

72-4921

LIN, Yen Kuang, 1939-
COMPRESSIBILITY, STRENGTH AND FROST
SUSCEPTIBILITY OF COMPACTED FLY ASH.

The University of Michigan, Ph.D., 1971
Engineering, civil

University Microfilms, A XEROX Company, Ann Arbor, Michigan

COMPRESSIBILITY, STRENGTH AND FROST SUSCEPTIBILITY
OF COMPACTED FLY ASH

by

Yen Kuang Lin

A dissertation submitted in partial fulfillment
of the requirements for the degree of
Doctor of Philosophy
(Civil Engineering)
in The University of Michigan
1971

Doctoral Committee:

Associate Professor Donald H. Gray, Chairman
Professor Frank E. Richart, Jr.
Associate Professor Egons Tons
Associate Professor Alan S. Wineman

PLEASE NOTE:

**Some Pages have indistinct
print. Filmed as received.**

UNIVERSITY MICROFILMS

To
My Mother
and
My Wife

ACKNOWLEDGMENTS

The author is deeply indebt to Professor D. H. Gray, Chairman of the Doctoral Committee, for his guidance and encouragement throughout. The author also wishes to extend his appreciation to the other committee members for their interest in this research. Review and suggestions on Sections 5.4 and 5.5 of this dissertation given by Professor A.S. Wineman are specially invaluable.

Special thanks are due to a number of individuals in the soils group, Department of Civil Engineering. The constant support and advice given by Professors F. E. Richart and R. D. Woods are much appreciated. The continuous encouragement and invaluable assistance from the author's colleagues, S. S. Afifi, R. P. Brenner and K. H. Stokoe, are sincerely acknowledged.

This research was made possible by Research Grant No. R01-EC-00317-01 from the U. S. Public Health Service. Fly ash samples were made available through arrangements of Mr. D. H. Brandt of Consumers Power Company and Mr. F. V. Zimmer of the Detroit Edison Company.

The author is also grateful for the financial assistance he received during his study from the University of Michigan.

The author greatly appreciates all the help and encouragement given by his wife, Huei-mei (Amy), whose support and assistance have been indispensable during his study at the University.

CONTENTS

LIST OF TABLES	vi
LIST OF FIGURES	viii
LIST OF SYMBOLS	xii
LOSSARY OF TERMS	xiv
 CHAPTER 1	
INTRODUCTION	1
CHAPTER 2	
REVIEW OF PAST WORK	10
CHAPTER 3	
INDEX PROPERTIES AND STRENGTH OF COMPACTED FLY ASH	15
3.1	
Introduction	15
3.2	
Variables which Affect the Strength Development of Compacted Fly Ash . . .	18
1. Material Variables	18
2. Environmental Variables	22
3.3	
Index Properties and Composition of Typical Michigan Fly Ashes	26
1. Source	26
2. Composition	27
3. Leachate Characteristics	28
4. Index Properties	28
5. Permeability	32
3.4	
Strength of Compacted Fly Ashes . . .	34
1. Compaction-Strength Characteristics	34
(1) Method of Compaction	34
(2) Compaction Characteristics and Unconfined Compressive Strength	35
(3) Shear Strength Parameters . . .	41
2. Effect of Lime Stabilization on the Strength of Compacted Fly Ash . . .	43
(1) Sample Preparation and Testing.	43
(2) Results	45
3. Effect of Curing Temperature on the Strength of Compacted Fly Ash . . .	56

CHAPTER 4	FROST SUSCEPTIBILITY OF COMPACTED FLY ASH	63
4.1	Summary of Frost Heave Mechanism . .	64
4.2	Physical Effects of Stabilization on Frost Heave	67
4.3	Test Procedure and Results	70
	1. Frost Heave Cabinet	70
	2. Sample Preparation and Test. Pro- cedure	71
	3. Results	74
	4. Conclusions	74
CHAPTER 5	A RHEOLOGICAL STUDY OF THE COMPRESSI- BILITY OF COMPACTED FLY ASH	80
5.1	Introduction	80
5.2	Thixotropy, Creep and Aging Processes	84
5.3	Experimental Investigations on the Effect of Aging on the Compressibility of Compacted Fly Ash	87
	1. Laboratory Set-Up	90
	2. Sample Preparation	93
	3. Test Procedure	93
	4. Discussion of Results	94
5.4	Viscoelastic Stress-Strain Law for Aging under One-Dimensional Strain Condition	106
5.5	Use of Viscoelastic Stress-Strain Law in the Construction of the Void Ratio vs. Effective Pressure Curve	112
5.6	Compression of Partially Saturated, Compacted Soils	124
5.7	Effect of Partial Saturation on the Compressibility of Compacted Fly Ashes	129
	1. Laboratory Set-Up	130
	2. Sample Preparation	133
	3. Test Program and Test Procedure .	133
	4. Discussion of Results	136
CHAPTER 6	SUMMARY AND CONCLUSIONS	145
	REFERENCES	151

LIST OF TABLES

<u>Table</u>	<u>Page</u>
1.1 Comparative Ash Utilization in the USA, 1966 Through 1969 (from Brackett, 1970)	3
1.2 Fly Ash Collection and Utilization in the USA, 1969 (from Brackett, 1970)	4
3.1 Unconfined Compression Strengths of Cement and Lime Stabilized P.F. Ashes (from Sutherland et al., 1968)	24
3.2 Ratios of Strength of 10 percent Stabilized Cement and Lime P.F. Ashes at Different Times (from Sutherland, et al., 1968)	24
3.3 Chemical Analysis of Fly Ash Solids from Some Coal-Fired Power Stations in Michigan	29
3.4 Partial Chemical Analysis of Leachate from Fly Ashes Obtained from Some Coal Fired Power Stations in Michigan (from Gray and Lin, 1971).	29
3.5 Index Properties of three Michigan Fly Ashes from Detroit Edison Company	30
3.6 Chemical and Physical Analysis of KEMIKAL Hydrate Lime	44
3.7 Unconfined Compression Strengths of four Michigan Fly Ashes with Various Lime Treatment after Various Periods of Moist Curing	46
3.8 Shear Strength Parameters of three Compacted Fly Ashes from Detroit Edison Company	47
3.9 Effect of Curing Temperature on the Compressive Strength of Treated and Untreated Fly Ashes from Trenton Channel Plant and Marysville Plant	58
3.10 Comparison of the Effect of the Strength of Lime Ash Stabilized Sands of Curing at a Constant Temperature of 20°C and Curing in a Temperature Cycle of 10°C and 30°C (from Sherwood and Ryley, 1966)	61

<u>Table</u>		<u>Page</u>
5.1	Calculations of the Void Ratio-Effective Pressure Relation by Using Creep Functions, Trenton Channel Fly Ash	121
5.2	Calculations of the Void Ratio-Effective Pressure Relation by Using Creep Functions, Marysville Fly Ash	122

LIST OF FIGURES

<u>Figures</u>	<u>Page</u>
1.1 Commercial Utilization of Ash from Pulverized Fired Plants, Central Electricity Generating Board, Britain, 1964-1969 (from Wilson and Barber, 1970)	6
3.1 Grain Size Distribution of Some Typical Michigan Fly Ashes	31
3.2 Permeability of Trenton Channel Fly Ash with Different Dry Densities (from Gray and Lin, 1971)	33
3.3 Miniature Impact Compactor and Modified Miniature Mold	36
3.4 Compaction Characteristics and Compressive Strength of Fly Ash from D.E. Karn Plant . . .	37
3.5 Compaction Characteristics and Compressive Strength of Fly Ash from Marysville Plant . . .	38
3.6 Compaction Characteristics and Compressive Strength of Fly Ash from St. Clair Plant . . .	39
3.7 Compaction Characteristics and Compressive Strength of Fly Ash from Trenton Channel Plant.	40
3.8 Relation between Maximum Dry Densities and Optimum Water Contents of Four Michigan Fly Ashes	42
3.9 Penetration Resistance of Compacted Fly Ash from Trenton Channel Plant	42
3.10 Effect of Lime Stabilization on the Compressive Strength of Fly Ash from D.E. Karn Plant (Samples Cured at 20°C)	48
3.11 Effect of Lime Stabilization on the Compressive Strength of Fly Ash from Marysville Plant (Samples Cured at 20°C)	49

<u>Figures</u>	<u>Page</u>
3.12 Effect of Lime Stabilization on the Compressive Strength of Fly Ash from St. Clair Plant (Samples Cured at 20°C)	50
3.13 Effect of Lime Stabilization on the Compressive Strength of Fly Ash from Trenton Channel Plant (Samples Cured at 20°C)	51
3.14 Effect of Lime Content on the Compressive Strength of four Michigan Fly Ashes after 28 Days of Curing (Samples Cured at 20°C)	53
3.15 The Failure Plane in a Compacted Sample Stabilized with Optimum Lime Content and Moist Cured at 20°C for 28 days. Fly Ash from Trenton Channel Plant	54
3.16 Effect of Lime Stabilization on the Penetration Resistance of Fly Ash from Trenton Channel Plant	55
3.17 Effect of Curing Temperature on the Compressive Strength of Treated and Untreated Fly Ash from Trenton Channel Plant	59
3.18 Effect of Curing Temperature on the Compressive Strength of Treated and Untreated Fly Ash from Marysville Plant	60
4.1 Curvature of Ice-water Interface	65
4.2 Effect of Lime and Cement Additions on the Permeability of Some Compacted, British Fly Ashes (from Sutherland and Gaskin, 1970) . . .	69
4.3 Schematic Drawing of the Set-up for Frost Heave Tests	72
4.4 Sample Installation in the Freezing Cabinet . .	73
4.5 Effects of Various Additions of Lime and Cement on the Frost Heave of Fly Ash from Trenton Channel Plant	75
4.6 Effects of Various Additions of Lime and Cement on the Frost Heave of Fly Ash from Marysville Plant	76
4.7 The Variation in Frost Heave of four Fly Ash Samples from Trenton Channel Plant in Response to Different Stabilization Treatments	77

<u>Figures</u>	<u>Page</u>
4.8 The Formation of Ice Lense in an Untreated Sample of Trenton Channel Fly Ash after 10 days of Frost Heave Test	78
5.1 The Comparison of Settlements Both on Road Surface and Beneath the Embankments of a Conventional Earthfill and a Compacted Fly Ash fill, Railroad Bridge Overpass, Staines, England (from Margason and Cross,1966)	81
5.2 The Aging and Non-aging Stress-Strain Behavior of Soils Described by the Burgers Rheological Model	88
5.3 Three-Dimensional Sketch of Strain-Loading Time-Aging Relation of An Aging Material . . .	88
5.4 Difference in Response of An Aging Material to Step Loads at Different Stages of Material Aging	89
5.5 Schematic Diagram of Laboratory Set-up for Consolidation Tests on Saturated Compacted Fly Ashes	91
5.6 Laboratory Set-up for Consolidation Tests on Saturated, Compacted Fly Ashes	92
5.7 The Deformation-Time Relationship of Compacted Fly Ash from Trenton Channel Plant	95
5.8 The Deformation-Time Relationship of Compacted Fly Ash from Marysville Plant	96
5.9 Time-Strain Curves for Lime Treated Trenton Channel and Marysville Fly Ashes	100
5.10 Effects of Lime Treatment and Loading-Time Interval on the Compressibility of Compacted Trenton Channel Fly Ash	101
5.11 Effects of Lime Treatment and Loading-Time Interval on the Compressibility of Compacted Marysville Fly Ash	102
5.12 Effects of Lime Treatment and Loading-Time Interval on the Compressibility of Compacted Trenton Channel Fly Ash. Void Ratios have Been Normalized for Comparison	103

<u>Figures</u>	<u>Page</u>
5.13 Effect of Lime Treatment and Loading-Time Interval on the Compressibility of Compacted Marysville Fly Ash. Void Ratios have Been Normalized for Comparison	104
5.14 Schematic Diagrams of the Loading Pattern Conventionally Used in the Consolidation Test.	114
5.15 Creep Functions of Lime Treated Trenton Channel Fly Ash with Different Aging before Tests . . .	118
5.16 Creep Functions of Lime Treated Marysville Fly Ash with Different Aging before Tests . . .	119
5.17 Effect of Previous Curing on The Magnitude of Creep Function of Lime-Treated Fly Ashes. Curves Extrapolated from Figures 5.15 and 5.16	120
5.18 Comparisons of Void Ratio vs. Compression Stress Curves between Those from Conventinnal Tests and Those Calculated From Creep Functions	123
5.19 Strain Gage Circuits for The Lateral Strain Detector	132
5.20 Schematic Diagram of Laboratory Set-Up for Compression Tests on Partially Saturated, Compacted Fly Ashes	134
5.21 Laboratory Sep-Up for Compression Tests on Partially Saturated, Compacted Fly Ashes . . .	135
5.22 Comparison of the Compressibility Behavior of Saturated and Partially Saturated, Compacted Fly Ashes from Trenton Channel Plant. Void Ratios Have Been Normalized	137
5.23 Comparison of the Compressibility Behavior of Saturated and Partially Saturated, Compacted Fly Ashes from Marysville Plant. Void Ratios Have Been Normalized	138
5.24 Effects of Lime Treatment and Loading-Time Interval on The Compressibility of Compacted Fly Ashes. Void Ratios Have Been Normalized .	140
5.25 Effect of Suction on The Volume Change Behavior of Compacted Fly Ash from Trenton Channel Plant	142
5.26 Effect of Suction on The Volume Change Behavior of Compacted Fly Ash from Marysville Plant . .	143

LIST OF SYMBOLS

C_v	: coefficient of consolidation
e	: void ratio
H	: height; the maximum distance of the pore flow path in soil
J	: creep function
J_1	: deviatoric creep function
J_2	: octahedral creep function
J_e	: equivalent creep function for one-dimensional strain condition
K_0	: coefficient of earth pressure at rest
k_1, k_2	: spring constants
L_h	: latent heat of fusion
r	: radius of curved surface of an ice-water interface
s_{ij}	: deviatoric stress components
T	: absolute temperature
ΔT	: freezing temperature depression
T_c	: time factor (in consolidation theory)
t	: time; current time
t_c	: creation time of material (or the time when the specimen is made)
t_L	: time when load is applied
u_a	: pore air pressure
u_w	: pore water pressure
ΔV_m	: difference in partial molar volume between ice and water

w	: water content
γ_d	: dry density
δ_{ij}	: Kronecker delta
ϵ	: strain
ϵ_{kk}	: octahedral strain components
ϵ_{ij}	: deviatoric strain components
η_1, η_2	: viscosities
ν	: Poisson's ratio
π	: equivalent osmotic pressure
ρ_i	: density of ice
σ	: stress (total stress)
σ'	: effective stress
σ_{iw}	: ice-water interfacial energy
σ_{kk}	: octahedral stress components
τ	: time variable
$\phi(x,t)$	: the environmental parameter that influence the mechanical response of a material
ϕ_f	: angle of internal friction
χ	: soil saturation parameter

GLOSSARY OF TERMS

- cenosphere - thin-walled hollow sphere in fly ash
- creep function - a function which represents the time-dependent response to a unit stress
- curing - storage of sample under specified temperature and moisture conditions to improve its strength and durability
- dry density - the amount of solids in a unit volume of soil:
also called dry unit weight
- optimum lime content - the amount of lime required in a fly ash to reach the highest strength under a specified curing condition
- optimum water content - the water content at which the maximum dry density of soil can be achieved by a specific compactive effort
- pozzolanic activity - a chemical reaction with which calcium hydroxide reacts chemically at ordinary temperature with any natural or artificial siliceous or aluminous material in the presence of water form compounds possessing cementing properties
- soil stabilization - the alteration of property of a soil to improve its engineering performance. This includes mechanical blending and chemical admixtures
- specific surface - surface area per unit volume of a particle
- void ratio - the ratio of the volume of voids in a soil to the volume of soil solids

CHAPTER 1

INTRODUCTION

The development of pulverized-coal-fired boilers and the advancements in combustion technique have been a great contribution to the world economy by providing electric power at low costs. During the same time, however, massive amounts of fly ash have been produced and the annual output continues to grow at a substantial rate. As a result, the disposal of fly ash has become a critical problem in many areas, and the public is also becoming increasingly conscious of the fly ash nuisance.

During 1969 the electric utility industry in the United States produced over 22 million tons of fly ash. Output will continue to increase, and by 1975 it is estimated by Brackett (1970) that the production rate will be about 29 million tons per year, and it could result in an annual tonnage of 36 million tons by 1980 in spite of the fact that an increasing percentage of power is expected to be produced by nuclear power plants. Furthermore, the output figures cited above do not include the bottom ash and boiler slag output which amounts to 11 million tons per year currently and is estimated to reach an annual production of 16 million tons by 1980.

The location of fly ash production is also of interest. Most of the output is concentrated in the Great Lakes region. The states of Illinois, Indiana, Michigan, Ohio, and Wisconsin contribute more than one third of the national fly ash production.

Because of the diversity in its physical and chemical properties and a lack of basic knowledge relating to this material, the utilization of fly ash has been quite limited. Consequently, fly ash has been usually considered as an unwanted waste byproduct. The most common method of disposal has been "dumping," and this will tend to be the traditional method as long as the cost of dumping is less than that required to develop a market. However, disposal sites have become scarcer and more costly, particularly in the large urban or metropolitan areas. As production rates continue to increase and as environmental controls on dumping of residuals become more stringent, fly ash disposal will be a major problem for coal-fired power plant operation in the immediate future.

In view of this problem, some attention has been devoted to the development of fly ash utilization in recent years. A limited effort has been made in the United States in this direction, however, the percent utilization is quite low compared to most European countries. Britain and France, for example, use 42 and 55 percent respectively of their output as opposed to only 9 percent in the United States. Also, in terms of total tonnage British and Germany each use more than does the United States. In addition, from the figures shown in

Table 1.1, it can be noted that the rate of fly ash utilization in this country has not increased in the period of 1968 through 1969 while the output increased 12.6 percent. It is even more disturbing to note that the total percentage of ash utilization (including bottom ash and boiler slag) has dropped from 17.5 percent in 1968 to 15.9 percent in 1969.

TABLE 1.1 - Comparative Ash Utilization in the USA, 1966 through 1969, Tons

	1966	1967	1968	1969
Produced:				
Fly ash	17,123,144	18,409,854	19,813,747	22,304,513
Bottom ash	8,065,683	9,131,453	7,259,212	8,042,017
Boiler slag	-	-	2,554,569	3,020,282
Total	25,188,827	27,541,307	29,627,528	33,366,812
Utilized	3,050,669	3,794,714	5,194,016	5,306,764
Percent ash Utilized:				
Fly ash	7.9	8.2	9.6	9.4
Bottom ash	21.0	25.0	25.0	27.3
Boiler slag	-	-	57.8	33.7

(from Brackett, 1970)

Major uses for fly ash at present in the United States are as follows:

- (1) Lightweight sintered aggregate.
- (2) Soil stabilizer for road and highway subbases.
- (3) Cement replacement in mass concrete.
- (4) Cement replacement in concrete products.
- (5) Mineral filler in asphalt paving.
- (6) Constituent in Portland cement.
- (7) Grouting agent in oil well cementing.

The ash collection and utilization in 1969 are shown in Table 1.2. More detailed information on the above uses has been

TABLE 1.2 - Fly Ash Collection and Utilization
in the USA, 1969

	Fly ash, tons	Bottom ash, tons	Boiler slag (if separated from bot- tom ash), tons
1. Ash collected	22,304,513	8,042,017	3,020,282
2. Ash utilized:			
a. Mixed with cement clinker or cement (pozzolan cement)	32,759	-	-
b. Mixed with raw material before forming cement clinker	132,226	5,000	-
c. Partial replacement of cement in:			
1. Concrete pro- ducts	175,086	5,686	-
2. Structural con- crete	194,322	-	-
3. Dams and other mass concrete	163,809	-	-
d. Stabilizer for road bases, parking areas, etc.	264,224	20,617	73,240
e. Lightweight aggre- gate	250,251	19,388	-
f. Fill material for roads, construction sites, etc.	73,440	1,102,240	548,606
g. Filler in asphalt mix	114,067	10,084	89,970
h. Miscellaneous (see table 2)	244,508	648,052	257,204
Total	1,644,692	1,811,067	969,020
3. Ash removed from plant sites at no cost to utility but not cover- ed in categories list- ed under "Ash Util- ized"	452,522	381,076	48,387
Total Utilized	2,097,214	2,192,143	1,017,407

(From Brackett, 1970)

presented in recent symposia on ash utilization (1967,1970).

The use of compacted, stabilized fly ash by itself in load-bearing fills has received very little attention in this country. This is largely because potential users lack knowledge and experience on the one hand and the producers are discouraged by some possible handling problems on the other. However, problems of the latter type either are exaggerated or have been solved by European ash producers. For example, the dust problem mentioned by Zimmer (1967) can be controlled. Fly ash used as fill is usually shipped in a damp condition, preferably at or near the optimum moisture content. Therefore, it is generally dust free and can be transported in open or sheeted trucks. Of course, this requires the ash producer to install a special "conditioning" plant. The problems of the former type can be overcome by capitalizing on knowledge and experience gained from countries with an active ash utilization research program such as Britain. Figure 1.1 shows the status of ash utilization in Britain. It should be noted that more than 50 percent of utilized fly ash was used as load-bearing fill material. This should not be considered as merely accidental but as a clear indication of its successful utilization in engineering practice.

The advantages of using compacted fly ash as a replacement for conventional earthfill material in load-bearing fills are as follows:

- (1) It solves two problems with one effort, viz., elimination of a solid waste problem on the one hand and

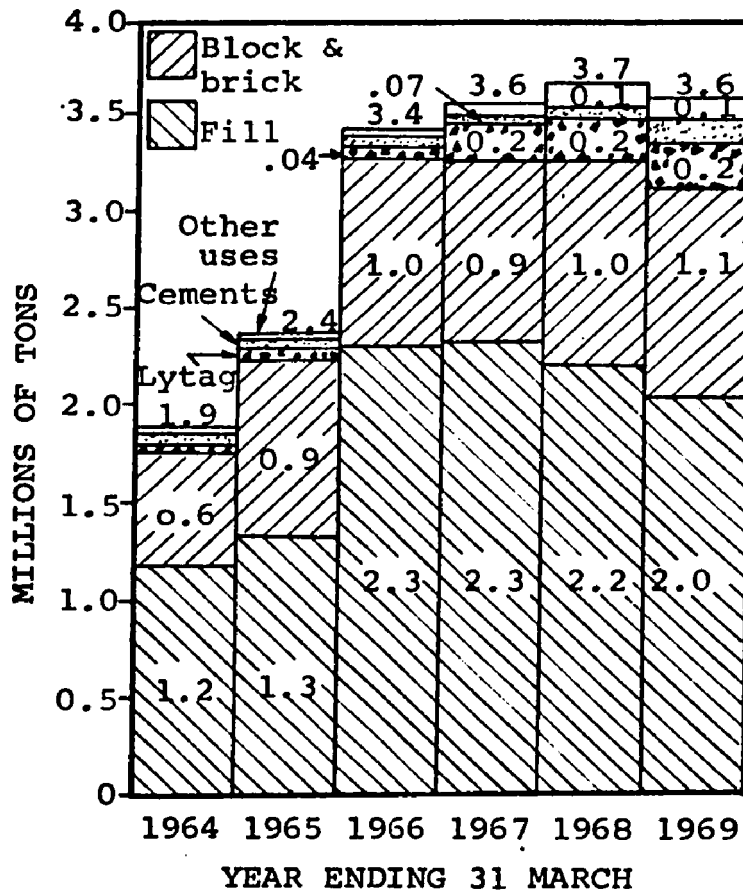


Figure 1.1 Commercial utilization of ash from pulverized fired plants, Central Generating Board. (from Wilson & Barber 1970)

provision of a needed construction material on the other. Both these two problems are becoming acute in the urban environment because it is here where most coal-fired generating plants are located and where the supply of natural earthfill material is scarce.

(2) Properly compacted and stabilized fly ash generally possesses strength comparable to that of conventional, compacted earthfill. Furthermore, because of its age hardening property, compacted, stabilized fly ash will exhibit a cohesion not found in normal soils and develop additional strength with time.

(3) The compacted fly ash has low dry density. This is advantageous if the fill or embankment must be placed on ground of low bearing capacity or where long term settlement is possible. This is also advantageous economically because one ton of compacted fly ash represents a greater volume than one ton of conventional earthfill.

(4) Fly ash can be readily conditioned (i.e. mixed with the right amount of water and lime or cement) at the plant so as to arrive on site in a state ready for compaction. It can be easily handled and placed by conventional compaction plants.

In addition, many of the bituminous coals that are produced throughout the world, and particularly in the United States, are relatively high in sulfur content. When these coals are burned in modern furnaces the amount of SO_2 that is released has created considerable concern about pollution of the atmosphere. The elimination of the SO_2 emission during the combustion process will become an important issue. Among a few trial processes currently being developed, Minnick (1970) suggested that the introduction of limestone or dolomite into the furnace may be an effective way of dealing with the problem. Based on the results of additive treatments available at the present time, there may be more than a stoichiometric requirement of lime needed to react with the sulfur gases. If this is so, fly ash will contain more than 50 percent by weight of lime compounds in the final mix, and the physical and chemical properties of these new ashes will differ substantially from

those of fly ashes produced at present. Minnick also pointed out from the studies made to date, that new fly ashes seem to have limited application in concrete, lightweight aggregate and ceramic brick. On the other hand, because a high lime content assists in producing self-hardening reactions in compacted fly ash, the use of this new fly ash as a structural fill may become increasingly attractive.

The main objective of this dissertation is to study the engineering properties of compacted fly ash as a material for load-bearing fills and subbases. This includes the study of index properties, strength, and compressibility behavior. Fly ash may be liable to frost heave because of its grain size distribution. It is important, therefore, to include a study of the frost heave characteristics of some typical local fly ashes in this dissertation because frost heave is one of the major concerns of soil engineers in regard to its use as a fill material.

Accordingly, this study can be divided into three parts: 1) index properties and strength, 2) compressibility, and 3) frost susceptibility. Each of these topics is described separately in three chapters. Chapter 3 contains information on the index properties and strength of some compacted Michigan fly ashes. A general discussion of the possible variables that affect the age hardening of fly ash is given, and the effects of lime stabilization and curing temperature on strength of fly ashes are also discussed. The frost susceptibility of some Michigan fly ashes is treated in Chapter 4. The effects

of cement and lime stabilizations on the frost heaving of these ashes are also described. Chapter 5 deals with the compressibility behavior of compacted fly ash. The load-deformation-time relationship of this material is studied theoretically and experimentally by taking into account the age hardening property. The effect of partial saturation on the compressibility of compacted fly ashes is also discussed in this chapter.

The scope of this study has been confined to laboratory investigations; no attempt has been made to carry out a field study. However, the importance of field trials should be emphasized and the conclusions drawn from laboratory investigations should be validated by a series of field trials.

CHAPTER 2

REVIEW OF PAST WORK

Fly ash investigations, particularly in this country, have focused primarily on chemical reactivity and strength development. Leonard and Davidson (1959) and Minnick (1967) attempted to assess the strength development in compacted ash samples in terms of their pozzolanic activity. Leonard and Davidson studied the pozzolanic reaction in terms of the rate of calcium absorption which was in turn determined by using a conductivity method. Minnick analyzed the reaction from X-ray diffraction and the differential thermal analysis results, from which the reactants and the reaction products were identified.

Soil engineers, on the other hand, are more interested in strength of fly ash than the pozzolanic reaction itself. Some investigators, e.g., Snyder (1967), believe that strength development is not a simple function of the extent of pozzolanic reaction. The long-term strength of fly ash cannot be determined solely by measuring the extent to which the fly ashes react with calcium hydroxide. Leonard and Davidson also recognized that there is a relationship between the absorption of lime and the compressive strength, but the total strength development of lime-pozzolan mortars is dependent on other

factors in addition to the pozzolanic reaction itself. Davidson et al. (1958), Vincent et al. (1961), Mateos (1964), Raymond (1965), Thorne and Watt (1965), Sherwood and Ryley (1966), Sutherland and Finlay (1967), and Sutherland et al. (1968) have all attempted to find statistically significant relationships between strength and compositional variables. Unfortunately, no general relationship that can serve as a guide for practicing engineers to assess the strength of this material has been found. In spite of the diverse nature of fly ash and the difficulty of establishing a unique relationship between fly ash properties and strength, the consensus of these studies is that many ashes, being pozzolanic materials, acquire a considerable amount of strength with time after compaction. It appears from these studies that compacted fly ash is a satisfactory structural fill material as far as its strength is concerned.

A considerable amount of information regarding strength and stabilization of compacted fly ash was also presented in the proceedings of two symposia on fly ash utilization (1967, 1970). A good summary of much published literature in this area was given in the fly ash stabilization handbook published by the British Central Electricity Generating Board (1967).

In the United States, fly ash is used more often as an admixture in base course stabilization and pozzolanic pavements than it is by itself in load-bearing fills. Examples of the former include many roadway projects in Alabama (Hester, 1967), pozzolanic pavements in the Chicago area (Savage, 1970), and

the runway construction at the Newark Airport, New Jersey (Engineering News-Record, 1969). In all these projects, fly ash was used only as a pozzolanic additive in a lime-fly ash-soil or cement-lime-fly ash-soil mixture. The use of compacted fly ash in structural fills is relatively unknown in the U.S.A. Consequently, the published information and engineering data on this subject are quite limited in U.S. journals. A relatively good summary on some design features of fly ash fills and field compaction properties of fly ash from Pennsylvania has been presented by DiGioia and Nuzzo (1970). However, no actual construction data nor descriptions of case studies were given therein. Some preliminary test results of engineering properties of some typical Michigan fly ashes have also been presented by Gray and Lin (1971).

On the other hand, a considerable number of British technical publications in this area are available. The use of compacted fly ash as a highway embankment and structural fill material has been studied by Raymond (1961), Raymond and Smith (1966), and Margason and Cross (1966). Raymond investigated some ashes from northwestern England and western Scotland and also reported the results of two field trials. Raymond and Smith examined a fly ash fill behind an abutment. Methods for calculating active pressure on fly ash retaining structures, bearing capacity and settlement of structural fill were also presented. Margason and Cross described the construction of two bridge sites where fly ash was used as fill behind one abutment of each bridge and conventional soil fill was used

for the other abutments. They reported that settlements beneath compacted fly ash embankments were considerably less than those beneath conventional granular fills. The use of stabilized fly ash in road construction was reported by Raymond and Smith (1964), Sherwood and Ryley (1966), and Sutherland et al. (1968). Sherwood and Ryley assessed those properties of fly ash likely to affect its use as a road construction material, on the basis of laboratory investigations on nine British fly ashes. Raymond and Smith described the construction and performance of a road sub-base made from stabilized fly ash at Halewood, England. Sutherland et al. gave a description of the successful use of fine and coarse fly ashes in road construction work in Warwickshire, Britain. The successful use of fly ash as bulk fill for airport runways at Birmingham airport was also reported by Woollard (1967). Field handling techniques and emplacement procedures have been discussed by Raymond (1961) and Raymond and Smith (1964, 1966). Engineering properties of compacted fly ash have been discussed in almost all the preceding references.

In general, the use of properly compacted and stabilized fly ash in engineering practice has been quite satisfactory. The main problem encountered appears to be the lack of correlation between predicted settlements based on standard laboratory consolidation tests and actual field settlements. Application of conventional consolidation theory to oedometer test results tends to give exaggerated settlements, Raymond and Smith (1966). An adequate characterization of the com-

pressibility of compacted fly ash and the development of reliable settlement predictions are still required.

Frost susceptibility of compacted fly ash has been studied by many British investigators. Raymond and Smith (1964) carried out a field and laboratory test on ash from the Bold power station in England by using the freeze-thaw test and suggested that ashes with 10 percent cement content can be regarded as non-frost susceptible. However, this recommendation was based on one ash. A more comprehensive study was made by Croney and Jacobs (1967) who investigated ashes from nine sources, using the British Road Research Laboratory Frost Test. They found that five of the ashes were frost susceptible but could be made non-frost susceptible by cement treatment. The percentage cement required depends on the individual ash. Sutherland et al. (1958) performed similar tests on four Scottish ashes and indicated that the percentages of required cement vary from 6 to 15 percent for samples prepared at maximum dry densities. Sutherland and Gaskin (1970) also reported studies of the effect of permeability and compressive strength on the frost susceptible characteristics of a number of fly ashes.

A more detailed review or commentary on specific aspects of previous fly ash studies will be presented at appropriate sections in subsequent chapters.

CHAPTER 3

INDEX PROPERTIES AND STRENGTH OF COMPACTED FLY ASH

3.1 Introduction

Fly ash is a byproduct which results from burning pulverized coal in power stations. The coal used in modern power stations is ground in mills prior to combustion to the size of which about 80 percent passes a No. 200 sieve. The pulverized coal is then injected into the furnace at high speed in a stream of air and burned almost instantaneously in suspension at a temperature of approximately 1,500°C which is above the melting point of most of the minerals present. As a result they undergo chemical and physical changes. The exact nature of the changes depends not only on temperature but also on type of coal, fineness of grinding and retention period in the hot zone of the furnace.

A high proportion of the ash remains as individual particles but the remainder forms agglomerates. The unburned inorganic materials form small molten globules and congeal into spherical particles as they leave the high temperature zone. Some partially burned organic substances also form larger and more irregular shaped particles, which are considered to be

mostly carbon. The larger agglomerates fall to the bottom of the furnace into a hopper and are removed therefrom. This portion is called bottom ash. The amount of the bottom ash production depends on the nature of power plant boilers. Dry bottom boilers only produce about 10 percent bottom ash whereas wet bottom boilers, 50 percent and cyclone facility, 85 percent.

The remaining substances are called "fly ash" and are separated from the exhaust gas stream by various methods depending on the design of power plant. The collection of fly ash in the flue gas is usually accomplished by using either electrical precipitators or mechanical collectors. The electrical precipitators are more efficient than the mechanical collectors and usually remove a higher percentage of the fly ash from the flue gases, especially finer particles. Therefore the fly ash collected by an electrical precipitator has a high specific surface which is considered conducive to high pozzolanic activity.

Therefore, it should be noted that fly ashes with relatively different characteristics can be produced from different power stations. These variations are due to:

- (a) the degree of pulverization of the coal,
- (b) the design of furnace, including the methods of combustion and the method of collection,
- (c) the type of coal supply,
- (d) the boiler load, hence the firing conditions.

In view of this variant nature, the importance of understanding the index properties and the possible factors which affect the engineering properties of fly ash from any specific power plant

is rather evident. This variability also tends to discourage more widespread use of fly ash in the construction of roads, embankments and fills. Establishment of simple and reliable criteria for fly ash utilization in engineering practice is an important goal in any engineering study of this material.

The variables that affect the engineering properties of a fly ash are generally believed to be those which affect the pozzolanic activity in fly ash. The term pozzolanic activity is defined as the activity with which the calcium hydroxide reacts chemically at ordinary temperature with the natural or artificial siliceous or aluminous material (which in itself possesses little or no cementitious values) in the presence of water to form compounds possessing cementing properties.

From the above definition, it is evident that the siliceous and aluminous component and free lime content are the most essential elements in the pozzolanic reaction. Experimental verifications in terms of compressive strength have been attempted by many investigators, such as Davidson et al. (1958), Thorne and Watt (1965), Sherwood and Ryley (1966) and Sutherland et al. (1968). Although $\text{SiO}_2 + \text{Al}_2\text{O}_3$ content appears to be an important determinant of compressive strength, other variables also play an important role. In fact, under certain conditions these variables can be even more important than $\text{SiO}_2 + \text{Al}_2\text{O}_3$ content. For example, lack of free lime can eliminate pozzolanic reaction in a fly ash; high carbon content tends to retard the reaction and consequently reduce strength gain in fly ash. Therefore, no unique relation between strength and these var-

variables is available to be used as a guide to assess the strength of fly ash. The other possible variables besides the $\text{SiO}_2 + \text{Al}_2\text{O}_3$ content are specific surface, loss on ignition, nature of compaction, moisture and temperature during curing. These factors are summarized and presented in the next section.

