202.8
	14	88.7	499.0
	18	215.9	439.9
32	10	256.2	544.1
	14	386.6	556.7
	18	474.6	839.7
33	10	451.0	634.6
	14	568.3	887.5
	18	634.5	952.3
34	10	484.3	884.2
	14	768.5	805.7
	18	916.0	981.5
35	10	759.5	1077.6
	14	772.7	1451.5
	18	1058.8	1514.7
36	10	508.5	764.9
	14	507.2	777.0
	18	759.3	1061.1

Table D.6:

UNCONFINED COMPRESSIVE STRENGTH, PSI, OF CEMENT
STABILIZED SHALES CURED FOR 90 DAYS

Shale Number	Cement Content, %	Curing Temperature	
		70°F	110°F
29	10	420.8	760.0
	14	615.0	1438.1
	18	852.8	1532.3
30	10	935.1	275.8
	14	1052.9	1204.1
	18	1299.2	1566.0
31	10	201.1	228.9
	14	272.3	477.5
	18	318.3	423.8
32	10	288.9	436.4
	14	619.2	611.9
	18	643.9	765.3
33	10	444.2	610.6
	14	531.0	938.8
	18	792.6	1278.3
34	10	579.2	999.7
	14	809.8	941.3
	18	1095.8	1974.1
35	10	991.5	1318.7
	14	824.8	1238.7
	18	991.4	1443.1
36	10	597.1	1085.5
	14	612.4	1286.7
	18	974.4	1161.5

Table D.7:

IMMERSED STRENGTH, PSI, OF CEMENT STABILIZED
SHALES CURED FOR 28 DAYS

Shale Number	Cement Content, %	Curing Temperature	
		70°F	110°F
29	10	210.5	334.6
	14	278.1	404.3
	18	262.3	856.3
30	10	495.8	498.3
	14	526.7	661.4
	18	562.6	735.5
31	10	206.1	597.4
	14	167.7	778.0
	18	35.4	372.0
32	10	143.9	398.7
	14	91.8	250.3
	18	144.9	449.9
33	10	310.3	655.5
	14	693.4	424.5
	18	628.1	670.4
34	10	406.4	745.7
	14	309.7	433.2
	18	453.2	714.1
35	10	565.1	802.0
	14	646.2	1438.0
	18	503.0	1477.5
36	10	285.0	428.8
	14	438.1	530.7
	18	480.3	720.3

Table D.8:

IMMERSED STRENGTH, PSI, OF CEMENT STABILIZED
SHALES CURED FOR 90 DAYS

Shale Number	Cement Content	Curing Temperature	
		70°F	110°F
29	10	329.4	674.7
	14	322.8	689.8
	18	440.0	819.3
30	10	716.75	548.8
	14	522.4	467.6
	18	645.0	701.8
31	10	391.3	306.0
	14	548.0	354.7
	18	1090.1	845.8
32	10	144.7	401.1
	14	118.8	276.8
	18	331.9	431.7
33	10	499.0	701.3
	14	552.1	873.9
	18	565.0	804.0
34	10	336.0	954.1
	14	359.5	897.3
	18	455.1	925.1
35	10	727.0	717.5
	14	493.0	609.0
	18	514.0	1301.0
36	10	305.6	582.7
	14	388.7	569.2
	18	311.8	687.0

Table D.9:
 UNCONFINED COMPRESSIVE STRENGTH, PSI, OF CLAPAK
 STABILIZED SHALES CURED FOR 28 DAYS

Shale Number	Plasticity Index	Clapak*	Curing Temperature	
			70°F	110°F
29	11	25	57.85	75.6
30	13	25	39.1	49.3
31	25	35	63.9	50.1
32	40	50	49.8	47.4
33	12	25	42.6	56.5
34	12	25	42.9	47.5
35	5	20	78.1	77.2
36	18	25	55.9	70.9

Table D.10:

DRY AND IMMERSED STRENGTH, PSI, OF CLAPAK
STABILIZED SHALES CURED FOR 90 DAYS

Shale Number	Curing Temperature			
	70 ^o F		110 ^o F	
	Dry	Immersed	Dry	Immersed
29	27.2	---	51.8	---
30	24.0	---	49.0	---
31	27.3	---	27.4	---
32	20.4	---	24.2	---
33	32.7	---	32.8	---
34	32.7	---	41.5	---
35	22.6	---	64.0	---
36	17.2	---	93.0	---

--- Sample broke on immersion in water.

Table D.11:

DRY AND IMMERSED STRENGTH, PSI, OF CHEMICAL (CLAPAK - CLASET)
STABILIZED SHALES CURED FOR 28 DAYS

Shale Number	Curing Temperature			
	70°F		110°F	
	Dry	Immersed	Dry	Immersed
29	27.4	-	106.6	-
30	24.0	-	27.3	-
31	22.5	-	33.8	-
32	24.2	-	85.9	-
33	46.6	-	61.8	-
34	17.3	-	32.7	-
35	78.1	-	77.2	-
36	20.8	-	41.4	-

- All samples broke on immersion in water.

Table D.12:

DRY AND IMMERSED STRENGTH, PSI, OF CLAPAK AND CLASET
STABILIZED SHALES CURED FOR 90 DAYS

Shale Number	Curing Temperature			
	70°F		110°F	
	Dry	Immersed	Dry	Immersed
29	13.7	---	72.5	---
30	17.3	---	121.1	---
31	28.9	---	100.1	---
32	20.7	---	144.3	---
33	51.5	---	101.7	---
34	31.0	---	101.8	---
35	+	---	+	---
36	46.5	---	98.0	---

+ Strength almost negligible

--- Sample broke on immersion in water

Table D.13:
 DRY AND IMMERSED STRENGTH, PSI, OF FLY-ASH
 STABILIZED SHALES CURED FOR 28 DAYS

Shale Number	Curing Temperature			
	70°F		110°F	
	Dry	Immersed	Dry	Immersed
Atok ² 29	325.5	159.6	305.9	221.0
Lebate 30	109.3	174.4	360.9	139.4
Claypool 31	96.6	51.4	104.4	62.7
Fredricksburg ² 32	126.1	---	186.8	72.0
JV 33	110.0	93.8	141.0	83.2
Bart 34	65.0	52.1	89.6	55.4
Aupt 35	149.5	120.7	238.7	108.7
Sanora ² 36	107.5	96.6	147.7	103.8

--- Sample broke on immersion in water.

Table D.14:

DRY AND IMMERSED STRENGTH, PSI, OF FLY-ASH
STABILIZED SHALES CURED FOR 90 DAYS

Shale Number	Curing Temperature			
	70°F		110°F	
	Dry	Immersed	Dry	Immersed
A10K2 29	367.8	221.7	569.0	500.6
30	278.1	159.4	381.6	361.3
31	124.9	62.1	117.9	106.9
32	178.0	-----	216.5	96.4
33	102.2	72.8	130.2	76.2
34	77.8	44.9	94.8	82.8
35	265.6	142.2	273.7	179.4
36	183.5	124.5	204.7	180.7

----- Sample broke on immersion in water.

APPENDIX E
WEATHERABILITY TEST DATA

Table E.1:
 UNCONFINED COMPRESSIVE STRENGTH, PSI,
 OF LIME STABILIZED (16%) SHALES
 AFTER WET-DRY CYCLES

Shale Number	28-Day Cured			90-Day Cured		
	Wet-Dry Cycles			Wet-Dry Cycles		
	0	5	15	0	5	15
29	73.2	58.9	--	131.4	98.3	--
30	120.3	463.7	380	238.5	432.6	434.0
31	337.8	175.0	--	360.7	153.7	--
32	248.4	--	--	258.5	--	--
33	87.8	275.3	249	309.6	421.6	210.0
34	161.3	236.9	192	370.2	327.3	513.0
35	659.0	402.3	194	698.0	399.5	180.0
36	145.1	122.7	195	396.6	278.3	438.0

-- Sample dispersed while subjected to wet-dry cycles.

Table E.2:

UNCONFINED COMPRESSIVE STRENGTH, PSI,
OF CEMENT STABILIZED (14%) SHALES
AFTER WET-DRY CYCLES

Shale Number	28-Day Cured			90-Day Cured		
	Wet-Dry Cycles			Wet-Dry Cycles		
	0	5	15	0	5	15
29	902.5	486.4	761.0	1438.1	513.2	861.0
30	1094.7	329.1	1005	1204.1	711.4	990.0
31	499.0	422.6	586	477.5	377.8	673.0
32	556.7	159.4	201.5	611.9	60	--
33	887.5	428.4	863.5	938.8	444.0	722.0
34	981.5	748.7	711	941.3	606.9	1042.0
35	1451.5	678.8	892	1238.7	627.0	816.0
36	777.0	327.0	641.5	1287.7	454.5	1149.0

-- Sample dispersed while subjected to wet-dry cycles.

Table E.3:

UNCONFINED COMPRESSIVE STRENGTH, PSI
OF FLY ASH STABILIZED (25%) SHALES
AFTER WET-DRY CYCLES

Shale Number	28-Day Cured			90-Day Cured		
	Wet-Dry Cycles			Wet-Dry Cycles		
	0	5	15	0	5	15
29	305.9	223.5	158.0	569.0	197.0	164.0
30	360.9	220.0	172.0	381.6	216.0	192.0
31	104.4	105.5	137.0	117.9	110.0	100.0
32	186.8	--	--	210.5	--	--
33	141.0	263.0	225.0	130.2	229.0	236.0
34	86.9	313.5	496.0	94.8	370.0	333.0
35	238.7	292.5	409.0	273.7	422.0	336.0
36	147.7	203.0	275.0	204.7	238.0	203.0

-- Sample Dispersed while subjected to freeze-thaw cycles.

Table E.4:

UNCONFINED COMPRESSIVE STRENGTH, PSI
OF LIME STABILIZED (6%) SHALES
AFTER FREEZE-THAW CYCLES

Shale Number	28-Day Cured			90-Day Cured		
	Freeze-Thaw Cycles			Freeze-Thaw Cycles		
	0	5	15	0	5	15
29	73.2	--	--	131.4	--	--
30	120.3	238.2	176.6	238.5	258.7	193.6
31	337.8	463.4	194.5	360.7	200.5	283.4
32	348.4	--	--	258.5	209.0	--
33	87.8	184.4	209.9	309.6	409.7	216.1
34	161.2	50.7	32.1	370.2	585.8	248.3
35	659.0	90.2	168.4	698.0	622.7	147.0
36	145.1	--	--	369.6	508.7	--

-- Sample dispersed while subjected to freeze-thaw cycles.

Table E.5:

UNCONFINED COMPRESSIVE STRENGTH, PSI
OF CEMENT STABILIZED (14%) SHALES
AFTER FREEZE-THAW CYCLES

Shale Number	28-Day Cured			90-Day Cured		
	Freeze-Thaw Cycles			Freeze-Thaw Cycles		
	0	5	15	0	5	15
29	902.5	486.4	556.0	1438.1	456.5	418.1
30	1094.7	329.1	444.6	1204.1	539.1	427.4
31	499.0	422.6	482.3	477.5	506.5	304.8
32	556.7	159.4	212.5	611.9	210.4	89.8
33	887.5	428.4	565.0	938.8	923.5	944.4
34	981.5	787.4	583.5	941.3	601.8	887.8
35	1451.5	678.8	364.8	1238.7	631.4	588.7
36	777.0	327.0	233.8	1287.7	675.9	648.1

-- Sample dispersed while subjected to freeze-thaw cycles.

Table E.6:

UNCONFINED COMPRESSIVE STRENGTH, PSI
OF FLY ASH STABILIZED (25%) SHALES
AFTER FREEZE-THAW CYCLES

Shale Number	28-Day Cured			90-Day Cured		
	Freeze-Thaw Cycles			Freeze-Thaw Cycles		
	0	5	15	0	5	15
29	305.9	--	--	569.0	--	--
30	360.9	--	--	381.6	136.2	--
31	104.4	--	--	117.9	--	--
32	186.8	--	--	216.5	--	--
33	141.0	--	--	130.2	65.2	--
34	86.9	--	--	94.8	127.5	--
35	238.7	--	--	273.7	136.7	--
36	147.7	--	--	204.7		--

-- Sample dispersed while subjected to freeze-thaw cycles.

Table F.1:

OPTIMUM MOISTURE CONTENT AND MAXIMUM DRY DENSITY OF LIME
STABILIZED (6%) SHALES AFTER DELAYED COMPACTION

Shale Number	Optimum Moisture Content, %				Maximum Dry Density, pcf			
	Delay Time, hr				Delay Time, hr			
	0	1	2	4	0	1	2	4
29	20.6	22.4	20.2	19.5	101.9	97.6	101.8	101.0
30	16.9	15.6	15.6	15.6	111.3	112.6	111.7	111.0
31	23.2	22.7	22.7	22.7	99.0	100.0	99.0	98.8
32	29.5	21	22.7	23.0	86.0	86.9	91.2	85.0
33	21.0	18.8	18.8	18.8	102.9	106.4	105.8	105.4
34	20.0	24.8	17.4	14.0	103.4	102.3	105.8	105.0
35	23.5	22.0	22.0	22.0	99.6	102.0	100.6	100.4
36	20.3	17.7	17.7	17.7	102.3	108.7	108.4	108.0

Table F.2:

UNCONFINED COMPRESSIVE STRENGTH, PSI, OF LIME
STABILIZED SHALES (6%) AFTER DELAYED COMPACTION

Shale Number	28-Day Cured				90-Day Cured			
	Delay Time, hr				Delay Time, hr			
	0	1	2	4	0	1	2	4
29	51.7	41.7	45.0	48.2	58.4	63.9	81.2	75.7
30	62.6	62.2	86.3	74.4	96.3	99.8	84.3	106.2
31	63.0	113.1	143.9	137.3	142.2	204.7	209.1	231.9
32	113.8	117.5	130.8	113.3	328.8	162.0	168.9	159.3
33	29.0	27.7	22.3	31.1	37.8	29.1	44.8	50.3
34	46.0	48.3	50.0	46.8	71.7	91.6	94.8	84.7
35	311.4	264.6	273.4	152.0	455.9	433.1	349.3	305.5
36	43.9	43.2	38.2	46.8	63.8	43.2	44.7	56.4

Table F.3:

IMMERSED STRENGTH, PSI, OF LIME STABILIZED
SHALES AFTER DELAYED COMPACTION

Shale Number	28-Day Curing				90-Day Curing			
	Delay Time, hr				Delay Time, hr			
	0	1	2	4	0	1	2	4
29	--	24.2	26.0	24.2	14.0	26.4	41.4	43.1
30	--	13.7	20.6	20.7	34.4	44.9	39.7	34.6
31	85.6	93.5	156.5	116.2	131.5	197.7	259.7	199.7
32	19.8	12.8	16.1	14.4	23.1	18.9	27.6	20.7
33	48.0	6.9	8.7	17.1	85.9	31.1	41.5	22.5
34	41.2	34.5	34.7	41.5	59.5	98.3	91.6	64.0
35	332.3	310.9	251.1	180.6	157.4	276.6	287.6	278.7
36	--	37.8	27.5	20.7	14.0	24.2	27.7	26.1

-- Sample dispersed after immersion in water.

Table F.4:

OPTIMUM MOISTURE CONTENT AND MAXIMUM DRY DENSITY OF FLY ASH
STABILIZED (25%) SHALES AFTER DELAYED COMPACTION

Shale Number	Optimum Moisture Content, %				Maximum Dry Density, pcf			
	Delay Time, hr				Delay Time, hr			
	0	1	2	4	0	1	2	4
29	17.2	23.5	15.3	16.5	110.0	94.1	106.8	95
30	18.0	14.6	19.6	20.7	110.0	97.5	93.5	94.2
31	26.1	22.2	16.2	16.0	84.8	91.1	97.2	86
32	26.2	25.4	27.7	25.6	87.6	85.5	90.6	85.4
33	16.7	25.0	19.0	21.9	92.6	91.5	91.2	89.3
34	19.4	17.8	13.3	12.5	90.7	94.2	95.2	94.0
35	20.6	25.5	21.5	23.5	86.4	93.6	97.5	83.5
36	16.0	20.0	13.2	15.0	94.1	91.9	99.3	94.0

Table F.5:
UNCONFINED COMPRESSIVE STRENGTH, PSI, OF FLY ASH
STABILIZED SHALES (25%) AFTER DELAYED COMPACTION

Shale Number	28-Day Cured				90-Day Cured			
	Delay Time, hr				Delay Time, hr			
	0	1	2	4	0	1	2	4
29	325.5	232.0	213.0	165.0	367.8	184.0	231.0	219.0
30	190.3	130.0	303.5	313.0	278.1	262.0	342.0	317.0
31	96.6	164.0	182.0	156.0	124.9	208.0	200.0	193.0
32	126.1	175.5	195.0	164.0	178.0	260.0	235.0	204.0
33	110.1	121.5	108.0	130.5	102.2	--	116.0	125.0
34	65.0	224.0	185.0	158.0	77.8	306.0	306.0	267.0
35	149.5	138.0	148.0	139.0	265.6	177.0	184.0	175.0
36	107.5	178.5	177.0	199.0	183.5	220.0	233.0	255.0

Table F.6:

IMMERSED STRENGTHS, PSI, OF FLY ASH STABILIZED
SHALES (25%) AFTER DELAYED COMPACTION

Shale Number	28-Day Cured				90-Day Cured			
	Delay Time, hr				Delay Time, hr			
	0	1	2	4	0	1	2	4
29	159.6	73	103.0	116.0	221.7	284.0	283.0	265.0
30	174.4	123.5	184.5	164.0	159.4	340.0	358.0	437.0
31	51.4	104.5	145.0	92.0	62.1	204.0	216.0	221.0
32	--	16.0	--	--	--	184.0	177.0	169.0
33	93.8	52.0	51.0	56.5	72.8	--	217.0	225.0
34	52.1	200.0	218.0	193.0	44.9	177.0	154.0	191.0
35	120.7	74.0	103.0	72.0	142.2	121.0	128.0	106.0
36	96.6	103.0	116.0	127.5	124.5	124.0	154.0	195.0

-- Sample dispersed after immersion in water.

Table F.7:

OPTIMUM MOISTURE CONTENT AND MAXIMUM DRY DENSITY OF
CEMENT STABILIZED SHALES (14%) AFTER DELAYED COMPACTION

Shale Number	Optimum Moisture Content, %				Maximum Dry Density, pcf			
	Delay Time, hr				Delay Time, hr			
	0	1	2	4	0	1	2	4
29	17.5	17.0	15.6	15.0	107.9	108.0	105.1	100.0
30	16.2	22.1	13.6	20.7	116.0	106.1	110.2	98.0
31	22.2	20	17.6	13.5	100.05	93.0	96.5	92.0
32	27.0	25.4	21.4	18.8	90.3	95.1	92.0	90.4
33	19.8	18.6	17.4	15.4	106.4	109.7	106.3	101.5
34	17.8	15.0	13.4	11.0	107.1	109.8	106.7	104.0
35	19.4	23.2	14.2	14.5	105.1	88.9	96.0	93.0
36	19.0	16.4	11.9	11.8	107.8	110.4	107.4	104.1

Table F.8:

UNCONFINED COMPRESSIVE STRENGTHS, PSI, OF CEMENT
STABILIZED (14%) SHALES AFTER DELAYED COMPACTION

Shale Number	28-Day Cured				90-Day Cured			
	Delay Time, hr				Delay Time, hr			
	0	1	2	4	0	1	2	4
29	515.5	390.4	365.6	195.8	615.0	385.2	418.0	244.5
30	895.4	305.3	389.0	383.1	1050.9	412.6	455.2	389.7
31	88.7	36.9	316.7	347.6	272.3	370.0	323.5	405.9
32	386.6	133.8	193.5	187.3	619.2	174.8	346.5	277.0
33	568.6	147.0	386.0	343.0	531.0	434.0	458.0	415.0
34	768.5	526.0	496.0	393.0	809.8	541.0	472.0	326.0
35	772.7	621.0	566.0	588.0	824.8	600.0	882.0	650.0
36	507.2	446.0	273.0	300.0	612.4	413.0	345.0	307.0

Table F.9:

IMMERSED STRENGTHS, PSI, OF CEMENT STABILIZED
(14%) SHALES AFTER DELAYED COMPACTION

Shale Number	28-Day Cured				90-Day Cured			
	Delay Time, hr				Delay Time, hr			
	0	1	2	4	0	1	2	4
29	278.1	297.9	303.3	155.9	322.8	284.5	353	190.6
30	526.7	265.0	280.6	261.1	522.4	278.8	336.9	343.5
31	167.7	353.2	333.5	280.8	548.0	431.3	375.9	452.6
32	91.8	55.0	72.1	76.2	118.8	60.1	83.4	93.0
33	693.4	153.0	349.0	372.0	552.1	448.0	316.0	351.0
34	309.7	310.0	321.0	272.0	359.5	426.0	373.0	386.0
35	646.2	614.0	576.0	501.0	493.0	756.0	549.0	428.0
36	438.1	462.0	367.0	309.0	388.7	347.0	284.0	228.0

Table G.1:
 DRY AND IMMERSSED STRENGTH OF SHALES STABILIZED
 WITH LIME (6%) AND NaCl (2%)
 AND CURED FOR 28 DAYS

Shale Number	UCS, psi	
	Dry	Immersed
29 ⁺	153.9 (1.10)*	115.4 (-)
30	53.7 (0.55)	343.0 (-)
31 ⁺	255.2 (3.05)	233.0 (1.72)
32	111.0 (-0.55)	58.8 (1.94)
33 ⁺	148.6 (4.10)	103.1 (1.14)
34 ⁺	158.6 (2.43)	105.8 (1.56)
35	327.6 (+0.05)	287.0 (-0.13)
36	47.0 (+0.09)	31.3 (-)

$$* \text{ Strength Beneficiation Index (SBI) } = \frac{b - a}{a}$$

a = compressive strength of lime treated 6% shale
 b = compressive strength lime (6%) + NaCl (2%)
 treated shale

- For lime stabilized case, there was no strength at all.
- + Lime + NaCl, Combination more beneficial than lime alone.

Table G.2:
 UNCONFINED COMPRESSIVE STRENGTH OF SOME SELECTED
 SHALES STABILIZED WITH LIME (6%)
 AND NaCl (2%) AND CURED FOR 90
 DAYS

Shale Number	UCS, psi	
	Dry	Immersed
29	83.1 (.42)*	58.9 (3.20)
31	298.5 (1.09)	274.4 (1.09)
32	143.5 (-0.34)	50.5 (1.18)
33	107.0 (1.27)	76.1 (-0.11)
34	200.4 (1.80)	165.1 (1.77)

*Strength Beneficiation Index

$$(SBI) = \frac{b - a}{a}$$

b = compressive strength of lime 6% + NaCl
2% stabilized shales.

a = compressive strength of lime stabilized
shales.

+ Lime 6% + NaCl 2% combination is more beneficial
than lime alone.

Table G.3:
 DRY AND IMMERSED STRENGTH OF CEMENT (10%) AND
 FLYASH (4%) STABILIZED SHALES

Shale Number		
	Dry	Immersed
29	278.2 (-0.46)*	178.2 (-0.35)
30	663.1 (-0.25)	545.7 (+0.036)
31+	543.6 (5.17)	502.9 (2.0)
32	331.7 (-0.14)	76.6 (-0.16)
33	356.8 (-0.37)	305.5 (-0.55)
34	496.1 (-0.35)	359.3 (0.16)
35	655.0 (-0.15)	520.6 (-0.19)
36	196.7 (-0.61)	149.4 (-0.65)

$$\text{*Strength Beneficiation Index (SBI)} = \frac{b - a}{a}$$

b = compressive strength of cement 10% + flyash 4% treated shales.

a = compressive strength of cement (14%) treated shales.

+Cement and flyash combination is better than cement alone.

Table G.4:

DRY AND IMMERSSED STRENGTH OF CEMENT (10%)
FLYASH (4%) AND Na₂CO₃ (0.5%) STABILIZED
SHALES CURED FOR 28 DAYS

Shale Number	UCS, psi	
	Dry	Immersed
29	250 (-0.51)	214.5 (-0.22)
30	596.6 (-0.33)	390.2 (-0.25)
31 ⁺	325.8 (2.69)	345.1 (1.06)
32	285.5 (-0.26)	73.3 (0.19)
33	364.0 (-0.35)	291.1 (-0.58)
34	383.5 (-0.50)	201.5 (-0.32)
35	428.6 (-0.44)	354.7 (-0.45)
36	451.1 (-0.11)	315.7 (-0.28)

$$\text{*Strength Beneficiation Index (SBI)} = \frac{b - a}{a}$$

b = compressive strength of cement 10% + flyash 4% + Na₂CO₃ 0.5% treated shales.

a = compressive strength of cement (14%) treated shale alone.

⁺ Cement, flyash and Na₂CO₃ Combination is better than cement treatment alone.

Table G.5:

DRY AND IMMERSSED STRENGTH OF CEMENT (8%),
LIME (2%) AND FLYASH (4%) STABILIZED
SHALES CURED FOR 28 DAYS

Shale Number	UCS, psi	
	Dry	Immersed
29	249.6 (-0.51)	166.7 (-0.40)
30	526.9 (-0.41)	362.0 (-0.31)
31 ⁺	413.5 (3.69)*	413.2 (1.47)
32 ⁺	524.6 (0.35)	132.6 (0.45)
33	349.2 (-0.38)	281.5 (-0.59)
34	410.7 (-0.46)	309.1 (0)
35	656.7 (-0.15)	494.1 (-0.23)
36	245.5 (-0.51)	127.5 (-0.71)

$$\text{*Strength Beneficiation Index (SBI)} = \frac{b - a}{a}$$

b = compressive strength of cement 10% + lime 2%
+ flyash 4%.

a = compressive strength of cement (14%) treated
shales alone.

⁺Combination of cement, lime and flyash (8%, 2%, 4%) is better than
cement (14%) alone.

Table G.6:

DRY AND IMMERSSED STRENGTH OF LIME (5%) AND
FLYASH (10%) STABILIZED SHALES
CURED FOR 28 DAYS

Shale Number	UCS, psi	
	Dry	Immersed
29	280 (-0.13)	215 (0.35)
30 ⁺	283.7 (0.48)	191.3 (0.09)
31 ⁺	247.2 (1.57)*	233.2 (3.56)
32 ⁺	231.4 (0.83)	86.8 (-)
33 ⁺	165.9 (0.5)	132.3 (0.37)
34 ⁺	200.6 (2.07)	131.2 (2.52)
35 ⁺	383.3 (1.55)	285.2 (1.37)
36	96.5 (-0.10)	45.2 (-0.53)

$$\text{*Strength Beneficiation Index (SBI)} = \frac{b - a}{a}$$

b = compressive strength of flyash 10% + lime 5% treated shales.

a = compressive strength of flyash treated shale alone (25%).

⁺Flyash and lime combination is better than flyash alone.

Table G.7:

DRY AND IMMERSED STRENGTH OF LIME (5%), FLYASH (10%)
AND CaCl₂ (0.5%) STABILIZED SHALE
CURED FOR 28 DAYS

Shale Number	UCS, psi	
	Dry	Immersed
29	249.3 (-0.23)	168.5 (0.05)
30 ⁺	361.9 (.9)	202.8 (0.16)
31 ⁺	274.5 (1.83)*	201.0 (2.94)
32	207.9 (0.64)	12.4 (-)
33 ⁺	232.1 (1.10)	126.7 (0.34)
34 ⁺	444.4 (5.83)	212.2 (3.07)
35 ⁺	555.8 (2.7)	321.6 (1.67)
36 ⁺	242.2 (1.26)	193.9 (1.01)

$$\text{*Strength Beneficiation Index (SBA)} = \frac{b - a}{a}$$

b = compressive strength of lime 5% + flyash 10% +
CaCl₂ 0.5%.

a = compressive strength of flyash (25%) treated shales.

⁺Lime, Flyash and CaCl₂ combination is better than flyash
treatment alone.

Table G.8:

DRY AND IMMERSSED STRENGTH OF LIME, (5%), FLYASH (10%)
AND NaoH (0.5%) STABILIZED SHALES
CURED FOR 28 DAYS

Shale Number	UCS, psi	
	Dry	Immersed
29	227.1 (-0.30)	130.1 (-0.16)
30 ⁺	414.1 (1.17)*	172.2 (0.0)
31 ⁺	308.8 (2.20)	163.6 (2.19)
32 ⁺	410.0 (2.25)	40.0 (-)
33	168.1 (0.52)	55.4 (-0.4)
34 ⁺	279.8 (3.29)	185.8 (2.55)
35 ⁺	543.1 (2.62)	302.5 (1.51)
36	201.8 (0.87)	90.1 (-0.00)

$$\text{*Strength Beneficiation Index (SBI)} = \frac{b - a}{a}$$

b = compressive strength of lime 5% + flyash 10% +
NaoH 0.5% treated shales.

a = compressive strength of flyash (25%) treated shales.

⁺Lime, flyash and NaoH combination is better than flyash
treatment alone.

Table G.9:

DRY AND IMMERSSED STRENGTH OF LIME, FLYASH
AND SODIUM CARBONATE (5%, 10%, and 0.5%)
STABILIZED SHALES CURED FOR 28 DAYS

Shale Number	UCS, psi	
	Dry	Immersed
29	236.1 (-0.27)	228.2 (0.43)
30 ⁺	434.4 (1.28)*	295.0 (0.69)
31 ⁺	219.5 (1.19)	208.4 (3.07)
32	235.0 (0.86)	46.5 (-)
33 ⁺	160.1 (0.45)	120.2 (0.27)
34 ⁺	235 (2.61)	221.7 (3.25)
35 ⁺	581 (2.67)	410.5 (2.41)
36 ⁺	158.9 (0.47)	149.8 (0.56)

$$\text{*Strength Beneficiation Index (SBI)} = \frac{b - a}{a}$$

b = compressive strength of lime 10% + flyash 10% +
sodium carbonate 0.5% treated shales.

a = compressive strength of flyash (25%) treated shales.

⁺Lime, flyash and sodium carbonate combination is better than
flyash alone.

Table 1

TABLE 1. ANALYTICAL DATA FOR RAW AND STABILIZED SHALES

Sample	Loss on ignition (%)	Total organic carbon (%)	Total nitrogen (%)	Total sulfur (%)
101	35.21	4.12	0.15	0.08
102	36.27	4.21	0.16	0.09
103	36.97	4.30	0.17	0.10
104	38.12	4.40	0.18	0.11
105	39.11	4.50	0.19	0.12
106	40.14	4.60	0.20	0.13
107	41.21	4.70	0.21	0.14
108	42.31	4.80	0.22	0.15

APPENDIX H

ELECTRON MICROSCOPY OF RAW AND STABILIZED SHALES

Figure 1 shows the electron micrographs of raw shale (101) and stabilized shale (108). The raw shale (101) shows a highly porous structure with large interconnected pores, while the stabilized shale (108) shows a more compact structure with smaller, more uniform pores.