An experimental study of the strength properties of some typical Michigan fly ashes has been made. Since the range of the variation in material properties of the ashes is narrow, the study of relations between intrinsic material variables and strength has not been attempted. However, the effects of external test conditions such as lime stabilization and temperature were studied because of their practical significance. The results are given in the last section of this chapter. The index properties and composition of these typical Michigan fly ashes are also given in this chapter.

3.2 Variables which Affect the Strength Development of Compacted Fly Ash

The possible variables that affect the strength, or the pozzolanic reaction, of a fly ash can be divided into two basic categories viz., material variables and environmental variables. They are each discussed in turn as follows:

1. Material Variables

Material variables are those properties which a fly ash possesses intrinsically. They include siliceous and aluminous composition, surface area, carbon content and free lime content.

- (a) Siliceous and aluminous composition: Finely divided silica and alumina are essential constituents in a pozzolanic material. Pozzolans possess little or no cementitious properties, but will react chemically with lime in the presence of water to form stable compounds possessing cementitious properties. The relationship between $\text{SiO}_2 + \text{Al}_2\text{O}_3$ content and strength of fly ash has been investigated by Thorne and Watt (1965). In a study of the crushing strength of some British fly ash specimens, they found that the pozzolanic activity of an ash resides principally in the glassy particles, and the glassy particles are composed primarily of SiO_2 and Al_2O_3 . A particularly good correlation between strength and pozzolanic content was obtained for samples with long periods of curing. Similar results were also obtained by Sherwood and Ryley (1966) based on the unconfined compression strength tests of twelve, British fly ashes.
- (b) Free lime content: Free lime in a fly ash plays a key role in pozzolanic reactions. It is a necessary ingredient along with siliceous and aluminous constituents in producing self-hardening or cementitious reaction. The self-hardening property of a fly ash would not exist if the presence of free lime is inadequate. The importance of free lime content in fly ashes as the cause of self-hardening has been verified experimentally by a few

investigators, such as Sherwood and Ryley (1966), Sutherland et al. (1968). Sherwood and Ryley performed unconfined compression tests on many sources of British fly ashes with and without stabilization treatments. They showed that because the free lime already present in the fly ash is probably more finely divided and is in more intimate contact with the ash particles than the lime which is added, it appears more effective in the strength development than added limes.

- (c) Surface area: The surface area of fly ash particles is not the cause of pozzolanic reaction but it affects the rate of the reaction. A material with a higher specific surface, i.e., with finer particles and denser packing, tends to have more grain contacts and more exposed area available for immediate chemical reactions. This can be best demonstrated by the investigation made by Thorne and Watt (1965) on the crushing strength of 14 fly ashes. They observed that for short periods of curing, the best correlation was between strength and specific surface. However, when the curing periods were longer, i.e., the pozzolan and lime had been exposed to contact sufficiently long enough, a better correlation was found between strength and the $\text{SiO}_2 + \text{Al}_2\text{O}_3$ content. A good correlation between specific surface and 28-day strength of some British fly ashes was also observed by Sherwood and Ryley (1966).

(d) Loss on ignition: Loss on ignition is usually expressed as a percentage of the total fly ash and approximately represents the carbon content. Carbon in fly ash appears to be harmful to the pozzolanic activity and the strength of fly ash. From the microscopic examinations, Davidson et al. (1958) suggested that the carbon tends to adhere to and partially cover the particle surfaces, and hence, reduces the interfacial area available for pozzolanic reactions with lime. Besides, carbon coatings also act like links between adjacent particles to produce a porous aggregated structure which reduces available surface area and attainable compacted density. The deleterious influence of loss on ignition on the strength development has been shown by Davidson et al. from the experiments on four Midwestern fly ashes. They concluded that the fly ashes containing less than 10 percent of carbon possess high compressive strength whereas the fly ash with carbon content near 30 percent showed no pozzolanic activity regardless of the amount of lime added. A similar effect was also found by Sherwood and Ryley (1966) in some British fly ashes with ignition losses above 16 percent. However, no clear relation exists between strength and loss on ignition for fly ashes with low ignition loss values. This again emphasizes the fact that no single factor can be uniquely related to the pozzolanic activity or strength development of a fly ash.

2. Environmental Variables

Environmental variables refer to the external factors that affect the pozzolanic reaction. These variables include lime addition, degree of compaction, and curing condition.

- (a) Lime addition: Lime additives often serve as stabilizing agents, especially when the inherent free lime in the fly ash is inadequate. The added lime will react with the available siliceous and aluminous components to produce bonding between the stabilizer matrix and particles. It is generally agreed that the strength of a fly ash increases with increase in lime addition until a certain limit is reached, at which the further addition of lime will either have no effect on or possibly decrease the strength. The amount of lime additive at this limit is called "optimum lime content". The lime added to the fly ash as a stabilizer is believed to be less effective than the inherent free lime due to the fact that the particles of added limes are less finely divided and not so uniformly distributed among ash particles. Admixed lime is even less beneficial when moisture is present before thorough mixing has occurred because the presence of water causes a premature aggregation of particles with a consequent decrease in surface area. On the other hand, the presence of free lime in fly ashes is often scant and therefore seldom sufficient for self-hardening process. Consequently, it is frequently found that the lime stabilization is necessary.

In addition to lime, cement is also widely used as a stabilizer. There are advantages to using either agent. It appears that cement possesses a higher stabilizing potential, especially in regard to the rate of strength increase. However, Sutherland et al. (1968) found that, although cement stabilized ashes are stronger than lime stabilized ashes at early ages, the difference has been eliminated for many ashes after 3 months of curing. The comparison of unconfined compression strengths of some lime and cement stabilized British fly ashes are shown in Tables 3.1 and 3.2. Furthermore, lime is generally available at a lower price than cement. Therefore, the use of lime as stabilizer is reasonable if the performance of lime in a fly ash is satisfactory and provided adequate curing conditions exist.

- (b) Degree of compaction: It is believed (Sutherland et al. 1968) that, in a given fly ash, the curing strength is in proportion to the compacted dry density. From the tests on some British ashes, they found that the strength of these ashes reduced to 75 and 60 percent of the value at maximum dry density (standard Proctor test) when the degrees of relative compaction were 90 and 85 percent respectively. This may be due to the more intimate contacts between particles of both ash and lime in a denser packed structure for the higher dry density condition. The maximum dry density of a compacted soil for a certain compactive effort is achieved by compacting it

TABLE 3.1 - Unconfined Compression Strengths of Cement and Lime Stabilized P.F. Ashes

Station	Test Series	Unconfined Compression Strength (lb/in ²) at age (days) stated											
		10 per cent Cement						10 per cent lime					
		7	28	56	91	182	581	7	28	56	91	182	581
Barony	1	670	1220	1320	--	--	--	538	2505	3123	--	--	--
Barony	2	1087	1780	--	3435	3790	4447	455	1905	--	3370	4010	5000
Braehead	1	187	334	444	--	--	--	66	270	410	--	--	--
Braehead	2	239	356	--	599	663	775	84	299	--	520	603	848
Kincardine	1	316	543	835	--	--	--	104	317	439	--	--	--
Kincardine	2	266	397	--	759	956	1296	90	212	--	486	739	1205
Portobello	1	392	936	1061	--	--	--	134	675	1604	--	--	--
Portobello	2	469	1027	--	1313	1517	1843	122	407	--	1316	1933	2620

(From Sutherland, et al., 1968)

(From Sutherland, et al., 1968)

TABLE 3.2 - Ratios of Strength of 10 percent Stabilized Cement and Lime P.F. Ashes at Different Times

Station	Test Series	Strength of 10 per cent cement stabilized ash Strength of 10 per cent lime stabilized ash at age(days)									
		7	28	56	91	182	581				
Barony	1	1.25	0.49	0.42	--	--	--				
Barony	2	2.38	0.93	--	1.02	0.94	0.89				
Braehead	1	2.84	1.24	1.08	--	--	--				
Braehead	2	2.84	1.19	--	1.15	1.10	0.91				
Kincardine	1	3.04	1.71	1.90	--	--	--				
Kincardine	2	2.96	1.87	--	1.56	1.30	1.08				
Portobello	1	2.92	1.39	0.66	--	--	--				
Portobello	2	3.84	2.52	--	1.00	0.78	0.70				

(From Sutherland, et al., 1968)

(From Sutherland, et al., 1968)

at the optimum water content. Therefore, to attain high curing strength, the degree of compaction, involving both compaction effort and moisture content, seems important.

- (c) Curing condition: Curing condition refers to two major considerations, namely, provision of moisture and curing temperature. It can be recognized from the definition of pozzolanic reaction that the presence of water is a necessary condition for the process. On the other hand, excessive supply of moisture may tend to reduce strength and cause sample deterioration. In the investigation of strength, settlement and compaction characteristics of some British fly ashes, Raymond and Smith(1966) suggested that the compacted fly ash will increase in strength with time provided that a partially saturated condition is maintained. However, this may be only true for untreated ashes and those stabilized ashes which have not yet reached an advanced stage of chemical hardening. Beyond this stage, an ample supply of moisture should be favorable to pozzolanic activity.

The influence of temperature on the pozzolanic activity is anticipated because many chemical reactions are temperature dependent. Leonard and Davidson (1959) studied the pozzolanic reactivity in terms of Ca(OH)_2 absorption, and observed that the reaction was endothermic, i.e., increasing the amount of heat drives the reaction faster to completion. Similar observations have been

made by Davidson et al. (1958), Thorne and Watt (1965), and Sherwood and Ryley (1966) from direct measurements of unconfined compressive strength. The consensus is that an increase in temperature causes an increase in reaction rate. However, all the samples were cured at more or less constant moisture content, and the temperature effect on the ultimate strength is also influenced by available moisture. Thorne and Watt even noted a reduction in the final strength for samples cured at higher temperature. This loss of strength is probably due to the insufficient supply of moisture during curing. It seems reasonable to expect that the fly ashes cured at higher temperature will reach a higher ultimate strength provided the supply of moisture is adequate.

3.3 Index Properties and Composition of Typical Michigan Fly Ashes

Description of four typical Michigan fly ashes used in this dissertation is given in this section.

1. Source of Fly Ashes

Fly ashes under investigation were obtained from four coal-fired, electric generating plants in Michigan. The fly ashes were selected as representative of the typical range of ashes produced by coal-fired power stations in the Great Lake region which accounts for the greatest annual tonnage of production of fly ash in the United States. The sources of the fly ashes are as follows:

<u>Plant Location (or Name)</u>	<u>Operator</u>
(1) Marysville	Detroit Edison Company
(2) St. Clair	Detroit Edison Company
(3) Trenton Channel	Detroit Edison Company
(4) Saginaw (D.E. Karn plant)	Consumers Power Company

Most of these plants obtain their coal by unit train from West Virginia coal field. The coal is the bituminous variety, and it typically contains about less than 1 percent by weight sulphur. Fly ashes are removed and collected from the stack gases in the plants by electrical precipitators.

2. Composition of Fly Ashes

A chemical analysis of the fly ash solids from these four Michigan power stations is shown in Table 3.3. The principal constituents of these fly ashes (reported as oxides) are similar to most fly ashes. They consist of silica, alumina and iron oxide. Minor amounts of calcium, magnesium and unburned carbon are also present. The typical range of chemical composition for most fly ashes reported by the Central Electricity Generating Board of England (1967) is as follows:

	<u>Max.</u>	<u>Percent</u> <u>Min.</u>	<u>Typical</u>
Silica (as SiO_2)	58	38	48
Alumina (as Al_2O_3)	40	20	26
Iron oxide (as Fe_2O_3)	16	6	10
Calcium (as CaO)	10	2	4
Magnesium (as MgO)	3.5	1	2
Total Sulphate (as SO_3)	2.5	0.5	1.2
Water Soluble Alkalis (as $\text{Na}_2\text{O}, \text{K}_2\text{O}$)	5.5	2.0	3.0

Comparison with the British fly ashes suggests that the Fe_2O_3 content of typical Michigan fly ashes is higher whereas the calcium content is considerably lower. This low calcium content may partly contribute to the lack of self-hardening properties of these fly ashes as will be shown in later sections.

3. Leachate Characteristics

The problem of leaching by ground water becomes important when a fly ash fill is emplaced in low lying or swampy terrain. The leachate composition, therefore, may be of considerable interest to those concerned with water quality. An analysis of leachate of typical Michigan fly ashes is shown in Table 3.4. Total dissolved solids were less than 2 percent on a weight basis. The principle dissolved ions appeared to be calcium, magnesium and sulphate. The boron content in all cases was well below the critical limit of 25 ppm. established by the Central Electricity Generating Board of England, hence the toxicity of boron concentrations of these fly ashes to the plants is not significant. The leachate was alkaline with a pH of approximately 11 in most cases.

4. Index Properties

Some important index properties of three fly ashes from Detroit Edison Company such as grain size distribution, specific gravity of ash solids, surface area (determined by the air permeability method) and compacted dry density are summarized in Table 3.5. Fly ash is a predominantly silt-sized material. Grain size distribution curves for these three fly ashes are shown in Figure 3.1. The curves are quite uniform and almost

TABLE 3.3 - Chemical Analysis of Fly Ash Solids from Some Coal-Fired Power Stations in Michigan

Fly Ash Source	Chemical Composition - Per cent by Weight									
	SiO ₂	Al ₂ O ₃	Fe ₂ O ₃	TiO ₂	CaO	MgO	Na ₂ O	K ₂ O	SO ₃	Carbon
D. E. Karn Plant	45.0	28.5	24.3	--	1.0	0.5	--	--	Trace	0.7
Marysville Plant	36.5	19.0	33.0	0.6	2.0	0.8	0.3	1.7	0.66	3.53
St. Clair Plant	36.0	29.0	19.0	1.0	3.8	0.9	0.4	1.6	0.97	4.75
Trenton Channel Plant	46.5	26.5	16.0	1.0	0.9	0.8	0.4	1.6	0.60	4.32

TABLE 3.4 - Partial Chemical Analysis of Leachate (1) from Fly Ashes Obtained from Some Coal Fired Power Station in Michigan

Plant Name or Location	Operator	Concentration - ppm						PH
		Ca++	Mg++	SO ₄ =	Boron	Tot. Dissolved Solids		
						ppm	% dry solids	
Marysville	Detroit Edison	200	266	230	.3	1029	1.0	11.4
Trenton Channel	Detroit Edison	288	402	220	.4	1448	1.4	7.4
D. E. Karn	Consumers Power	245	357	200	.4	1051	1.1	10.8
B. C. Cobb	Consumers Power	856	184	960	10.0	1697	1.7	11.2

(1) Supernatant obtained by shaking 100gms. of dry ash with 1 liter of distilled water for 24 hours

(From Gray and Lin, 1971)

TABLE 3.5 - Index Properties of Three Michigan Fly Ashes
From Detroit Edison Company

Properties		Ash Source		
		Marysville	St. Clair	Trenton Channel
Specific Gravity		2.61	2.48	2.36
Specific Surface (cm ² /gm)		2620	3050	3050
Max. Dry Density (lb/ft ³) (Mod. ASSHO Test)		74.2	90.5	92.8
Optimum Water Content (%)		32.0	23.0	19.5
Particle Size Distribution	>2mm (Gravel Size)	0	0	0
	.074mm-2mm (Sand Size)	12	6.5	10.5
	.002mm-.074mm (Silt Size)	85	88.5	82.5
	<.002mm (Clay Size)	3	5	7

fall completely within the conventional limits for frost susceptible soils; this is perhaps one of the main reasons why engineers in the United States have been reluctant to use fly ash in load-bearing fills. Frost susceptibility of fly ash and the relevancy of conventional frost criteria based on gradation alone will be discussed separately in the following chapter.

In spite of containing high percentage of ferric oxide, the average specific gravity of Michigan fly ashes is about 2.45 or some 90% that of quartz; the major constituent of most soils. One possible explanation of this lower average specific gravity is the fact that a significant proportion of fly ash particles are "cenospheres", Raask (1968). In accord with Watt and Thorne's work (1965) on some British ashes,

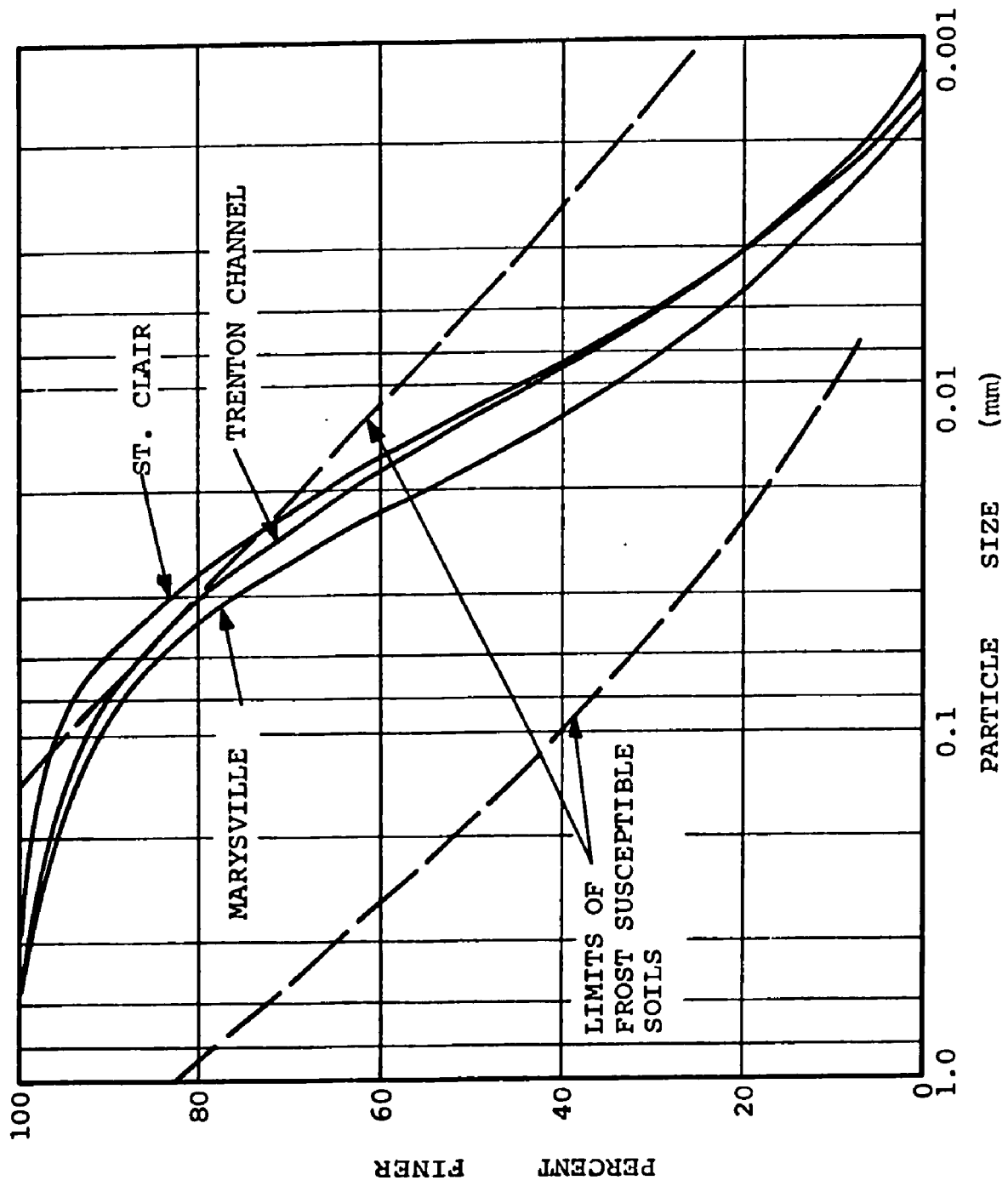


Figure 3.1 Grain size distribution of three typical Michigan fly ashes

virtually all the particles of Michigan ashes appear rather spherical or spongy when examined under the microscope.

The characteristics of hollowness, spherical and spongy particle shape, and uniform gradation may in turn contribute to the lower compacted density of fly ash relative to conventional earth fill compacted to the same effort.

5. Permeability

Laboratory determinations of permeability of British fly ash compacted at 95% to 100% of the Proctor maximum dry density have shown values ranging from 0.5×10^{-6} cm/sec to 4×10^{-4} cm/sec, Raymond (1961) and the Central Electricity Generation Board (1967). These correspond to drainage characteristics ranging from practically impervious to medium, which are similar to those of soils of similar particle sizes and gradings. The permeability of fly ash from Trenton Channel plant was measured. A theoretical curve was also calculated according to the Kozeny-Carman relationship and is shown in Figure 3.2. The good correlation may be due to the spherical shape of particles, on which the shape factor of the Kozeny-Carman equation is based. The curve shows that the permeability of this particular fly ash is about 6×10^{-5} cm/sec at Modified AASHO maximum dry density.

The importance of permeability of fly ash is its influence on leaching and runoff. Low permeability will lessen the probability of extensive ground water percolation and the consequent danger of soluble material being leached out of the fill. On the other hand, low permeability also means a high

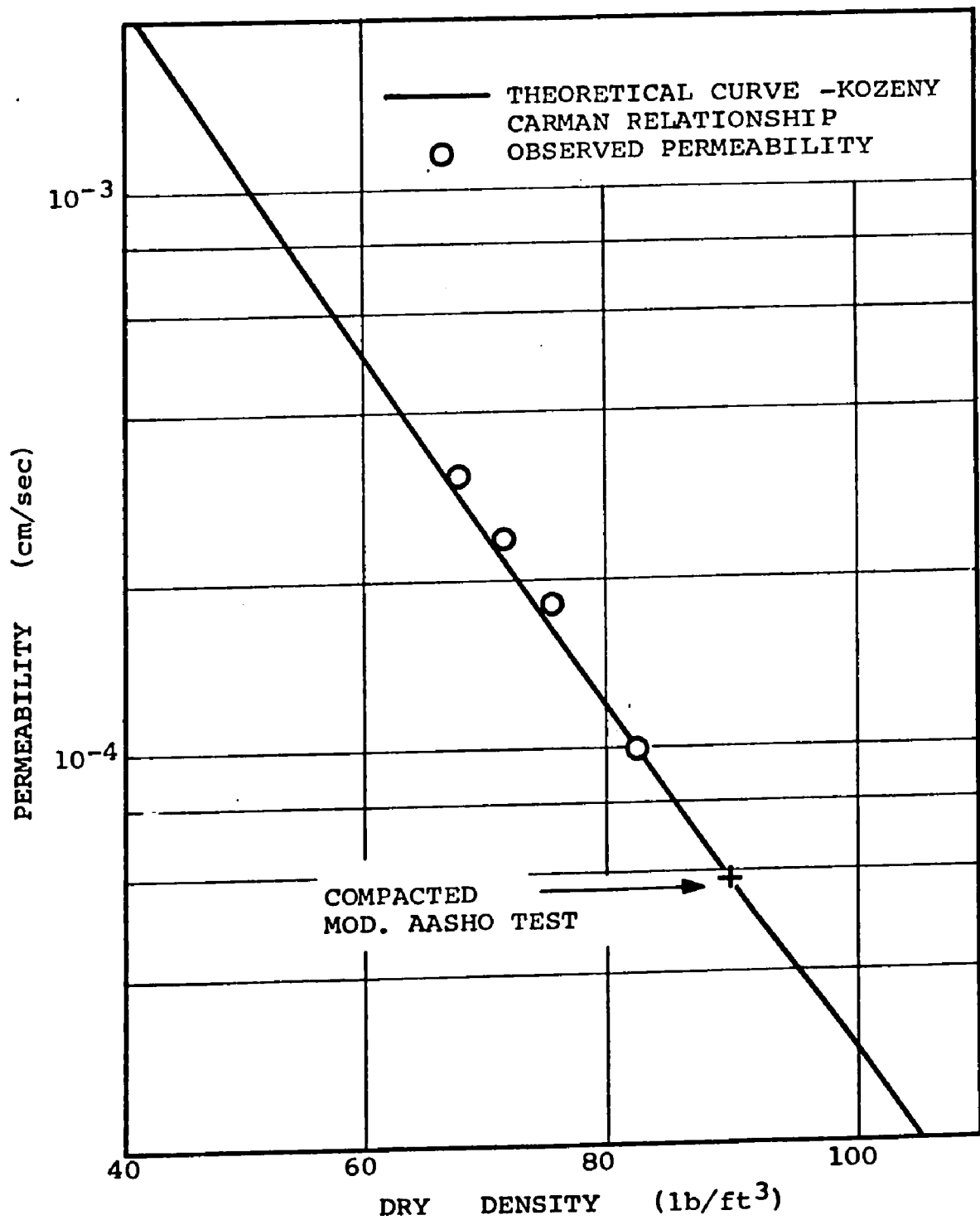


Figure 3.2 Permeability of Trenton Channel fly ash with different dry densities (from Gray & Lin, 1971)

degree of runoff, therefore precautions to prevent erosion of side slopes should be taken. Furthermore, permeability will also effect the heaving characteristics of frost susceptible soils. This will be discussed in more detail in the subsequent chapter.

3.4 Strength of Compacted Fly Ashes

The compressive strength characteristics of compacted Michigan fly ashes and the effects of lime stabilization and curing temperature on the age hardening properties are discussed in this section. The method of compaction employed is also reported herein.

1. Compaction-Strength Characteristics

(1) Method of Compaction: From the laboratory tests on compacted sewage ash, Gray (1970) showed that Kneading type compaction introduced excessive shear strain into the ash and failed to compact it properly. To avoid the same difficulty, only impact compaction was used. Compactions were performed according to the modified AASHTO test with various compaction efforts. However, to maintain consistency in sample size for unconfined compression tests, triaxial tests and K_0 compressibility tests, a standard triaxial size sample was used. All the samples were compacted in a modified miniature compaction mold (with inside diameter of about 1.4 inches and a height of about 2.8 inches). A special miniature compactor was designed and made in such a way that it produces the same compaction energy in the miniature mold as a standard compactor does in a standard mold with the same compactive effort. The

weight of the hammer was 1.12 lbs. and the height of drop was 12 inches. The compactor and the mold are shown in Figure 3.3.

The compactive effort was varied by using either 15, 25 or 35 blows per layer (except for Trenton Channel fly ash which was compacted at 20, 25, and 30 blows). A total of 5 lifts or layers were used for all compaction tests.

(2) Compaction Characteristics and Unconfined Compressive Strength: The compaction curves obtained by impact compaction of four Michigan fly ashes are shown in Figure 3.4 through Figure 3.7. It can be observed that these fly ashes respond to impact compaction in a rather similar way as any fine grained soil would behave. The optimum water contents for samples compacted at 25 blows per layer ranged from 17 to 32 percent and the maximum dry densities ranged from 74 to 97 lbs/ft³, which are considerably lower than those of ordinary soils. The water content at which the maximum dry density can be achieved by a specific compactive effort is called "optimum water content". An inverse relationship between maximum dry density and optimum water content was observed confirming results obtained earlier by Raymond (1961) on British fly ashes. Figure 3.8 indicates that the higher maximum dry densities are associated with lower optimum water contents. Nevertheless, no relation between maximum dry density and specific gravity, as found by Raymond from some British fly ashes, appears to exist.

Immediately following compaction the sample strengths were determined by unconfined compression tests. Low unconfined compressive strengths of these compacted fly ashes were observed.



Figure 3.3 Miniature impact compactor and modified miniature mold.

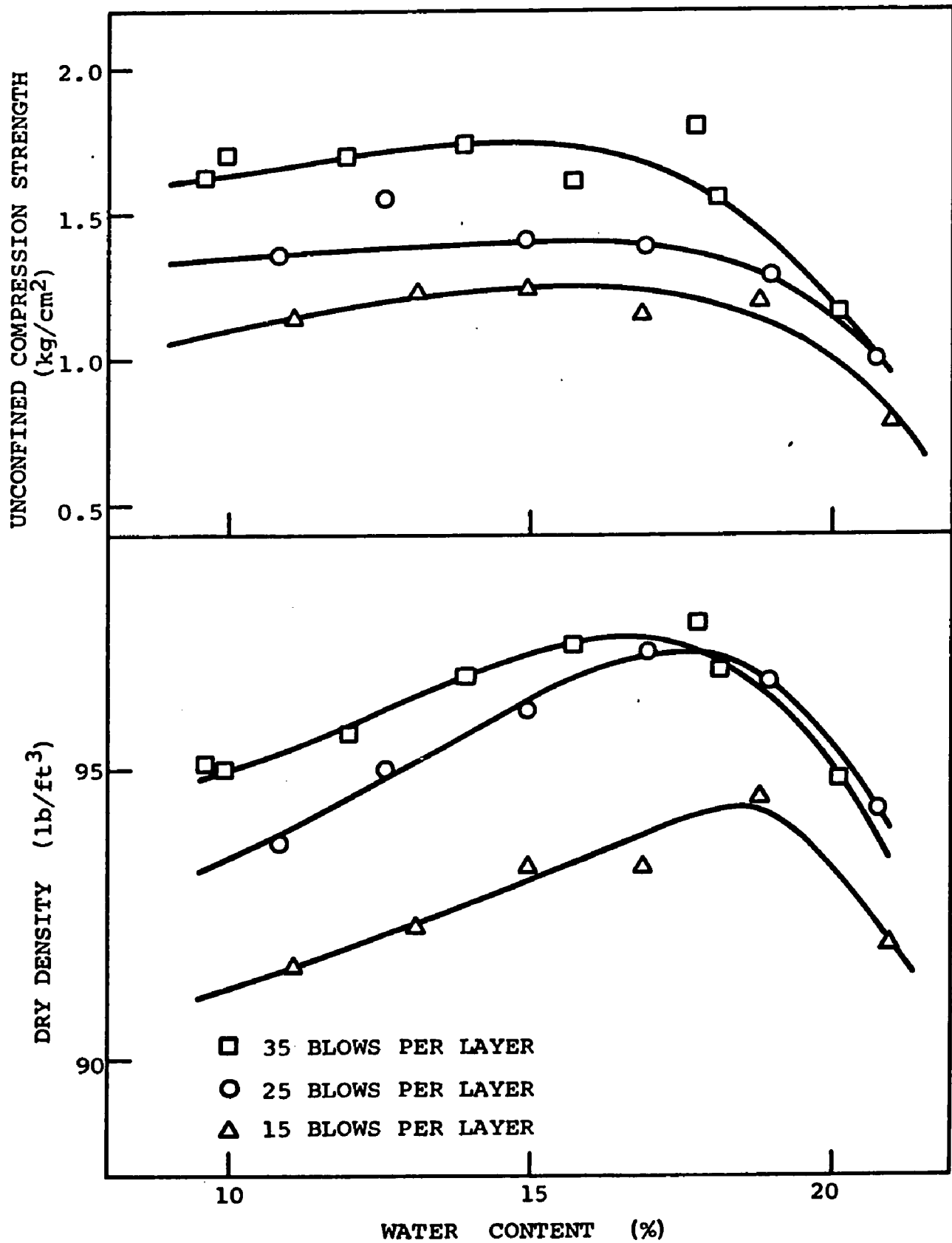


Figure 3.4 Compaction characteristics and compressive strength of fly ash from D.E.Karn plant.

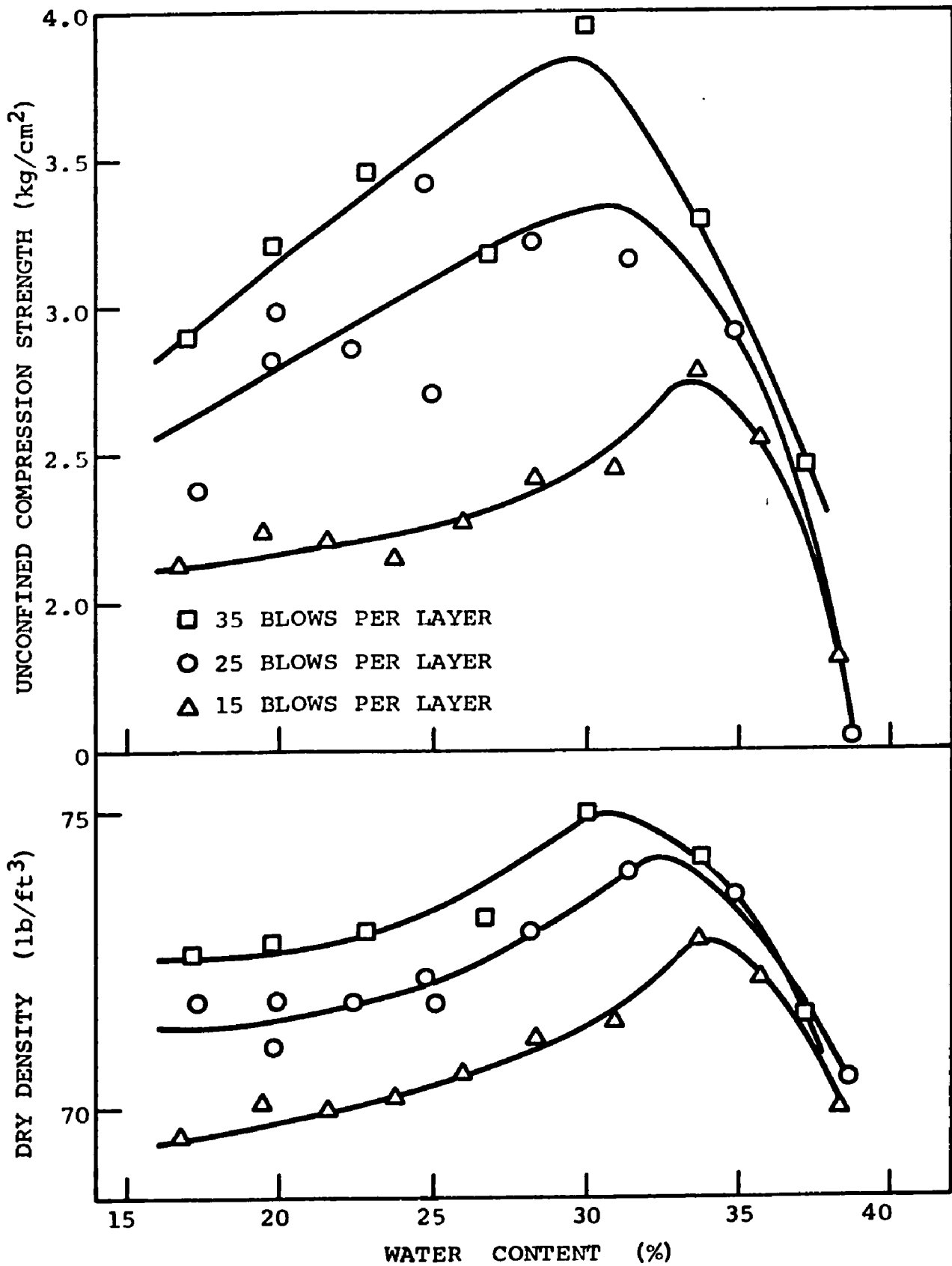


Figure 3.5 Compaction characteristics and compressive strength of fly ash from Marysville plant.