Table H.1:

ELECTRON MICROSCOPIC STUDY OF RAW SHALES

Shale Number	Unconfined Compressive Strength, psi	Void Cross-Sectional Area, %	Largest Pore Intercept, mm
29	35.4	39.68	.018
30	27.3	56.54	.015
31	36.9	29.06	.022
32	40.2	64.48	.020
33	32.8	27.77	.011
34	36.3	36.70	.011
35	24.7	92.26	.019
36	29.5	72.42	.024

Total Sample Cross-Sectional Area = $6.7 \times 10^{-4} \text{ mm}^2$

Magnification 3000 times

Sample dried by evaporation $7 \times 10^{-5} \text{ mm Hg}$ vacuum

Table H.2:

ELECTRON MICROSCOPIC STUDY OF LIME (6%) STABILIZED
SHALES CURED FOR 28 DAYS AT 110°F

Shale Number	Unconfined Compressive Strength, psi	Void Cross-Sectional Area, %	Largest Pore Intercept, mm
29	73.2	32.73	.015
30	120.3	32.73	.011
31	337.8	43.65	.014
32	248.4	36.78	.017
33	87.8	42.65	.012
34	161.2	69.94	.028
35	659.0	60.01	.014
36	145.1	59.52	.013

Total sample cross sectional area = $6.7 \times 10^{-4} \text{ mm}^2$

Magnification 3000 times

Sample dried by evaporation at $7 \times 10^{-5} \text{ mm Hg}$ vacuum

Table H.3:

ELECTRON MICROSCOPIC STUDY OF LIME (6%) STABILIZED
SHALES CURED FOR 90 DAYS AT 110°F

Shale Number	Unconfined Compressive Strength, psi	Void Cross-Sectional Area, %	Largest Pore Intercept, mm
29	131.4	25.90	.010
30	238.5	29.10	.011
31	360.7	31.40	.012
32	258.8	26.90	.010
33	309.8	31.10	.012
34	370.2	36.80	.015
35	698.0	49.80	.011
36	369.6	16.50	.009

Total sample cross sectional area = $6.7 \times 10^{-4} \text{ mm}^2$

Magnification 3000 times

Sample dried by evaporation at $7 \times 10^{-5} \text{ mm Hg}$ vacuum

Table H.4:

ELECTRON MICROSCOPIC STUDY OF CEMENT (14%) STABILIZED
SHALES CURED FOR 28 DAYS AT 110°F

Shale Number	Unconfined Compressive Strength, psi	Void Cross-Sectional Area, %	Largest Pore Intercept, mm
29	902.5	29.00	.014
30	1094.7	42.90	.013
31	499.0	20.70	.025
32	556.7	32.10	.018
33	887.5	18.60	.008
34	805.7	30.30	.021
35	1451.5	15.90	.012
36	777.0	27.90	.016

Total Sample Cross-Sectional Area = $6.7 \times 10^{-4} \text{ mm}^2$

Magnification 3000 times

Sample dried by evaporation 7×10^{-5} Hg vacuum

Table H.5:

ELECTRON MICROSCOPIC STUDY OF FLYASH (25%) STABILIZED
SHALES CURED FOR 90 DAYS AT 110°F

Shale Number	Unconfined Compressive Strength, psi	Void Cross-Sectional Area, %	Largest Pore Intercept, mm
29	1438.1	24.70	.0115
30	1204.1	22.00	.012
31	477.5	20.50	.014
32	611.9	19.00	.008
32	938.3	33.00	.018
34	941.3	28.00	.013
35	1238.7	28.50	.012
36	1286.7	18.00	.016

Total Sample Cross-Sectional Area -- $6.7 \times 10^{-4} \text{ mm}^2$

Magnification 3000 times

Sample dried by evaporation at $7 \times 10^{-5} \text{ mm Hg}$ vacuum

Table H.6:

ELECTRON MICROSCOPIC STUDY OF FLYASH (25%) STABILIZED
SHALES CURED FOR 28 DAYS AT 110° F

Shale Number	Unconfined Compressive Strength, psi	Void Cross-Sectional Area, %	Largest Pore Intercept, mm
29	305.9	34.90	.012
30	360.9	21.20	.010
31	104.4	35.10	.020
32	186.8	38.00	.016
33	141.0	35.60	.013
34	89.6	30.40	.014
35	238.7	21.10	.009
36	147.7	39.10	.016

Total Sample Cross-Sectional Area = $6.7 \times 10^{-4} \text{ mm}^2$

Magnification 3000 times

Sample dried by evaporation at $7 \times 10^{-5} \text{ mm Hg}$ vacuum

Table H.7:

ELECTRON MICROSCOPIC STUDY OF FLYASH (25%) STABILIZED
SHALES CURED FOR 90 DAYS AT 110°F

Shale Number	Unconfined Compressive Strength, psi	Void Cross-Sectional Area, %	Largest Pore Intercept, mm
29	569.0	26.80	.015
30	381.6	34.40	.015
31	117.9	43.90	.019
32	216.5	29.40	.015
33	130.2	31.90	.012
34	94.8	22.70	.012
35	273.7	25.20	.011
36	204.7	27.30	.011

Total Sample Cross Sectional Area = $6.7 \times 10^{-4} \text{ mm}^2$

Magnification 3000 times

Sample dried by evaporation at 7×10^{-5} mm Hg vacuum

APPENDIX I

WEATHER CYCLES OF OKLAHOMA

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Chapter V	6
Chapter VI	7
Chapter VII	8
Chapter VIII	9
Chapter IX	10
Chapter X	11
Chapter XI	12
Chapter XII	13
Chapter XIII	14
Chapter XIV	15
Chapter XV	16
Chapter XVI	17
Chapter XVII	18
Chapter XVIII	19
Chapter XIX	20
Chapter XX	21
Chapter XXI	22
Chapter XXII	23
Chapter XXIII	24
Chapter XXIV	25
Chapter XXV	26
Chapter XXVI	27
Chapter XXVII	28
Chapter XXVIII	29
Chapter XXIX	30
Chapter XXX	31
Chapter XXXI	32
Chapter XXXII	33
Chapter XXXIII	34
Chapter XXXIV	35
Chapter XXXV	36
Chapter XXXVI	37
Chapter XXXVII	38
Chapter XXXVIII	39
Chapter XXXIX	40
Chapter XL	41
Chapter XLI	42
Chapter XLII	43
Chapter XLIII	44
Chapter XLIV	45
Chapter XLV	46
Chapter XLVI	47
Chapter XLVII	48
Chapter XLVIII	49
Chapter XLIX	50
Chapter L	51
Chapter LI	52
Chapter LII	53
Chapter LIII	54
Chapter LIV	55
Chapter LV	56
Chapter LVI	57
Chapter LVII	58
Chapter LVIII	59
Chapter LIX	60
Chapter LX	61
Chapter LXI	62
Chapter LXII	63
Chapter LXIII	64
Chapter LXIV	65
Chapter LXV	66
Chapter LXVI	67
Chapter LXVII	68
Chapter LXVIII	69
Chapter LXIX	70
Chapter LXX	71
Chapter LXXI	72
Chapter LXXII	73
Chapter LXXIII	74
Chapter LXXIV	75
Chapter LXXV	76
Chapter LXXVI	77
Chapter LXXVII	78
Chapter LXXVIII	79
Chapter LXXIX	80
Chapter LXXX	81
Chapter LXXXI	82
Chapter LXXXII	83
Chapter LXXXIII	84
Chapter LXXXIV	85
Chapter LXXXV	86
Chapter LXXXVI	87
Chapter LXXXVII	88
Chapter LXXXVIII	89
Chapter LXXXIX	90
Chapter LXXXX	91
Chapter LXXXXI	92
Chapter LXXXXII	93
Chapter LXXXXIII	94
Chapter LXXXXIV	95
Chapter LXXXXV	96
Chapter LXXXXVI	97
Chapter LXXXXVII	98
Chapter LXXXXVIII	99
Chapter LXXXXIX	100

Table I.1:

EXPLANATION OF LOCATION

<u>Symbol</u>	<u>Location</u>
A	Ada
B	Altus
C	Anadarko
D	Antlers
E	Ardmore
F	Great Salt Plains
G	Guymon
H	Kingfisher
I	McAlester
J	Muskogee
K	Pawhuska
L	Perry
M	Poteau
N	Seminole
O	Talooga
P	Vinita

Table I.2:

WET-DRY CYCLES

LOCATION^a

YEAR	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P
1958	38	33	31	39	39	35	30	37	44	45	34	39	40	43	31	47
1960	42	36	41	44	42	34	26	40	39	39	39	38	45	43	37	38
1961	41	29	33	40	35	28	26	29	36	42	36	33	43	37	34	46
1962	35	28	35	41	32	37	29	32	34	38	37	36	44	40	30	37
1963	25	23	29	28	22	33	16	27	28	30	23	37	30	26	30	26
1964	28	23	29	32	27	27	18	26	34	34	42	36	27	33	25	38
1965	36	31	31	35	38	28	31	33	33	32	33	30	38	34	28	34
1966	31	23	30	37	30	23	27	28	35	37	27	27	37	35	27	27
1967	29	20	29	39	36	32	23	31	37	40	40	36	38	35	32	45
1968	43	31	37	49	40	35	25	35	46	41	40	36	42	38	36	39
Third Highest	41	31	35	41	39	35	29	35	39	41	40	37	43	40	34	45
Mean	34.8	27.7	32.5	38.4	34.1	31.2	26.1	18.8	36.6	37.8	35.1	34.8	38.4	36.4	31.0	37.7
Standard Deviation	6.0	5.0	3.9	5.8	6.1	4.2	4.7	4.3	5.1	4.5	5.8	3.7	5.7	4.9	3.6	5.3

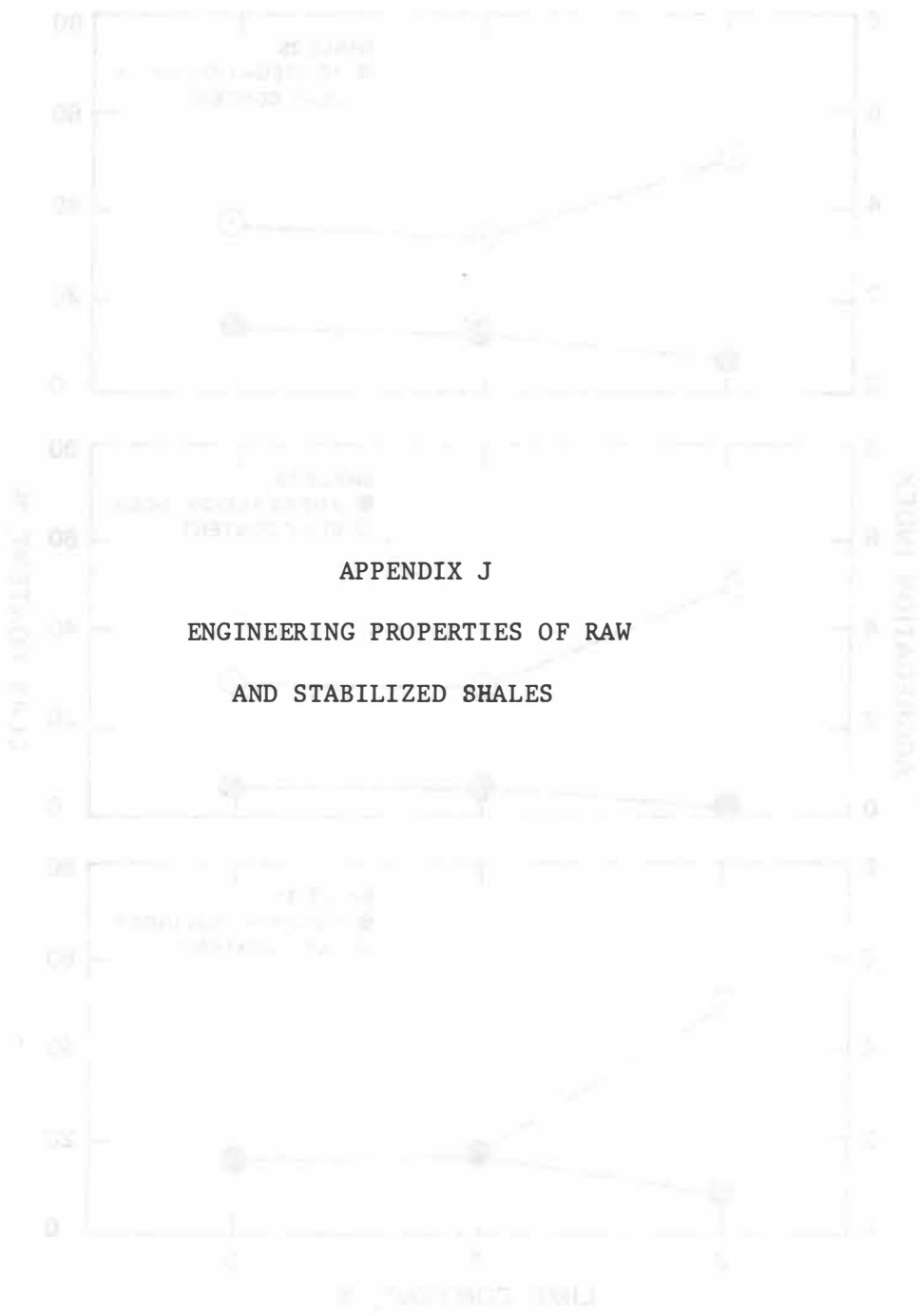
^aSee Table No. I-1 for Identification of location.

Table I.3:
 FREEZE-THAW CYCLES
 LOCATION^a

YEAR	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P
59-60	8	7	7	7	6	9	14	7	10	9	7	9	9	9	7	10
60-61	5	4	6	3	3	7	11	5	5	5	7	8	4	5	7	9
61-62	8	7	7	7	6	8	11	9	12	9	11	9	10	7	9	11
62-63	7	7	7	6	6	7	9	8	8	9	8	9	7	7	7	11
63-64	5	3	5	4	3	7	15	7	5	5	7	7	5	6	7	11
64-65	10	9	10	6	8	15	18	14	12	12	13	12	9	12	13	14
65-66	4	2	2	2	4	6	8	4	4	4	4	4	3	4	4	6
66-67	8	7	8	6	4	10	10	8	9	12	12	10	8	7	11	14
67-68	8	8	10	6	5	9	12	7	10	7	7	7	7	8	8	10
68-69	7	7	7	5	4	11	11	9	8	9	12	8	7	7	12	11
Third Highest	8	7	8	6	6	10	14	9	10	9	12	9	9	8	11	11
Mean	7	6.1	6.8	5.2	4.9	8.9	11.9	7.8	8.3	8.1	8.8	8.3	6.9	7.2	8.5	10.7
Standard Deviation	2.7	2.2	2.3	1.6	1.5	2.5	2.8	2.6	2.7	2.5	2.8	2.0	2.2	2.1	2.6	2.2

^aSee Table No. I-1 for Identification of location.

6/10/71
A



APPENDIX J
 ENGINEERING PROPERTIES OF RAW
 AND STABILIZED SHALES

Figure J-1. Engineering properties of raw and stabilized shales.

Atok 2

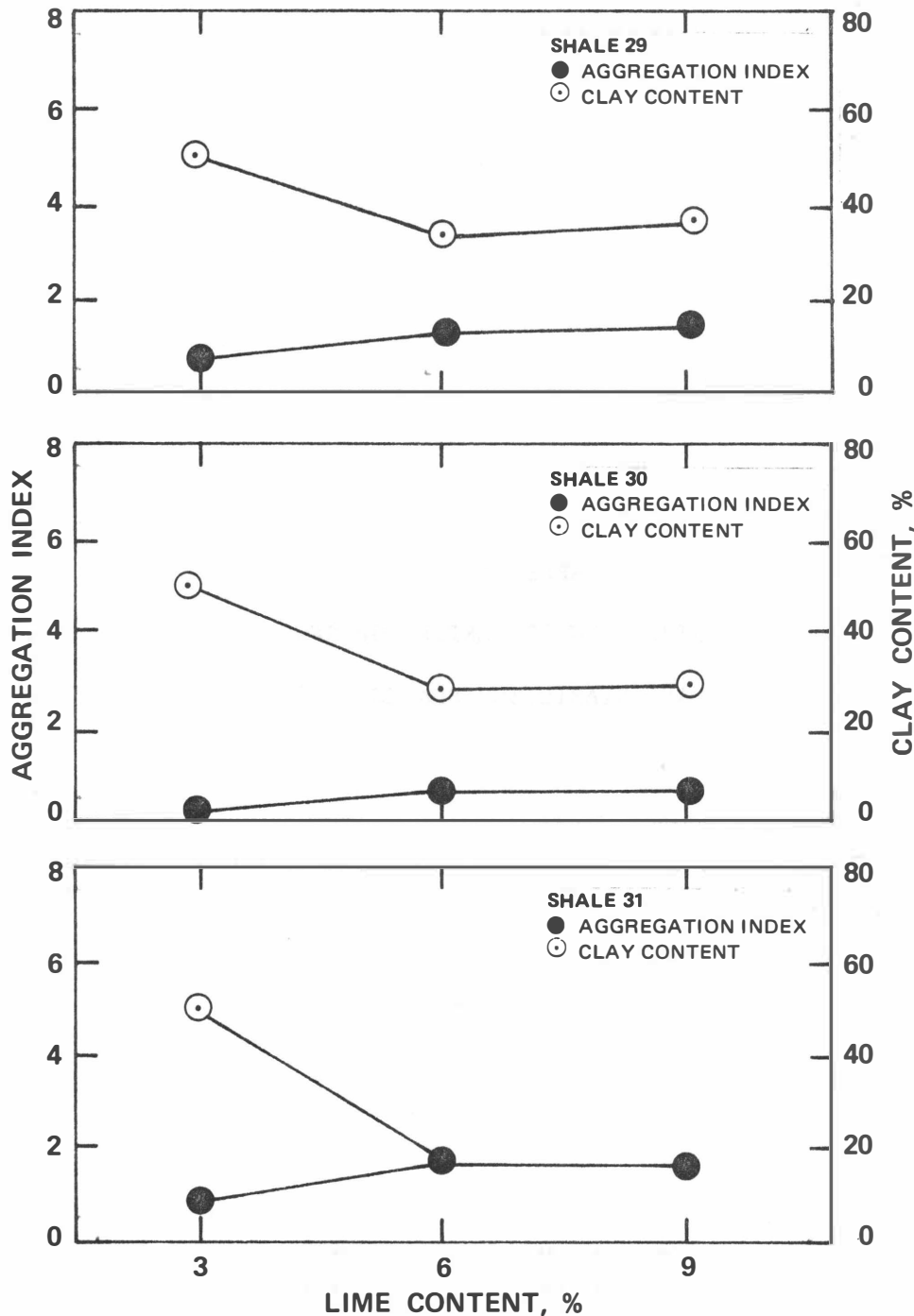


Figure J-1: Aggregation index and clay content relationship for lime stabilized shales.

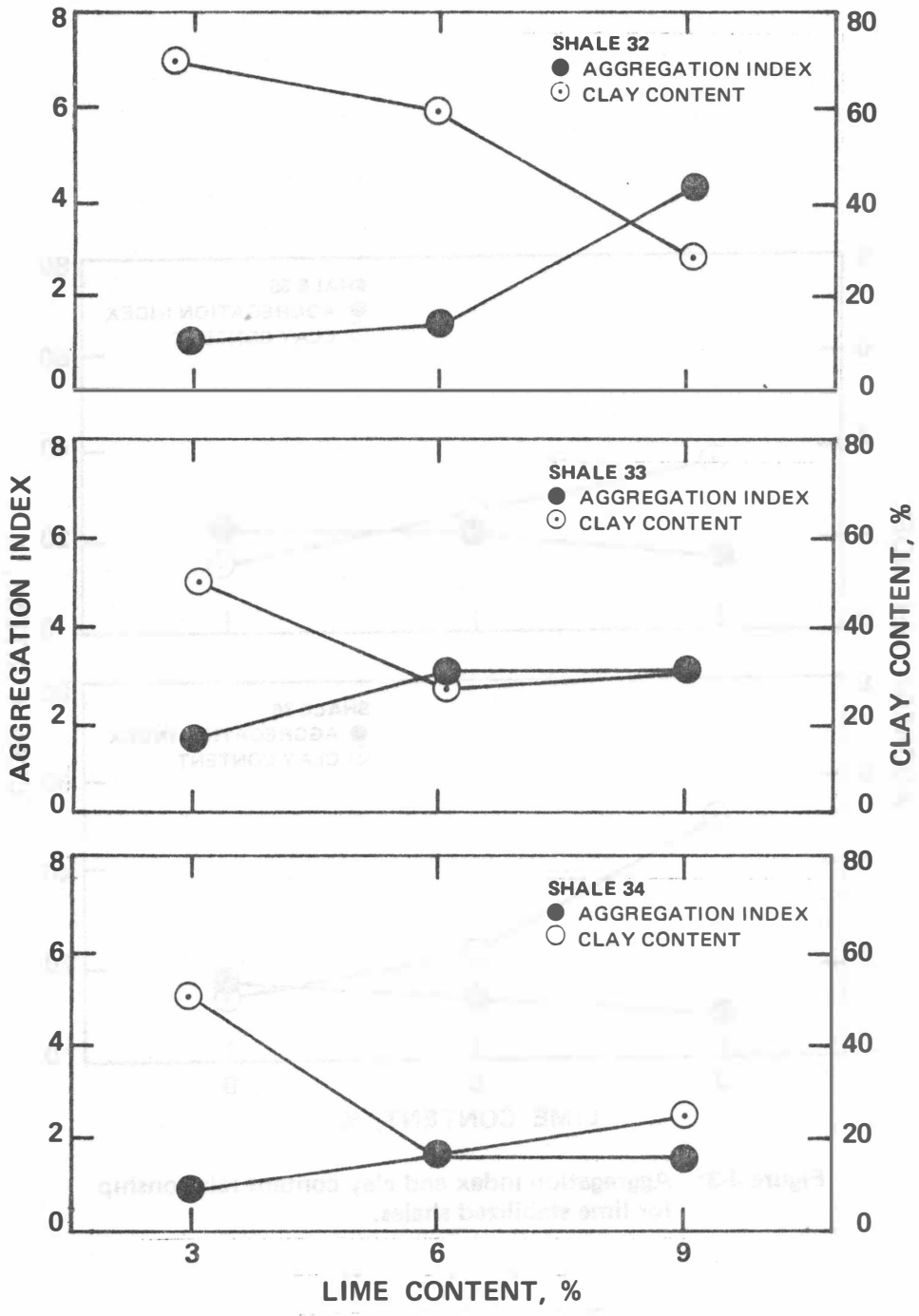


Figure J-2: Aggregation index and clay content relationship for lime stabilized shales.

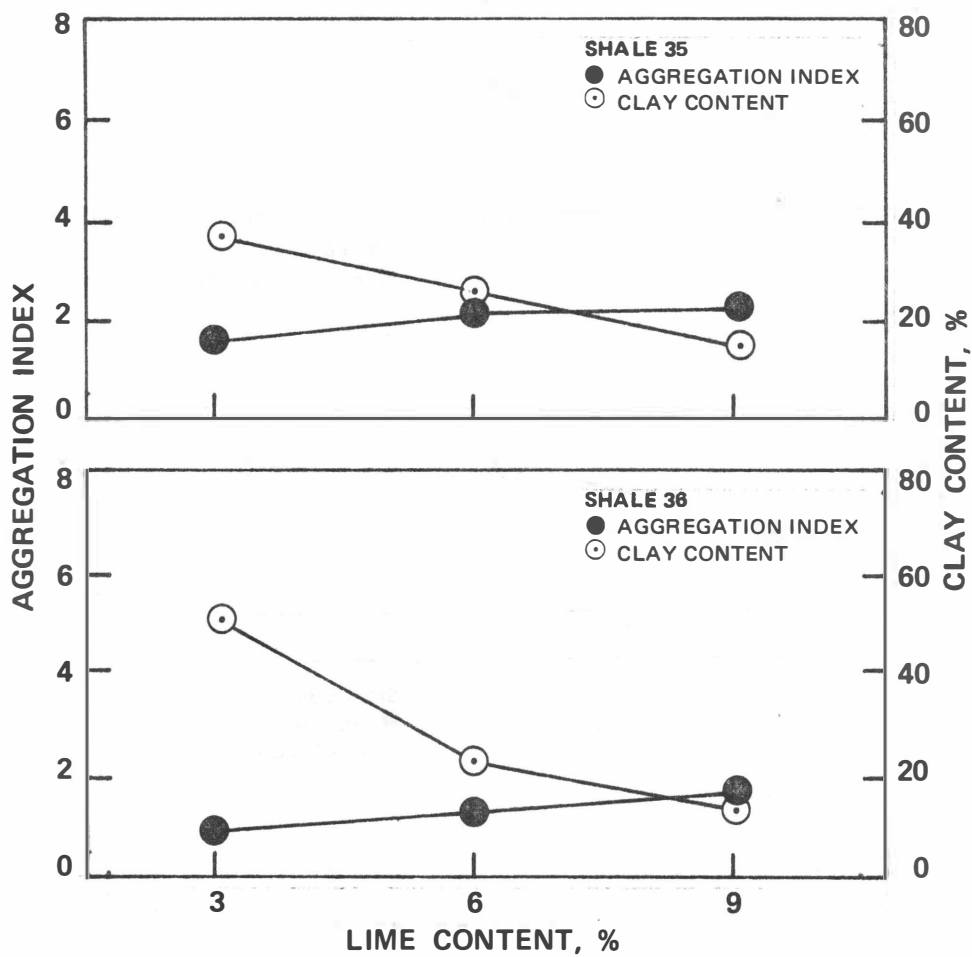


Figure J-3: Aggregation index and clay content relationship for lime stabilized shales.

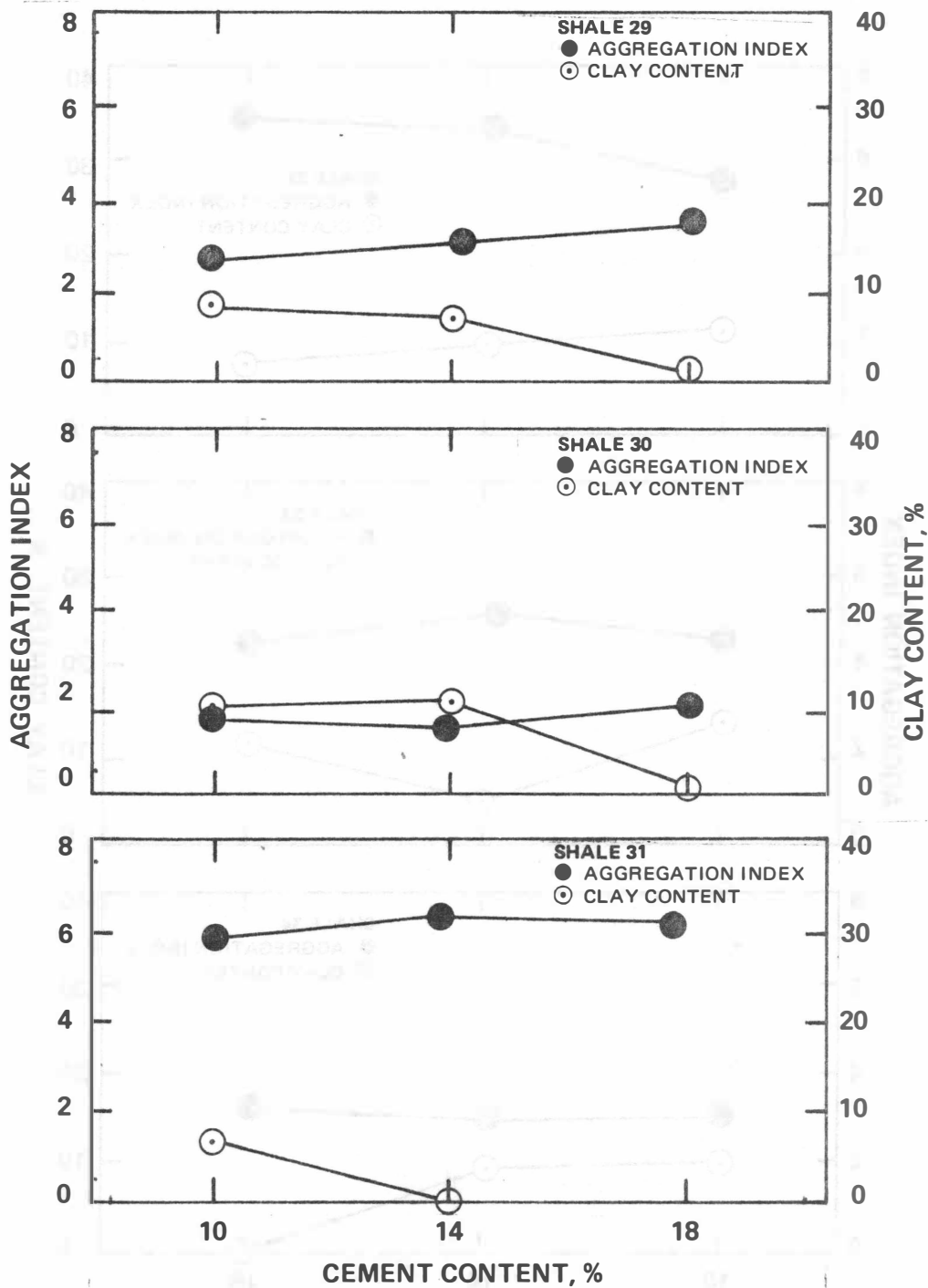


Figure J-4: Aggregation index and clay content relationship for cement stabilized shales.

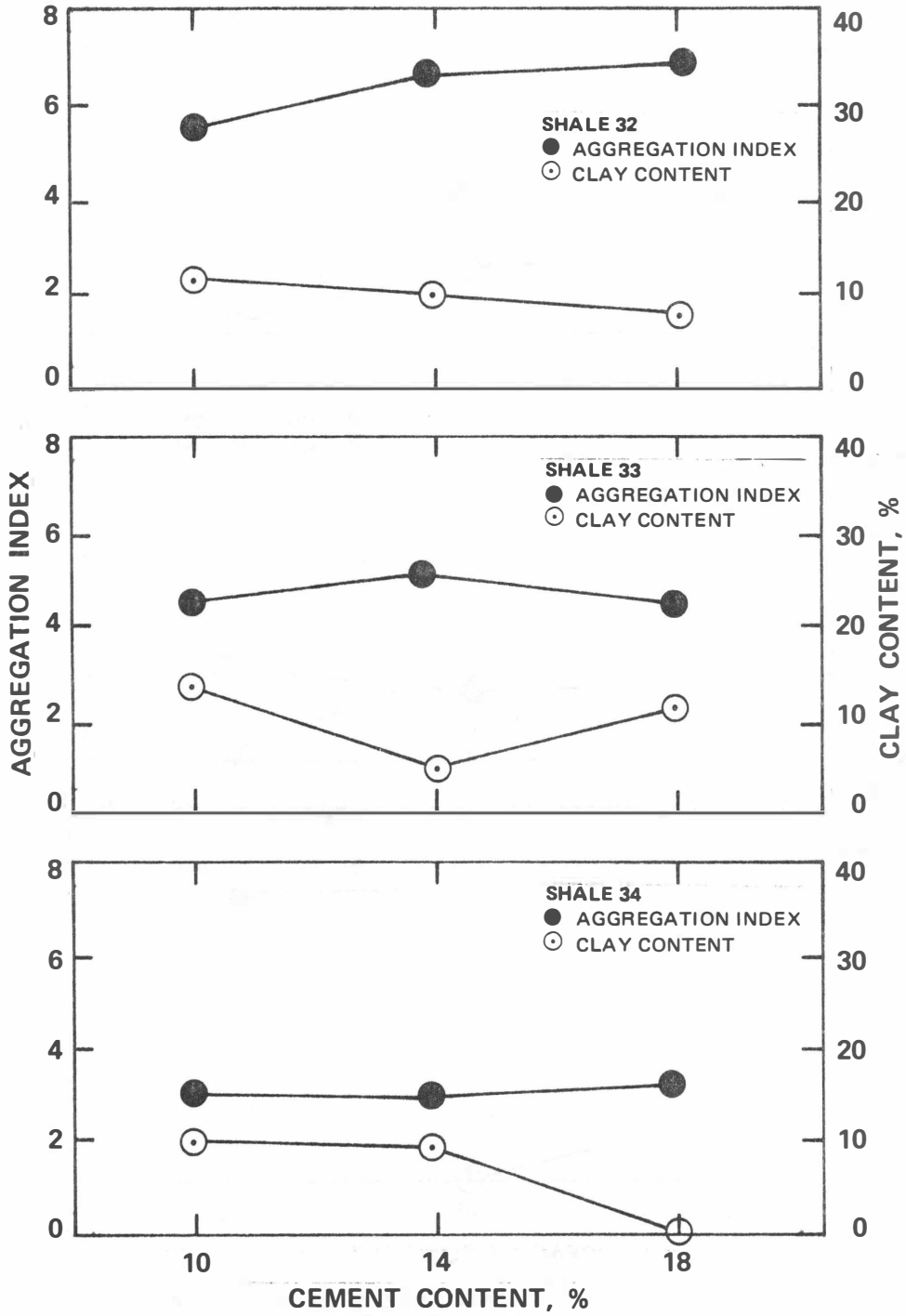


Figure J-5: Aggregation index and clay content relationship for cement stabilized shales.