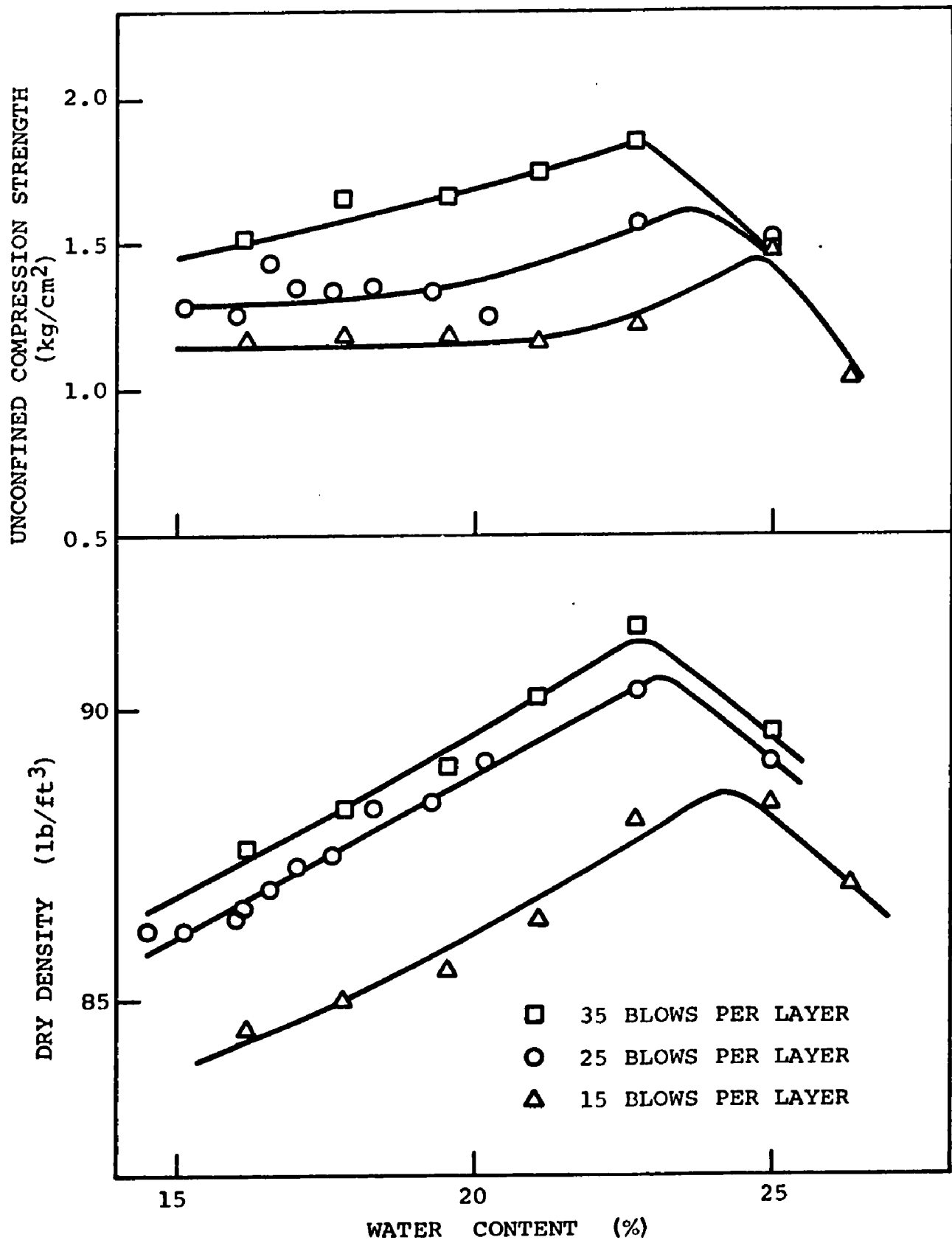


Figure 3.6 Compaction characteristics and compressive strength of fly ash from St. Clair plant.

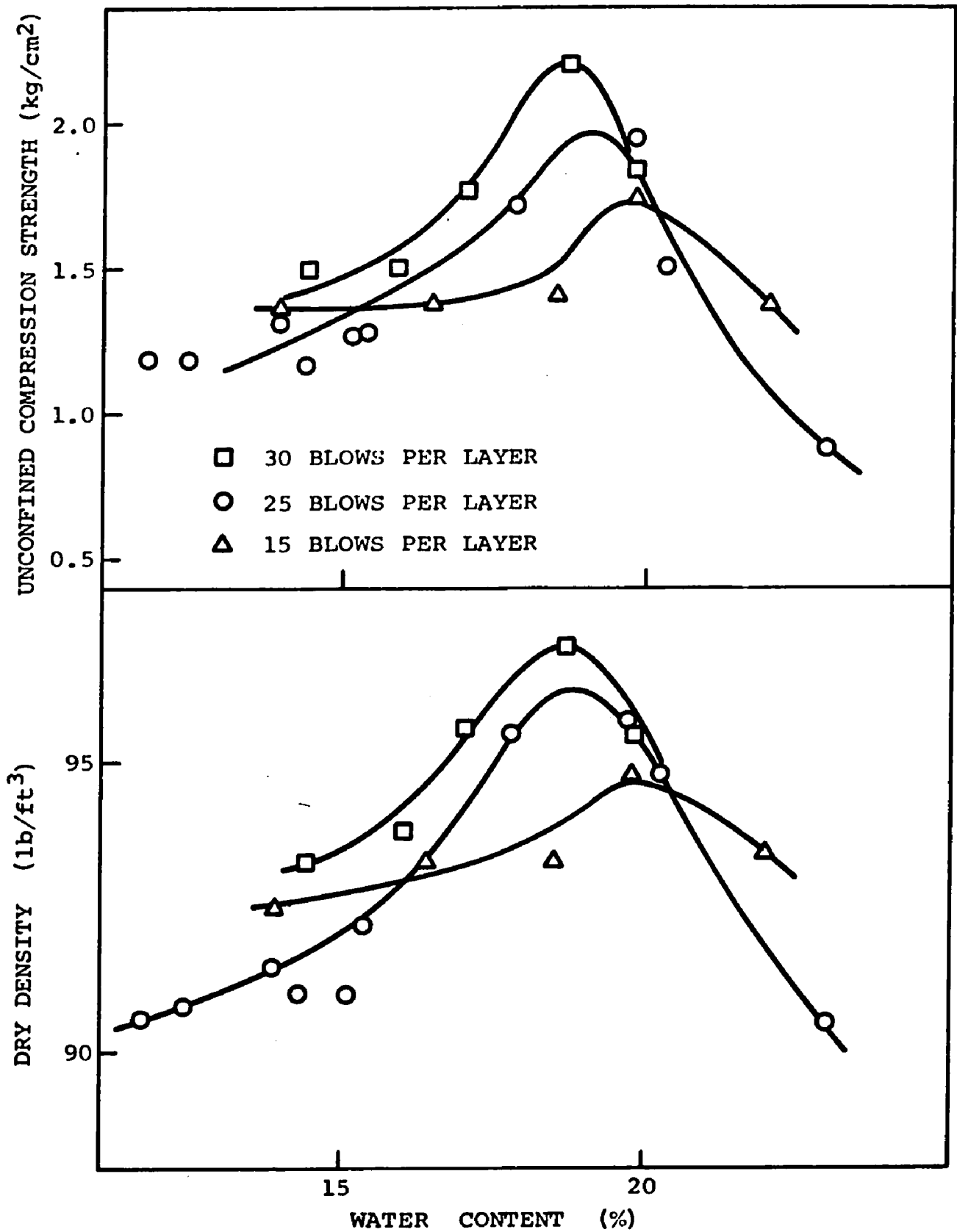


Figure 3.7 Compaction characteristics and compressive strength of fly ash from Trenton Channel plant.

The strengths ranged from only 1.2 to 3.9 kg/cm² even when samples were compacted to maximum dry density. However, it must be noted that these samples were neither treated with any lime or cement nor allowed to cure or age-harden before being tested. The results of compressive strengths are also shown in Figure 3.4 through Figure 3.7.

The unconfined compressive strength of the samples dropped off sharply on the wet side of optimum water content, whereas on the dry side, the strength was affected less by molding water content and more by dry density, in other words, it was affected more by the blow count or compactive effort used than by molding water content. The same phenomenon was also found from the penetration resistance in compacted Trenton Channel fly ash, which is shown in Figure 3.9.

Compacted samples for all four Michigan fly ashes exhibit fairly brittle behavior during unconfined compression. Failure normally occurred abruptly at a strain ranging from 1 to 4 percent. All the samples compacted at the neighborhood of optimum water contents failed at about 2 to 2.5 percent strain.

(3) Shear Strength Parameters: Drained triaxial tests were run on samples of freshly compacted fly ashes from three plants of Detroit Edison Company. All samples were compacted to maximum dry density at optimum water content according to AASHTO specifications. Friction angles ranged from 38 to 43 degrees and cohesion intercepts were about 0.7 to 1 kg/cm². These results are comparable to the strength of a number of British fly ashes compacted according to standard Proctor specifications, Sutherland et al. (1968). In virtually all cases

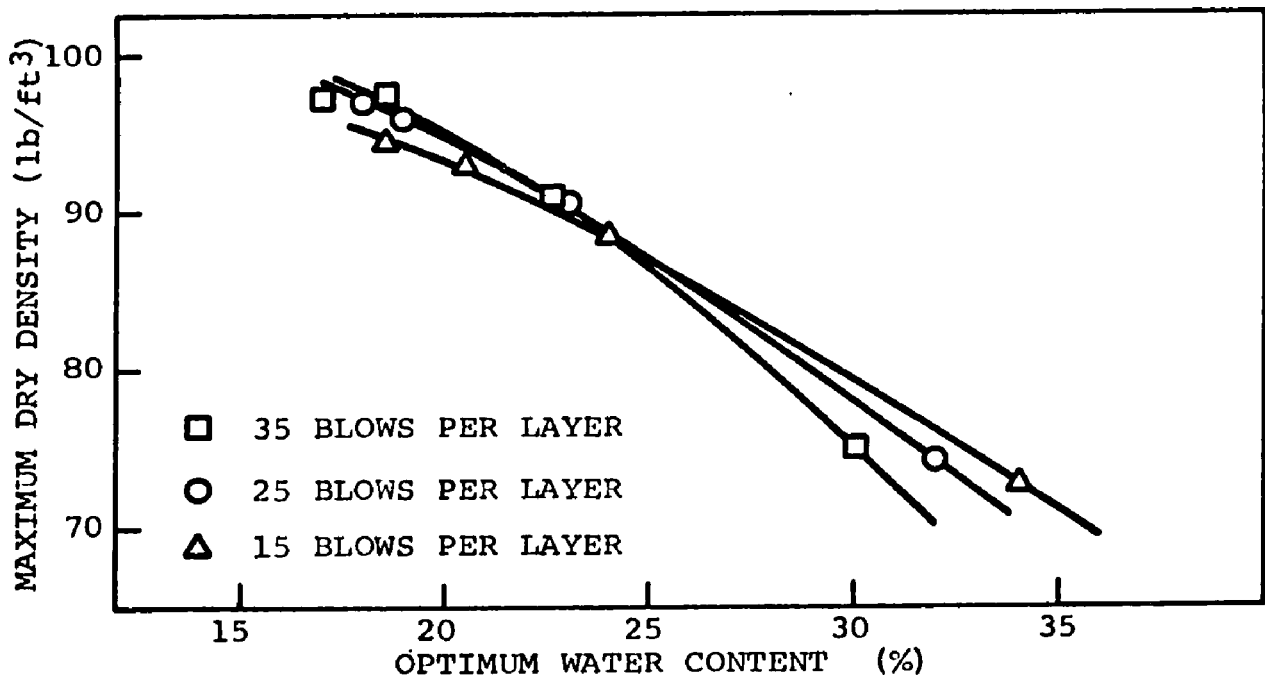


Figure 3.8 Relation between maximum dry densities and optimum water contents of four Michigan fly ashes.

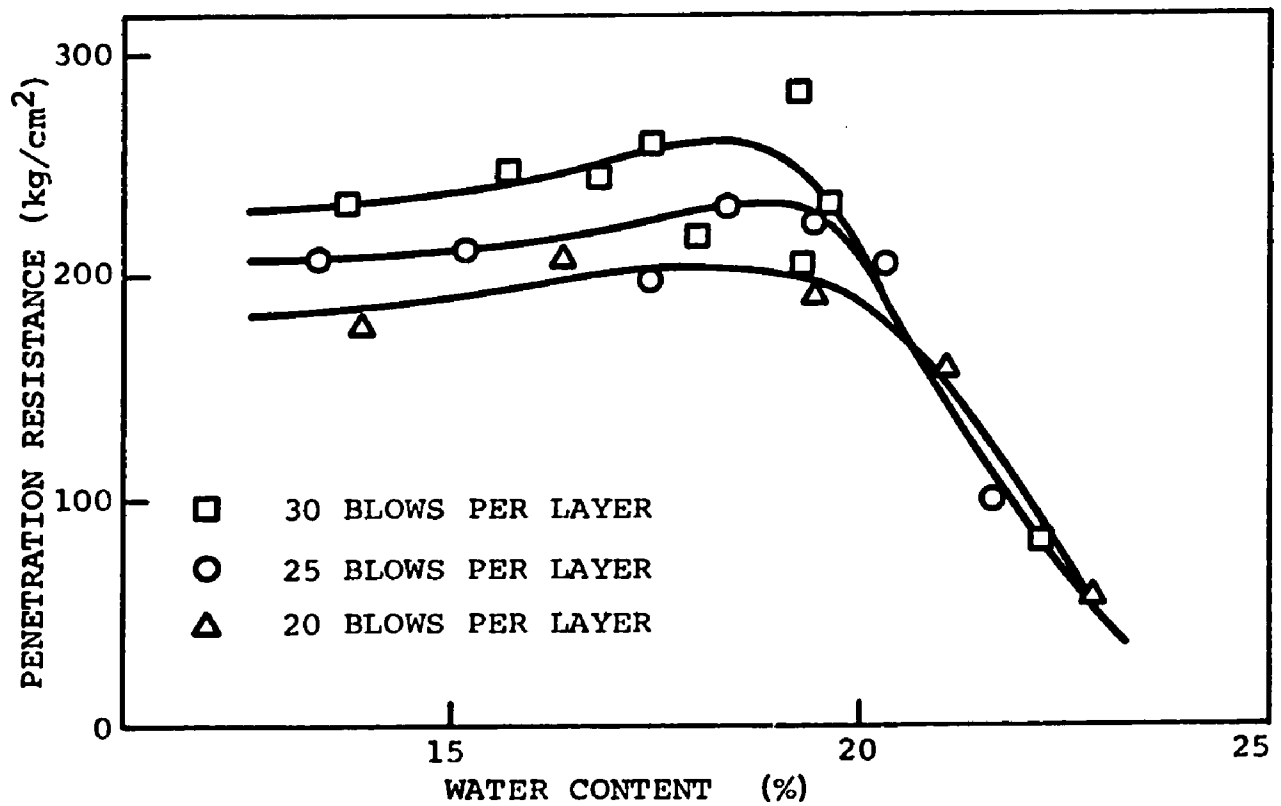


Figure 3.9 Penetration resistance of compacted fly ash from Trenton Channel plant.

the observed shear strength parameters would be sufficient to provide adequate bearing capacity and stability for a load-bearing fill or embankment.

2. Effect of Lime Stabilization on the Strength of Compacted Fly Ash

As it has been discussed in Section 3.2, lime or cement stabilization is one of the most important external factors that influence the pozzolanic reaction, especially when free lime content in a natural fly ash is low. The effect of lime stabilization on the strength of compacted Michigan fly ashes was studied and is presented herein.

(1) Sample Preparation and Testing: Samples from all four sources were prepared by either adding various amounts of lime or without any lime addition, mixing thoroughly at optimum water contents, and compacting according to the modified ASSHO specifications (25 blows per layer). Lime was used as the stabilizing agent because of the following reasons:

- (a) The laboratory experience in England showed that, although cement stabilized ashes are stronger than lime stabilized ashes at early stages, the difference was generally eliminated in 3 months (see Tables 3.1 and 3.2).
- (b) Lime is cheaper* than cement, hence should be considered with priority.

The lime used was "KEMIKAL" hydrate lime from United States Gypsum Company. The chemical and physical analysis provided by the manufacturer are shown in Table 3.6. After compaction,

*Cost per ton of lime and cement in Michigan is \$19 and \$41 respectively.

TABLE 3.6 - Chemical and Physical Analysis of KEMIKAL Hydrate Lime

Composition	% Content by Wt.
Total calcium oxide (CaO)	74.4
Available lime as CaO	73.0
Magnesium oxide (MgO)	0.1
Silica and insolubles	0.2
Iron and alumina	0.1
Sulfur	0.03
Loss on ignition	25.3
Finess: Through 100 mesh 99.8%	
200 mesh 98.5%	
325 mesh 97.5%	

the samples were then stored in a humidity compartment in which a moisture mist was provided continuously by a humidifier. Samples were cured in this manner for 7, 14 or 28 days before unconfined compression tests were run. The curing temperature was 20°C. Precaution had to be taken with untreated samples during curing. Because of insufficient cementing that developed in untreated fly ashes, excessive moisture tended to disintegrate the samples. Therefore, those untreated samples were stored on the lower shelves where the humidity was lower than other portions of the compartment, though the relative humidity still ranged from 90 to 95 degrees.

At the end of the curing periods (7, 14 or 28 days), samples were then tested in unconfined compression using a deformation rate of 0.5 mm/min. A Tinius-Olsen 12,000 lb.

testing machine was used as a compression apparatus. The tests were duplicated in nearly every case, and the results reported represent the compression stresses at failure.

(2) Results: The results of unconfined compression tests on samples with various amounts of lime treatments after various periods of moist curing are summarized in Table 3.7. The effect of lime treatment on the unconfined compressive strength of these fly ashes is shown in Figure 3.10 through Figure 3.13.

It can be noted that none of the Michigan fly ashes which were tested exhibited significant self-hardening properties. The same phenomenon was also found from the results of drained triaxial tests on three compacted fly ashes from Detroit Edison Company. These samples were not treated; they were compacted to maximum dry densities and moist cured for 28 days. The comparison of shear strength parameters of these untreated fly ashes tested immediately after compaction and those tested after 28 days of curing is shown in Tables 3.8. Based on laboratory experience in England, the finding that untreated fly ashes lack self-hardening properties is not surprising. Sherwood and Ryley (1966) found that only one third of the nine fly ashes tested exhibited self-hardening properties. This fact suggests that the self-hardening phenomenon may be less common than it is generally supposed. If this is the case for fly ashes taken directly from plant hoppers, it is even less likely that lagooned ashes will exhibit self-hardening since their hardening properties must be further dissipated by the water which is used to convey them to the lagoon. Therefore,

TABLE 3.7 - Unconfined Compression Strengths of Four Michigan Fly Ashes with Various Lime Treatment after Various Periods of Moist Curing*

Source	Days of Curing	Unconfined compression strength (kg/cm ²)									
		0% lime		5% lime		8% lime		10% lime		15% lime	
		1**	2	1	2	1	2	1	2	1	2
D.E. Karn	7	2.51	2.43	9.90	9.56	11.03	10.83	7.65	8.83		
	14	2.96	3.66	18.00	22.40			16.40	19.40		
	28	2.98		23.40	23.30	24.40		20.40	19.10		
Marysville	7	2.57	3.12	9.26		9.13	9.76	12.60	13.85	12.95	12.30
	14									17.65	18.50
	28	2.84		17.75	19.15	19.30	23.70	25.90	25.70	25.06	24.54
St. Clair	7	1.24	1.65	9.56	9.60	10.70	10.50	9.00	9.25		
	28	1.16	1.28	16.90	18.30	17.60	18.75	16.60	16.60		
Trenton Channel	7			16.70	16.75	18.50		24.35	22.05		
	(10) 28	2.74 2.05	2.34 1.52	27.60	27.80	27.00		27.60	28.40		

*Cured at 20°C

**Test number

TABLE 3.8 - Shear Strength Parameters of Three
Compacted Fly Ashes from Detroit
Edison Company

	Marysville		St. Clair		Trenton Channel	
Age	c	ϕ_f	c	ϕ_f	c	ϕ_f
Fresh	0.9	43.0	0.7	40.5	0.7	38.5
28 days	1.0	43.0	0.5	40.0	1.1	37.5

the self-hardening property cannot be relied upon to give appreciable strength to fly ash, and where a durable material is required, it would be necessary to stabilize the ash with additional lime or cement.

Another problem one must bear in mind in dealing with the untreated fly ashes is the phenomenon of disintegration or softening due to the excessive supply of water as indicated previously. The same problem was also found in the field by Raymond and Smith (1966). He suggested that when an embankment is constructed on ground having a high water table, it is advisable to put down a layer of fill with good drainage beneath the fly ash (untreated). This layer should be thick enough to prevent the ground water rising into the fly ash.

The substantial effect of lime stabilization on the age hardening properties of this four ashes can be also observed from Table 3.7 and Figure 3.10 to 3.13. The addition of a few percent by weight of hydrated lime increased the compressive strength of these compacted ashes more than 10-fold after four weeks of moist curing at 20°C. The optimum amount of lime

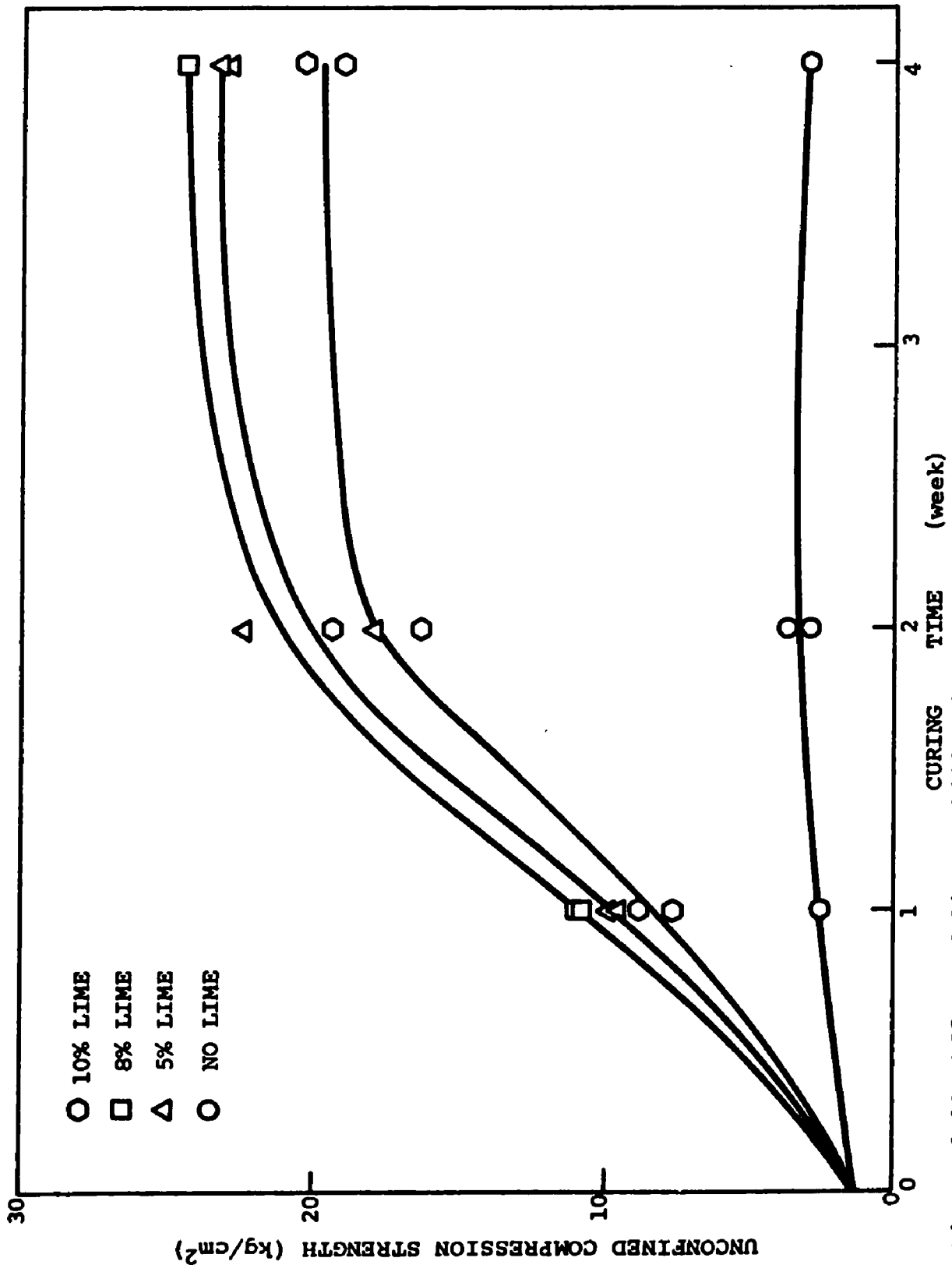


Figure 3.10 Effect of lime stabilization on the compressive strength of fly ash from D.E.Karn plant. (Samples cured at 20°C)

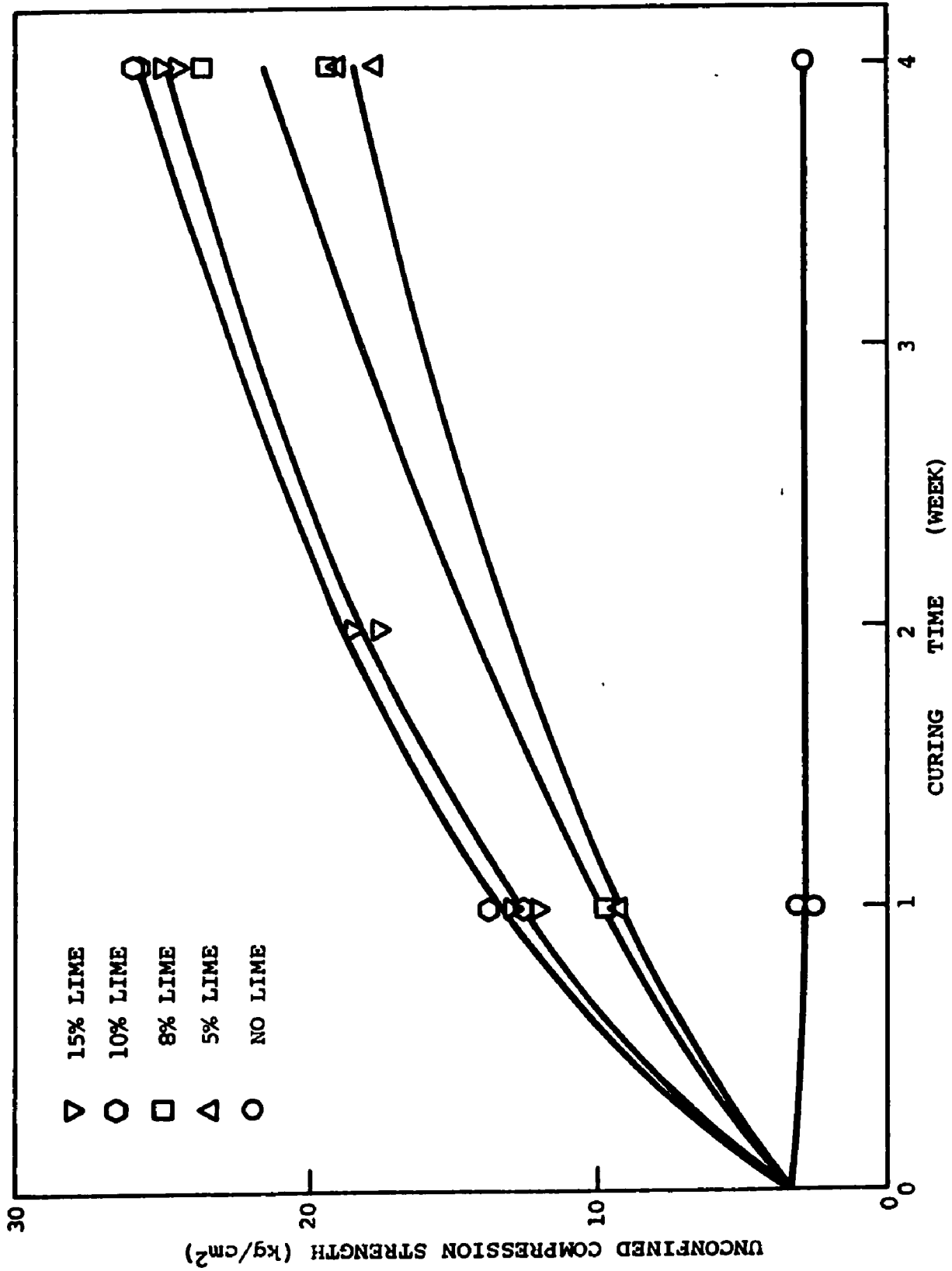


Figure 3.11 Effect of lime stabilization on the compressive strength of fly ash from Marysville plant. (Samples cured at 20°C)

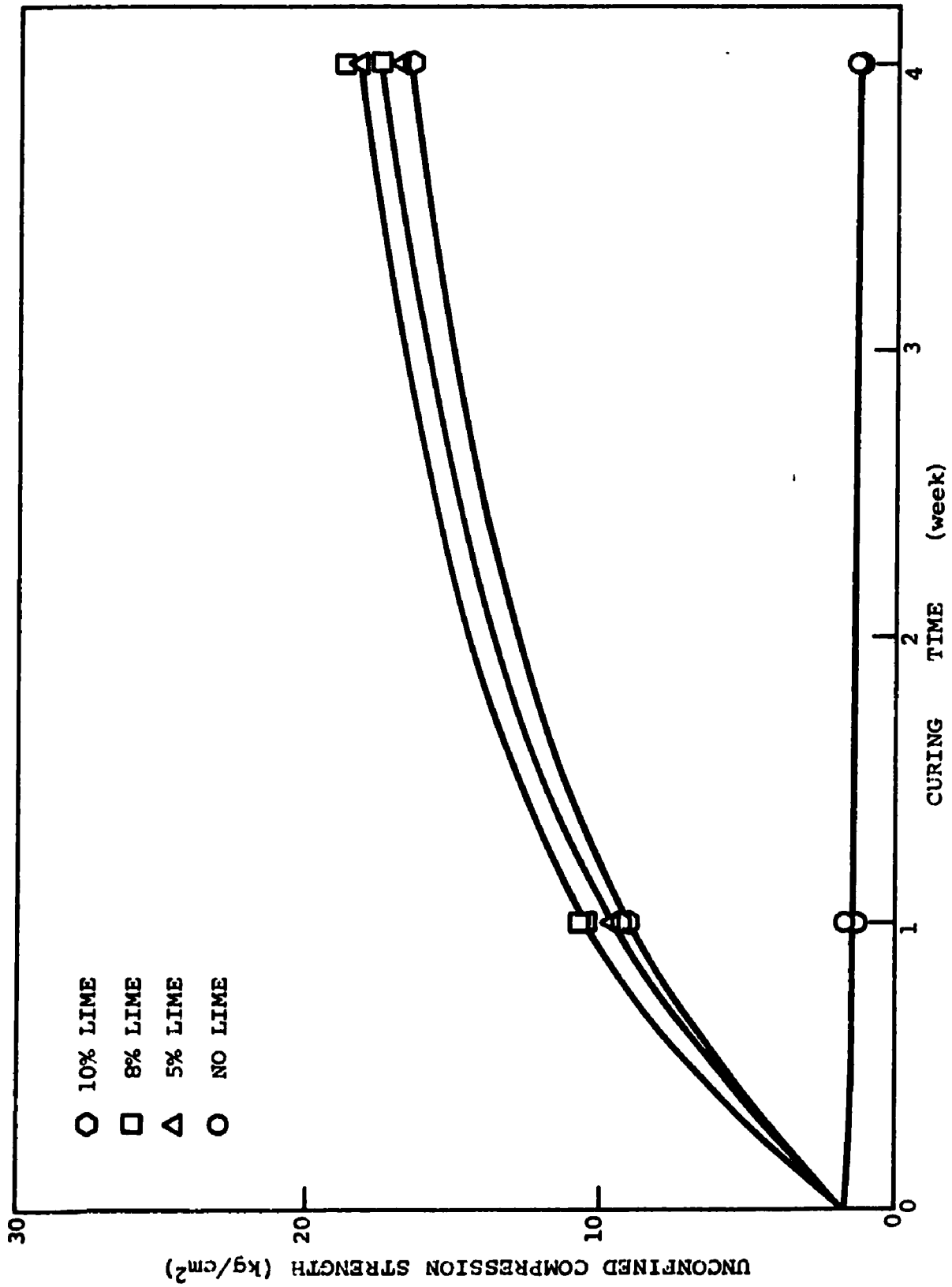


Figure 3.12 Effect of lime stabilization on the compressive strength of fly ash from St. Clair plant. (Samples cured at 20°C)

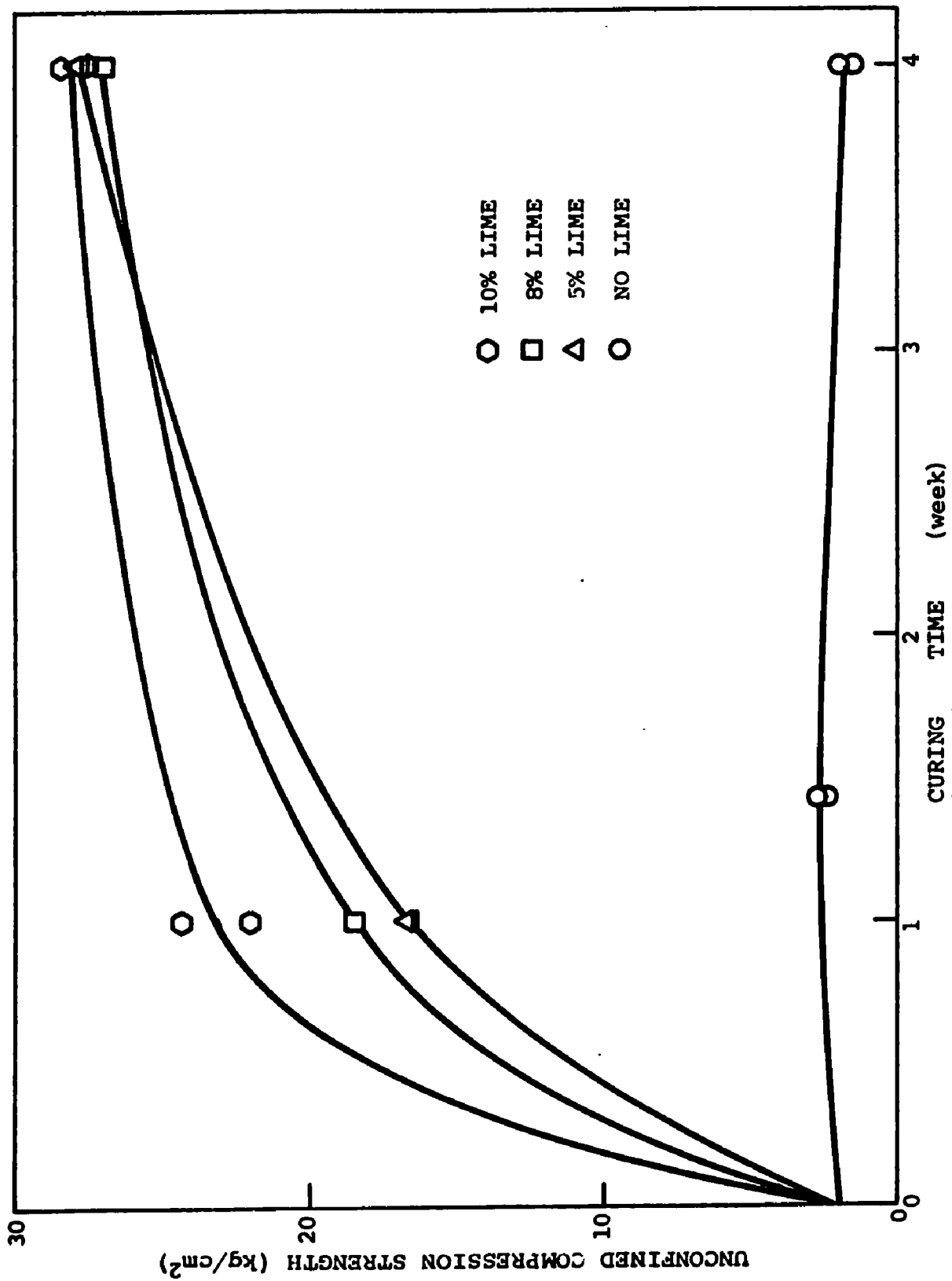


Figure 3.13 Effect of lime stabilization on the compressive strength of fly ash from Trenton Channel plant. (Samples cured at 20°C)

addition, defined as the amount of lime addition in a fly ash .. to reach the highest strength after 28 days of moist curing, varies with different ashes. It ranged from 7 to 13 percent for the Michigan fly ashes under study. The effect of hydrated lime content on the compressive strength of these four ashes after 28 days of curing is shown in Figure 3.14. Figure 3.15 shows a typical failure plane in a compacted sample of the Trenton Channel ash stabilized with optimum amount of lime and moist cured at 20°C for 28 days. The influence of lime stabilization on the age hardening of Trenton Channel fly ash in terms of penetration resistance was also studied. The same proportion of strength development was also observed. However, the optimum lime content appeared to be somewhat different from that for unconfined compressive strength. The stabilization affect on the penetration resistance of this fly ash is shown in Figure 3.16.