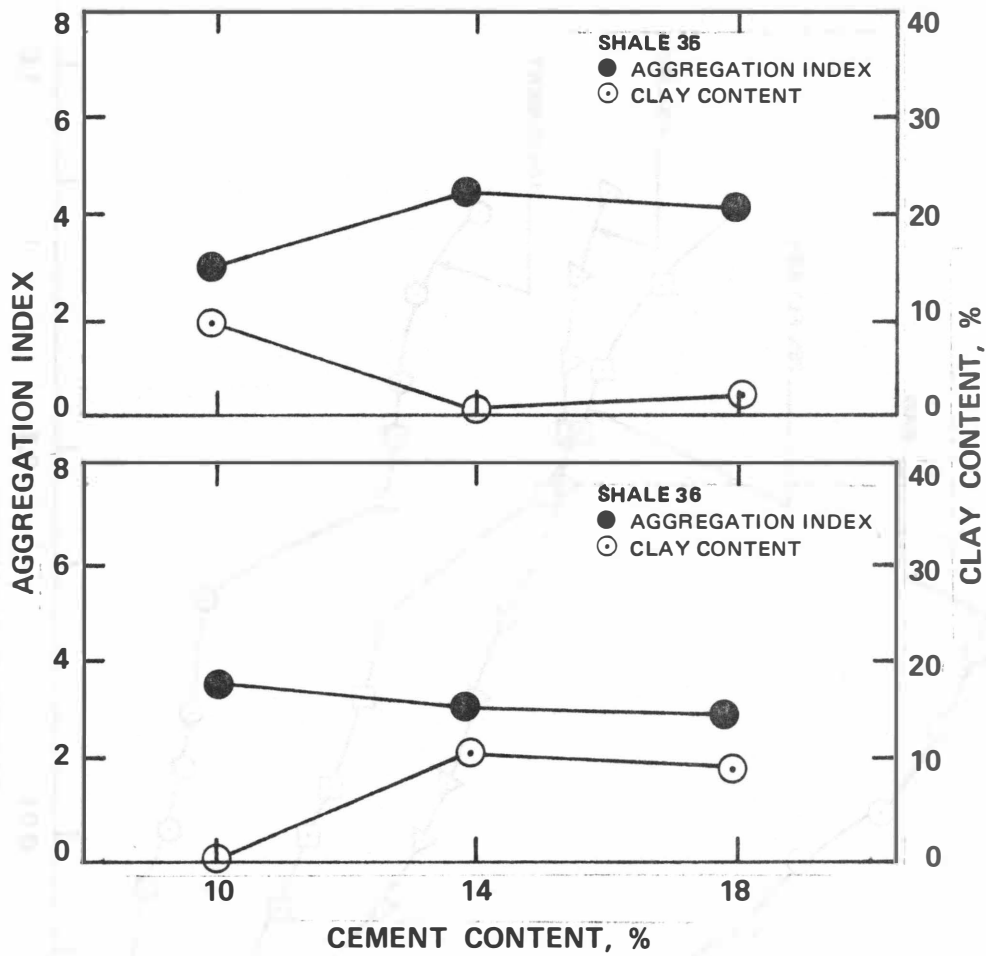


Figure J-6: Aggregation index and clay content relationship for cement stabilized shales.