The shear strength parameters, viz., cohesion and the angle of internal friction, are expected to increase with lime stabilization because the curing would produce bonding between the stabilizer matrix and particles and also cause greater friction between particles of fly ash which after reaction with lime would have a rough surface texture. However, being limited by the capacity of the triaxial test facility available in the Department of Civil Engineering at the University of Michigan, the effect of lime stabilization in the shear strength parameters of compacted fly ashes has not been studied.

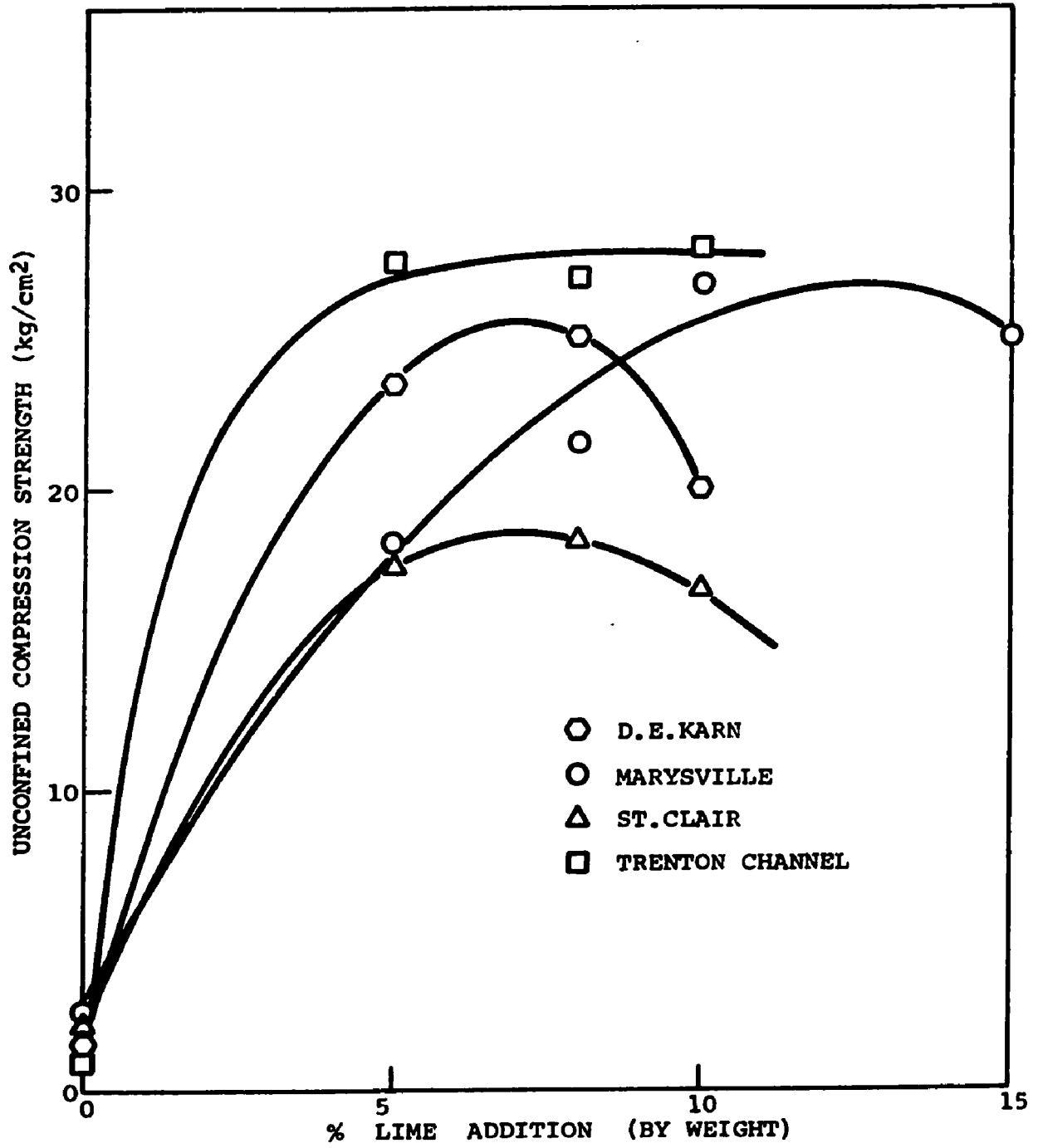


Figure 3.14 Effect of lime content on the compressive strength of four Michigan fly ashes after 28 days of curing. (Samples cured at 20°C)



Figure 3.15 The failure plane in a compacted sample stabilized with optimum lime content and moist cured at 20°C for 28 days. Trenton Channel fly ash.

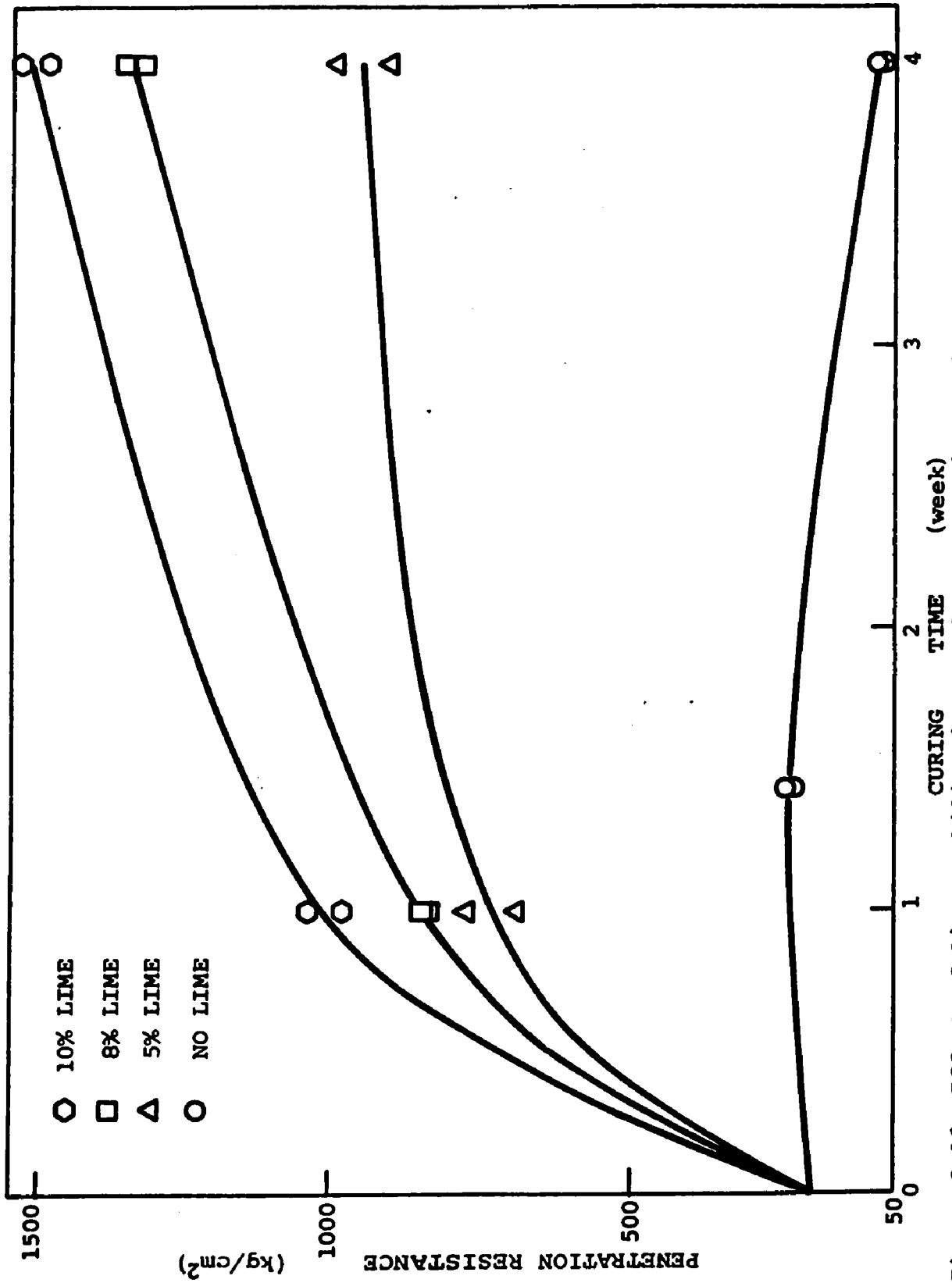


Figure 3.16 Effect of lime stabilization on the penetration resistance of fly ash from Trenton Channel plant. (Samples cured at 20°C)

Lime stabilized, compacted Michigan fly ashes appeared to be even more brittle than untreated samples. The strains at failure for samples compacted in the vicinity of optimum water contents were about 0.8 to 1.5 percent.

Strength requirements for compacted, stabilized base courses vary. Often quoted specifications are 400 psi at 7 days for cement treated materials and 550 psi at 28 days for lime stabilization. According to this specification for lime stabilization, none of Michigan ashes appear to be satisfactory. However, it is important to point out that the test results shown in Table 3.7 and Figures 3.10 through 3.11 were for samples cured at 20° C. The sensitivity of the rate and ultimate strength of lime treated fly ashes to curing temperature has been emphasized by many investigators; and was discussed in Section 3.2. The effect of variation in curing temperature on the compressive strength of some Michigan fly ashes was studied. The results are given in the next subsection.

3. Effect of Curing Temperature on the Strength of Compacted Fly Ash

Two Michigan fly ashes, namely, Trenton Channel fly ash and Marysville fly ash, were used to study the effect of variation in curing temperature on age hardening of fly ashes. The samples were prepared in the same manner as those cured at 20°C. However, they were only prepared with the optimum amount of lime or with no lime at all. After compaction, samples were placed in a desiccator which was stored in an oven with controlled temperatures. The temperature was set either

at 40°C or at 60°C. At the end of the curing periods (also 7, 14 or 28 days), samples were tested in unconfined compression in the Tinius-Olsen testing machine.

The results are tabulated in Table 3.9 and are also shown in Figures 3.17 and 3.18. It can be observed that not only the rate but also the ultimate magnitude of strength of lime treated, compacted fly ashes increased with increase in curing temperatures. The Trenton Channel fly ash treated with optimum amount of lime and cured at 60°C for 28 days exhibited strength of about 125 kg/cm², and the Marysville fly ash, about 100 kg/cm², which were approximately 4 times higher than those cured at 20°C. Even for those samples cured at 40°C the strengths increased more than three-fold. Therefore, it is important to note that a slight increase in curing temperature may cause a substantial rise in strength for treated fly ashes. It also implies that the favorable time for road base, fill or embankment construction using stabilized fly ashes is during the warmer seasons of the year.

One of the most practical problems in the field is the fluctuation of temperature. The effect of temperature fluctuation on the curing strength of treated fly ash mixtures has been studied by Sherwood and Ryley (1966). The strength of samples cured in a temperature cycle of 10°C and 30°C were reported higher than that of samples cured at a constant temperature of 20°C. The comparison of strength for these two cases are shown in Table 3.10. In practice the temperature of a stabilized fill or embankment is likely to fluctuate but it

TABLE 3.9 - Effect of Curing Temperature on the Compressive Strength of Treated and Untreated Fly Ashes from Trenton Channel Plant and Marysville Plant

Source		Unconfined compression strength (kg/cm ²)									
		No lime addition					Added with Optimum lime*				
		20°C		60°C		20°C		40°C		60°C	
		1+	2	1	2	1	2	1	2	1	2
Trenton Channel	++ 7 (10) 14	2.74	2.34	3.80	3.47	18.50		25.20	32.30	95.30	94.50
				3.98	3.68			73.90	74.90	96.00	86.30
	28	2.05	1.52	3.69	3.69	27.00		95.70	96.20	132.50	122.00
Marysville	7	2.57	3.12	4.56	4.75	13.20**		42.00	42.20	77.50	77.70
	14			4.44	5.07	18.80**		84.00	86.50	87.60	87.40
	28	2.84		4.84	5.09	25.80**		107.80	117.00	99.20	94.60

* 8% for Trenton Channel fly ash; 13% for Marysville fly ash

** Extrapolated from Figure 3.11 and Figure 3.14

+ Test number

++ Curing period

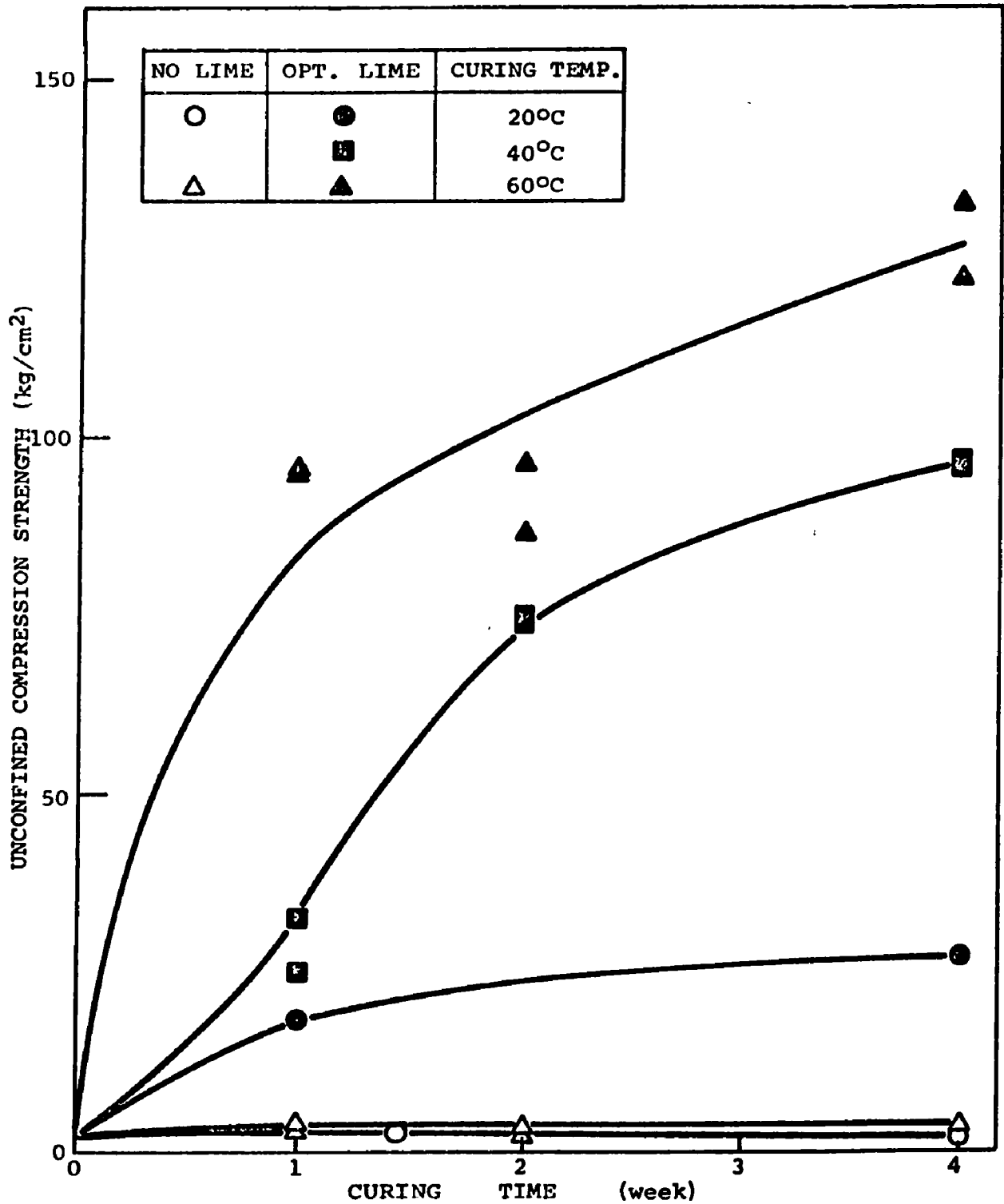


Figure 3.17 Effect of curing temperature on the compressive strength of treated and untreated fly ashes from Trenton Channel plant.

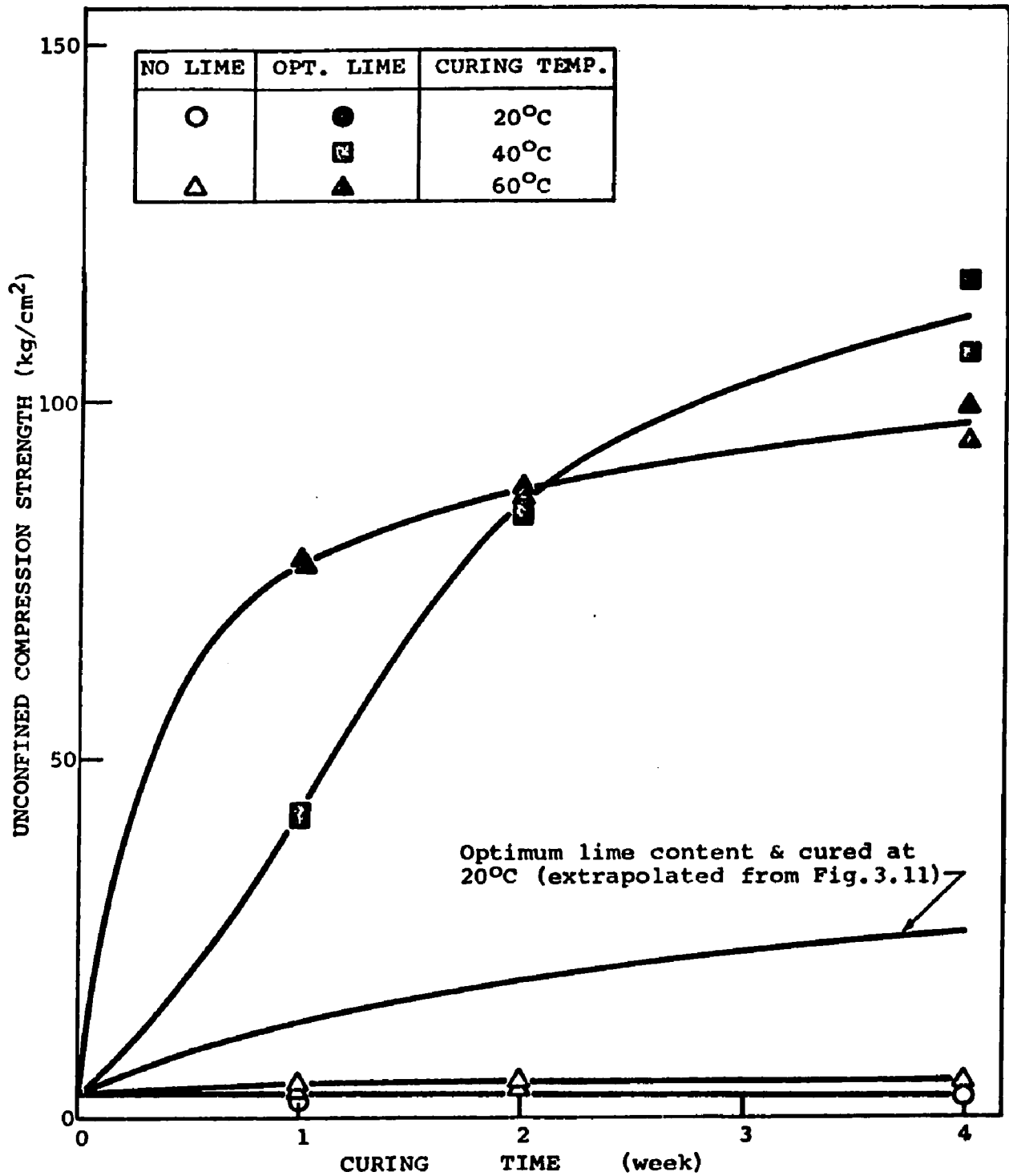


Figure 3.18 Effect of curing temperature on the compressive strength of treated and untreated fly ashes from Marysville plant.

seems that the fluctuation of temperature is not deleterious.

TABLE 3.10 - Comparison of the Effect of the Strength of Lime-Ash Stabilized Sands of Curing at a Constant Temperature of 20° and Curing in a Temperature Cycle of 10°C and 30°C

Batch No.	Unconfined compressive strength (lb/in. ²) after:			
	7 days		28 days	
	20°C	10/30°C	20°C	10/30°C
A	100	180	1060	1310
B	230	510	1165	1340
C	130	280	1080	1370
D	85	170	890	1360
E	140	300	1070	1340
F	105	260	965	1220
G	130	275	1050	1230
H	110	230	940	1150
J	110	240	900	1200
K	75	140	845	1110

(From Sherwood and Ryley, 1966)

Figure 3.18 shows that the strength for the treated Marysville fly ash cured at temperature of 60°C is lower than that cured at 40°C after 28 days of curing. One of the common explanations, Sherwood and Ryley (1966), for strength reduction in stabilized soil with high alumina content is that lime first reacts with alumina to form the compounds $\text{CaO} \cdot \text{Al}_2\text{O}_3 \cdot 10\text{H}_2\text{O}$ and $2\text{CaO} \cdot \text{Al}_2\text{O}_3 \cdot 8\text{H}_2\text{O}$ which are known to change into the compound $3\text{CaO} \cdot \text{Al}_2\text{O}_3 \cdot 6\text{H}_2\text{O}$ on heating. This compound has weaker cementing

properties and there is a consequent reduction in strength. However, the alumina content in Marysville fly ash does not appear much different from other Michigan fly ashes (see Table 3.3), thus the strength reduction might have been merely due to the insufficient moisture supply during curing.

The untreated samples of both Trenton Channel fly ash and Marysville fly ash did not exhibit any significant response in strength to the variation in curing temperature. This seems self-evident since temperature causes strength-increase in fly ash by accelerating the pozzolanic reaction, and none of the Michigan fly ashes appeared to possess the potential of pozzolanic activity without external lime addition.

CHAPTER 4

FROST SUSCEPTIBILITY OF COMPACTED FLY ASH

One of the engineering properties of compacted fly ash that has been an obstacle to the widespread use of this material in load-bearing fill is frost susceptibility. Because it contains predominantly silt size grains, this material is generally considered liable to frost heave.

The mechanism of frost heave was first studied by Taber (1916), who suggested that the migration of water in the soil to the freezing front, thereby forming ice lenses, is the cause of frost heave. This hypothesis was further expounded by Beskow (1935) and is still being used as the basic concept of the frost heave mechanism. The mechanical properties of frost action have been studied by a number of investigators since then. However, most of the studies have been relatively specific. For example, Tsytoovich (1957) studied the relationship between frost heave and unfrozen water content, frost penetration, dry density of soil and velocity of water migration. Williams (1964) investigated the relation between suction and the initial freezing point, temperature and unfrozen water content. He also proposed the use of air-intrusion value as a criterion of frost susceptibility (1966). Freden (1965)

related frost heave to the capillary potentials and the temperature gradient. Osler (1967) examined the relation between frost heave and the pore size of the soil. Sutherland and Gaskin (1967) demonstrated the use of the strength and the permeability as the indication of the frost heave potential. Comprehensive reviews and studies of frost action have been made by Penner (1959), Osler (1967), and Hoekstra (1969).

In order to provide some understanding of the effect of stabilization of frost heave, the mechanism of frost heave is summarized briefly in the following section.

4.1. Summary of Frost Heave Mechanism

There are two major potentials that affect water in soils, namely, the matric potential and the osmotic potential (Osler, 1967). The matric potential is caused by the adsorptive forces and the surface tension while the osmotic potential is due to the differences in the ion concentration between the bound water and the bulk water.

When a freezing front is advancing through a soil-water-air system, the ice front will be restricted by the soil pores and forced to form a curved face to advance into the pore (see Figure 4.1). This will cause a freezing temperature depression according to the Thompson equation (Penner, 1959, Osler, 1967):

$$\Delta T = - \frac{2 \sigma_{iw} T}{\rho_i L_h r}$$

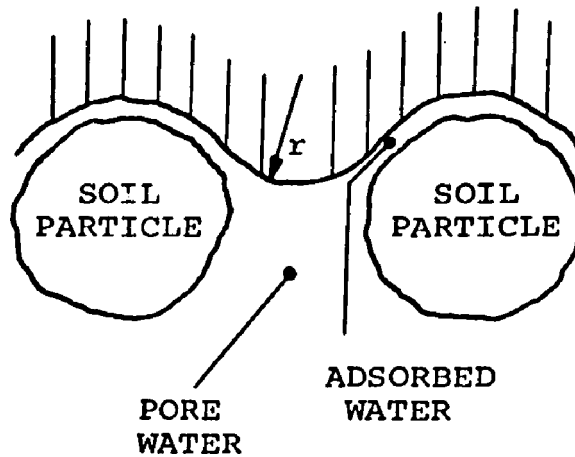


Figure 4.1 Curvature of ice-water interface.

where ΔT = freezing point depression, K
 σ_{iw} = ice-water interfacial energy, ergs/cm²
 T = freezing temperature at an interface without curvature, K
 ρ_i = density of ice, gms/cm³
 L_h = latent heat of fusion, ergs/gm
 r = radius of curved surface, cm.

Therefore, the crystallization of ice at the freezing front will be halted temporarily until the temperature reaches the depressed freezing point. Due to the effect of suction, or capillary potential, the adsorbed water closer to the particle surfaces will have a lower freezing point than that of bulk water. Thus in these regions, unfrozen water can exist in contact with the ice front.

During the process of ice crystallization, water is continuously removed near the ice front. Thus water acquired by

the surface film near the front must be constantly drawn from the warmer side. The water is transported to this region by means of diffusion. According to Hoekstra (1969), the freezing point depression can be converted into a driving force, in terms of an equivalent osmotic pressure π , for water diffusion by the equation

$$\pi = \frac{L_h}{T \Delta V_m} \Delta T$$

where ΔV_m is the difference in partial molar volume between ice and water. By this suction, water is constantly transported to the ice front and the ice crystalization proceeds continuously provided that the temperature gradient is sufficient and other necessary conditions, such as permeability, are satisfied.

The above discussion of the mechanism of frost action illustrates that the major factors which are essential to the frost heave are the below-freezing temperature, the accessibility of moisture, and the susceptibility of the soil. Generally little control can be exercised on the first two factors, at least within the usual economic limitations; on the other hand, the alteration of soil properties to prevent or eliminate heaving is a possible solution. The soil properties that are related to susceptibility are pore size, grain size, mineralogical composition, permeability and strength. The pore size constricts the advancing ice front; this causes freezing point depression and affects suction energy. Grain size and mineralogical composition influence the electrical charge and this

likewise affects the soil suction (Osler, 1967). The permeability affects the rate of flow of water to the freezing front in response to the suction. Strength resists the expansive forces arising from frost action.

Among these factors, there are certain combinations of soil properties or conditions which lead to the most favorable frost heave conditions. For example, the suction is far greater in clays than in silts, but the much lower permeability of clays tends to limit the supply of the unfrozen water to the freezing front. On the other hand, sands have much higher values of permeability but have low matric potentials which draw much less unfrozen water to the ice front. Besides, the pore size constriction to the ice front advancement is not so significant in sands and the freezing front often advances uniformly without the growth of ice lenses. Silts have the right combination of suction and permeability to cause the most significant frost effects.

4.2 Physical Effects of Stabilization on Frost Heave

Several methods have been used to deal with the problem of frost heaving; they include thermal insulation, water cut-off, surcharging, and soil stabilization. The last named includes mechanical blending and chemical admixtures. This technique not only tends to reduce frost susceptibility, it also increases strength and reduces compressibility. Based on information reported in the literature (Raymond, 1961; Croney and Jacobs, 1967), the addition of low levels ($\leq 10\%$ by weight) of either

Portland cement or lime appears to be the most promising and practical approach in this regard.

The physical effects of stabilization involve:

1. Increasing strength

An experimental study on some British fly ashes has been made by Sutherland and Gaskin (1970). The authors reported that the frost heave decreased as the curing strength increased. This may be due to the pozzolanic activity between additives and ashes which creates binding among ash particles. This cementing strength resists internal expansive forces and thus prevents the ash from heaving.

2. Lowering permeability

During stabilization, the additives fill the pores and aggregate the particles of the ash; therefore, the flow of unfrozen water to the ice front is restricted and the heaving process is retarded. The effect of stabilization on the permeability of some British ashes has been also reported by Sutherland and Gaskin (1970). Figure 4.2 shows the effect of lime and cement additions on the permeability of these compacted, British fly ashes. It can be seen that the addition of 10% cement by weight decreases the permeability by an order of magnitude in one week. Lime treatment also achieved substantially the same results.

The above discussion enables us to distinguish those materials that might be frost susceptible, the favorable conditions for frost heaving, and the effect of stabilization on

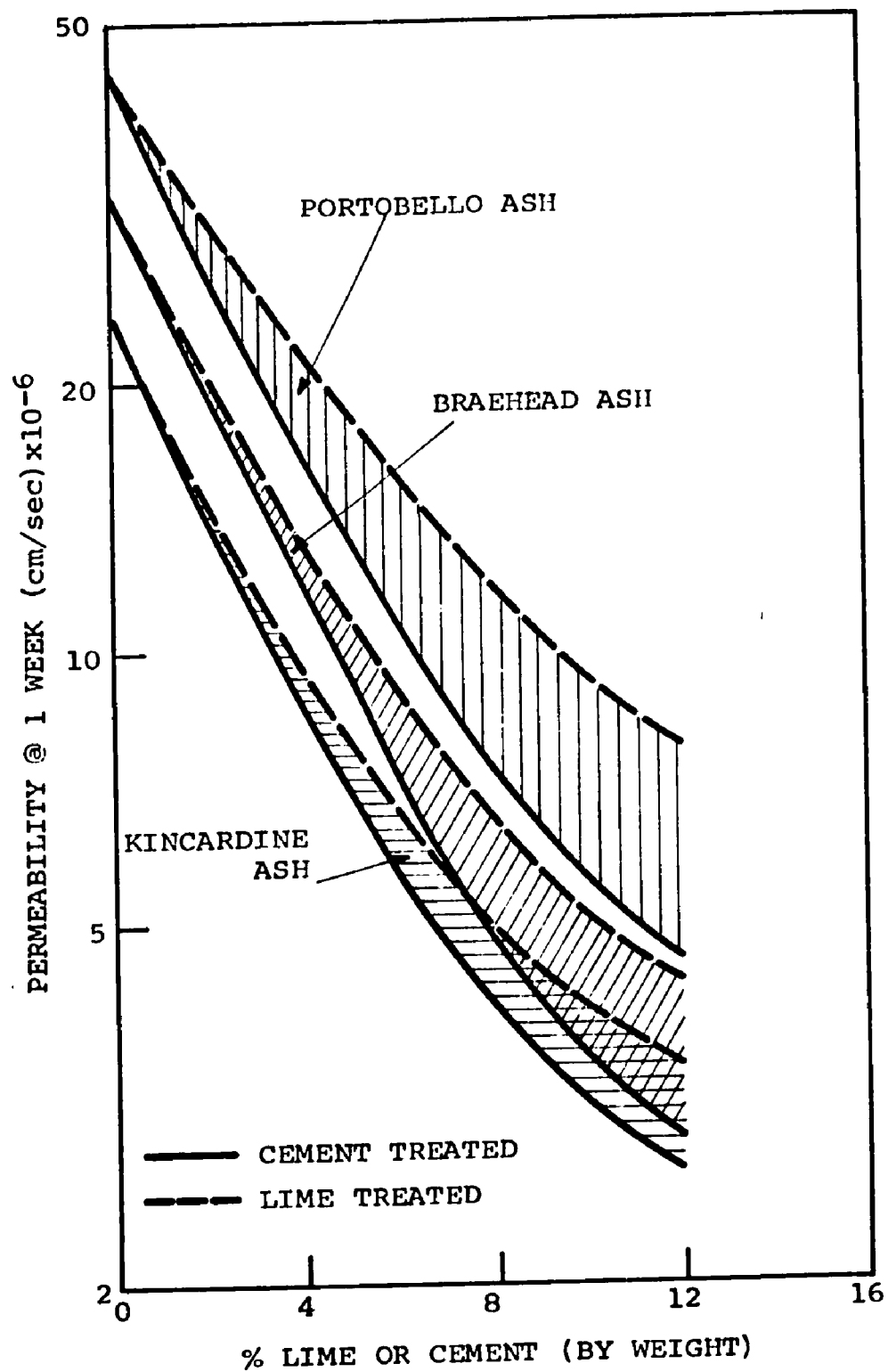


Figure 4.2 Effect of lime and cement additions on the permeability of some compacted, British fly ashes (from Sutherland & Gaskin, 1970)

a soil. A practicing engineer, however, is faced with the need for a reliable test for assessing the frost susceptibility of a proposed construction material — be it soil or fly ash. Evidence reported in the literature (Croney and Jacobs, 1967; Sutherland and Gaskin, 1970) suggest that conventional criteria of frost susceptibility (e.g., grain size distribution) do not always apply in the case of compacted fly ashes. The best approach appears to be a test which measures the extent of heave under simulated freezing conditions in the field. A simple test procedure based on this approach has been developed by Croney and Jacobs (1967) at the British Road Research laboratory. This test makes it possible to identify frost susceptible materials and to assess the effectiveness of various stabilization treatments in reducing frost heave to tolerable levels.

4.3 Test Procedure and Results

The test adopted at the University of Michigan is based on the experiments performed by Croney and Jacobs (1967). Compacted fly ash samples with different treatments are frozen from the top and given access to free water at the bottom in order to simulate actual freezing conditions in the field. The set-up was designed and built by Professor D.H. Gray and Mr. In-kuin Kim.

1. Frost Heave Cabinet

A freezing cabinet was modified to accommodate eight samples which are frozen from the top while the bottom part

are kept unfrozen. This was achieved by installing a heating coil in the water with which the samples were in contact. The temperature of the water was maintained at $+2^{\circ}\text{C}$ by a temperature controller. A magnetic stirrer was placed in the water to insure uniform water temperature. The specimens were placed in brass specimen carriers which fit into the holes on a wooden board that was located above the water. The space between the samples was filled with styro foam to eliminate radial temperature gradients in the samples. Cover disks with a vertical rod attached were placed on the top of each sample. The displacements of the rods, equivalent to heave of the samples, were measured with respect to a fixed reference bar by a micrometer depth gage. The schematic drawing of this set-up is shown in Figure 4.3 and the installation of the sample in the cabinet is shown in Figure 4.4.

2. Sample Preparation and Test procedure

Two sources of fly ashes, Trenton Channel plant and Marysville plant, have been tested. Eight specimens with four different treatments were prepared in duplicate each time. Duplicated specimens were compacted to their maximum dry densities at the optimum water contents either with 5% cement, 10% cement, 10% lime (by weight) or without additives. Then they were moist cured at 20°C for two weeks before being installed in the freezing cabinet. The modified AASHTO compaction method was used and the size of the specimens was 4 inches in diameter and 4.6 inches in height.