290sd. $\frac{4820}{5020}$
 4490sd. $\frac{56}{20}$
 $\frac{3620}{5:14}$

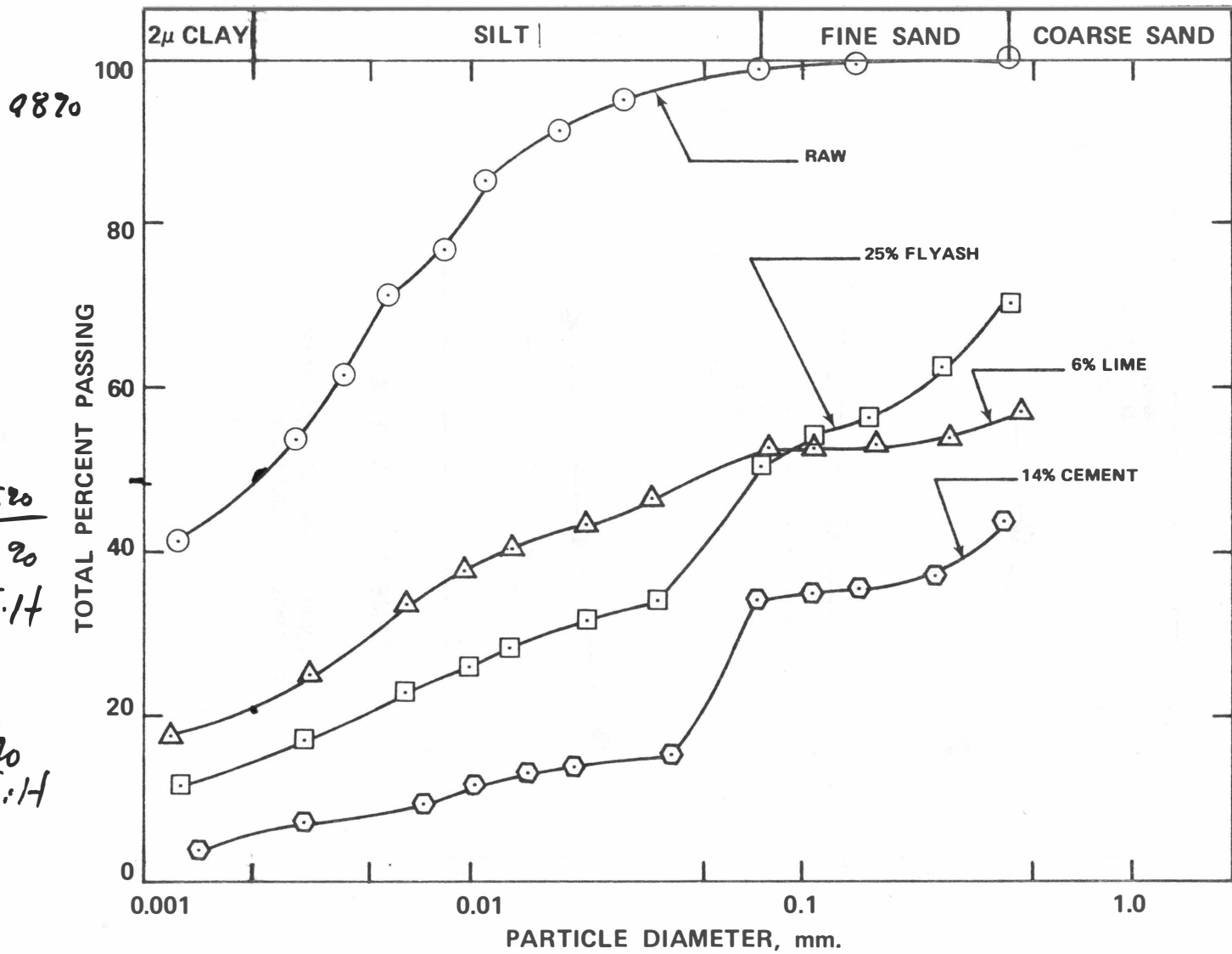


Figure J-7: Grain size distribution curve for shale 29. *Atoka*

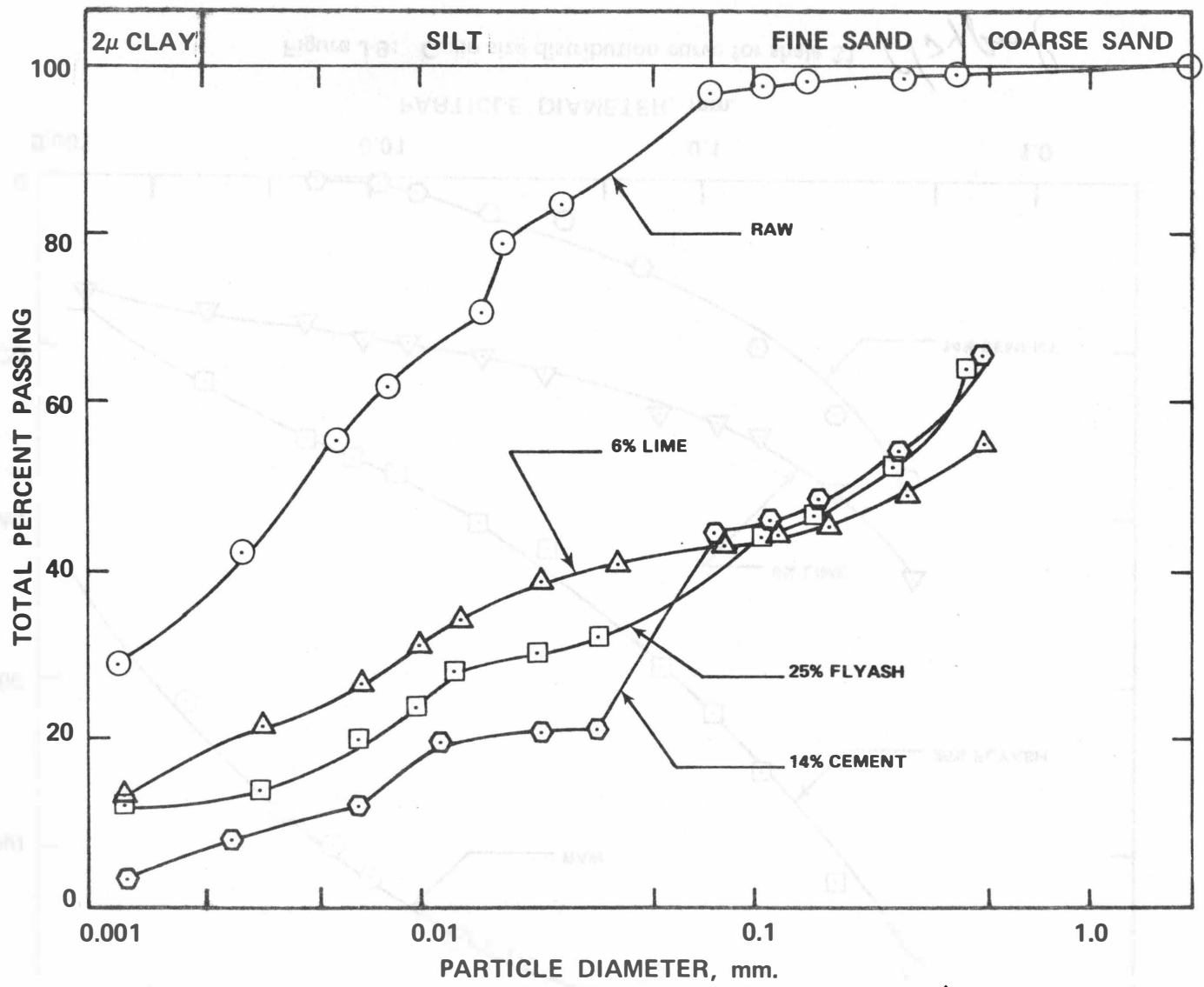


Figure J-8: Grain size distribution curve for shale 30. *webette*

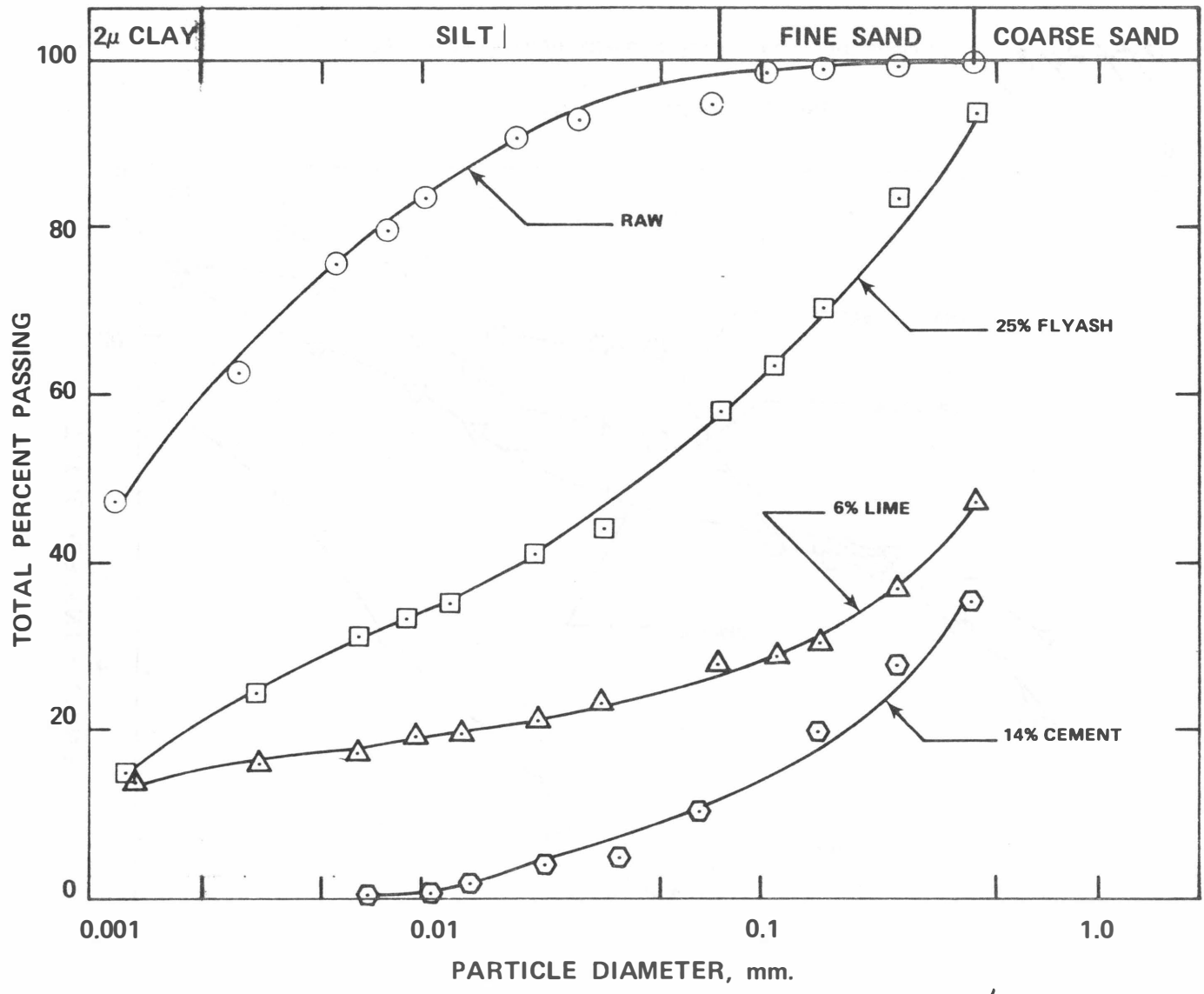


Figure J-9: Grain size distribution curve for shale 31. *claypool*

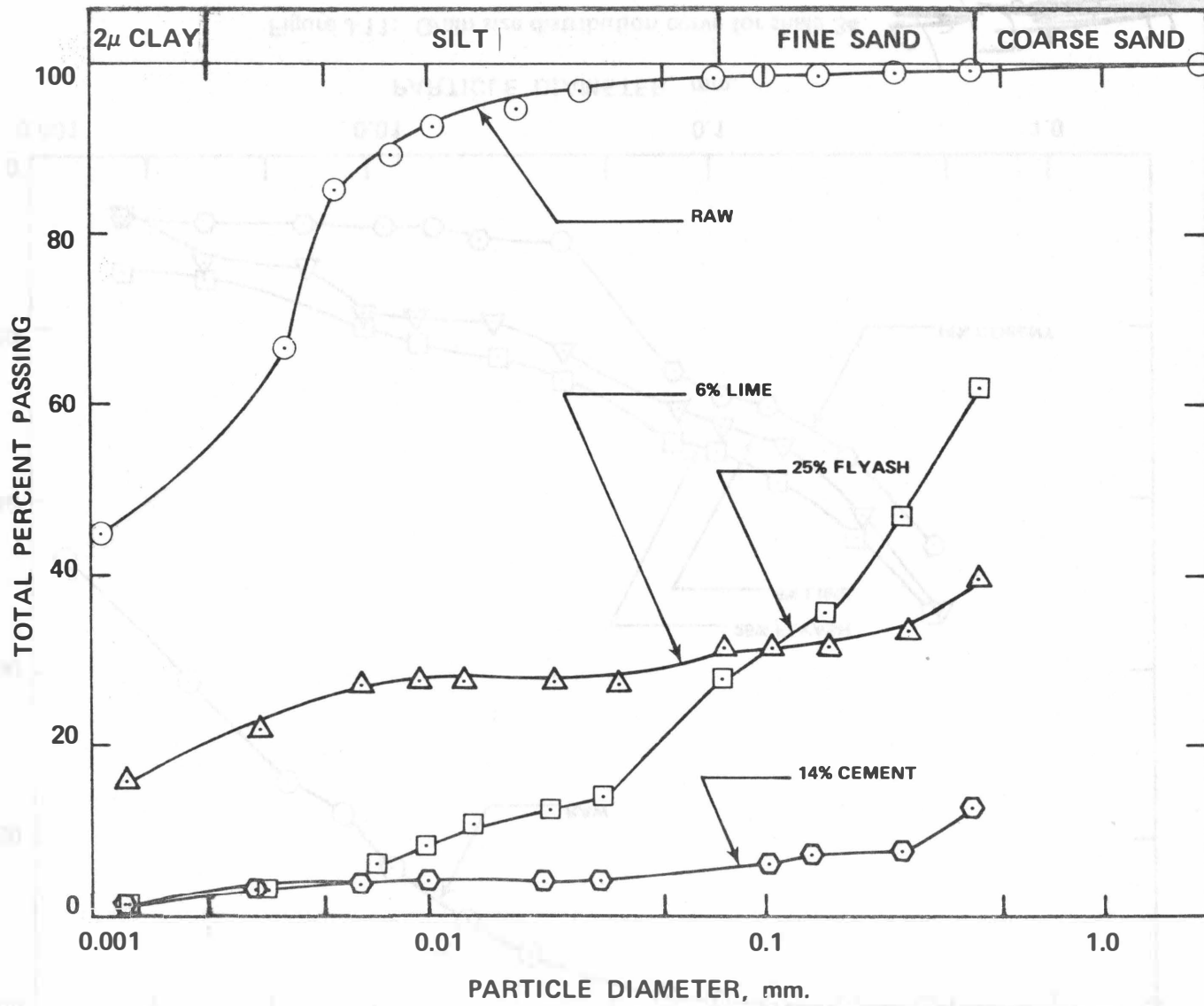


Figure J-10: Grain size distribution curve for shale 33. *Fredericksburg*

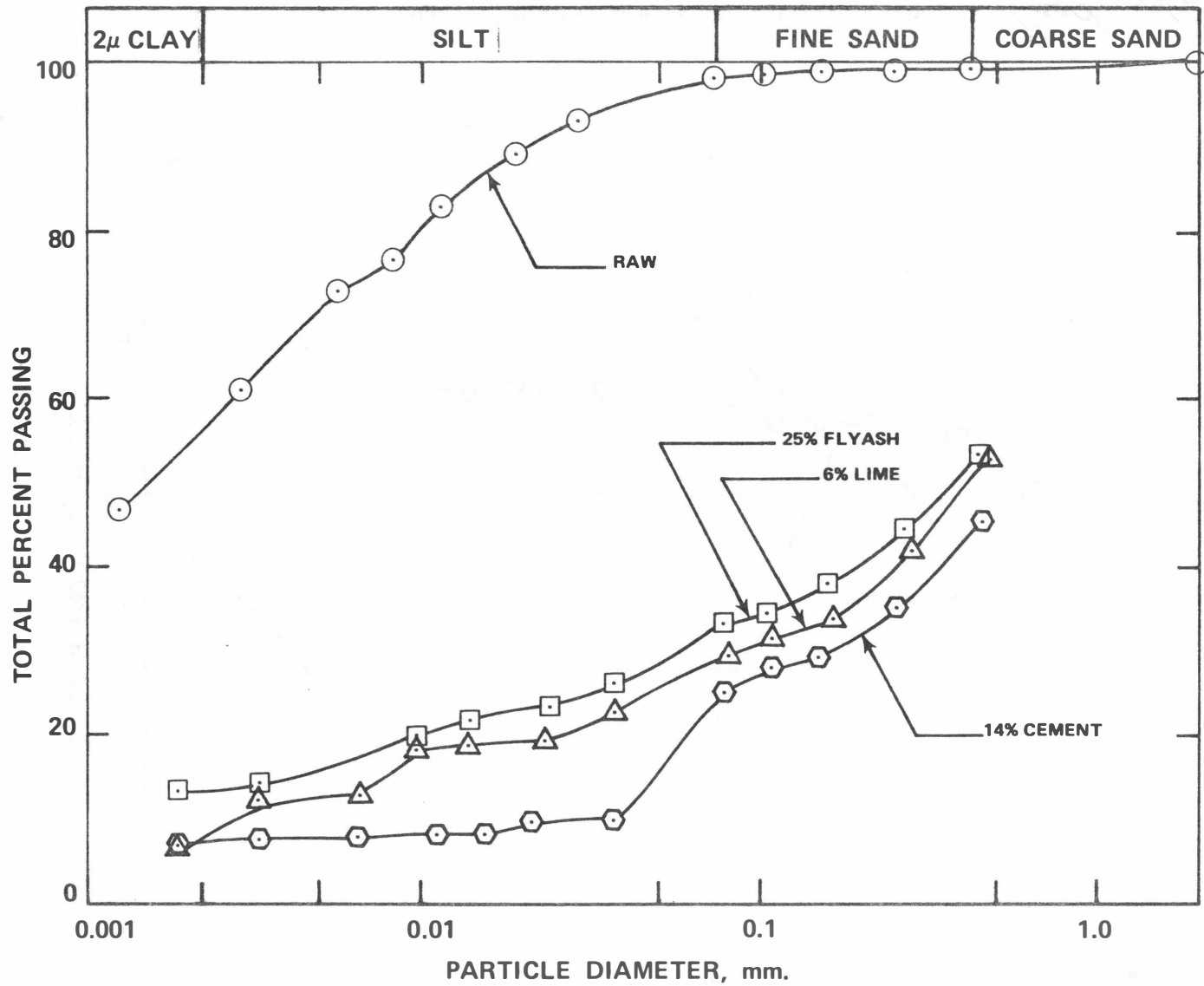


Figure J-11: Grain size distribution curve for shale 34.

~~Clay John W. Hay~~
BOG664

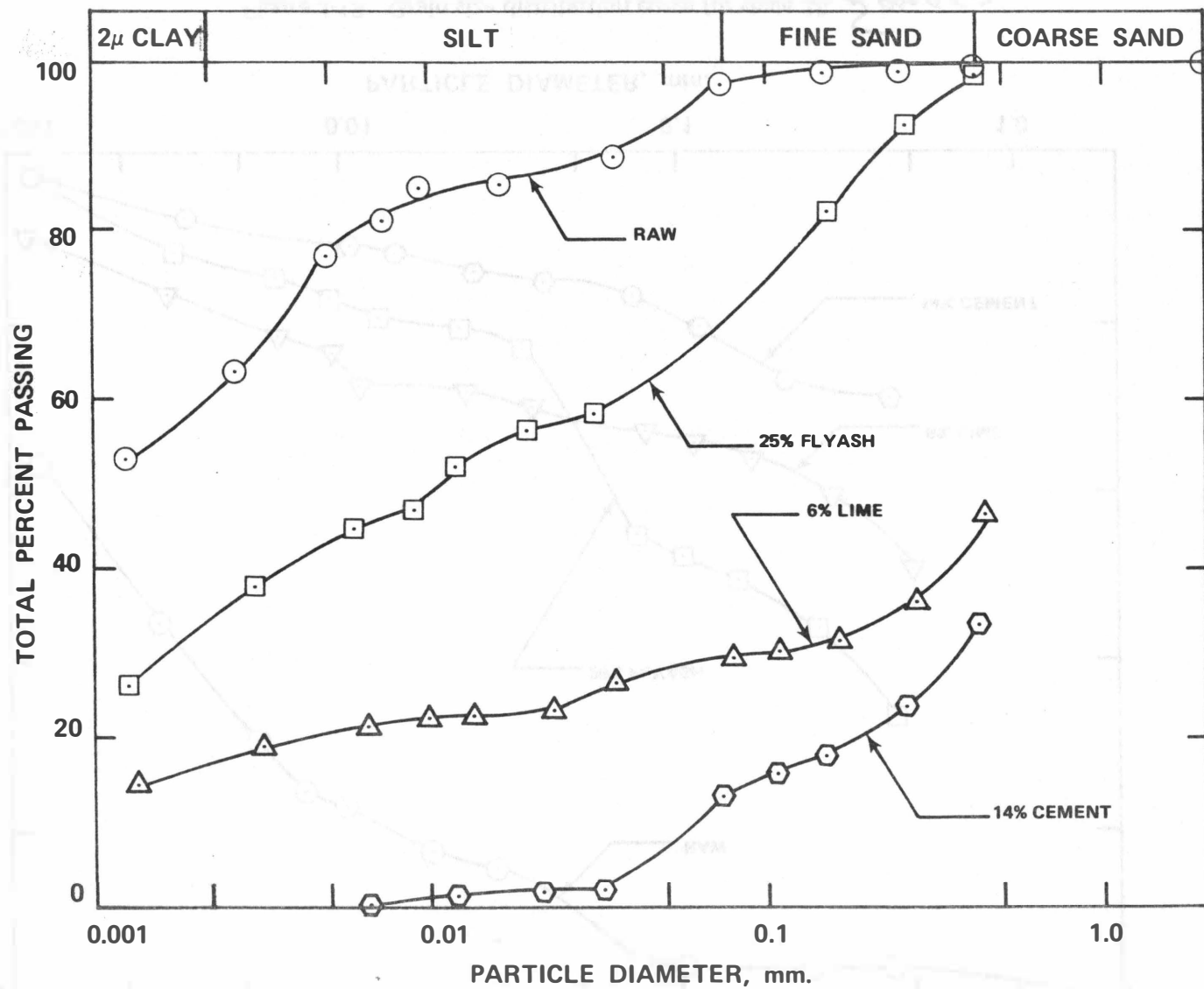


Figure J-12: Grain size distribution curve for shale 35.