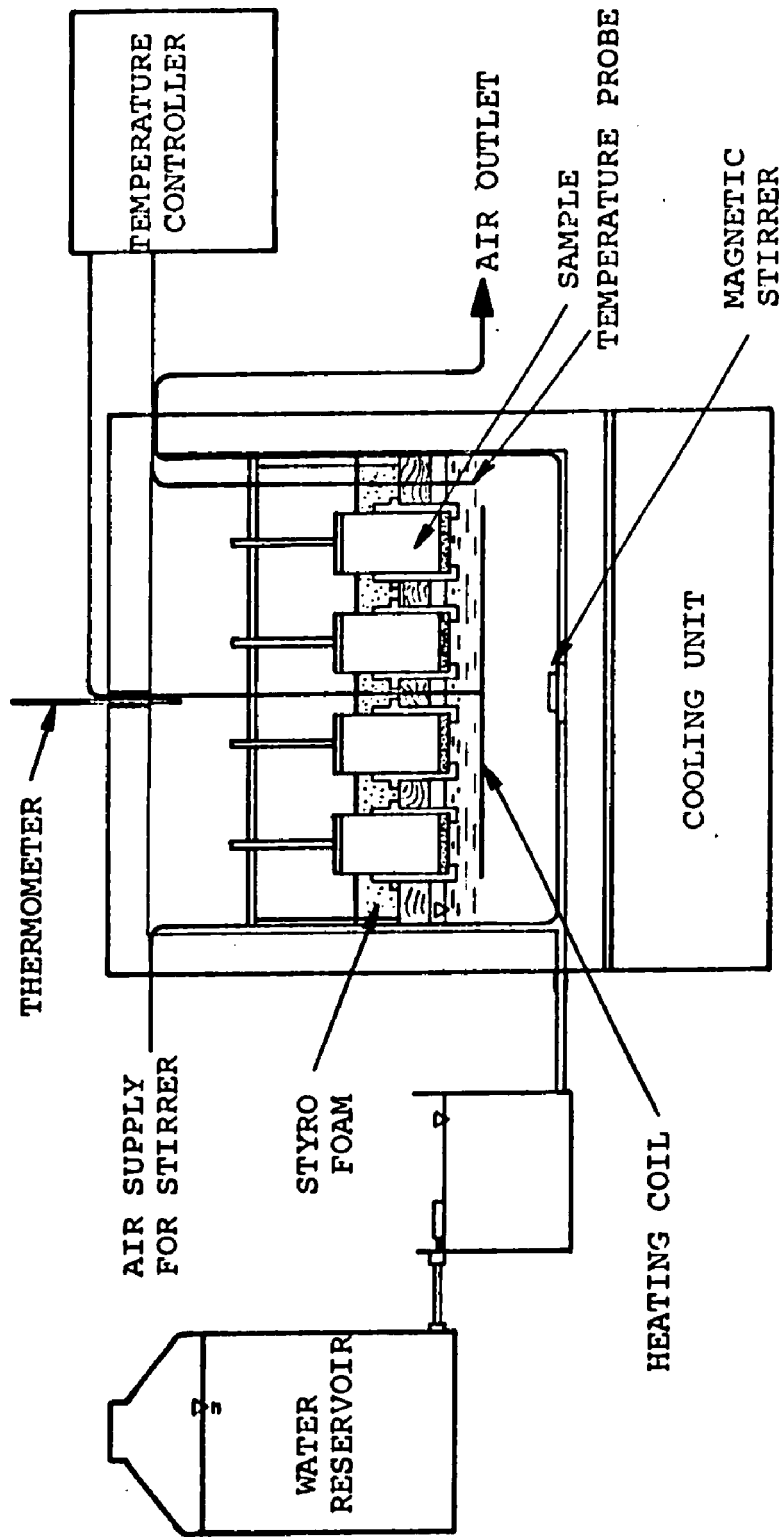


Figure 4.3 Schematic drawing of the set-up for frost heave tests.



Figure 4.4 Sample installation in the freezing cabinet.

The samples were placed in the cabinet in which the air temperature ranged between about -6°C and -9.5°C ; the temperature of the unfrozen water was set at $+2^{\circ}\text{C}$. Heave readings were taken once every 24 hours and the test lasted for 10 days.

3. Results

Figure 4.5 and Figure 4.6 show the results of the frost heave tests on these two ashes. The effect of various additions of lime and Portland cement on the frost heave characteristics is illustrated. The amount of heave was reduced to less than 7% with 5% of cement addition. This amount of cement rendered the Marysville fly ash completely non-frost susceptible. The samples treated with lime did not appear to reduce heaving substantially. This may be due to the lower rate of pozzolanic activity which failed to reduce permeability enough to limit the unfrozen moisture supply and which did not develop sufficient cementing strength in time to resist heaving pressure. A longer curing time seems necessary for lime-treated fly ash.

The variation in frost heaving of four samples with different stabilization treatments is also illustrated photographically in Figure 4.7. The formation of ice lense in an untreated sample is shown quite clearly in Figure 4.8.

4. Conclusions

From the results discussed in the previous subsection, it can be concluded that unstabilized Michigan fly ashes tend to be frost susceptible as might be predicted from their grain size distribution. They do not contain sufficient amounts of free lime to provide age hardening reactions which would tend

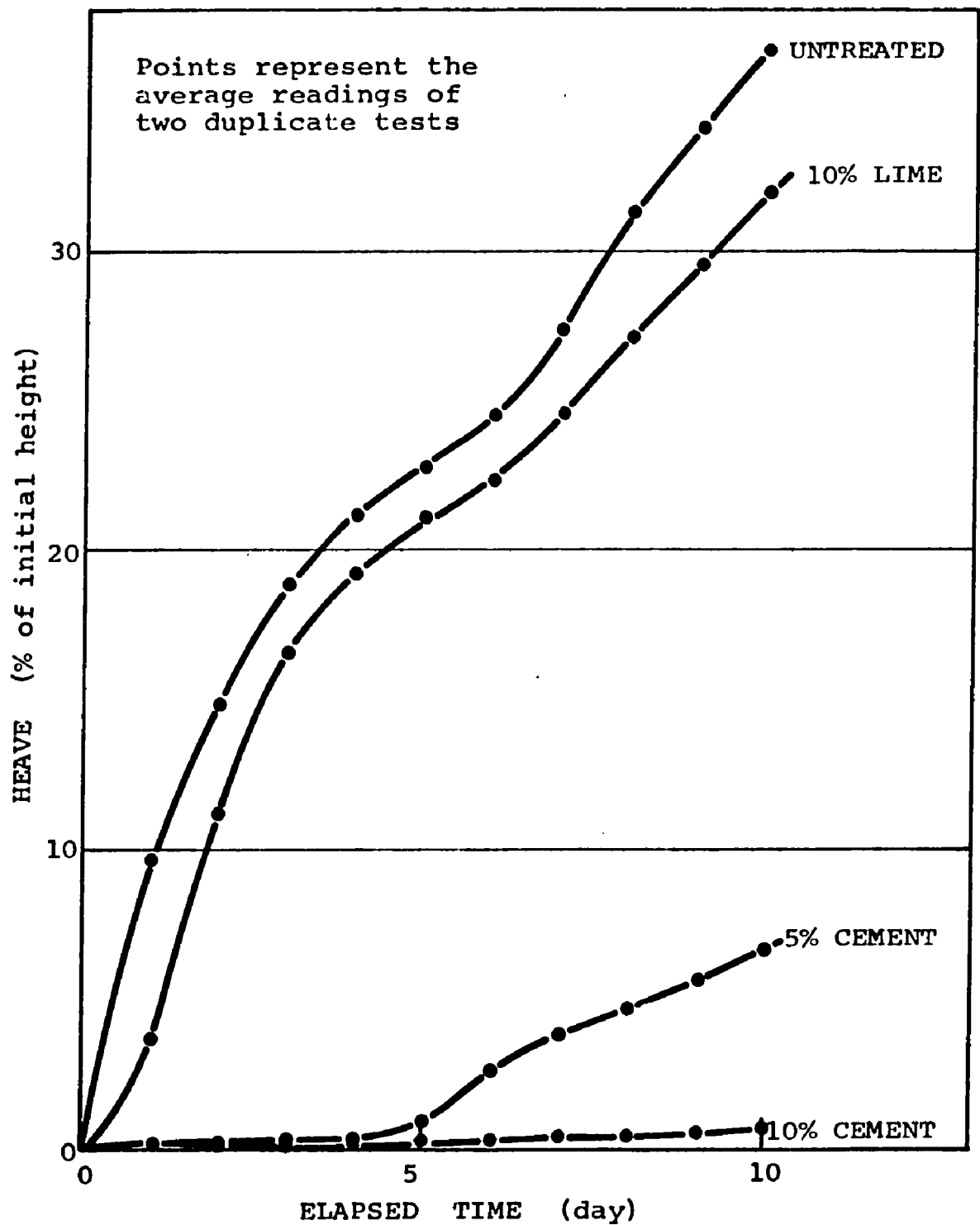


Figure 4.5 Effects of various additions of lime and cement on the frost heave of fly ash from Trenton Channel plant.

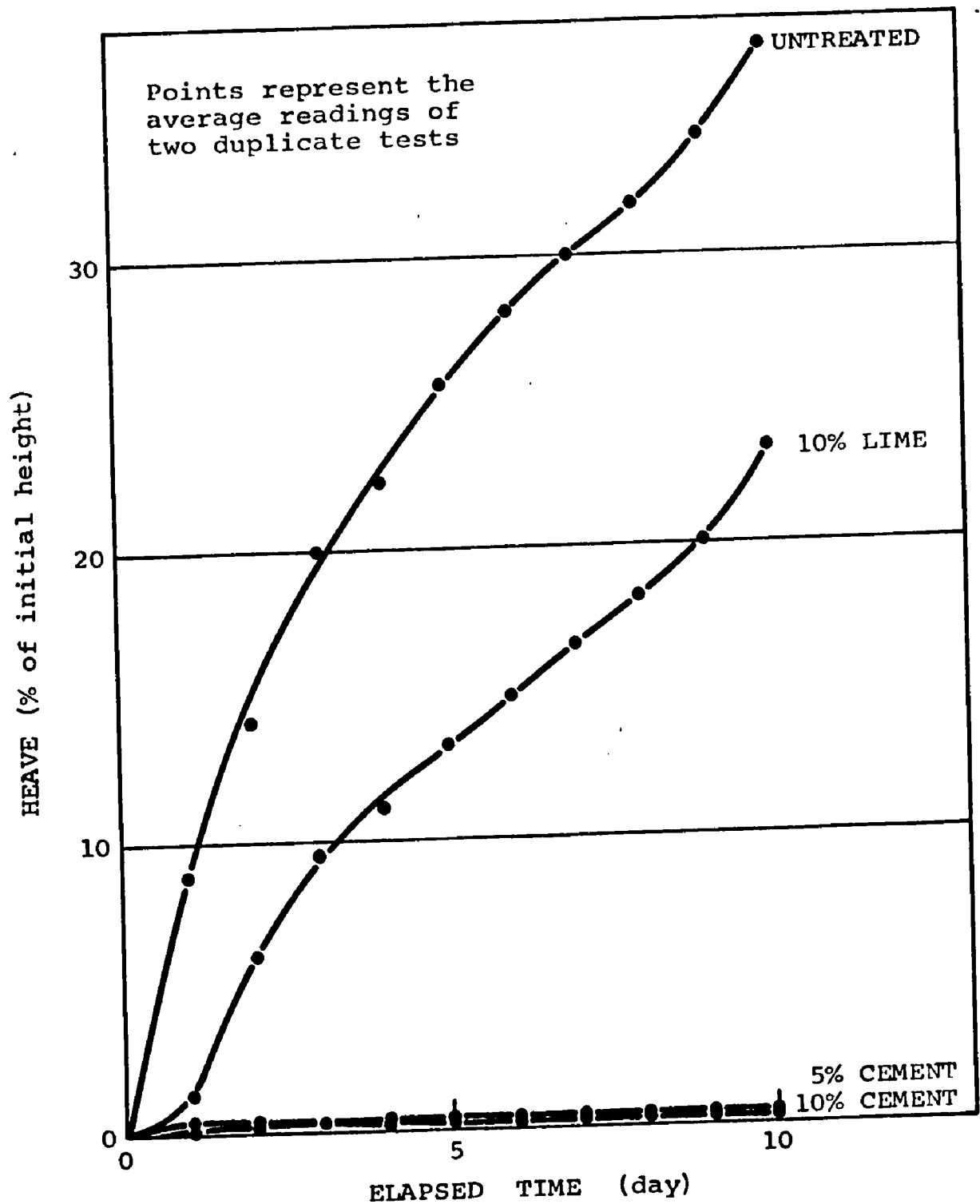


Figure 4.6 Effects of various additions of lime and cement on the frost heave of fly ash from Marysville plant.

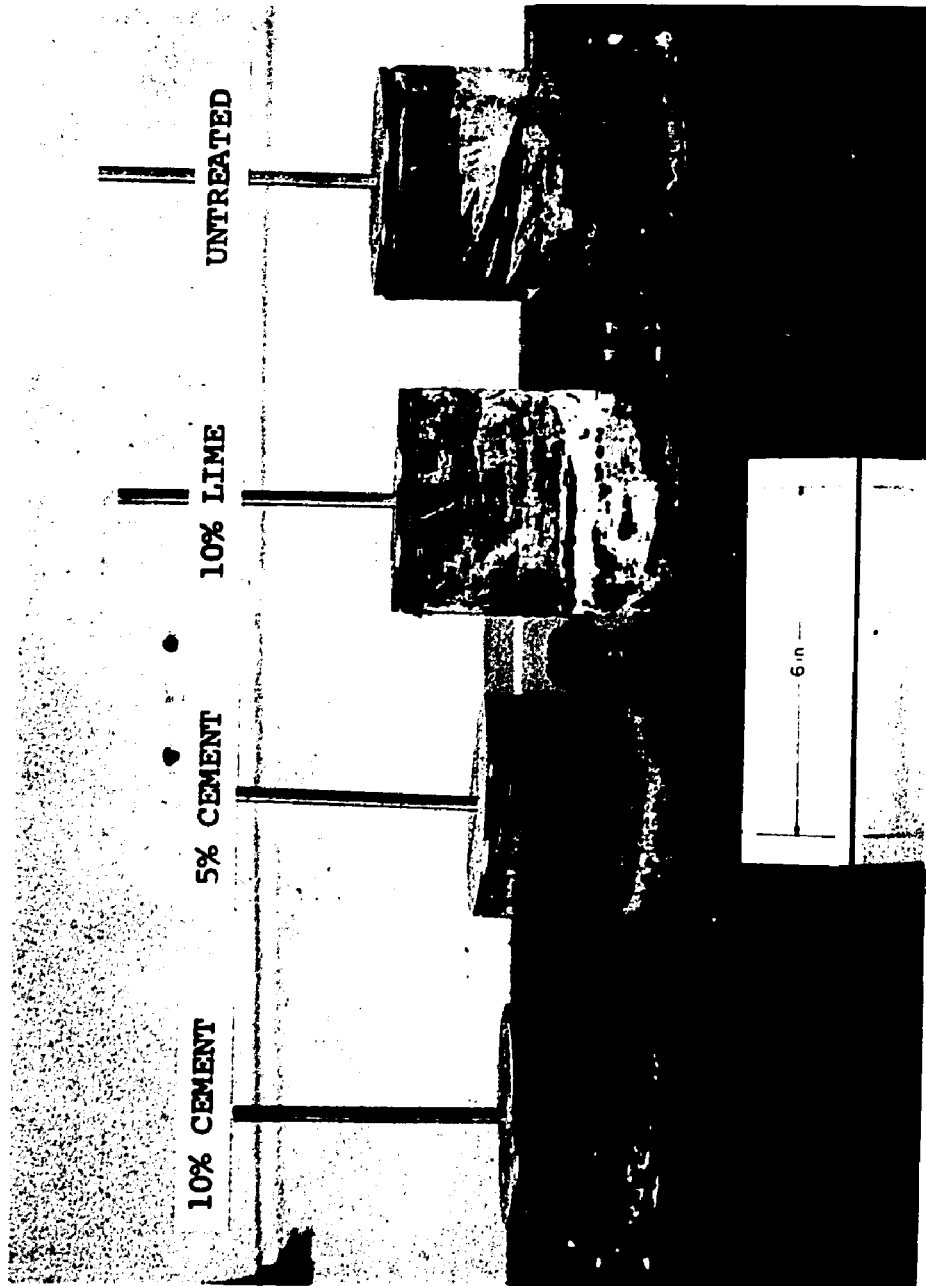


Figure 4.7 The variation in frost heave of four fly ash samples from Trenton Channel plant in response to different stabilization treatments.

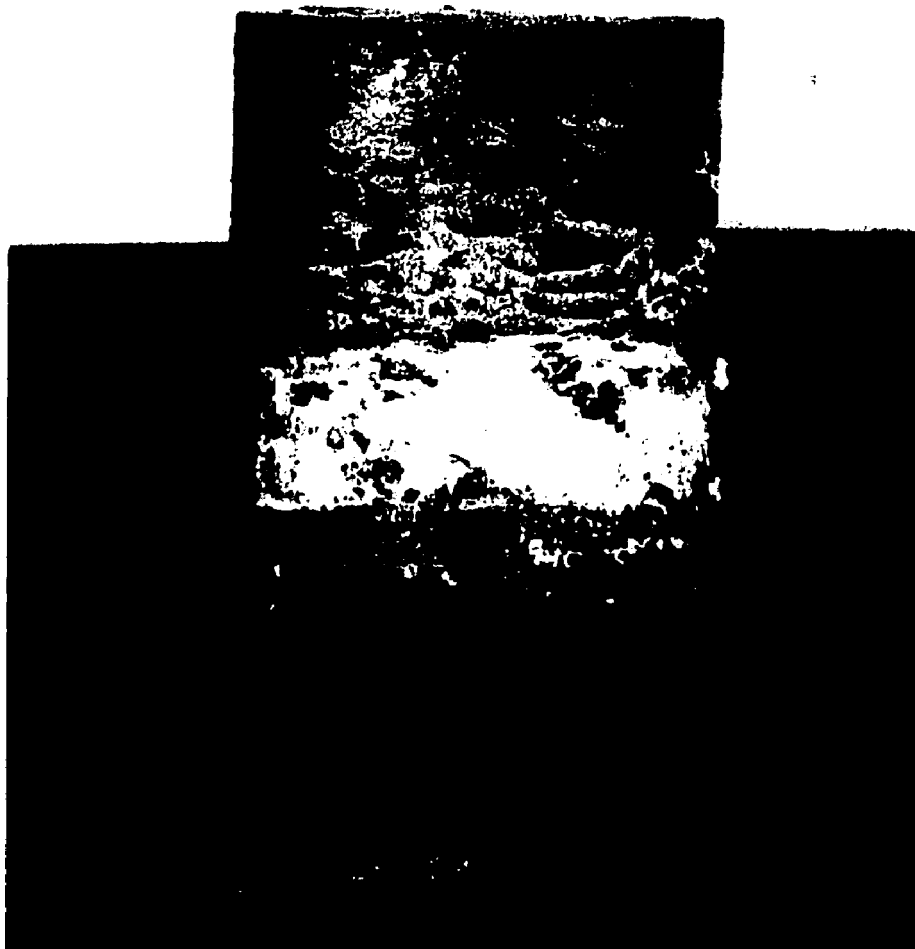


Figure 4.8 The formation of ice lense in an untreated sample of Trenton Channel fly ash after 10 days of frost heave test.

to limit frost heave. Nevertheless, heave can be reduced to a tolerable level by the addition of only a few percent by weight of Portland cement.

The fact that lime-treatment at similar levels failed to reduce heaving as much may be due to the lower rate of pozzolanic activity. Longer curing time or higher curing temperatures appears to be necessary for satisfactory performance of lime-stabilized fly ashes.

CHAPTER 5

A RHEOLOGICAL STUDY OF THE COMPRESSIBILITY OF COMPACTED FLY ASH

5.1. Introduction

One of the main problems which has been encountered in the use of compacted fly ash as a construction material is the prediction of settlements.

Field experience in England has shown that actual settlements are usually well within tolerable limits. The difficulty appears to lie in the prediction of field settlements. Margason and Cross (1966) reported on the use of compacted fly ash as the backfill behind an abutment. They found that the settlement of fly ash fill at this abutment was smaller than that observed with a coarse, granular fill. Furthermore, because of the low dry unit weight of compacted fly ash, the settlement beneath a compacted fly ash embankment was also considerably less than that beneath a conventional granular fill emplaced over the same ground. The comparison of settlements both on the surface and beneath the embankments of a conventional earthfill and a compacted fly ash fill is shown in Figure 5.1. Raymond and Smith (1966) studied a fly ash fill at the Agecroft power station and showed that the settlements computed from laboratory consolidation tests did

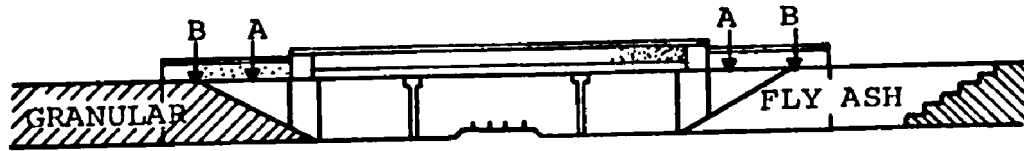
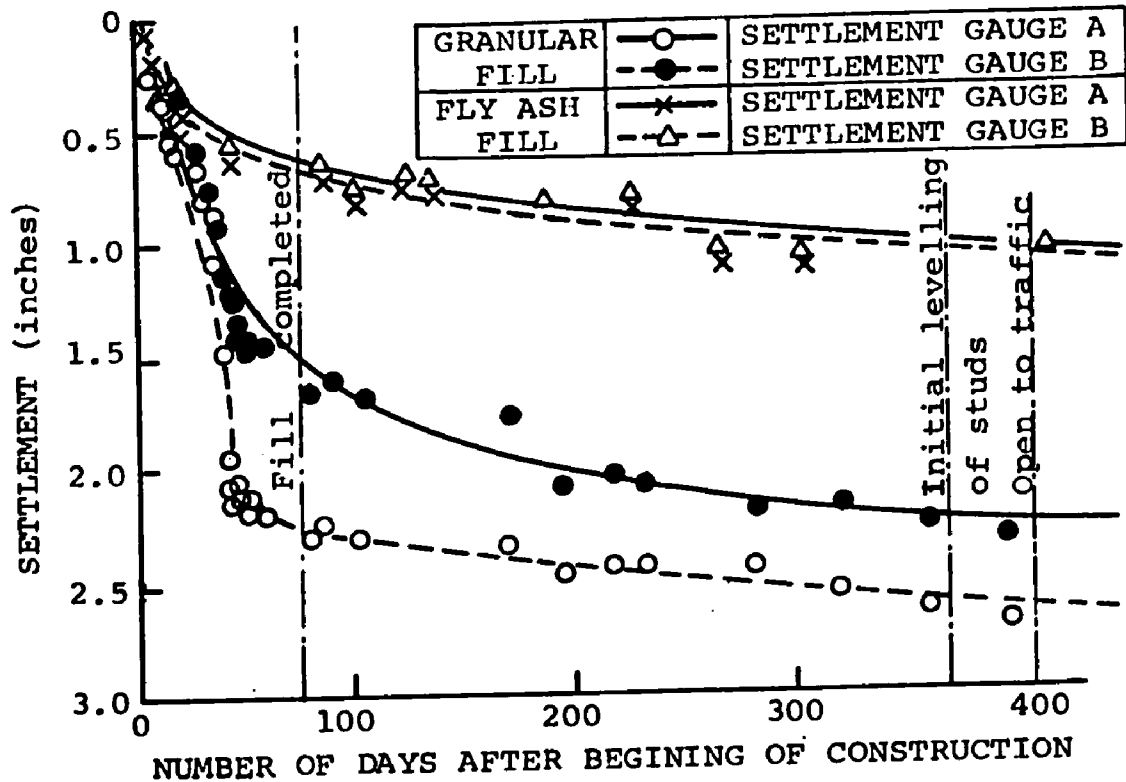
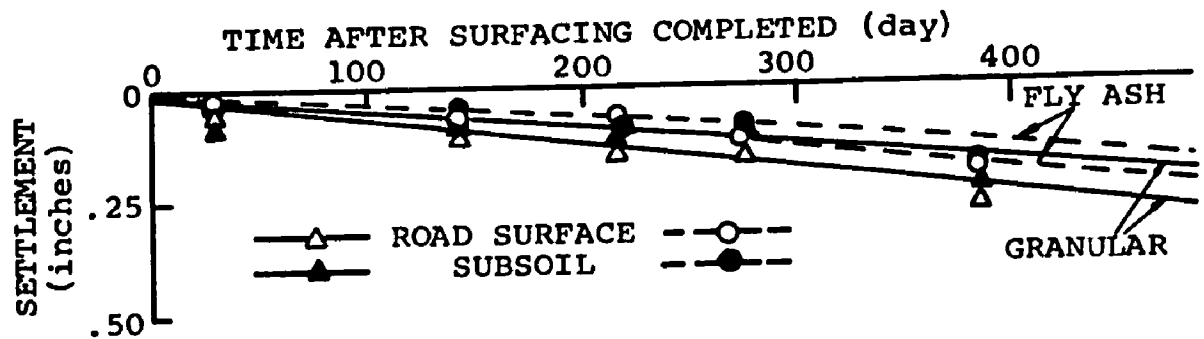
ELEVATIONSETTLEMENT OF SUBSOILSETTLEMENT OF ROAD SURFACE AND SUBSOIL

Figure 5.1 The comparison of settlements both on road surface and beneath the embankments of a conventional earthfill and a compacted fly ash fill, RR bridge overpass, Staines, England (from Margason & Cross, 1966)

not correlate with observations made on actual foundations resting on the compacted ash. The actual settlements were about half an inch but the computed settlements based on the results of laboratory consolidation tests were in the range of 12 to 16 inches. This deviation could be due to improper extrapolation of laboratory consolidation results to the field conditions. Unlike regular soils, the compressibility behavior of fly ash is affected by an additional time dependent component caused by its pozzolanic properties. As a result, its compressibility and predicted field settlements are likely to differ considerably from those of conventional soils. Besides, the compacted fills in most occasions are likely to be partially saturated. The error in settlement calculations can be also caused by failing to take into account the saturation condition in fills.

The viscoelastic properties of soils have been studied theoretically and experimentally by a number of investigators, for example, Schiffman (1959), Murayama and Shibata (1961), Lo (1961), Tan (1964), Wu and Neukrichner (1966), Barden (1969) and Šuklje (1969). However, the study of stress-deformation-time relation of stabilized soils including the consideration of aging effects caused by intrinsic pozzolanic activity does not seem to have been attempted. In the literature, the aging-viscoelastic response has been studied either by using a differential equation or an integral type of constitutive equation. A limited application of the viscoelastic theory for aging materials has been made to the aging behavior of

of concrete. The employment of this theory to the volume change characteristics of aging soils has not been reported in the literature.

The volume change processes of compacted fly ashes are further complicated by the effect of partial saturation. The volumetric behavior of a partially saturated soil appears to be different from that in fully saturated condition; moreover, the complex nature of the physical properties of a three-phase material makes an accurate description of the compressibility behavior rather difficult. During the past two decades considerable work has been done to explain shear strength and volume change behavior of partially saturated soils in terms of effective stresses, e.g., Bishop (1960). However, it has been found that the principle of effective stress alone is not adequate to account for the volumetric behavior of soils in this condition; the variation in saturation condition of the soil and the corresponding variation in stress components can be equally important.

In view of these problems discussed in the preceding paragraphs, the objective of the study reported in this chapter is broken down into three parts; (1) to examine experimentally the effect of aging, distinguished as a separate variable from the time-dependent deformation of an ordinary soil, on the compressibility of the stabilized fly ash, (2) to employ the existing viscoelastic theory for aging material to interpret the volumetric behavior of the stabilized fly ash, (3) to study experimentally the effects of saturation,

and change in stress components on the volume change properties of partially saturated fly ash.

An experimental investigation of the aging effect on the compressibility of compacted fly ashes is described in the first part of this chapter. A review and the extension of viscoelastic stress-strain law for aging materials under one-dimensional strain condition are presented next, and the use of this equation to construct a theoretical void ratio vs. applied pressure relation is attempted. A study of the effect of partial saturation on the volume change characteristics of compacted fly ash with aging property is also made and presented in the last part of this chapter.

Unlike ordinary soils, the rheological behavior of aging materials, e.g., lime-stabilized fly ash, is complicated by the age hardening process itself. Therefore, the effects of material aging should be always taken into account in examining the volumetric behavior of a stabilized material such as fly ash.

5.2 Thixotropy, Creep and Aging Processes

To deal with the rheological behavior of pozzolanic materials, the mechanism of thixotropy, creep and aging processes should be precisely distinguished.

"Thixotropy" has been defined by Burgers and Scott-Blair (1948) as a "process of softening caused by remolding, followed by a time-dependent return to the original harder state." The following fundamental aspects of thixotropy need to be emphasized:

1. It is a strength re-gain process due to the natural response of a soil structure to a change in ambient conditions. According to the hypothesis advanced by Mitchell (1960), the internal energy in a thixotropic soil immediately after remolding or compaction is not in an equilibrium condition. The soil will change its property with time to reach the energy equilibrium by changing particle arrangement, adsorbed water structure, and distribution of ions in the fluid phase.

2. It is a reversible process. Subsequent remoldings should not affect the characteristic and magnitude of the strength or stiffness re-gain.

Creep is defined as a slow development of deformation under a constant stress state. In soils creep occurs due to the change in the solid soil skeleton when it is subjected to a load. At the contact surfaces the load is transferred to the grains of the solid skeleton. In order to reach the new equilibrium condition commensurate to the new energy state given by this load, particles will be occasionally released from the original contacts or bonds and move to the more stable positions. This deformation characteristic is time-dependent and it occurs in all structurally sensitive soils regardless whether the soil itself possesses aging properties or not.

Aging, strictly speaking, is the change of soil behavior due to the internal change in soil properties. The process is intrinsic to the material regardless of the existence of such external factors as loading and temperature, although these factors may affect the rate of aging to a certain extent. It

is not a strength or stiffness re-gain process by means of the structural rearrangement of soil particles but rather a cementitious reaction among soil constituents which alters the gross material properties. In addition, the process of aging is not reversible, at least not by the mechanical means. After remolding, because of the change in soil properties during aging, the return of soil behavior to the original state is not likely.

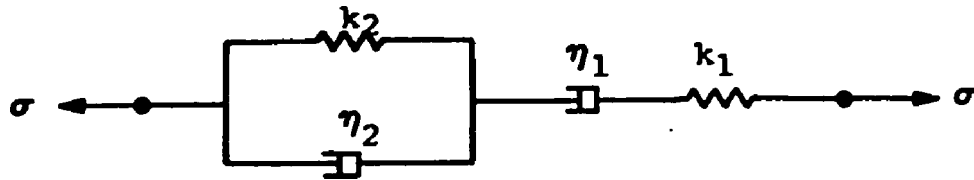
Creep may occur in aging soils. However, unlike ordinary soils, the rate and magnitude of the creep will depend on the age of the soil. This can be accounted for in rheological models. For example, if the Burgers model is chosen to describe the stress-strain behavior of the soils, as shown in Figure 5.2.a, then the law which governs the stress-strain behavior is the same but the creep parameters for the aging soils will not be constant but instead will vary with time. In other words, the springs will become stiffer and the dashpots will become more viscous with time, (as indicated in Figure 5.2.b). Therefore, the rate and magnitude of the creep will decrease with the increase in age of the soil. On the other hand, if the aging is caused by pozzolanic reaction, these cementitious reactions will assist in linking adjacent particles of the material and form a rather less compressible, aggregated structure. Consequently the flow of pore water will not act the same way as it would in ordinary soils during primary consolidation. In other words, the volume change characteristics will be governed by rheological processes rather than hydrodynamic ones. Therefore, the volume change of aging soil will be mainly caused by the changes or "re-arrangements"

in the solid soil skeleton, i.e., by "creep." It should be noted that the term "creep" used herein is employed in a broader sense as defined in the mechanics of continua. Any deformation of soil caused by the change in soil skeleton due to loading is considered as creep and the deformation does not necessarily have to be shear distortion.

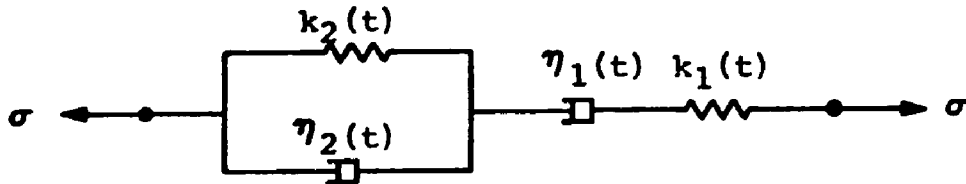
In summary, thixotropy, which is primarily a physical process and reversible, may be considered as special case of aging but not equivalent to aging. Because of the increase in stiffness of soil structure due to the aging effect, the hydrodynamic process of pore fluid expulsion is less important in controlling volume change of aging soils subjected to compressive loads. Instead, volume change will tend to be governed by rheological considerations.

5.3 Experimental Investigations on the Effect of Aging On the Compressibility of Compacted Fly Ashes

The volume change characteristics of an aging material can be demonstrated by compressibility tests on lime-treated, compacted fly ash. Because of its intrinsic pozzolanic properties, the aging effect becomes very significant and should be taken into account in evaluating volume change behavior under load. The response of an aging material at various ages to a constant load can be illustrated by a three-dimensional sketch as shown in Figure 5.3. The older the material, the lower is the compressibility. The reduction in compressibility with age in turn depends on the pozzolanic reactivity of the



a. Non-aging soil



b. Aging soil

Figure 5.2 The aging and non-aging stress-strain behavior of soils described by the Burgers rheological model.

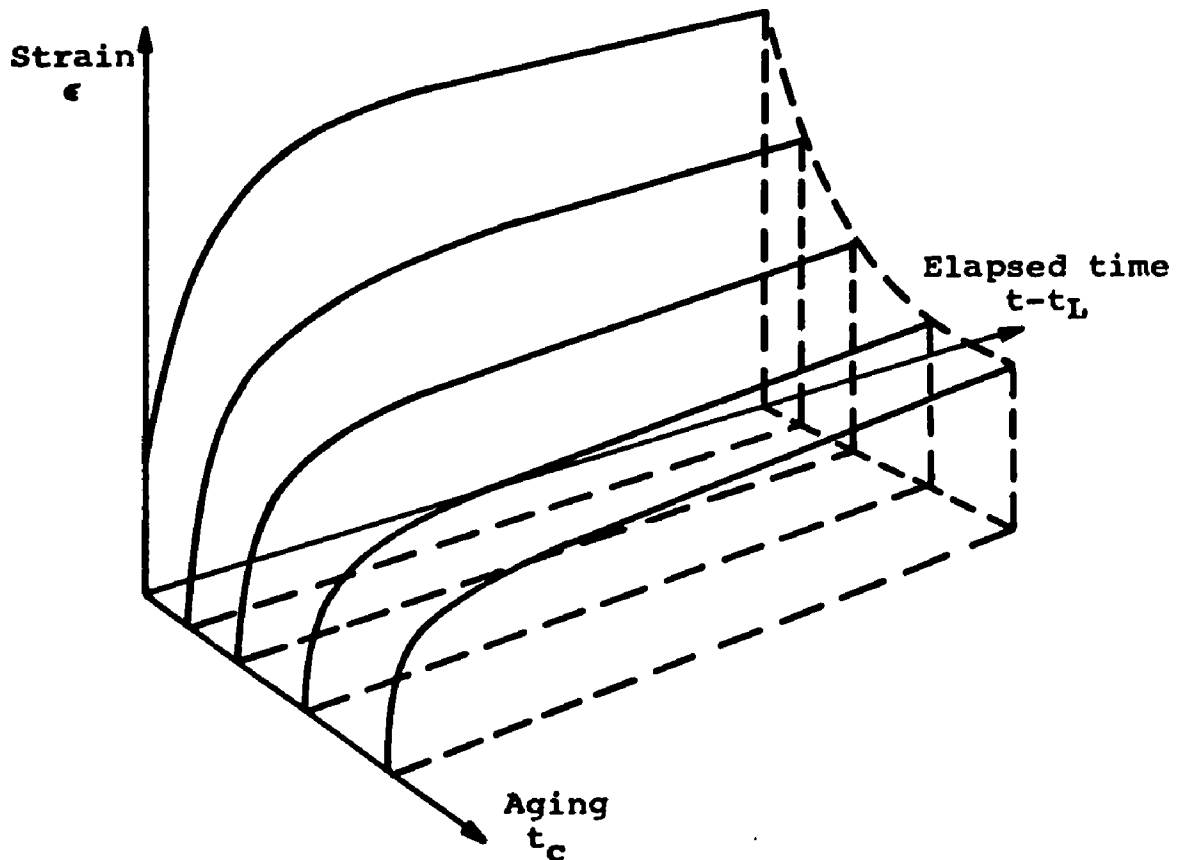


Figure 5.3 Three-dimensional sketch of strain-loading time-aging relation of an aging material.

material and the ambient condition, primarily temperature.

The variation in the rate of loading increments on the aging material can be quite important because the consecutive load increments would be applied at longer and longer elapsed aging times. Therefore, variations in the rate of loading could alter the magnitude of the deformation.

The laboratory experimental program was planned with these factors taken into consideration. The program has been divided into two parts:

Series 1. The effects of aging prior to the load application on the compressibility of compacted fly ash.

As was indicated in the preceding discussion, the variation in aging would affect the response of the soil to the load. This concept is illustrated in Figure 5.4. Tests were conducted in fresh samples and on samples cured for two days, four

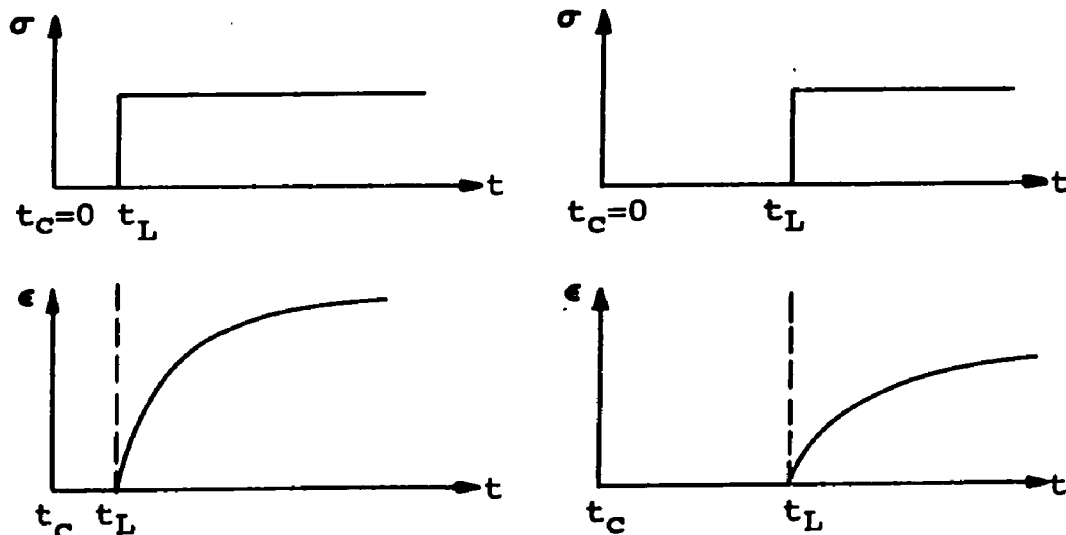


Figure 5.4 Difference in response of an aging material to step loads at different stages of material aging.

days and ten days respectively. All the samples were stabilized with the optimum amount of lime.

Series 2. The effect of loading time interval on the compressibility of compacted fly ash.

Four load increments, viz., 1, 2, 4 and 8 kg/cm², were applied on each sample. The load-increment duration of two hours was used in the tests on the fresh samples and the samples which were cured for ten days. The load-increment duration of one day was used only on the fresh samples. To investigate the effect of lime stabilization on the aging and compressibility of the compacted fly ash, the tests are conducted both on the samples made of natural fly ashes and on those treated with optimum amounts of lime.

The laboratory set-up, sample preparation and the test procedure are described in the subsequent sections.

1. Laboratory Set-Up

Since the deformation of compacted land fills or structural fills can be reasonably approximated by one-dimensional volume change, one-dimensional compression tests should serve this purpose adequately. A sensitive oedometer, Anteus Consolidometer Model A-1, was used. Back pressure can be applied in this consolidometer for sample saturation. A pressure transducer with a strain indicator was connected in the bottom drainage outlet for the pore water pressure measurement. The set-up is illustrated by a schematic diagram in Figure 5.5 and also shown in Figure 5.6. Because of the limitation of the test facility, it was not possible to conduct tests at

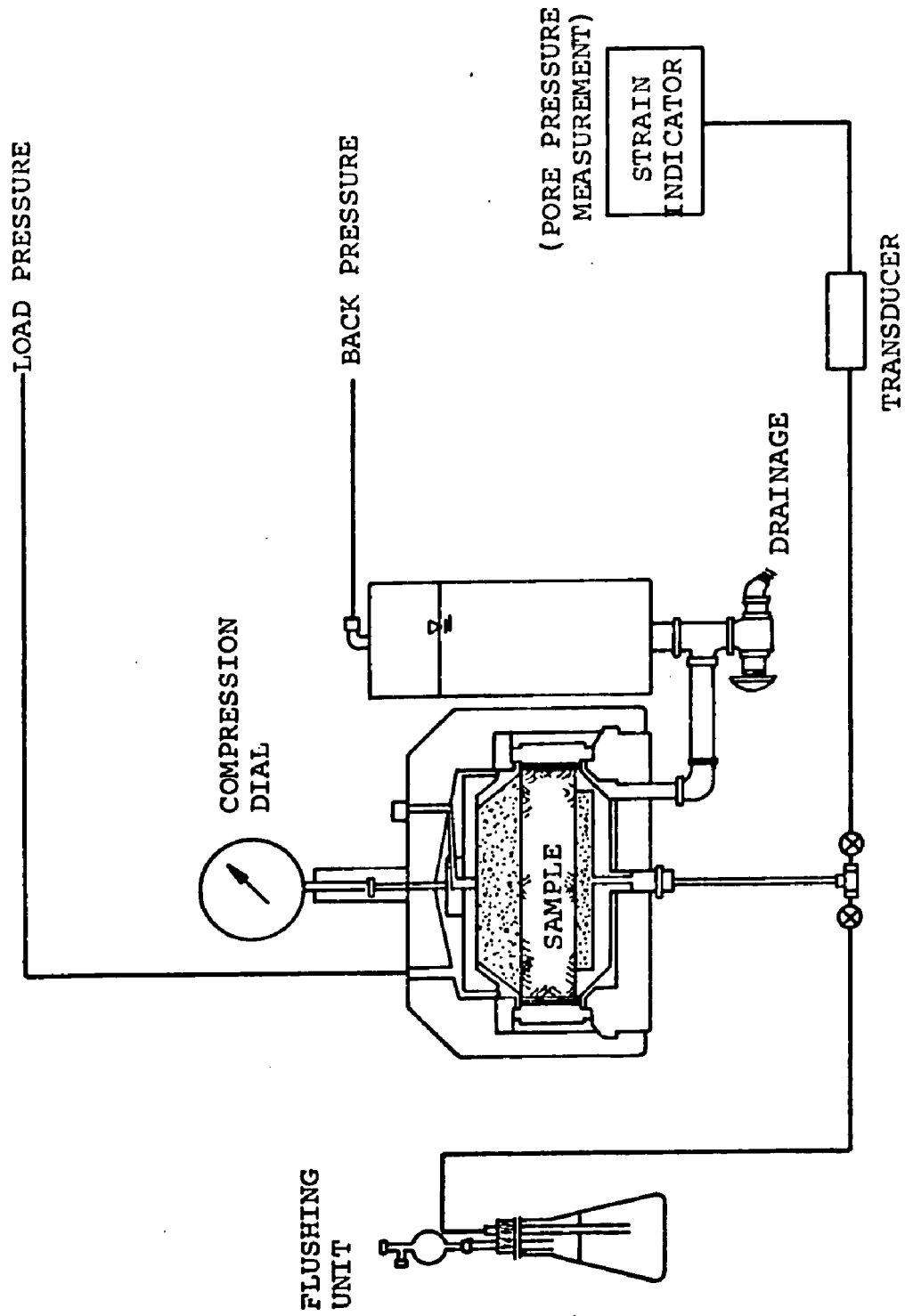


Figure 5.5 Schematic diagram of laboratory set-up for consolidation tests on saturated, compacted fly ashes.

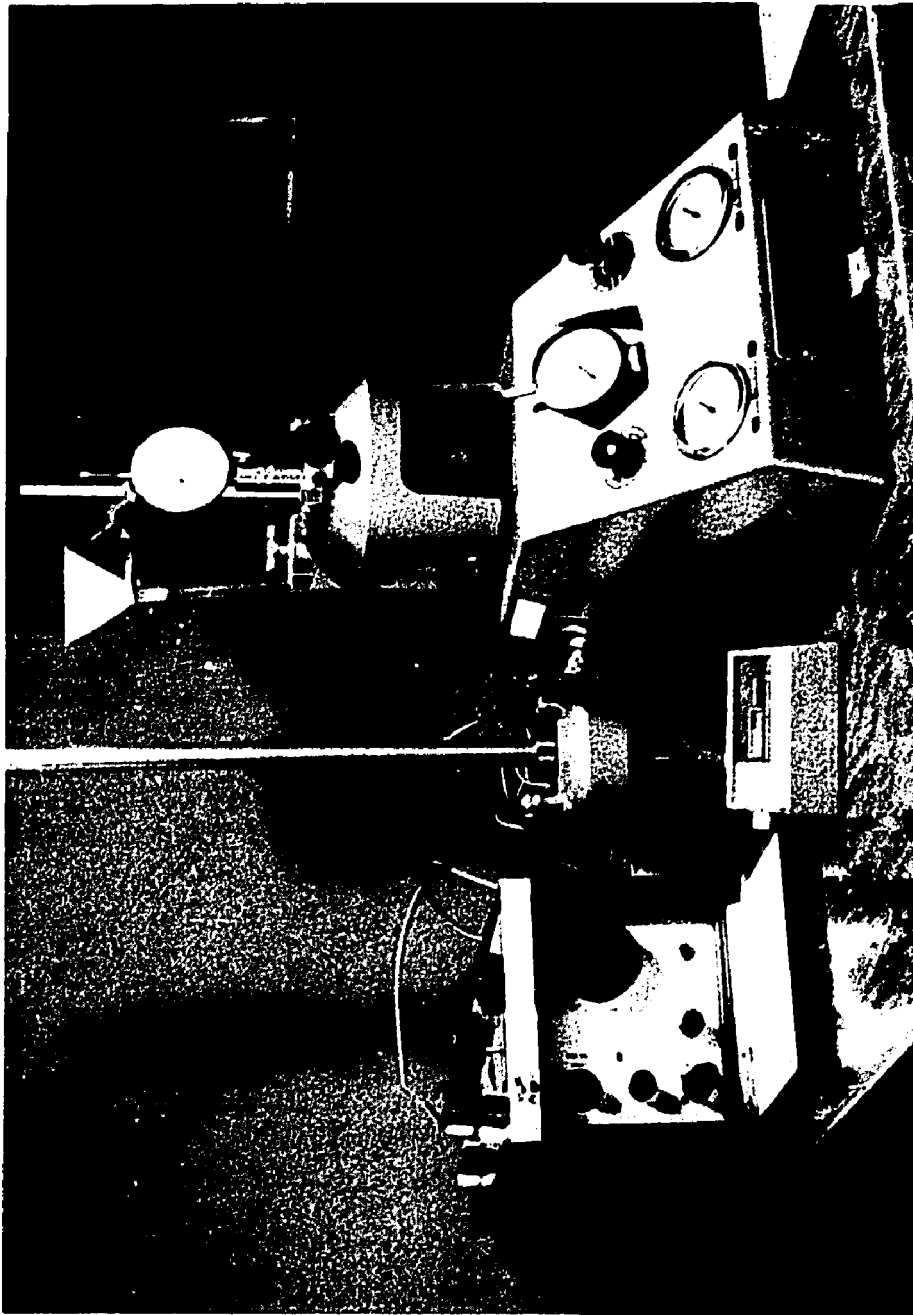


Figure 5.6 Laboratory set-up for consolidation tests on saturated, compacted fly ashes.

various temperatures. However, all the experiments were performed in a constant temperature room, and this eliminated temperature as one of the variables.

2. Sample Preparation

Fly ashes from two sources were used, namely Trenton Channel plant and Marysville plant. Both the natural fly ash and lime-treated ash samples were mixed in a mechanical blender with the optimum amount of water. After thorough mixing, the ashes were then put in the modified AASHO compaction mold and compacted in five layers with 25 blows on each layer. The compacted samples were then trimmed to the size of 4 inches in diameter and about 1 inch in height; this size can be accommodated in the Anteus Consolidometer. Then the samples were either tested immediately or cured at 24°C for 2, 4, and 10 days respectively before the tests.

3. Test Procedure

Prior to the test, a small amount of water was flushed from the bottom drainage to insure complete saturation in the drainage line and the bottom disk. Then the sample was placed in the chamber of the consolidometer, submerged in water and saturated by applying a back pressure of 1.5 kg/cm². After the completion of saturation, a constant load was applied on the sample. Immediately thereafter, deformation and pore pressure measurements were made constantly throughout the test. These tests lasted either 2 hours or 24 hours before subsequent load increments were applied. All the tests were conducted at a temperature of 24°C.

4. Discussion of Results

The deformation-time relationship and the pore pressure measurements for the first two load increments are summarized and shown in Figure 5.7 and Figure 5.8. Elapsed times were plotted on a logarithmic scale. All the points represent the average readings of two duplicate tests. To calculate the strain the "corrected zero points" were used as the zero readings to avoid the deviation in the initial readings due to the improper seating between samples and the loading cap or the base. The corrected zero point is defined by assuming that the early portion of the curve is a parabola, and is located by scribing a distance equal to the vertical distance between the intersection of curve with 0.1 minute time coordinate point and at a time co-ordinate which is four times greater.

permeability of compacted fly ashes is some three to four orders-of-magnitude higher than that of clays. Therefore, the pore pressure also dissipates substantially faster. Within the limitation of the test facilities, the readings could not be made any sooner than about 0.05 minute, at which time the pore pressure had already dissipated to a great extent.

From these two figures, the following phenomena can be observed:

- (a) Both strain time curves and pore pressure-time curve show that the primary consolidation of the compacted fly ash from both Trenton Channel plant and Marysville plant was completed in less than one minute.

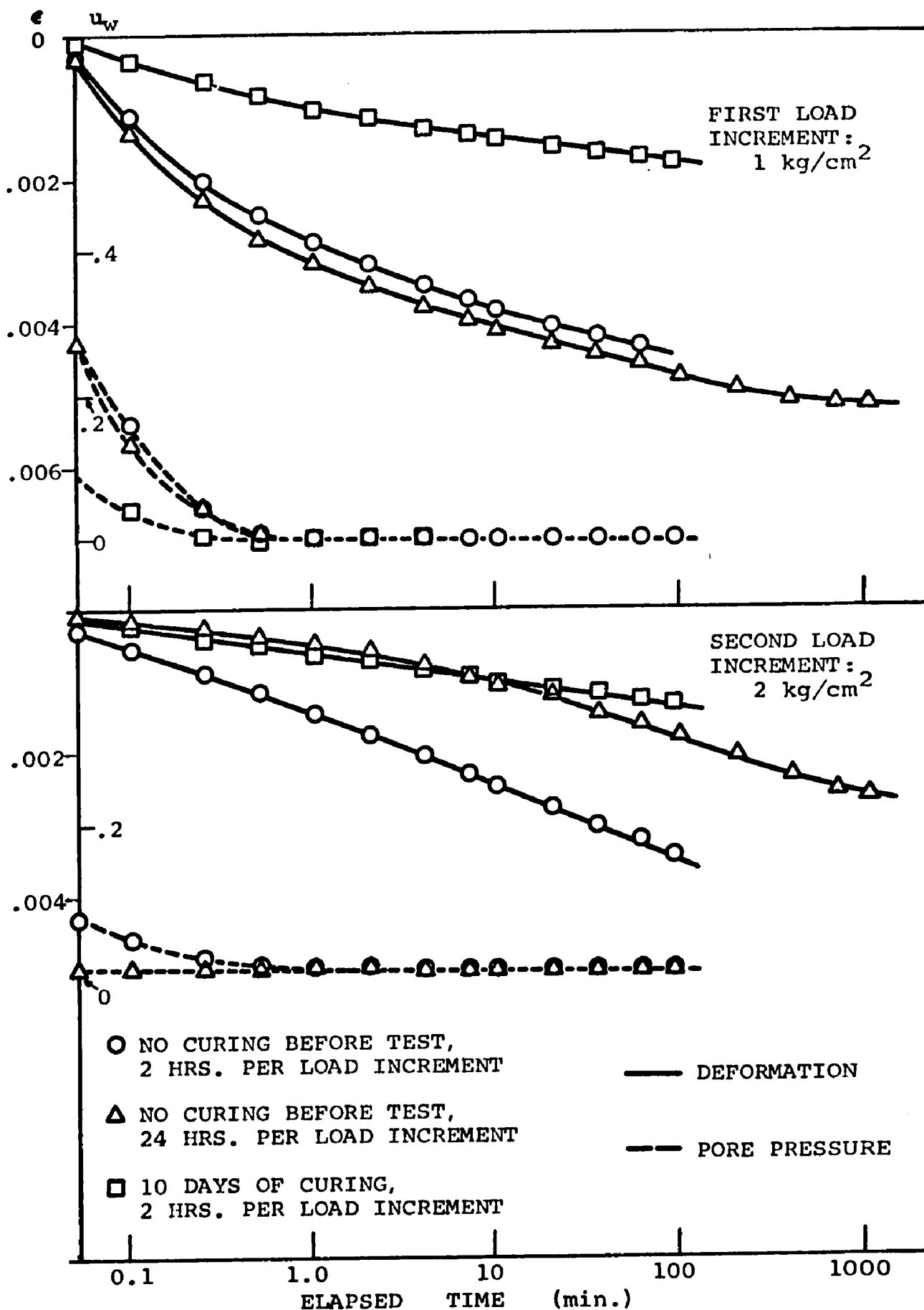


Figure 5.7 The deformation-time relationship of compacted fly ash from Trenton Channel plant.

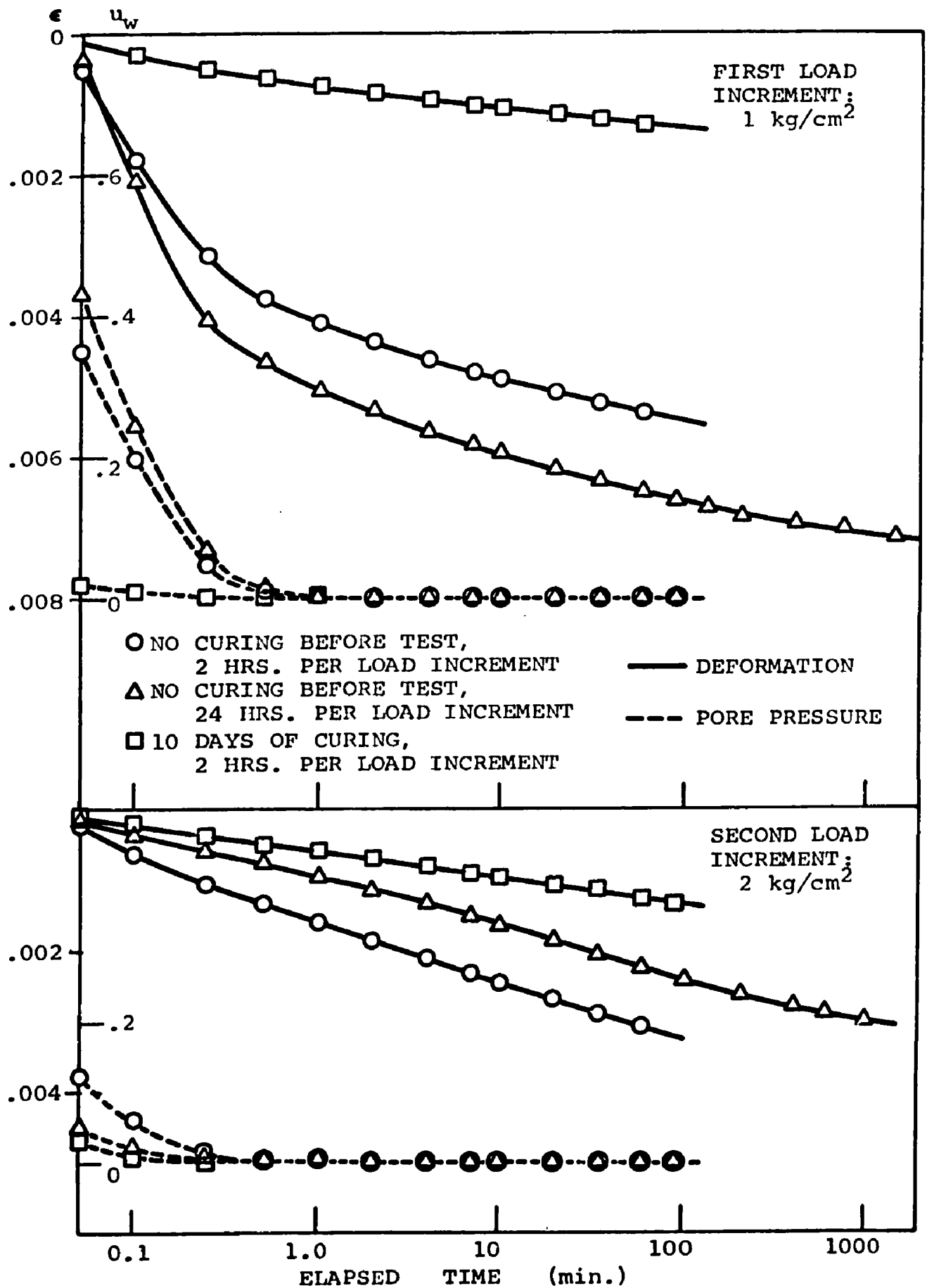


Figure 5.8 The deformation-time relationship of compacted fly ash from Marysville plant.

(b) At the second load increment, the consolidation curves appeared quite different from those for ordinary soils. The portion which represents primary consolidation was not distinct from the portion of secondary compression. The magnitudes of the strains were also considerably smaller. This can be attributed to aging. Cementing forces between particles might have been developed to such an extent that the soil skeleton was no longer so compressible. Therefore, the soil structure was stiff enough to carry part of the initial load that was supposed to be carried by pore fluid alone. Consequently, the out-flow of pore water and dissipation of pore pressure did not act as a governing mechanism for soil deformation within the period of primary consolidation; instead secondary compression, or creep process, played a major role in the volume change characteristics of aging soil subjected to a load. This aging effect becomes more obvious with increasing age of the samples. The long term sample, e.g., the samples with 24-hour load-increment duration, has much less compression than the short term samples at the second load increment although they have comparable compressibility at the first load increment. For the samples with 10 days of curing before tests (at which the samples reach at least 60 percent of their ultimate curing strengths) the test results show little evidence of the existence of primary consolidation, and the curves indicate little change from one load increment to the other.

- (c) The portions of the curves which represent the secondary compression are not straight lines but curve upward. This may be also due to the aging effect. The secondary compression does not proceed at a constant rate per log cycle of time but decreases with increasing age of the fly ash.

These factors are very important in field settlement predictions. Because of these properties, both the magnitude of the actual settlement and the time required to complete the consolidation would be quite different from those computed according to the conventional methods. The ultimate settlement would be considerably less than that of non-aging soils and the amount of reduction in total settlement depends upon the aging property of the individual material and the age of the material at which the load is applied. The time required to complete the primary consolidation is conventionally calculated from the following relation:

$$t = \frac{(\frac{H}{C_v})^2}{T_c}$$

where H = the maximum distance of the pore flow path in soil.

C_v = coefficient of consolidation.

T_c = time factor.

It can be seen that the required time is proportional to the second power of the thickness of the soil layer. However, the importance of soil thickness diminishes with time because the primary consolidation becomes less significant for the

soil after a short period of aging. Thus the rate of settlement would tend to be independent of the thickness of the soil layer.

The effect of aging prior to the load application on the compressibility of these two stabilized, compacted fly ashes are shown in Figure 5.9. This underscores the importance of proper correlation between the age of the samples tested and the age of the compacted fly ash in the field when the settlement prediction is made.

The influence of loading-time interval and the effect of lime treatment on the compressibility of compacted fly ashes from both plants are illustrated in Figure 5.10 and Figure 5.11. For the convenience of comparisons, the plotted void ratios were normalized with respect to the void ratios at the load of 1 kg/cm^2 . The results are shown in Figure 5.12 and Figure 5.13.

Based on these results, two points of interest are worth noting:

- (a) For untreated fly ashes, the effects of aging and loading-time intervals on the compressibility behavior are not significant. These natural ashes simply behave as the ordinary soils. In this case, the age of the ash and the rate of loading need not be considered.
- (b) For lime treated fly ashes, the influence of aging becomes substantial. It can be noted from both Figure 5.12 and 5.13 that the compressibilities of the samples either tested in long term loading (curve B) or cured before

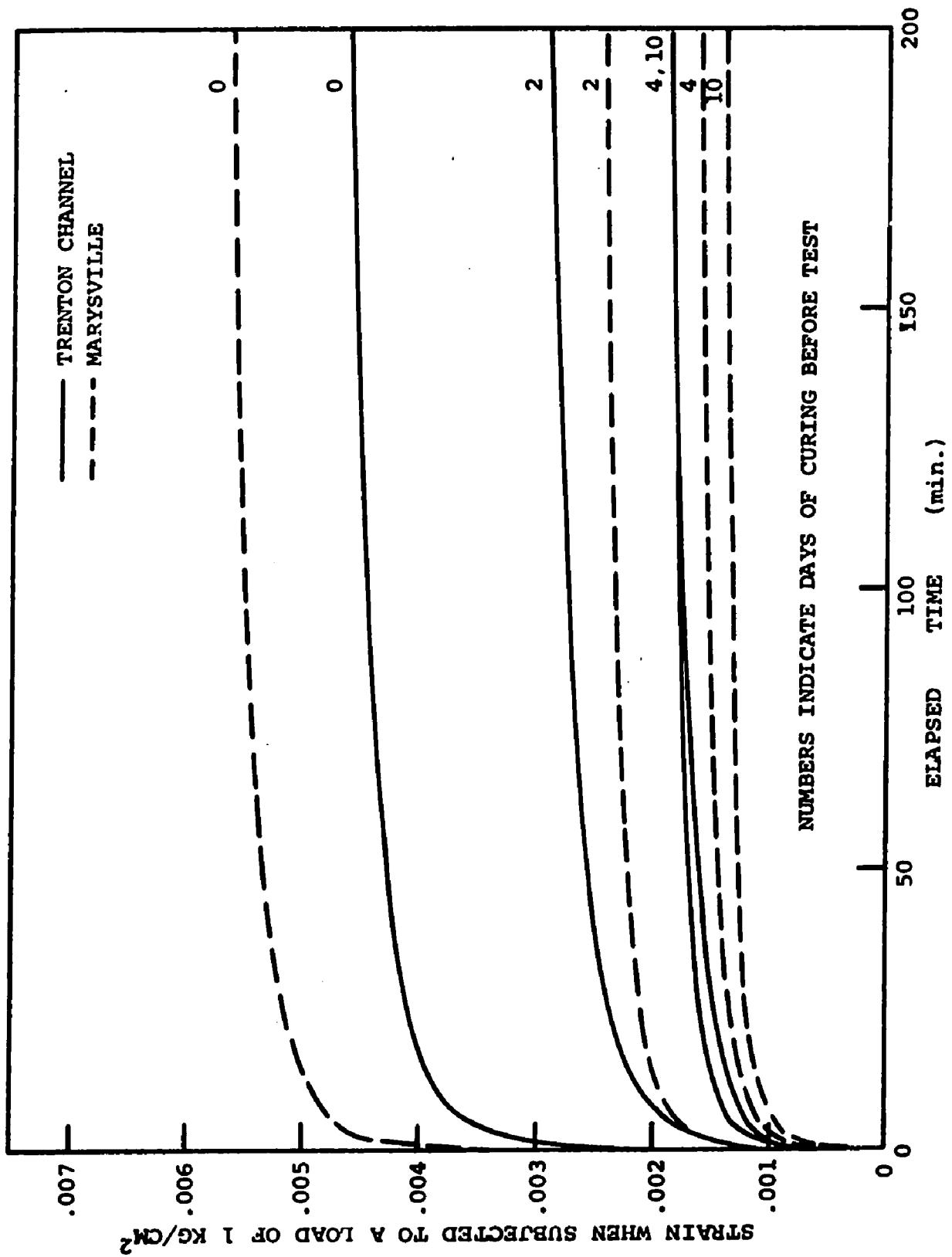


Figure 5.9 Time-strain curves for lime treated Trenton Channel and Marysville fly ashes.

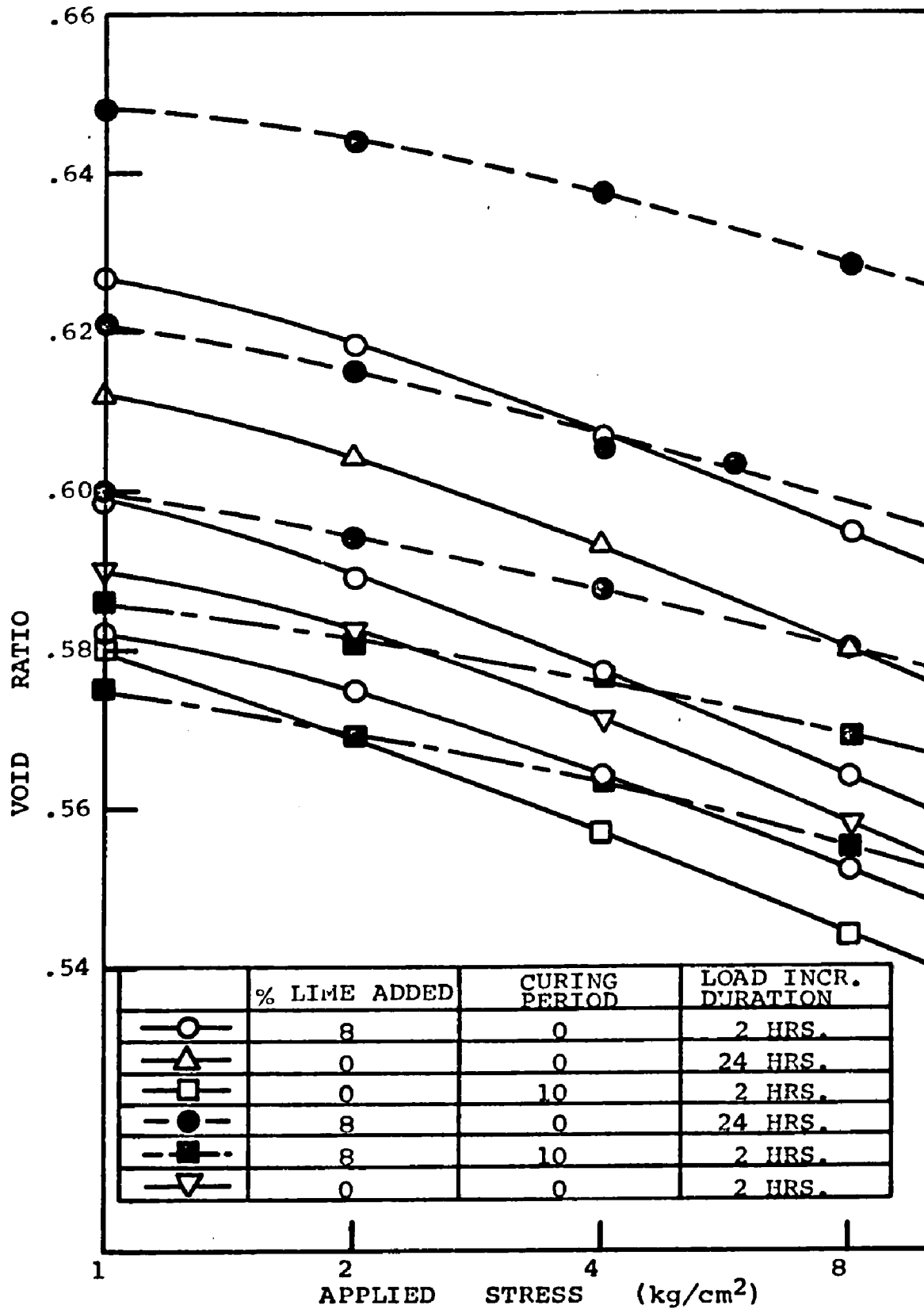


Figure 5.10 Effects of lime treatment and loading-time interval on the compressibility of compacted Trenton Channel fly ash.

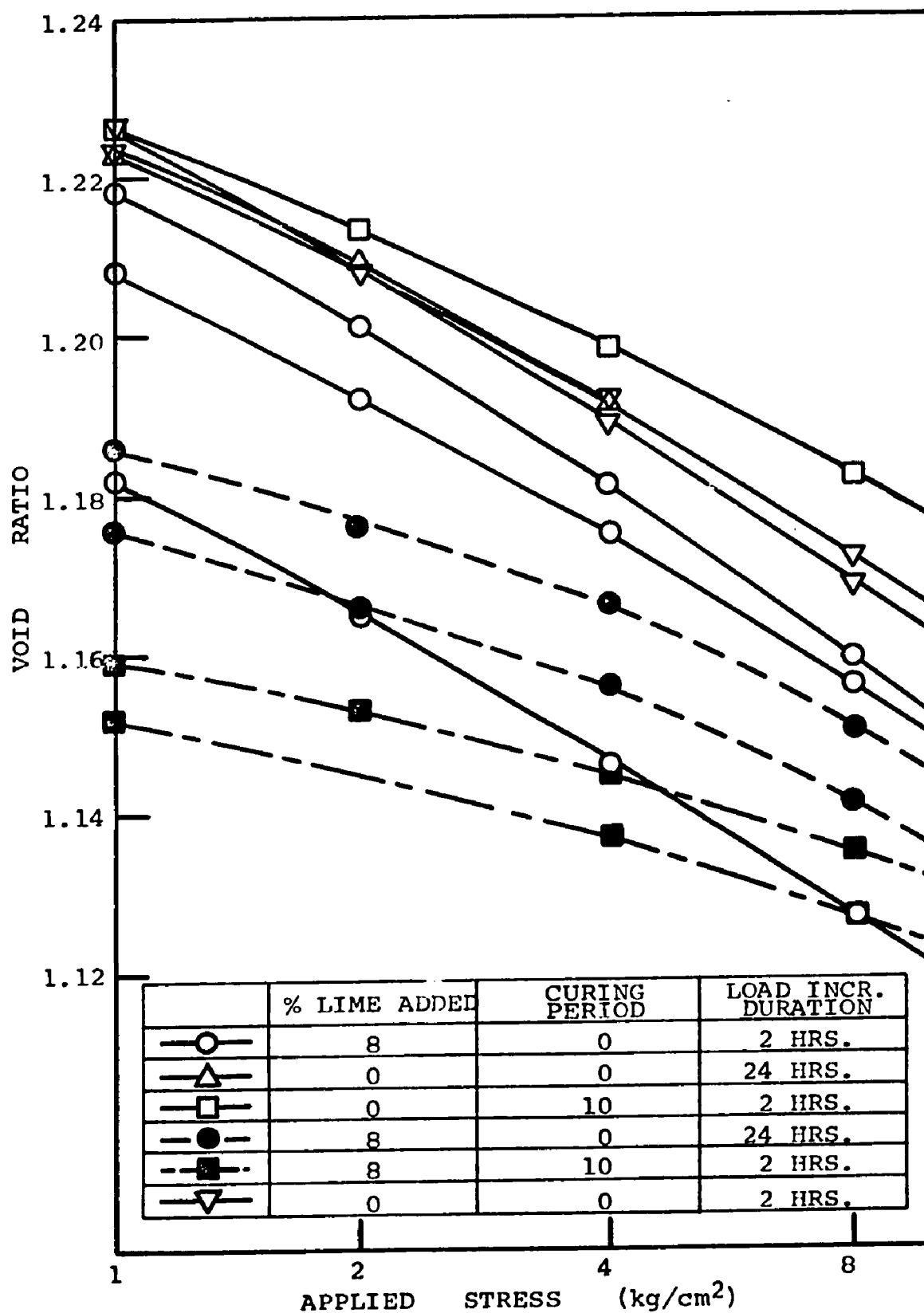


Figure 5.11 Effects of lime treatment and loading-time interval on the compressibility of compacted Marysville fly ash.