Flowerpot

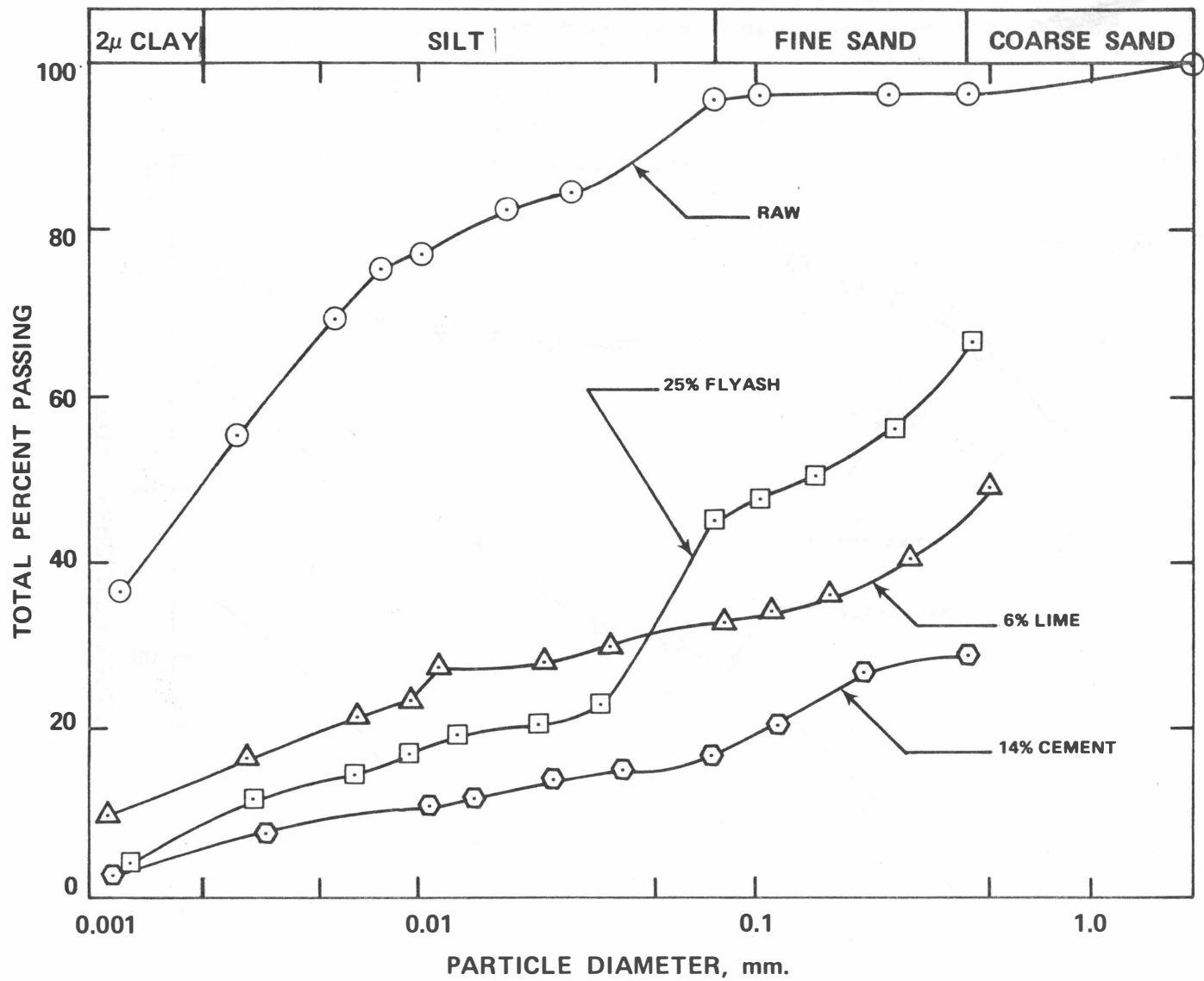


Figure J-13: Grain size distribution curve for shale 36. *Senora*

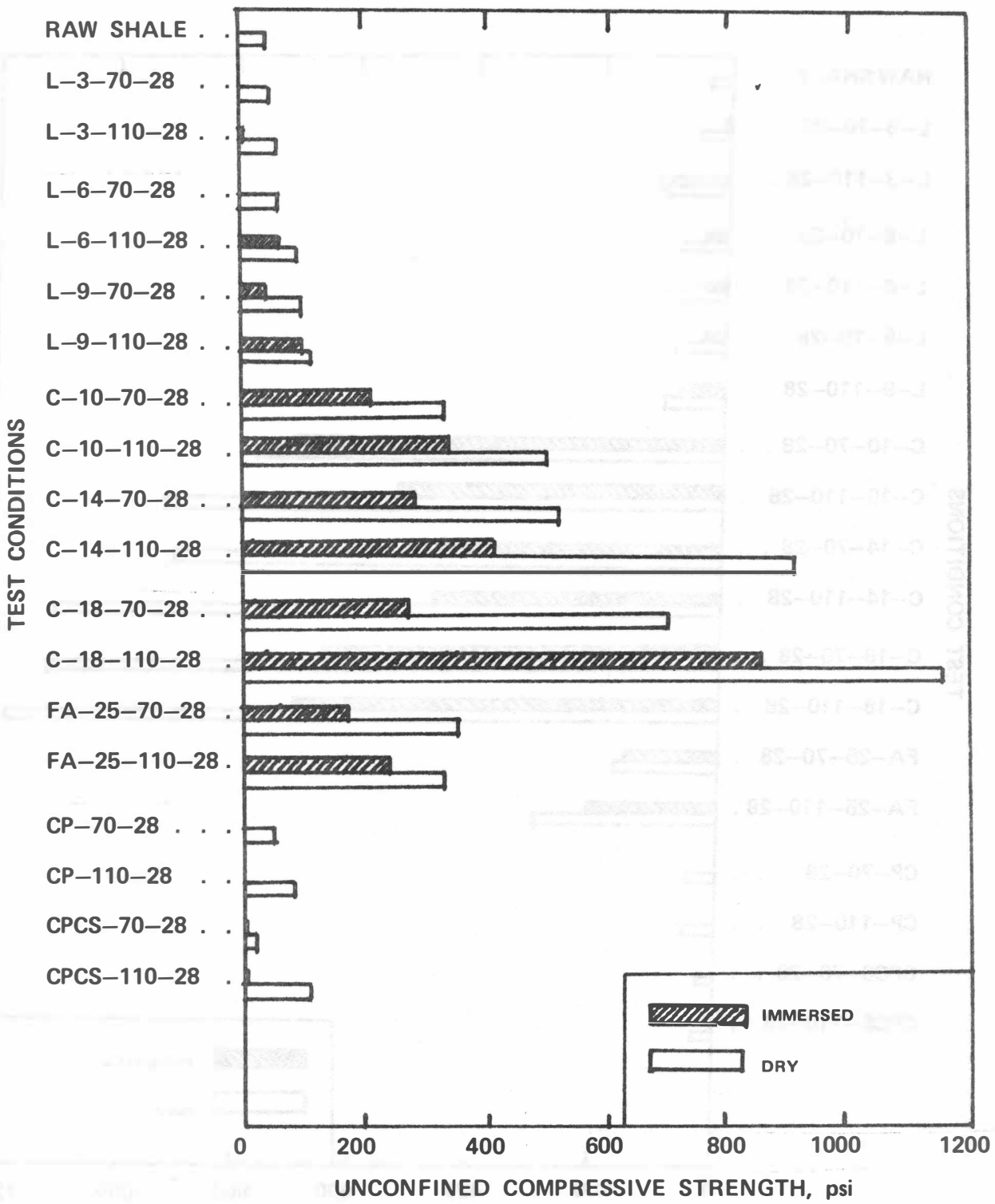


Figure J-14: Comparison of strength characteristics for shale 29.

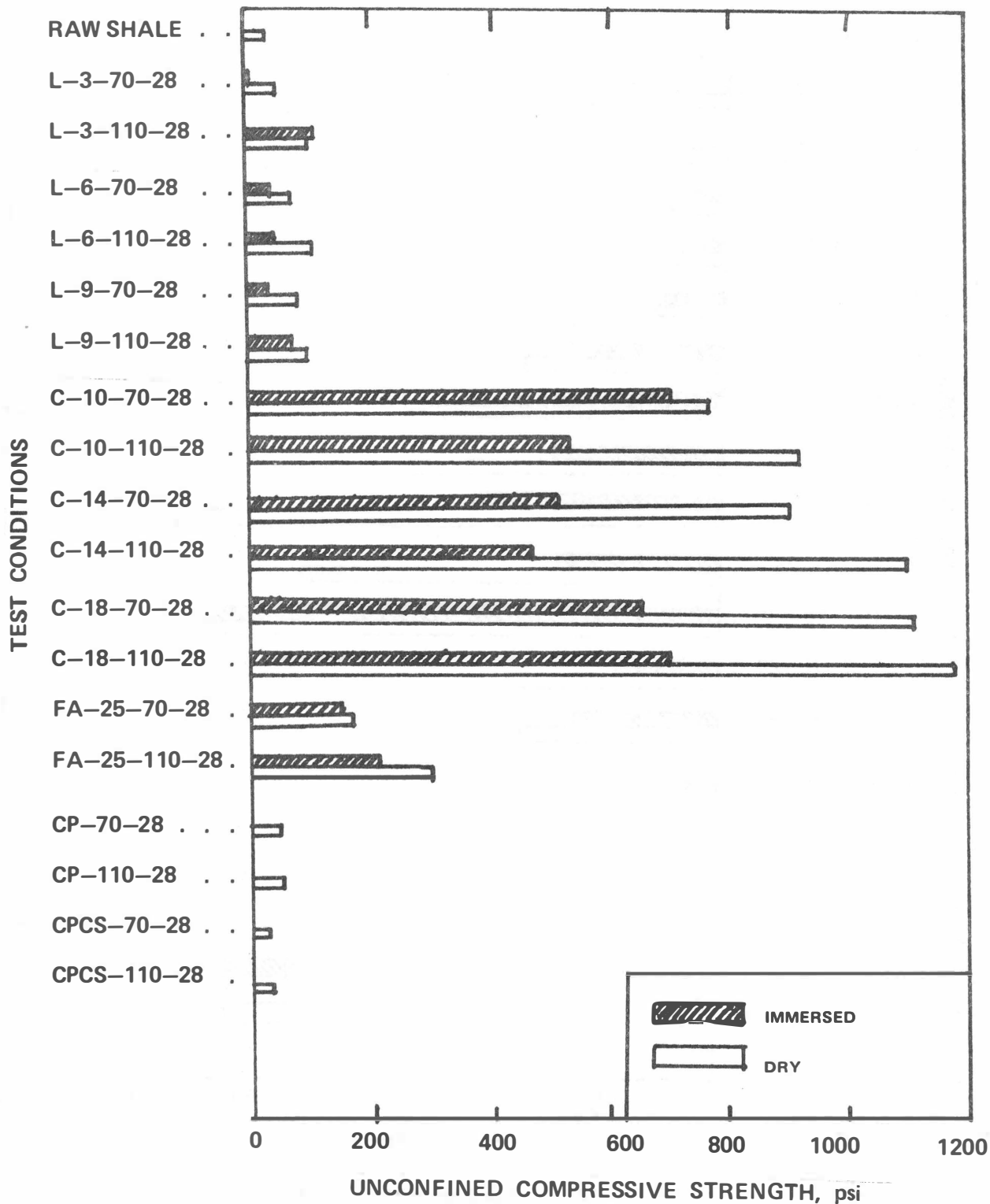


Figure J-15: Comparison of strength characteristics for shale 30.

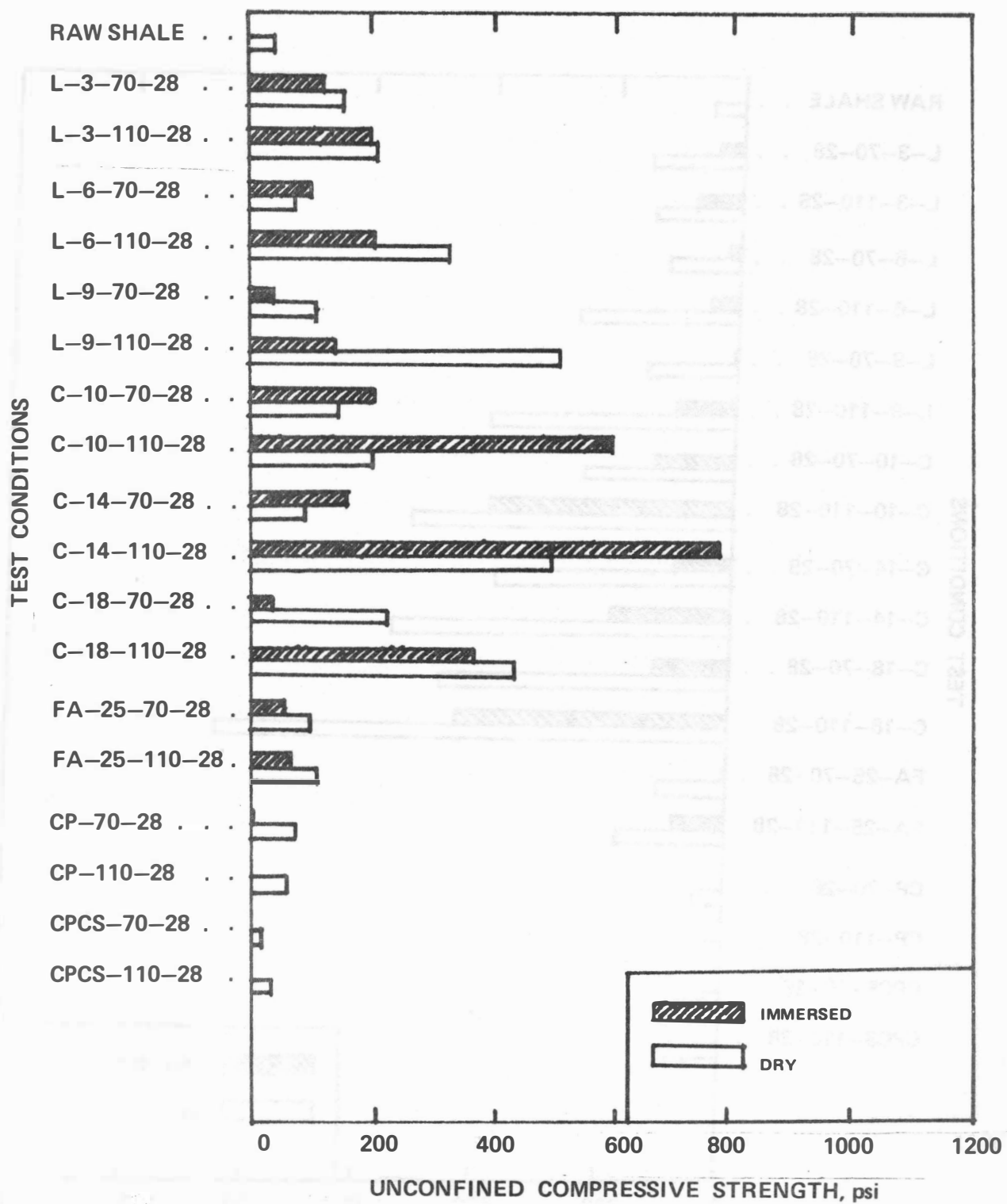


Figure J-16: Comparison of strength characteristics for shale 31.

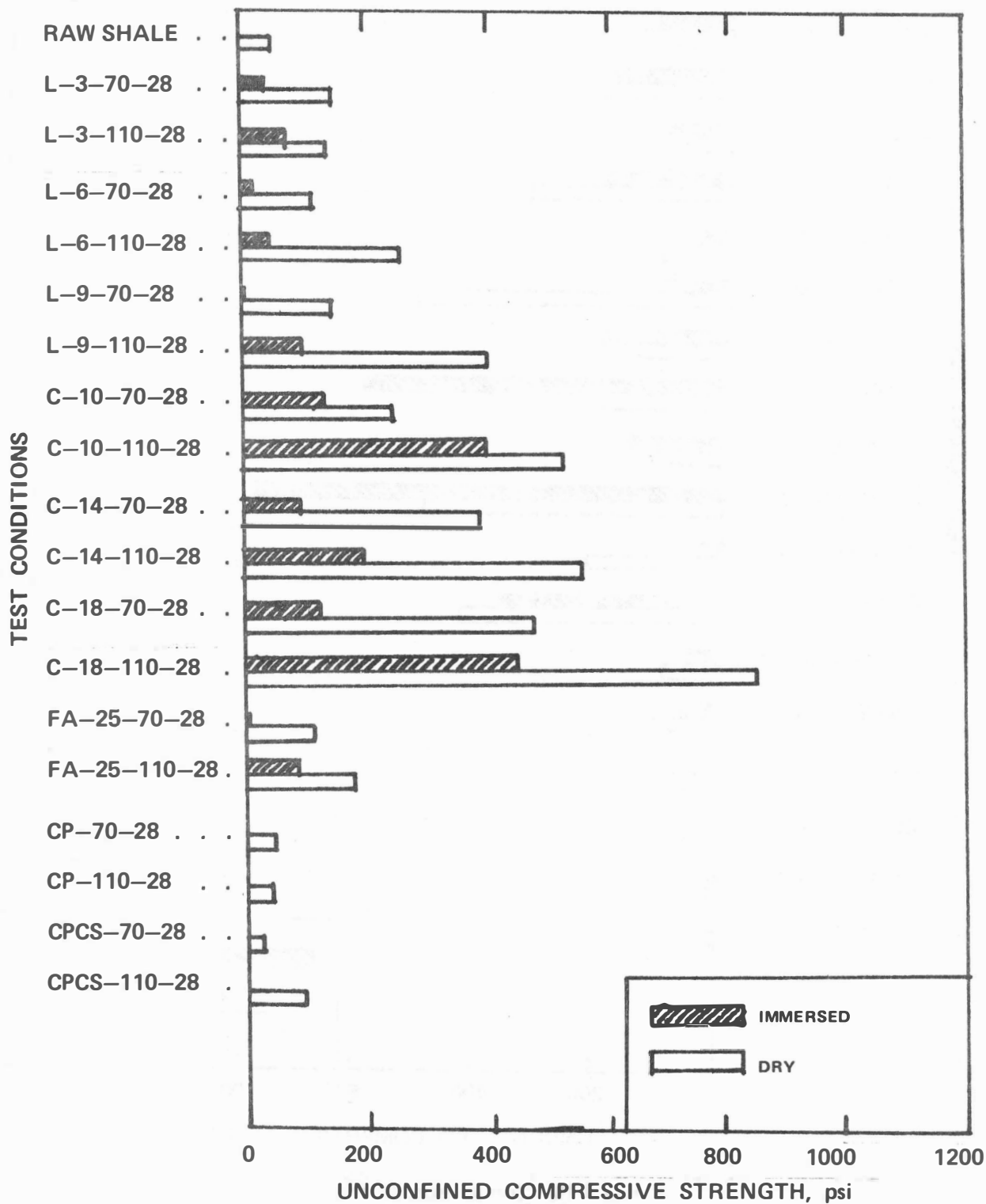


Figure J-17: Comparison of strength characteristics for shale 32.