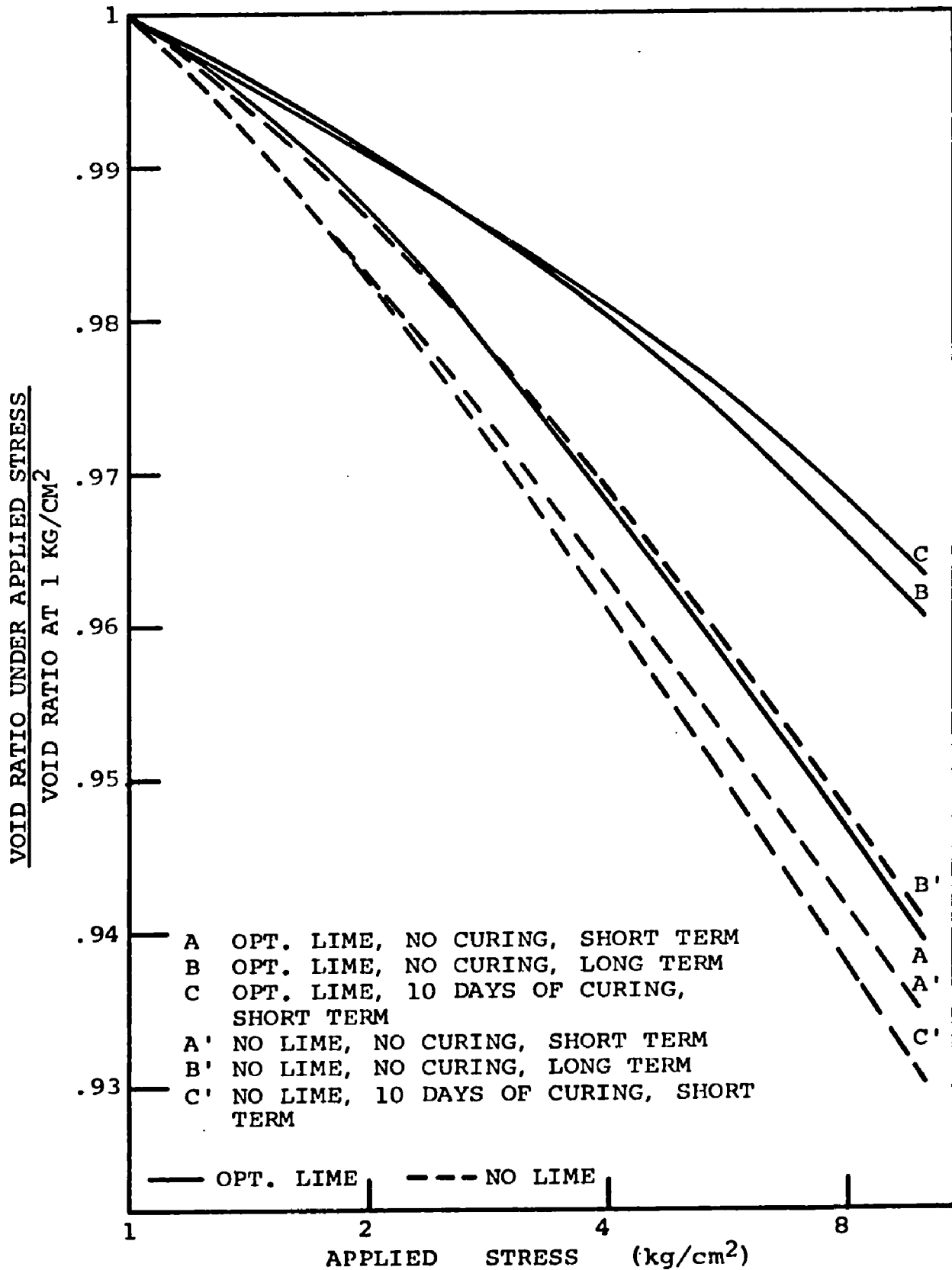


Figure 5.12 Effects of lime treatment and loading-time interval on the compressibility of compacted Trenton Channel fly ash. Void ratios have been normalized.

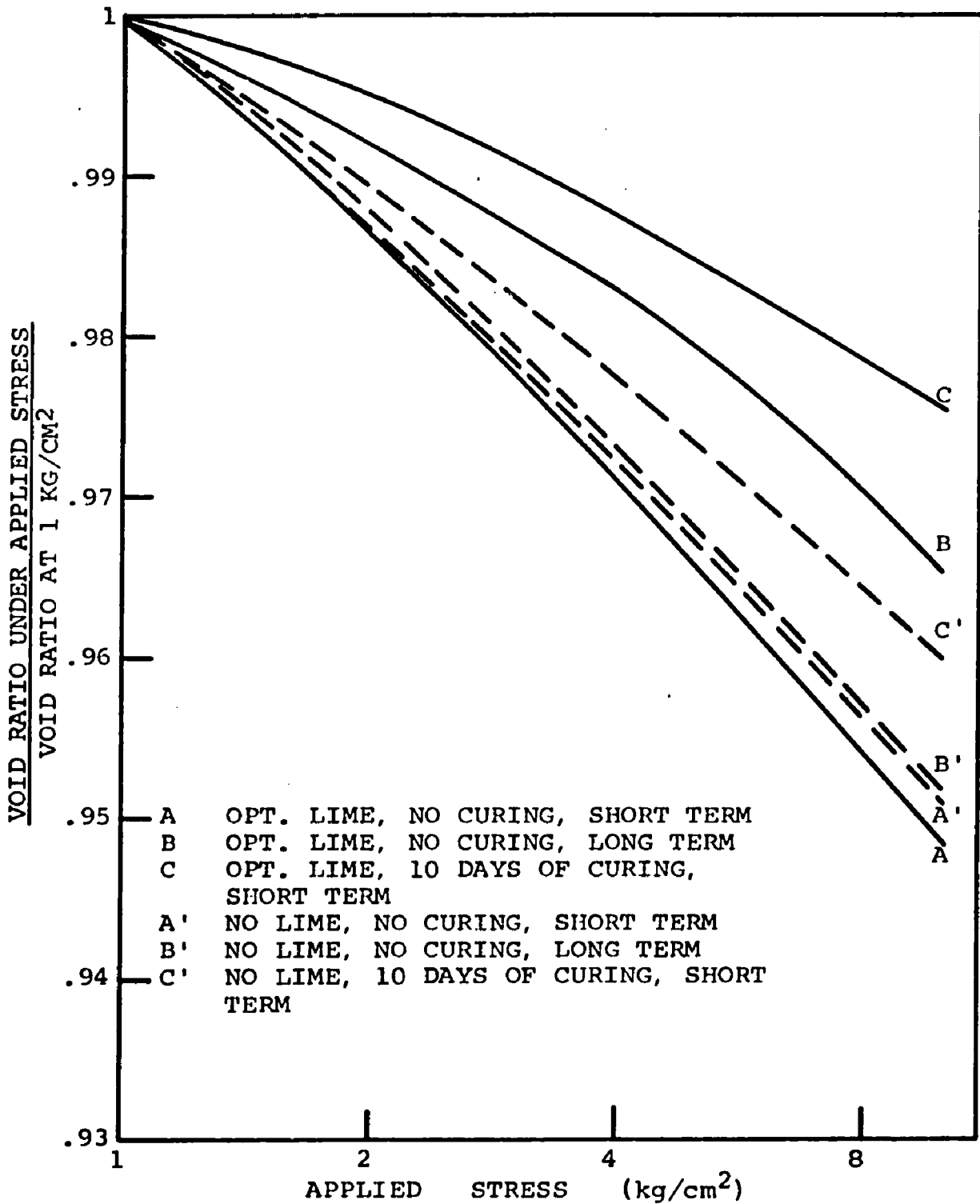


Figure 5.13 Effects of lime treatment and loading-time interval on the compressibility of compacted Marysville fly ash. Void ratios have been normalized.

testing (curve C) are comparable and considerably lower than the other samples. Therefore, in the course of evaluating field settlement from the laboratory testing results, one must be aware that the samples and the nature of the tests should be representative of existing field conditions.

Since one testing program only serves one specific field condition, the same set of test result cannot be applied to evaluate the settlement of fills under different circumstances. More generalized laboratory test procedure which is valid for a wider spectrum of field applications would be desirable. There are two approaches which might serve to achieve this objective, namely,

- (a) to conduct a large scale laboratory testing program on samples with different aging times and a wide range of loading-time intervals. A rather complete void ratio vs. loading "chart", which consists of void ratio vs. loading curves with different loading-time intervals, may be made from these test results. Then the settlement calculation of a fill in any specific condition and under any specific rate of loading can be done by interpolation from the chart.
- (b) To use the viscoelastic theory and the principle of superposition and reduce the laboratory testing to a minimum amount -- namely, a single short term loading test on samples at different stages of aging.

There are advantages and disadvantages for both approaches. The first method can be expected to give more accurate results

provided that a complete information on void ratio-loading relations is given. However, to obtain this complete information involves rather large scale laboratory testing. This would be expensive and time consuming and hence, not desirable. The second approach requires only a small number of laboratory test. However, the accuracy of the work depends on the validity of the principle of superposition, and the validity of this principle depends on the sensitivity of the soil structures. Since high structural sensitivity of compacted fly ash is not anticipated, this approach seems reasonable.

Based on this judgment, an attempt has been made to construct the aging and loading-rate dependent void ratio vs. pressure curves by using the viscoelastic theory. The discussions of viscoelastic stress-strain law for aging materials and its application are given in the following sections.

5.4 Viscoelastic Stress-strain Law for Aging Materials under One-dimensional Strain Condition

To serve as the background knowlege of the application of viscoelastic theory to the valuation of compressibility behavior of compacted fly ash, the viscoelastic stress-strain law for aging materials is discussed in this section.

Let t_c = the creation time of the material (or the time when the specimen is made).

t_L = the time when load is applied.

t = the current time.

τ = the time variable ($t_c \leq \tau \leq t$).

$\phi(x, t)$ = the environmental parameters that influence the mechanical response of a material. (For the case of compacted fly ash, temperature is the most important environmental factor because of pozzolanic activity.)

Strain may depend on stress and environment which may also influence aging. The viscoelastic behavior of an environment-dependent, aging material can be expressed as the following constitutive relation:

$$\epsilon(t) = L \left[\sigma \left(\tau_1 \right)_{\tau_1=t_c}^t, \phi \left(\tau_2 \right)_{\tau_2=t_c}^t, t, t_c \right].$$

At a specific environment, such as constant temperature, the relation reduces to

$$\epsilon(t) = L \left[\sigma(\tau)_{\tau=t_c}^t, t, t_c \right].$$

If all events are shifted in time by t time units, there should be no change in response. Then the constitutive equation simplifies to

$$\epsilon(t) = L \left[\sigma \left(\tau + t_c \right)_{\tau=0}^{t-t_c}, t - t_c \right].$$

By using t_c as the origin of the time coordinate, then

$$\epsilon(t) = L \left[\sigma \left(\tau \right)_{\tau=0}^t, t \right]. \quad (5.1)$$

Analogous to linear elasticity, the general linear viscoelastic constitutive relation can be written in the form of a fourth

order tensor

$$\epsilon_{ij} = L_{ijkl} \left[\sigma_{kl} \left(\tau \right)_{\tau=0}^t, t \right] . \quad (5.2)$$

If the functional L_{ijkl} is linear in the $\sigma_{kl}(t)$ component of stress history, non-retroactive, and continuous, it has been stated by Stouffer (1968) that, by employing Konig and Meixner's work, a linear hereditary aging stress-strain law can be established and Equation (5.2) can be expressed as a Riemann-Stieltjes integral as follows:

$$\epsilon_{ij} = \int_{-\infty}^t \sigma_{kl} (t - \tau) dJ_{ijkl} (\tau, t) , \quad (5.3)$$

or in convolution form,

$$\epsilon_{ij} = \sigma_{kl} * dJ_{ijkl} \quad (5.3-A)$$

Where $J(\tau, t)$ is the creep function which represents the strain at age t due to a unit step stress τ units earlier. For an isotropic material, it can be conveniently divided into octahedral and deviatoric components and written as

$$\left. \begin{aligned} \hat{\epsilon}_{ij} (t) &= s_{ij} * dJ_1 (\tau, t) \\ \epsilon_{kk} (t) &= \sigma_{kk} * dJ_2 (\tau, t) \end{aligned} \right\} \quad (5.4)$$

where $\hat{\epsilon}_{ij}$ and s_{ij} are deviatoric strain and stress tensors, and ϵ_{kk} and σ_{kk} are octahedral strain and stress tensors. Also, J_1 and J_2 are deviatoric and octahedral creep functions. Any strain tensor ϵ_{ij} can be expressed as

$$\epsilon_{ij} = \hat{\epsilon}_{ij} + \epsilon_{ij}^{\text{oct}} = \hat{\epsilon}_{ij} + \frac{1}{3} \epsilon_{kh} \delta_{ij} \quad (5.5)$$

Substituting Equation (5.4) into Equation (5.5)

$$\begin{aligned} \epsilon_{ij} &= s_{ij} * dJ_1 + \frac{1}{3} \delta_{ij} \sigma_{kh} * dJ_2 \\ &= J_1 * d s_{ij} + \frac{1}{3} \delta_{ij} J_2 * d \sigma_{kh} \\ &= J_1 * d (\sigma_{ij} - \sigma^{\text{oct}}) + \frac{1}{3} \delta_{ij} J_2 * d \sigma_{kh} \\ &= J_1 * d (\sigma_{ij} - \frac{1}{3} \sigma_{kk}) + \frac{1}{3} \delta_{ij} J_2 * d \sigma_{kh} \\ &= J_1 * d \sigma_{ij} + \delta_{ij} \frac{(J_2 - J_1)}{3} * d \sigma_{kh} \end{aligned}$$

$$\text{or} \quad \epsilon_{ij} = \sigma_{ij} * dJ_1 + \delta_{ij} \sigma_{kh} * d \left(\frac{J_2 - J_1}{3} \right) \quad (5.6)$$

For applying to the compressibility behavior of compacted fly ash when it is used as mass land fills, it is proper to confine this treatment to the physical condition which is similar to this field condition, namely, the condition of one-dimensional volume change. Two types of laboratory experiments are conventionally used to meet this condition, namely, the oedometer test and the K_0 triaxial test. In the oedometer test the cylindrical sample is physically confined in the radial direction, in the K_0 triaxial test the ratio of the lateral confining pressure to the axial pressure is so adjusted that the lateral deformation of the sample is prevented. In these tests the sample as a whole can move only in the direction of its vertical axis and the radial displacements do not occur. Therefore, the study of this volume change character-

istic by these types of tests is limited to the stress states corresponding to this displacement condition. The stress states for this case are

$$\sigma_{22} = \sigma_{33} = K_0 \sigma_{11} \quad (5.7)$$

Where K_0 is the coefficient of earth pressure at rest.

If a vertical stress σ_{11} is applied on the top of the cylindrical sample, the vertical strain component due to this load, according to Equation (5.6), will be

$$\epsilon_{v1} = \epsilon_{11} = \sigma_{11} * dJ_1 + \sigma_{11} * d\left(\frac{J_2 - J_1}{3}\right)$$

$$\text{or } \epsilon_{11} = \sigma_{11} * d\left(\frac{J_2 + 2J_1}{3}\right), \quad (5.8-A)$$

$$\sigma_{11} = \epsilon_{11} * d \ 3 [J_2 + 2J_1]^{-1}. \quad (5.8-B)$$

where the subscript v represents the plane normal to vertical axis. The notation used in Equation (5.8-B) represents a standard method of numerical solution for solving integral equation, Lee and Rogers (1963). Also from Equation (5.6), lateral strains ϵ_{22} , ϵ_{33} due to σ_{11} are

$$\epsilon_{22} = \epsilon_{33} = \sigma_{11} * d\left(\frac{J_2 - J_1}{3}\right) \quad (5.9)$$

Substituting Equation (5.8-B) into Equation (5.9) yields

$$\epsilon_{22} = \epsilon_{33} = (\epsilon_{11} * d \ 3 [J_2 + 2J_1]^{-1}) * d\left(\frac{J_2 - J_1}{3}\right)$$

$$\text{or } \epsilon_{22} = \epsilon_{33} = - \epsilon_{11} * d \nu \quad (5.10)$$

where $\nu(\tau, t) = (J_2 + 2J_1)^{-1} * d(J_1 - J_2)$ called vis-

coelastic Poisson's ratio for aging material. However, in one-dimensional strain conditions, lateral expansion is prevented, thus, it induces lateral stresses σ_{22} , σ_{33} in accordance with Equation (5.7), (or, alternatively these lateral stresses have to be applied to meet this physical condition). Therefore, the vertical strain components due to the induced stresses σ_{22} , σ_{33} will be, by using Equations (5.8) and (5.10),

$$\epsilon_{v2} = \epsilon_{v3} = -K_0 \sigma_{11} * d \left(\frac{J_2 + 2J_1}{3} \right) * d\nu.$$

Thus, the total vertical strain due to σ_{11} is

$$\epsilon_v = \sigma_{11} * d \left(\frac{J_2 + 2J_1}{3} \right) - 2K_0 \sigma_{11} * d \left(\frac{J_2 + 2J_1}{3} \right) * d\nu \quad (5.11)$$

If the change in values of K_0 with time is small and can be considered constant, then

$$\begin{aligned} \epsilon_v &= \sigma_{11} * d \left(\frac{J_2 + 2J_1}{3} \right) [1 - 2K_0 * d\nu], \\ \text{or } \epsilon_v &= \sigma_{11} * d \left(\frac{J_2 + 2J_1}{3} \right) [1 - 2K_0 * d(J_2 + 2J_1)^{-1} * d(J_1 - J_2)]. \end{aligned} \quad (5.12)$$

It should be noted that the quantity

$$\left(\frac{J_2 + 2J_1}{3} \right) [1 - 2K_0 * d(J_2 + 2J_1)^{-1} * d(J_1 - J_2)]$$

is only a function of two creep functions J_1 and J_2 if K_0 is a constant.

Let J_e be denoted as an equivalent creep function for one-dimensional strain condition and be defined as

$$J_e \equiv \left(\frac{J_2 + 2J_1}{3} \right) [1 - 2K_0 * d(J_2 + 2J_1)^{-1} * d(J_1 - J_2)],$$

then

$$\epsilon_v = \sigma_{11} * d J_e \quad (5.13)$$

If a unit step load is applied, the stress can be expressed as $\sigma_{11}(t) = 1(t - t_L)$

$$\begin{aligned} \text{where } 1(t-t_L) &= 0, \quad t-t_L < 0 \\ 1(t-t_L) &= 1, \quad t-t_L \geq 0. \end{aligned}$$

$$\begin{aligned} \text{Then, } \epsilon_v(t) &= \sigma_{11} * d J_e = J_e * d \sigma_{11} \\ &= J_e * d 1(t-t_L, t) \end{aligned}$$

$$\text{or } \epsilon_v(t) = J_e(t-t_L, t) \quad (5.14)$$

thus, the equivalent creep function $J_e(t-t_L, t)$ here represents the vertical strain at present time t (measured with respect to t_c) due to a step stress that was applied since t_L (also measured from t_c). Therefore, J_e can be determined experimentally from a material with different aging times subjected to a unit load. Then a three-dimensional plot or a chart can be obtained which represents the creep function-time-aging relations. Figure 5.9 is an example of this plot since it was constructed according to a series of responses to a unit load. Therefore, Figure 5.9 can be also called the aging-dependent creep functions of two stabilized, compacted fly ashes.

5.5 Use of Viscoelastic Stress-strain Law in the Construction Of the Void Ratio vs. Effective Pressure Curve

The creep function-time-aging relation discussed in the previous section can serve as a tool for constructing the

void ratio vs. effective pressure curves from which the settlement of a fill of aging materials can be calculated.

Considering a stress pattern that is usually used for the laboratory consolidation tests (shown in Figure 5.14.a), this stress diagram can be divided into a group of step loads by virtue of superposition (as shown in Figure 5.14.b). Note that all the effective stresses are approximated by step loads. Strictly speaking, the effective stress does not reach its full magnitude instantaneously. However, for a soil or material with relatively high permeability, this assumption should be valid.

Since the stress history is constant with time over a certain time interval, the integral equation will be reduced to a series of simple multiplications of stress and the creep functions. This can be shown by the following derivation.

Assuming the stress-strain behavior of compacted and stabilized fly ashes under study satisfies the viscoelastic constitutive relation, the vertical strain of a sample can be expressed by Equation (5.13), or

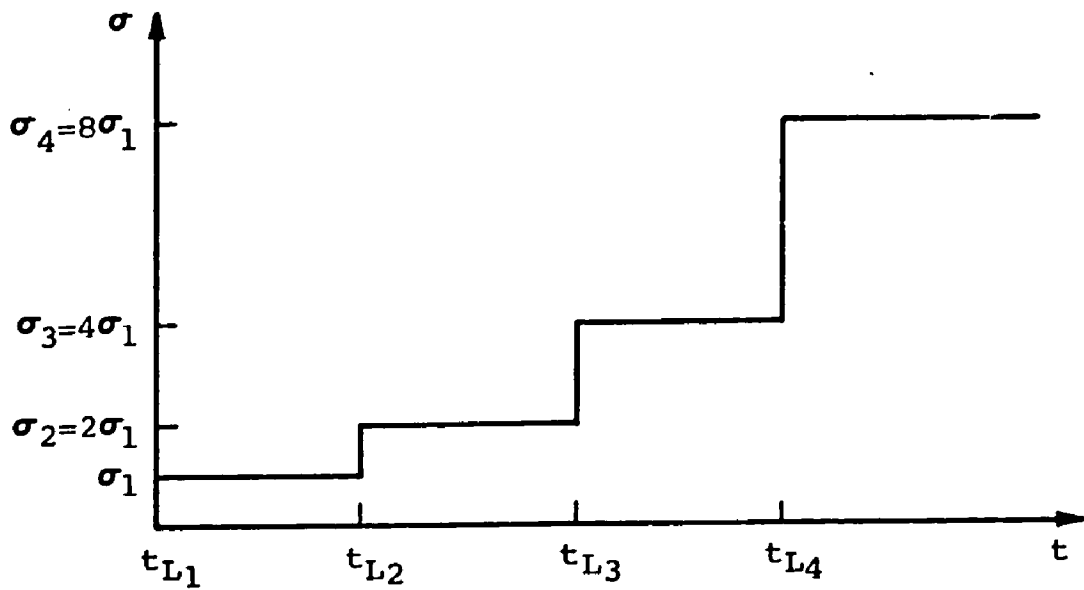
$$\epsilon_v = \int_{-\infty}^t \sigma(t-\tau) dJ_e(\tau, t) .$$

When it is subjected to a stress function

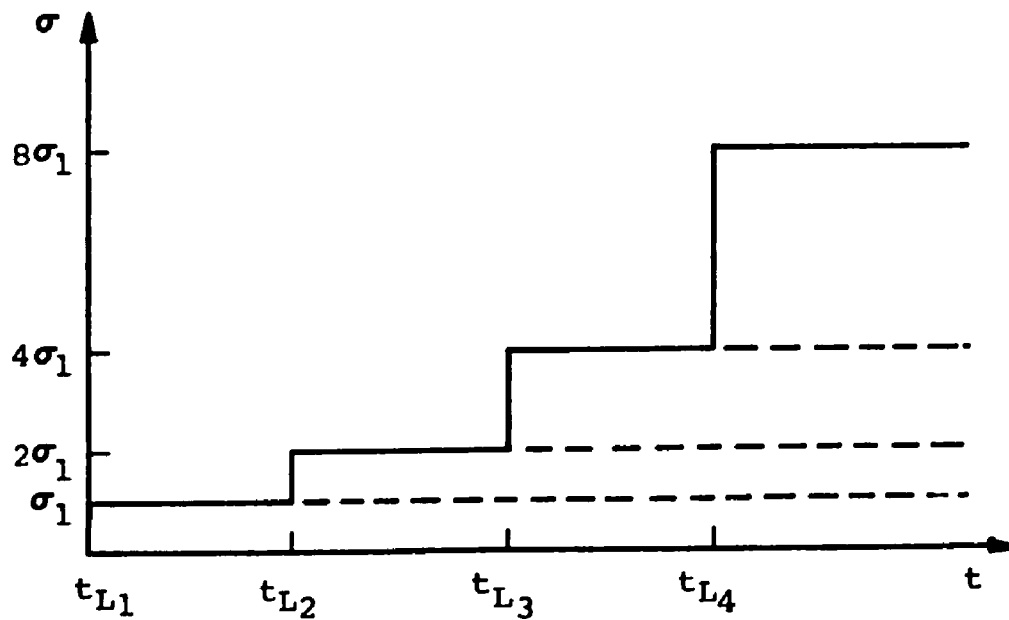
$$\sigma = \sigma_0 1(\tau - t_0) ,$$

where σ_0 is constant and t_0 is the time when load is applied,

$$\epsilon_v = \sigma_0 \int_{-\infty}^t 1(t-t_0-\tau) dJ_e(\tau, t)$$



a. Conventional loading pattern for consolidation tests



b. Superposition of step loads

Figure 5.14 Schematic diagrams of the loading pattern conventionally used in the consolidation test.

$$\begin{aligned}
&= \sigma_0 \int_{-\infty}^{t-t_0} dJ_e(\tau, t) \\
&= \sigma_0 \left\{ J_e(t-t_0, t) - J_e(-\infty, t) \right\}
\end{aligned}$$

using the notation of unit step function, $1(t)=0, t < 0$;

$$1(t)=1, t \geq 0.$$

In the case of this study,

$$J_e(-\infty, t) = 0$$

because creep functions start not sooner than the samples are made. Therefore,

$$\epsilon_v = \sigma_0 J_e(t-t_0, t).$$

The stress pattern shown in Figure 5.14.b can be expressed as

$$\begin{aligned}
\sigma(\tau) = & \sigma_1 1(\tau - t_{L1}) + (\sigma_2 - \sigma_1) 1(t - t_{L2}) \\
& + (\sigma_3 - \sigma_2) 1(t - t_{L2}) + (\sigma_4 - \sigma_3) 1(t - t_{L2}).
\end{aligned} \tag{5.15}$$

By linearity of the integral, the response to $\sigma(\tau)$ is the sum of response to each term. Considering the response to the first term,

$$\sigma(\tau) = \sigma_1 1(\tau - t_L),$$

it can be expressed as

$$\epsilon_v(t) = \int_{-\infty}^t \sigma_1 1(t - t_{L1} - \tau) dJ_e(\tau, t).$$

The quantity $t - t_{L1}$ is fixed and τ is a variable, thus the unit step $1(t) = 0$ for $\tau > t - t_{L1}$ by its definition.

Then

$$\begin{aligned}
 \epsilon_v(t) &= \sigma_1 \int_{-\infty}^{t-t_{L1}} dJ_e(\tau, t) \\
 &= \sigma_1 \left\{ J_e(t-t_{L1}, t) - J_e(-\infty, t) \right\} \\
 &= \sigma_1 J_e(t-t_{L1}, t).
 \end{aligned}$$

The subsequent terms in Equation (5.15) can be treated in the same manner. Consequently the sum of responses can be expressed as

$$\begin{aligned}
 \epsilon_v(t) &= \sigma_1 J_e(t-t_{L1}, t) + (\sigma_2 - \sigma_1) J_e(t-t_{L2}, t) \\
 &\quad + (\sigma_3 - \sigma_2) J_e(t-t_{L3}, t) + (\sigma_4 - \sigma_3) J_e(t-t_{L4}, t),
 \end{aligned}$$

$$\begin{aligned}
 \text{or } \epsilon_v(t) &= \sigma_1 \left\{ J_e(t-t_{L1}, t) + J_e(t-t_{L2}, t) \right. \\
 &\quad \left. + 2J_e(t-t_{L3}, t) + 4J_e(t-t_{L4}, t) \right\} \quad . \quad (5.16)
 \end{aligned}$$

Therefore, at the end of the first loading-time interval,

$$\epsilon_v(t_{L2}) = \sigma_1 J_e(t_{L2}-t_{L1}, t_{L2}) ;$$

at the end of the second loading-time interval,

$$\epsilon_v(t_{L3}) = \sigma_1 \left\{ J_e(t_{L3}-t_{L1}, t_{L3}) + J_e(t_{L3}-t_{L2}, t_{L3}) \right\} ;$$

at the end of the third loading-time interval,

$$\begin{aligned}
 \epsilon_v(t_{L4}) &= \sigma_1 \left\{ J_e(t_{L4}-t_{L1}, t_{L4}) + J_e(t_{L4}-t_{L2}, t_{L4}) + 2J_e(t_{L4} \right. \\
 &\quad \left. - t_{L3}, t_{L4}) \right\} ,
 \end{aligned}$$

etc. Proceeding in this manner, strain, hence the void ratio, at the end of each loading-time interval can be found.

By using this technique and obtaining creep functions from Figure 5.9, the void ratio vs. effective pressure curves for two Michigan fly ashes with different loading-time intervals can be constructed. For the convenience of obtaining creep function values, Figure 5.9 was replotted on logarithmic time scale and is shown in Figures 5.15 and 5.16. The curves of creep functions vs. aging for different elapsed loading time were also extrapolated and are shown in Figure 5.17. The calculations of void ratio-effective pressure relation by using creep functions were tabulated in Tables 5.1 and 5.2. The comparisons of the calculated curves and the experimental curves constructed by using conventional consolidation test results are shown in Figure 5.18.

This figure shows relatively good agreement between the calculated curves and the conventional curves for Marysville fly ash. The results for Trenton Channel fly ash were less satisfactory. The choice of either approach, viz., to calculate from creep functions or to obtain directly from compression tests, depends upon the priority between the accuracy of the results and the scale of the test program. The assessment of compressibility of a soil by using the creep response approach will not be exact, however the advantage of using this approach can be recognized when the load is neither constant with time nor a simple step pattern. The void ratio-load relation still can be obtained directly by using Riemann-Stieltjes

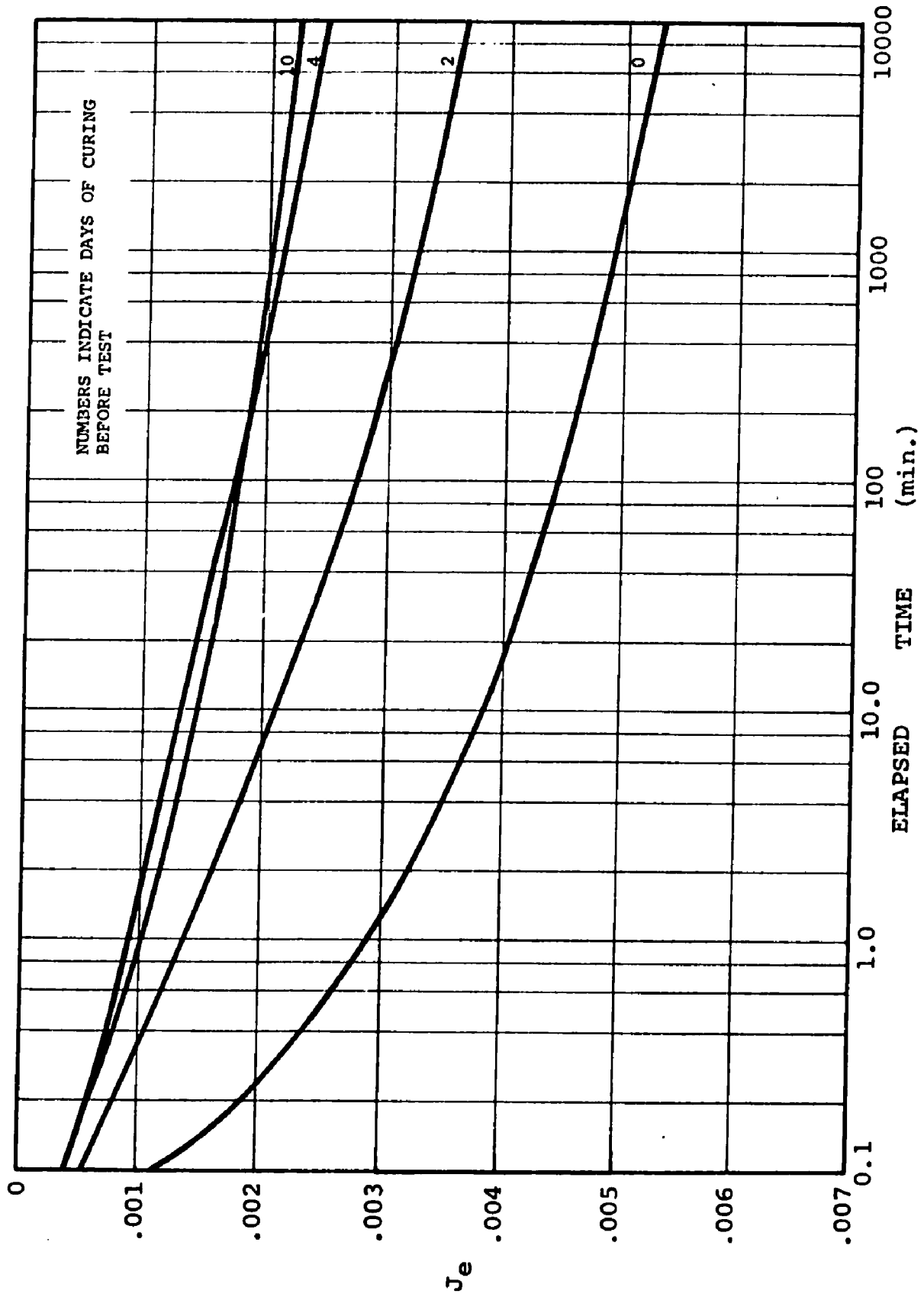


Figure 5.15 Creep functions for lime treated Trenton Channel fly ash with different aging before tests.