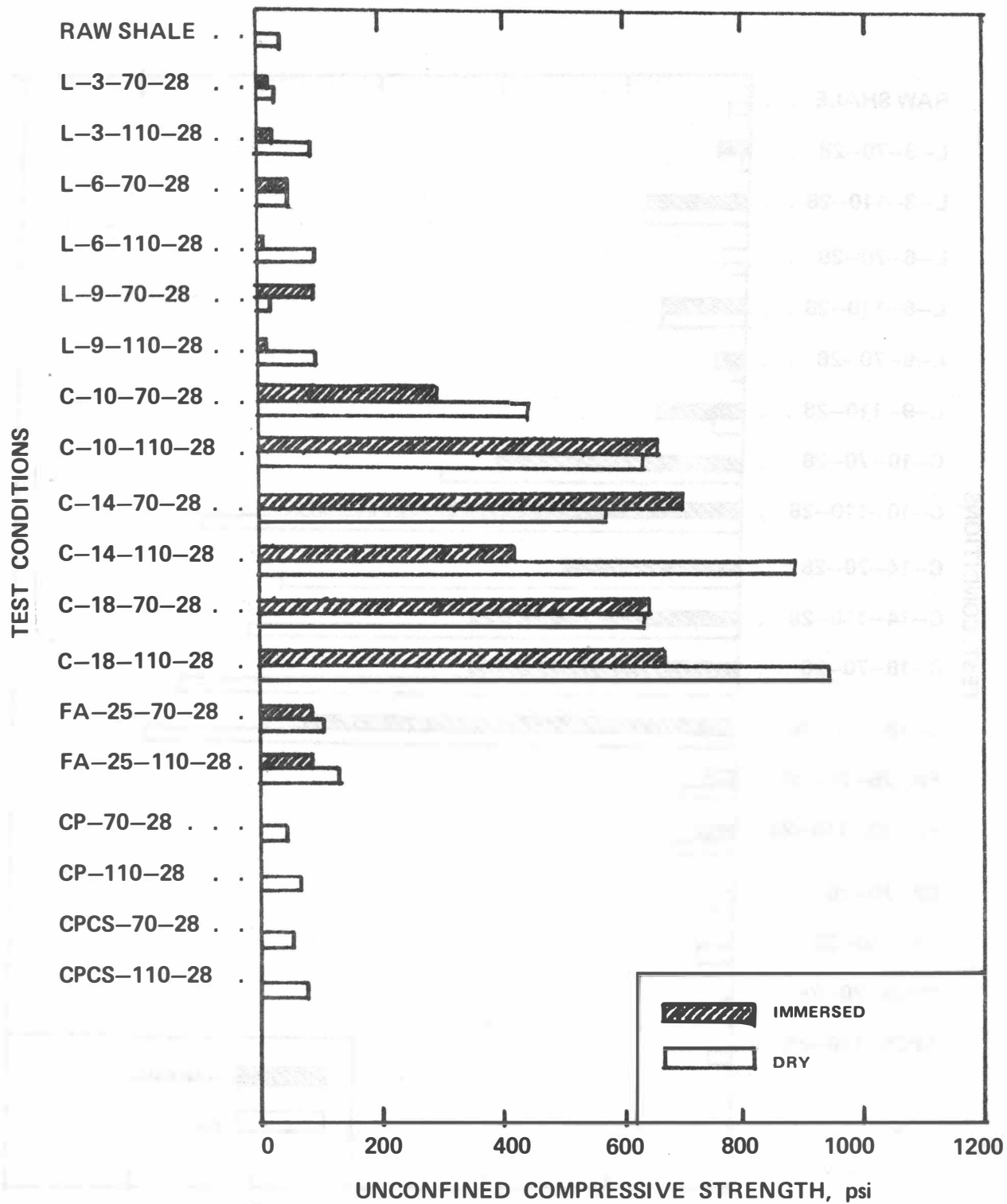


Figure J-18: Comparison of strength characteristics for shale 33.

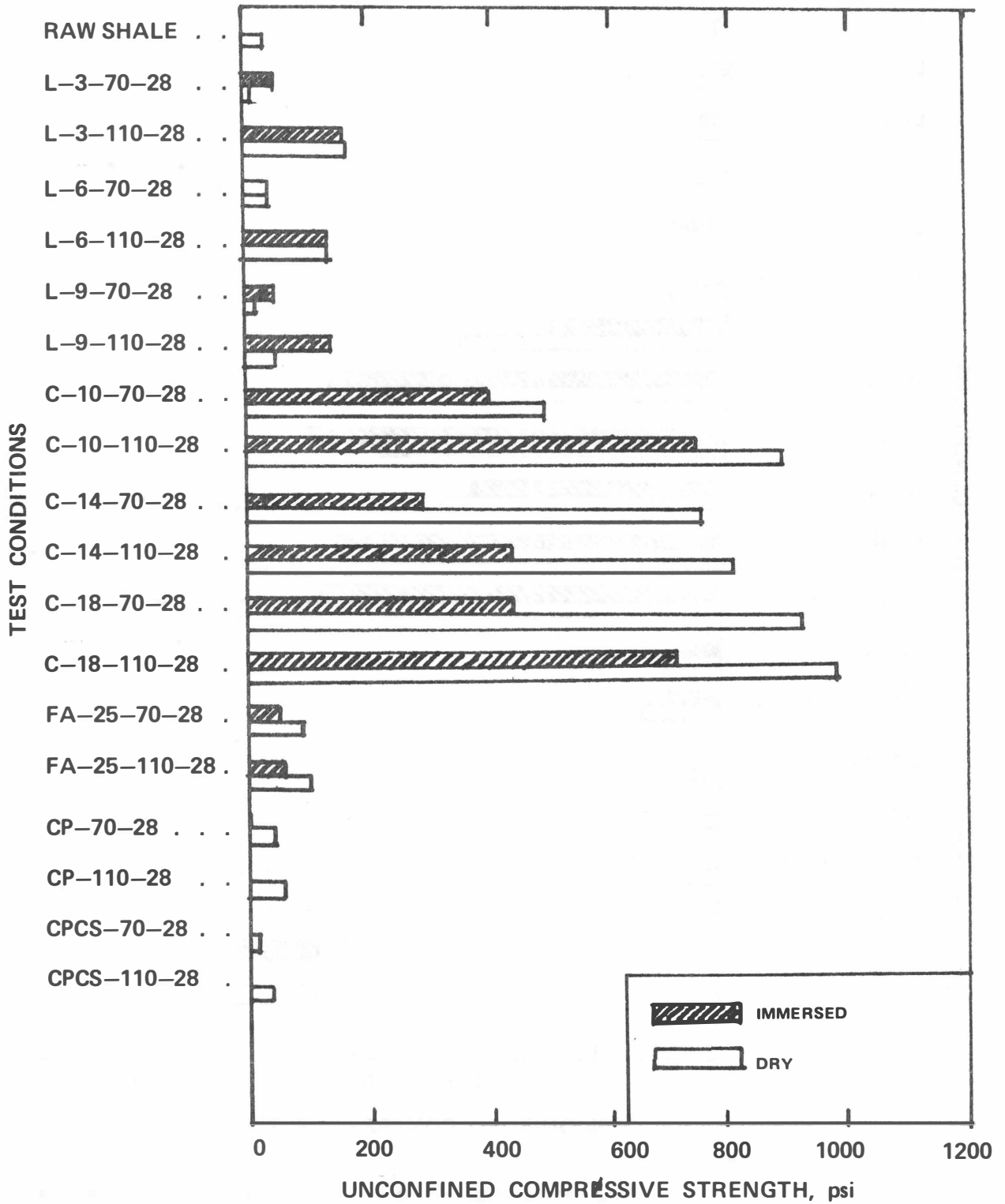


Figure J-19: Comparison of strength characteristics for shale 34.

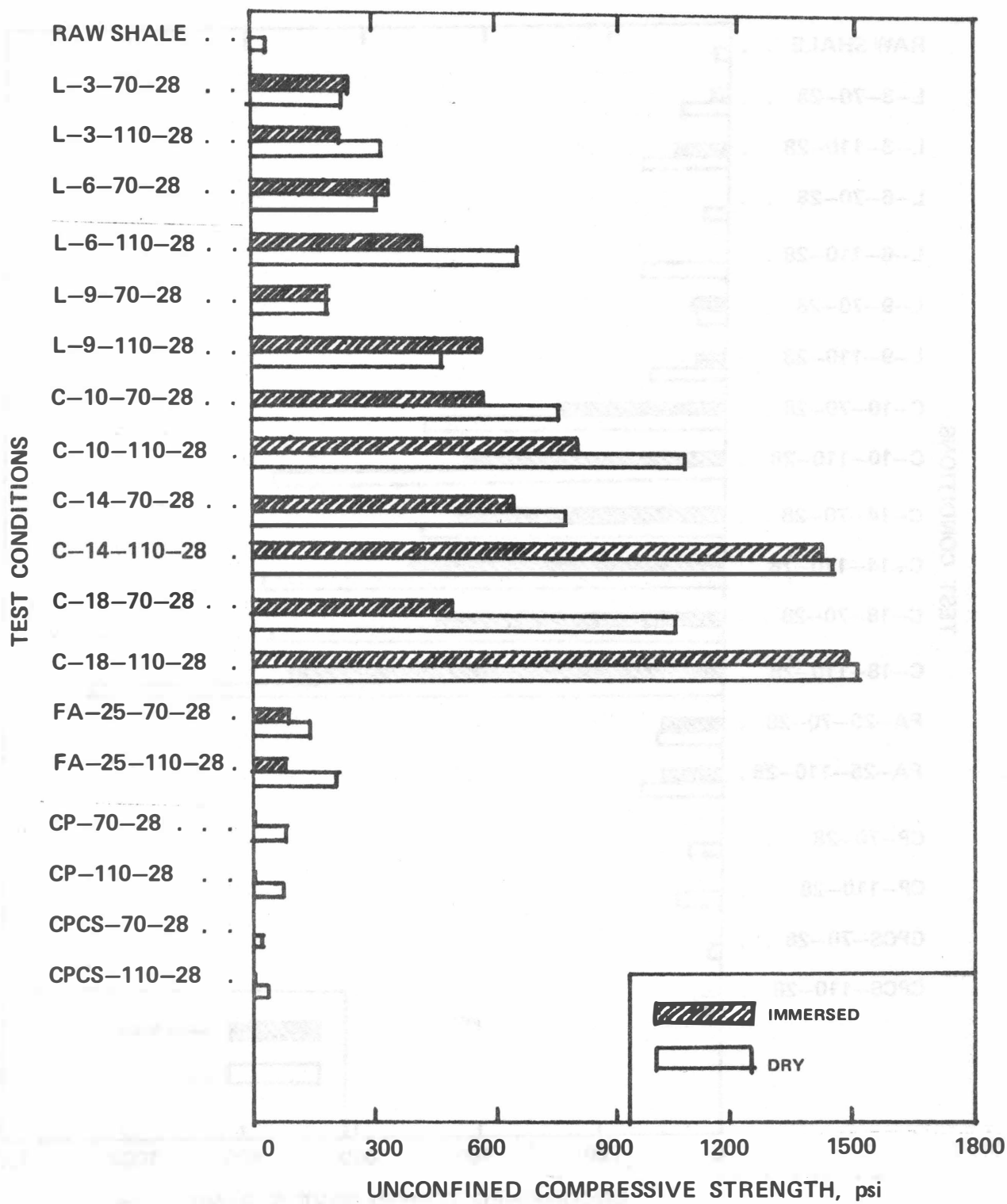


Figure J-20: Comparison of strength characteristics for shale 35.

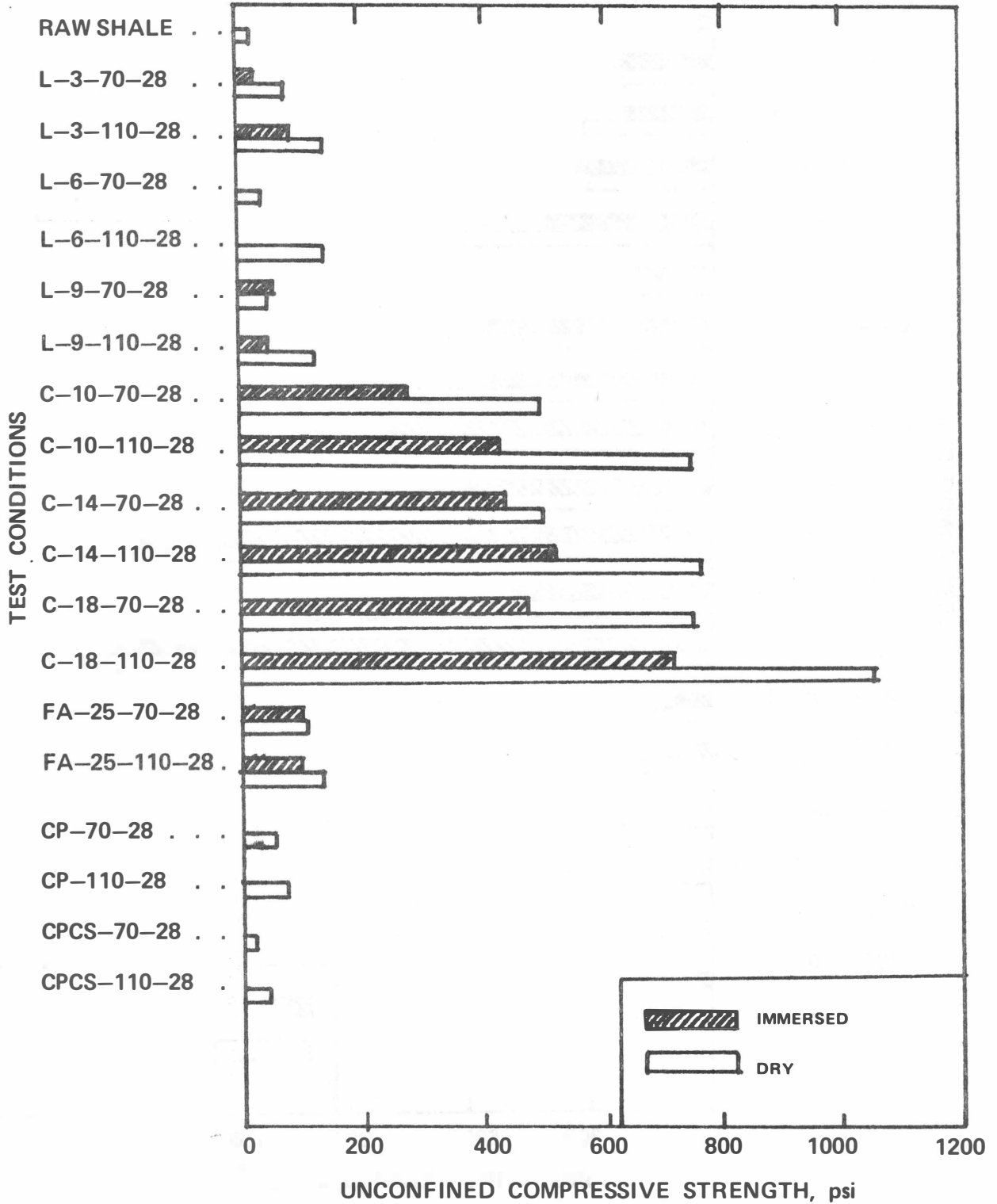


Figure J-21: Comparison of strength characteristics for shale 36.