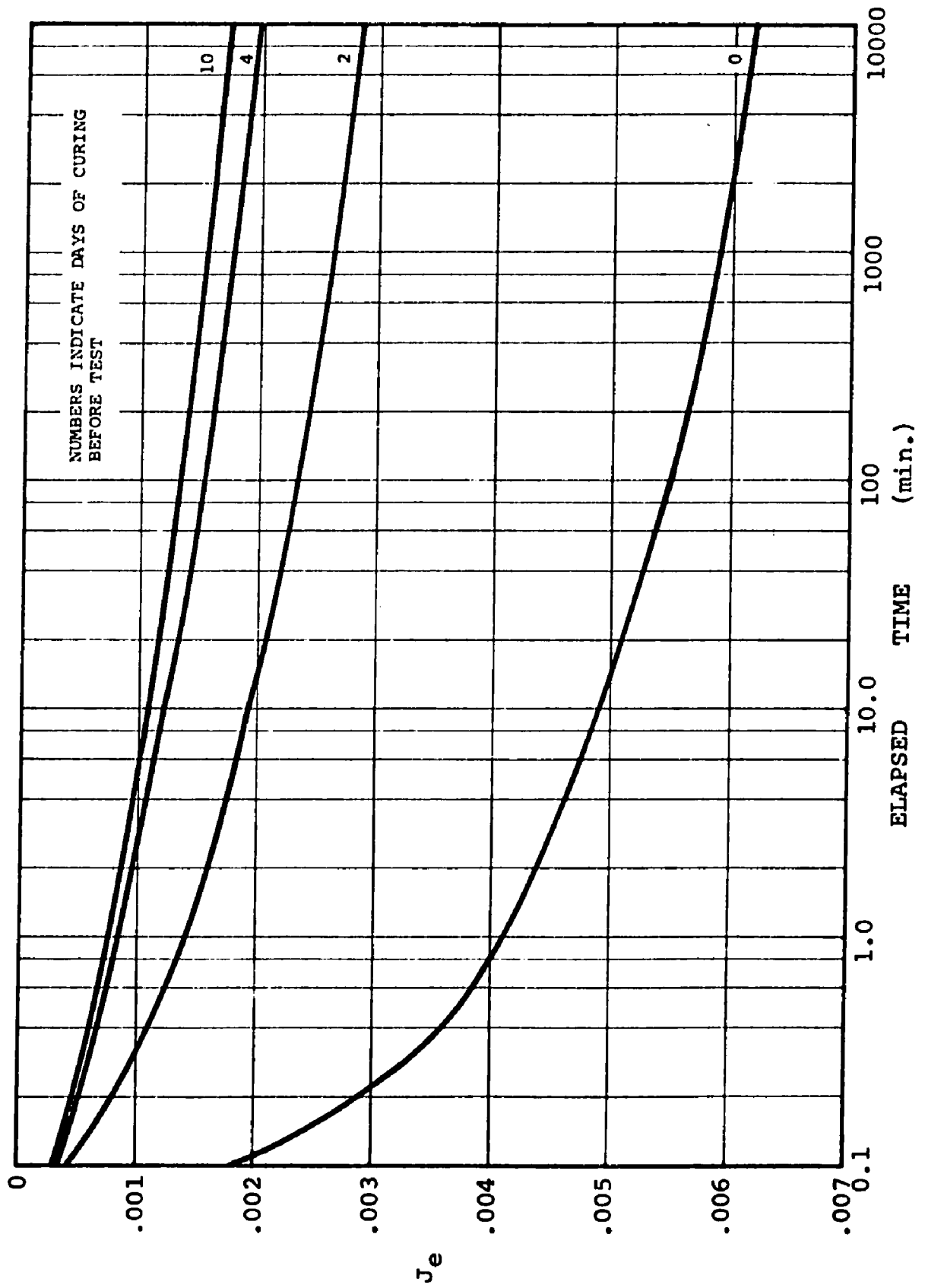


Figure 5.16 Creep functions of lime treated Marysville fly ash with different aging before tests.

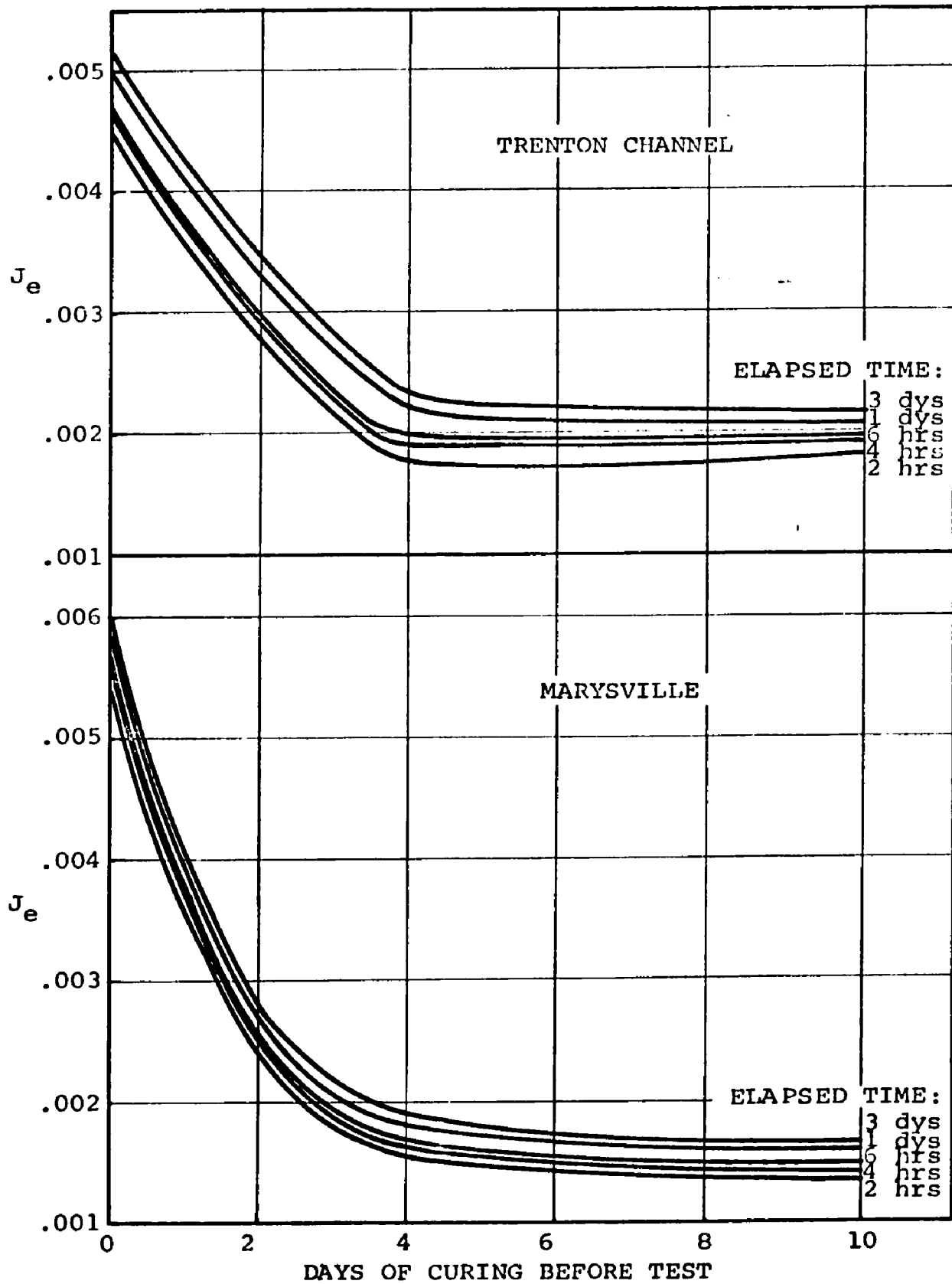


Figure 5.17 Effect of previous curing on the magnitude of creep function of lime-treated fly ashes.

TABLE 5.1 - Calculations of Void Ratio-Effective Pressure Relation by Using Creep Functions, Trenton Channel Fly Ash

Short Term Loading (2 hours per load increment):

Load Increments	1st.	2nd.	3rd.	4th.
Time (hr.)	0-2	2-4	4-6	6-8
J_{e1}^*	.00450	.00463	.00472	.00479
J_{e2}		.00443	.00456	.00465
$2J_{e3}$			.00864	.00894
$4J_{e4}$				.01700
$\epsilon(t)$	.00450	.00906	.01792	.03538
$\Delta e(\text{for } e_0 = .6)$	.007	.014	.029	.057
e	.593	.586	.571	.543

$$*J_{e1} = J_e(t-t_{L1}, t), J_{e2} = J_e(t-t_{L2}, t), J_{e3} = J_e(t-t_{L3}, t), \\ J_{e4} = J_e(t-t_{L4}, t)$$

Long Term Loading (24 hours per load increment):

Load Increment	1st.	2nd.	3rd.	4th.
Time (day)	0-1	1-2	2-3	3-4
J_{e1}	.00500	.00512	.00518	.00522
J_{e2}		.00407	.00417	.00426
$2J_{e3}$			.00660	.00684
$4J_{e4}$				.0160
$\epsilon(t)$	.00500	.00919	.01595	.02692
$\Delta e(\text{for } e_0 = .6)$	.008	.015	.026	.043
e	.592	.585	.574	.557

TABLE 5.2 - Calculations of Void Ratio-Effective
Pressure Relation by Using Creep
Functions, Marysville Fly Ash

Short Term Loading (2 hours per load increment):

Load Increment	1st.	2nd.	3rd.	4th.
Time (hrs)	0-2	2-4	4-6	6-8
J_{e_1}	.00552	.00565	.00572	.00577
J_{e_2}		.00525	.00545	.00550
$2J_{e_3}$			.01006	.01036
$4J_{e_4}$				.01928
$\epsilon(t)$	.00552	.01090	.02123	.04091
$\Delta e(\text{for } e_0=1.2)$	.012	.024	.047	.090
e	1.188	1.176	1.153	1.110

Long Term Loading (24 hours per load increment):

Load Increment	1st.	2nd.	3rd.	4th.
Time (day)	0-1	1-2	2-3	3-4
J_{e_1}	.00594	.00602	.00607	.00610
J_{e_2}		.00396	.00404	.00408
$2J_{e_3}$			.00530	.00544
$4J_{e_4}$				.00812
$\epsilon(t)$	.00594	.00998	.01541	.02374
$\Delta e(\text{for } e_0=1.2)$	.013	.022	.034	.052
e	1.187	1.178	1.166	1.148

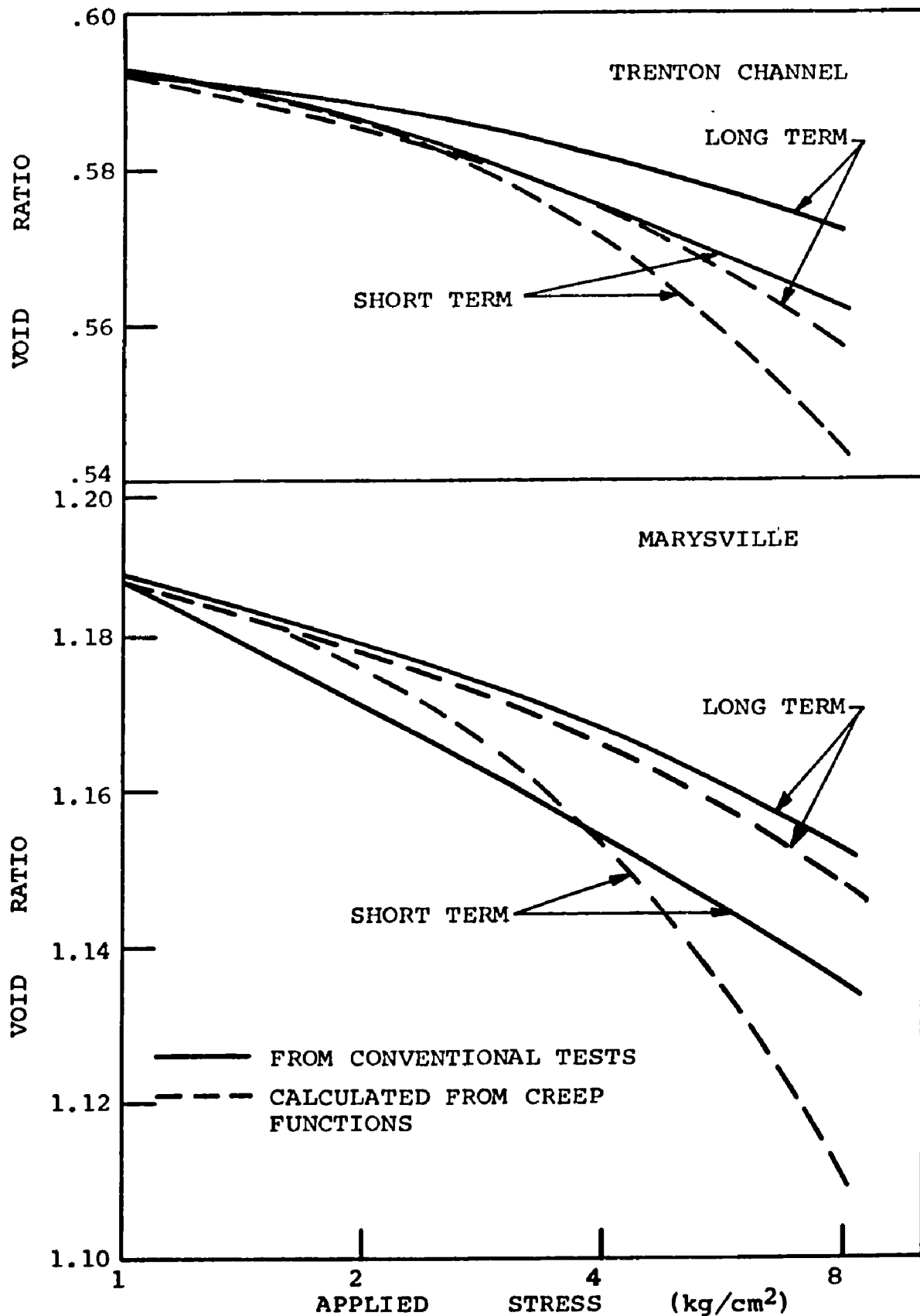


Figure 5.18 Comparisons of void ratio vs. compression stress curves between those from conventional tests and those calculated from creep functions.

integral, e.g., Equation (5.13), without any additional laboratory test. Of course, the calculation will involve solving a integral equation because the stress will no longer be a constant but a function of time. Consequently, the computation will be more complicated.

Figure 5.18 also shows that the calculated curves tend to give higher compressibility at higher pressure levels. This indicates that the influence of previous stress history on the compressibility of these fly ashes is more significant than previously anticipated. To obtain more accurate results, a non-linear viscoelastic stress-strain law may have to be employed. In this case, the functional relationship between stress and strain would lose its property of non-retroactivity. Hence the validity of using the Riemann-Stieltjes integral would no longer hold. Lubliner and Sackman (1966) suggested that the extension of linear constitutive relations of aging viscoelastic material into the region of nonlinear behavior is probably easier to accomplish by using the differential equation, rather than the integral, representation. Therefore, a further investigation on the use of nonlinear differential equations to represent aging, nonlinear viscoelastic behavior may be worthwhile. The extension of the study to the non-linear level has not been attempted in this dissertation but such extension might prove useful.

5.6. Compression of Partially Saturated Compacted Soils

The volume change characteristics of compacted fly ashes in a partially saturated condition are in reality more important

than those at full saturation due to the fact that compacted fills, embankments and road subgrades are more likely to be partially saturated.

The compression process of partially saturated soils is complicated by the nature of three-phase physical properties. According to Yoshimi and Osterberg (1963) the surface tension and vapor distillation cause water and water vapor to move from larger capillaries to smaller ones. Consequently the pore water fills the smaller voids whereas air is collected in the larger pores. The mobility of water and air depends on the degree of saturation. When the degree of saturation is low, air in the larger pores is continuous and free to flow through the soil; however, at higher degree of saturation, the water fills most of the small capillaries and occludes the air. The relative permeability of both water and air, hence the consolidation behavior of a partially saturated soil, is a function of, in addition to the porosity and the structure of the soil, the degree of saturation.

An analytic treatment on the consolidation of compacted and partially saturated soils has been attempted by Barden (1965). A formulation of a set of general equations by taking into account the variables of pore air pressure, pore water pressure, degree of saturation and porosity was made. In view of the complexity of these equations, he simplified the treatment by sub-dividing the consolidation process into a number of less general cases, each being relevant to a particular range of saturation values. Then the four complex

functions of porosity, soil structure and degree of saturation in his general treatment were replaced by simple functions of a single variable, viz., the pore pressure.

However, the classification of a consolidation process of a soil into a particular case implies that the treatment can be satisfactory only when it follows one of these idealized processes alone. The solutions would be complicated when more than one particular case has to be employed as the saturation changes from one range to the other during compression. Another shortcoming of this treatment is that the analysis was developed based on the assumption that there is no secondary compression involved in the consolidation process. In other words, his work essentially deals with the primary consolidation behavior, which may not be the governing mechanism of stress-strain relation for partially saturated soils. An insight into the over-all volume change characteristics of soils at partially saturated conditions appears to be more desirable.

During the past two decades considerable work has been done to explain shear strength and volume change behavior of partially saturated soils in terms of effective stresses.

Bishop (1960) suggested that the effective stress in an unsaturated soil can be expressed as

$$\sigma' = \sigma - u_a + \chi(u_a - u_w) \quad (5.17)$$

where σ' = effective stress

σ = total stress

u_a = pore air pressure

u_w = pore water pressure

$$u_a - u_w = \text{suction}$$

χ = parameter which depends on the degree of saturation, soil type and hysteresis effects due to wetting, drying or stress change.

The effective stress principle has been used quite successfully in predicting shear strength. However, the use of this principle in interpreting volume change characteristics has been rather controversial. Unlike shear strength which is usually analyzed at a failure state, volume change behavior is a continuous process of deformation. Therefore, while it is sufficient to compare the partially saturated and fully saturated soils at their critical state of equilibrium for the study of shear strength, it is necessary that they are comparable at every stage of deformation in examining the volume change behavior. In other words, because of complexity induced by the state of partial saturation, the volume change characteristics of partially saturated soils are much more path dependent than those of fully saturated soils.

The validity of effective stress principle in partially saturated soils was first questioned by Jennings and Burland (1962) who demonstrated experimentally that Equation (5.17) did not account for the behavior of many soils below a critical degree of saturation. Burland (1965) also maintained that in a partially saturated soil, in which the pore water exists mainly within menisci at small pores, a change in applied stress cannot be uniquely related to the change in pore pressure as it is in saturated soils. This is because the applied stress

and the pore water pressure are transmitted through the soil structure in a different manner than they are in the fully saturated soils.

The necessary conditions of the validity of the effective stress principle has been summarized by Matyas and Radhakrishna (1968) as follows:

- (a) It should satisfy the boundary cases for full saturation and for a completely dry state.
- (b) The behavior of a soil exposed to a change in stress should be predictable in terms of effective stress and should be independent of the manner in which the total stress and the pore pressure change.
- (d) It should be verified experimentally.

In partially saturated soils it is practically impossible to satisfy all three requirements and the use of the effective stress principle is necessarily limited to those cases which can be verified experimentally.

Bishop and Blight (1963) also recognized the limitations of Equation (5.17) and concluded that, in the case of partially saturated soil, it was not the effective stress only but the paths of the two components ($\sigma - u_a$) and ($u_a - u_w$) which had to be taken into account. In view of these limitations in the use of the effective stress principle, the prediction of the volume change behavior of a partially saturated soil requires a laboratory simulation of applied pressure and pore pressure conditions existing in the field.

An alternative and more general approach was developed by Matyas and Radhakrishna (1968). $(\sigma - u_a)$ and the suction were used directly as two independent variables to express soil behavior. They showed that variations in void ratio and degree of saturation could be represented by unique surfaces in a stress-void ratio and stress-degree of saturation space respectively. A similar approach was also made by Barden et al. (1969).

From the above literature study, it appears that the separation of $(\sigma - u_a)$ and $(u_a - u_w)$ as two independent variables is the most desirable method at the present time for studying volume change properties of partially saturated soils. Therefore, a limited experimental study on the effects of applied pressure and pore pressure on the volume change characteristics of compacted fly ashes was made according to this method and is discussed in the following section.

5.7 Effect of Partial Saturation on the Compressibility of compacted Fly Ashes

The compressibility behavior of partially saturated, compacted fly ashes was studied experimentally with special emphasis on

- (a) the comparison of the compressibility between fully saturated and partially saturated fly ashes,
- (b) the effects of applied pressure and suction on the volume change properties of partially saturated fly ashes, and

- (c) the effect of aging on the compressibility of partially saturated fly ashes.

The aging effect was studied in a similar manner as for the fully saturated case. The effects of applied stress and pore pressure were studied according to the method adopted by Matyas and Radhakrishna, namely, the separation of $(\sigma - u_a)$ and $(u_a - u_w)$. However, the initial condition of the compacted fly ashes was restricted to the "as compacted" condition at the optimum water contents. The study of the effect of any variation in the initial degree of saturation on the compressibility behavior was not attempted.

The laboratory set-up, sample preparation, test program and the discussion of the results are described subsequently in this section.

1. Laboratory Set-Up

In regard to the experimental technique, a few problems which do not exist in the tests on saturated samples are associated with the tests on partially saturated samples, namely,

- (a) desiccation of the specimens during tests,
- (b) side friction of specimens against the confining ring, and
- (c) capability of pore-air pressure application.

Although the desiccation of the specimens could be prevented by using the Anteus consolidometer, it does not seem possible to overcome all three problems when the tests were to be performed on the ordinary oedometer.

In view of these difficulties, the tests were conducted by using a triaxial cell with a modified anisotropic loading device. To maintain the one-dimensional volume change, the K_0 condition was imposed upon the sample by properly adjusting confining pressure. The lateral deformation was detected by a strain gage. The strain gage circuitry is shown schematically in Figure 5.19. To avoid the malfunctioning and deterioration of the strain gage, air was used to exert the confining pressure in the triaxial cell. The air leakage through membrane was minimized by using double membranes with high vacuum silicone grease in between, though traces of air leakage were still detected at higher confining pressures. The regulated pore air was provided at the top of the sample. A special porous ceramic disk with an air-entry value of 80 psi. was placed between the base cap and the bottom of the sample. The disk and the base cap were sealed circumferentially with silicone grease and a rubber sleeve to prevent air from going through the gap at the contact. Single drainage was provided from the base cap, from which a drainage line was connected to a dish filled with water at the same level as the bottom of the sample.

Because of limitations of the available test facility, neither pore water pressure nor suction was directly measured. Since the permeability of these fly ashes is high, it was assumed that, with the provision of single drainage, the pore water pressure is negligible and the suction is equivalent to pore air pressure. The vertical load was applied by using a

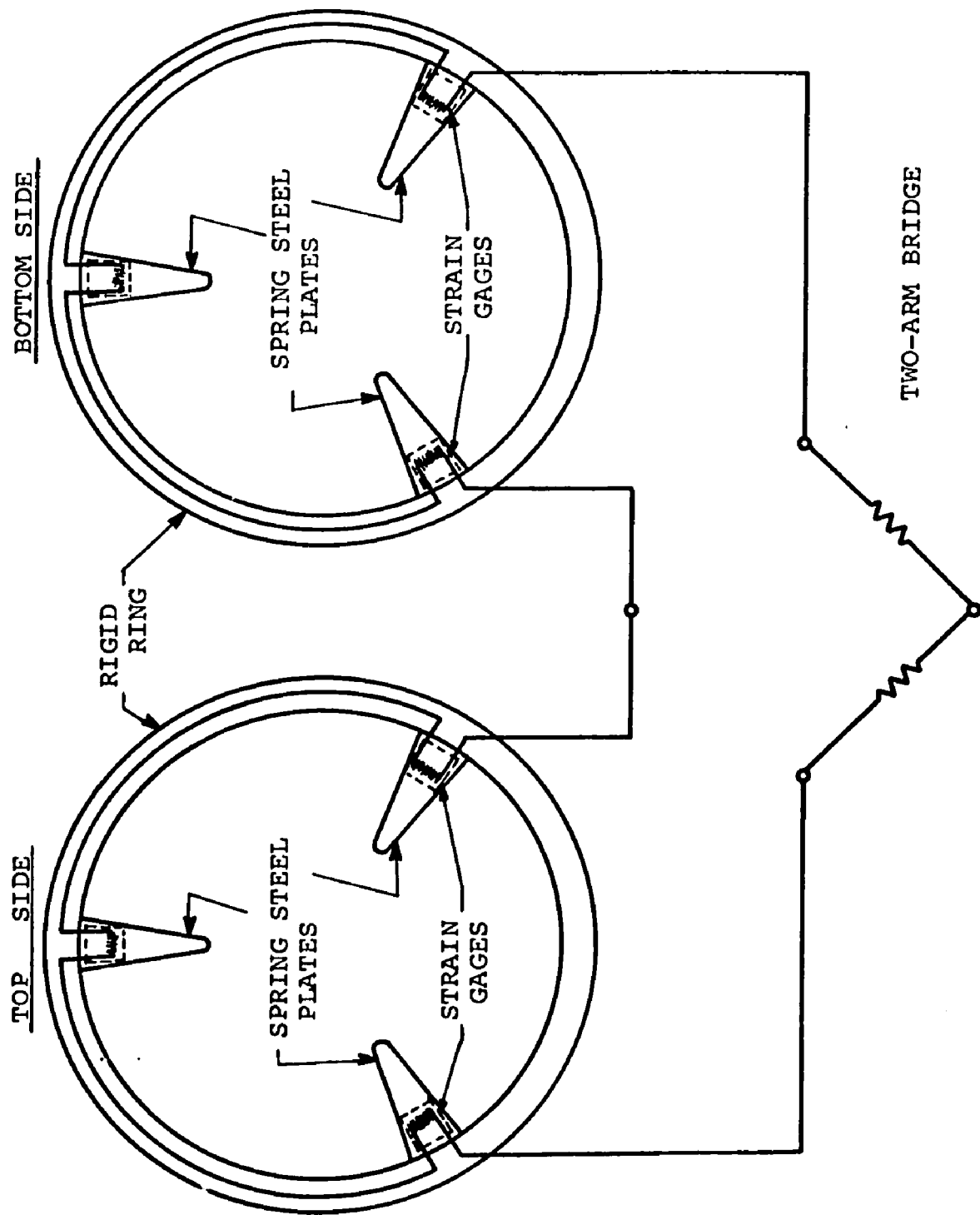


Figure 5.19 Strain gage circuits for the lateral strain detector.

lever which is capable of imposing a load four times larger than the weight hung at the free end. The vertical deformation was measured with a dial indicator attached to the top of the load piston. The set-up is illustrated by a schematic diagram in Figure 5.20 and also shown in Figure 5.21.

2. Sample Preparation

The source of the fly ashes and the method of preparing sample are identical to those for the saturated case. However, instead of compacting sample to a size of 4 inches in diameter, the size of the standard triaxial sample was adopted. All the samples were compacted with the miniature impact compactor in a compaction mold of triaxial-sample size.

3. Test Program and Test Procedure

The tests can be divided into the following three series.

Series A: Tests were run on the specimens without previous curing. The loading-time interval was 2 hours per load increment.

Series B: Samples were not cured before tests, as those in Series A. The Loading-time interval was 24 hours per load increment.

Series C: Samples were moist cured for 10 days before tests then run at 2 hours per load increment.

In each series samples were prepared either by adding the optimum amount of hydrated lime or without any lime addition. The tests were conducted either with a pore air pressure of 0.8 kg/cm^2 or without applying pore air pressure. Most of the tests were duplicated.

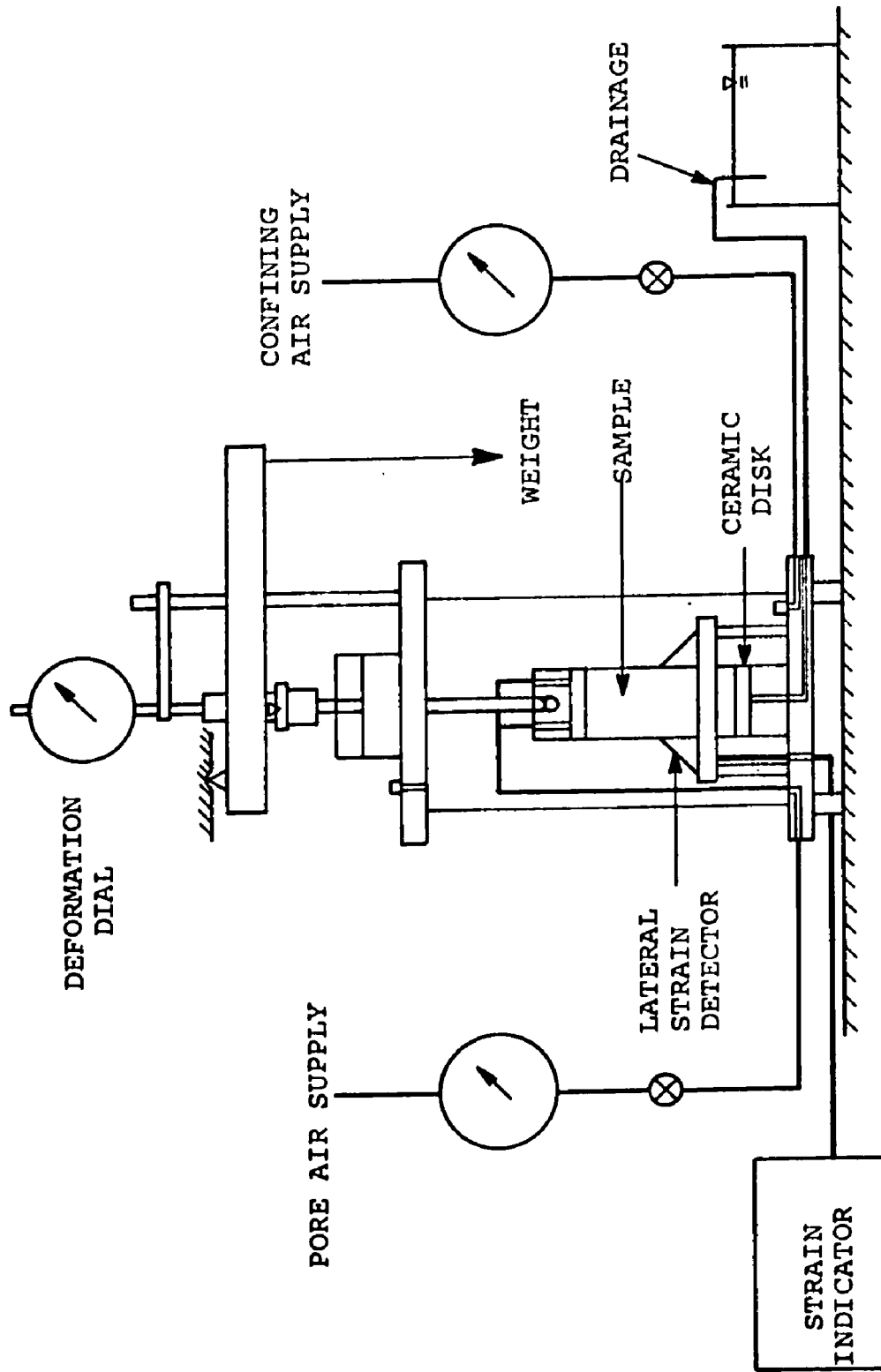


Figure 5.20 Schematic diagram of laboratory set-up for compression tests on partially saturated, compacted fly ashes.

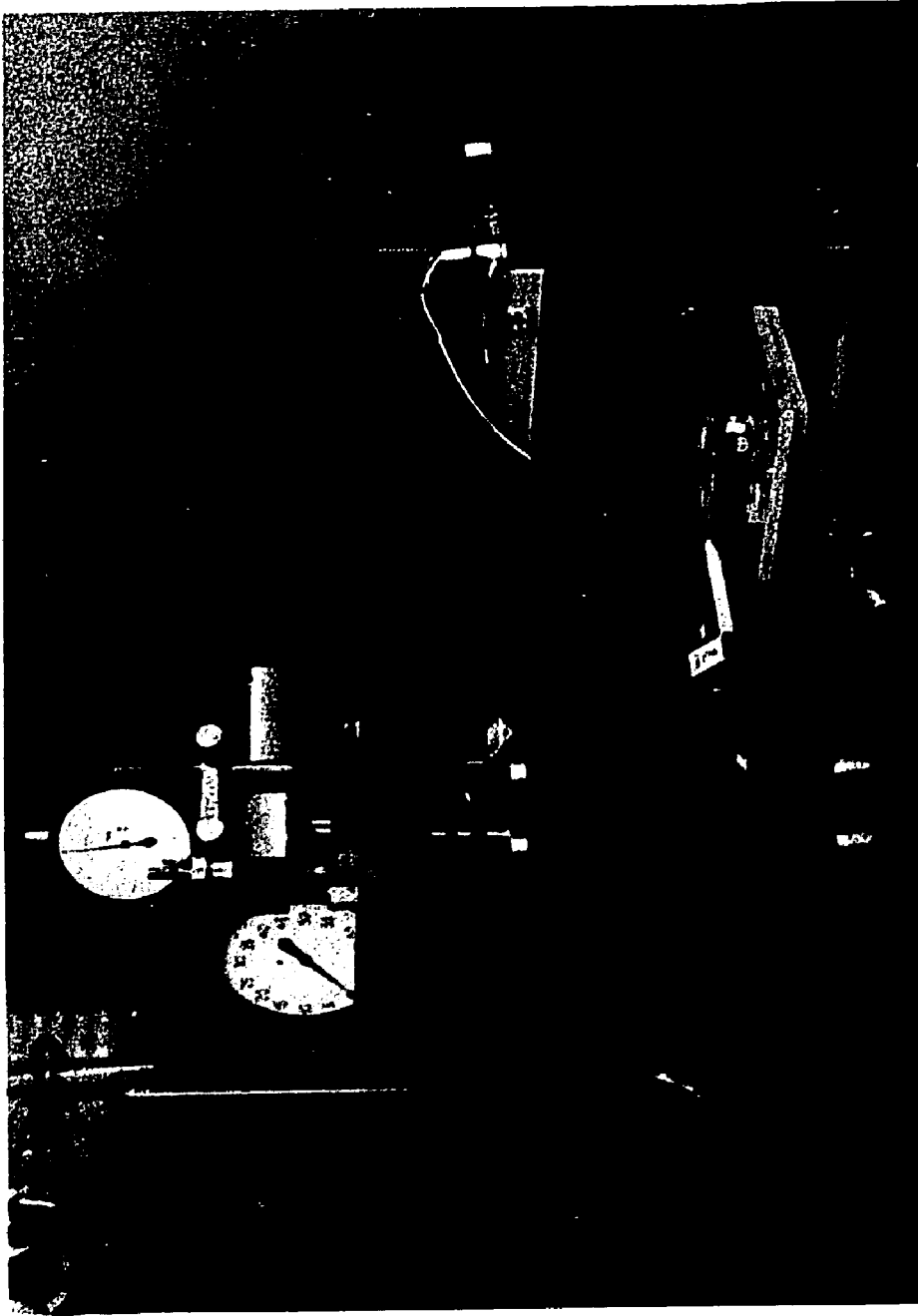


Figure 5.21 Laboratory set-up for compression tests on partially saturated, compacted fly ashes.

Before a sample was placed on the base cap, the drainage line including the ceramic disk were thoroughly saturated to insure free movement of water into or out of the sample without possible interference from air bubbles. Double membranes were put on the sample after it was set in place. The strain gage was then placed in contact with the membrane to detect the lateral deformation. For the case in which the pore air pressure was applied, the regulated air pressure was supplied through the top cap and inserted into the sample; however, for the case of zero suction, the pore-air pressure supply line was open to the atmosphere and the pore water in the sample could drain from either the top or bottom. The sample assembly was placed in a triaxial cell. To start the test, the cell pressure, the axial load and the pore air pressure were applied simultaneously. A clock was also started at the same time. The vertical deformation was recorded constantly throughout the test.

4. Discussion of Results

From the test results, the following observations and conclusions can be drawn:

(1) The deformation-time behavior of partially saturated fly ashes is similar to that in a fully saturated condition, but the magnitude of deformation in the partially saturated condition is considerably smaller. This behavior can be compared in the void ratio vs. applied pressure plots for both saturated and "as compacted" conditions shown in Figure 5.22 and 5.23. Similar phenomena have been observed by Burland

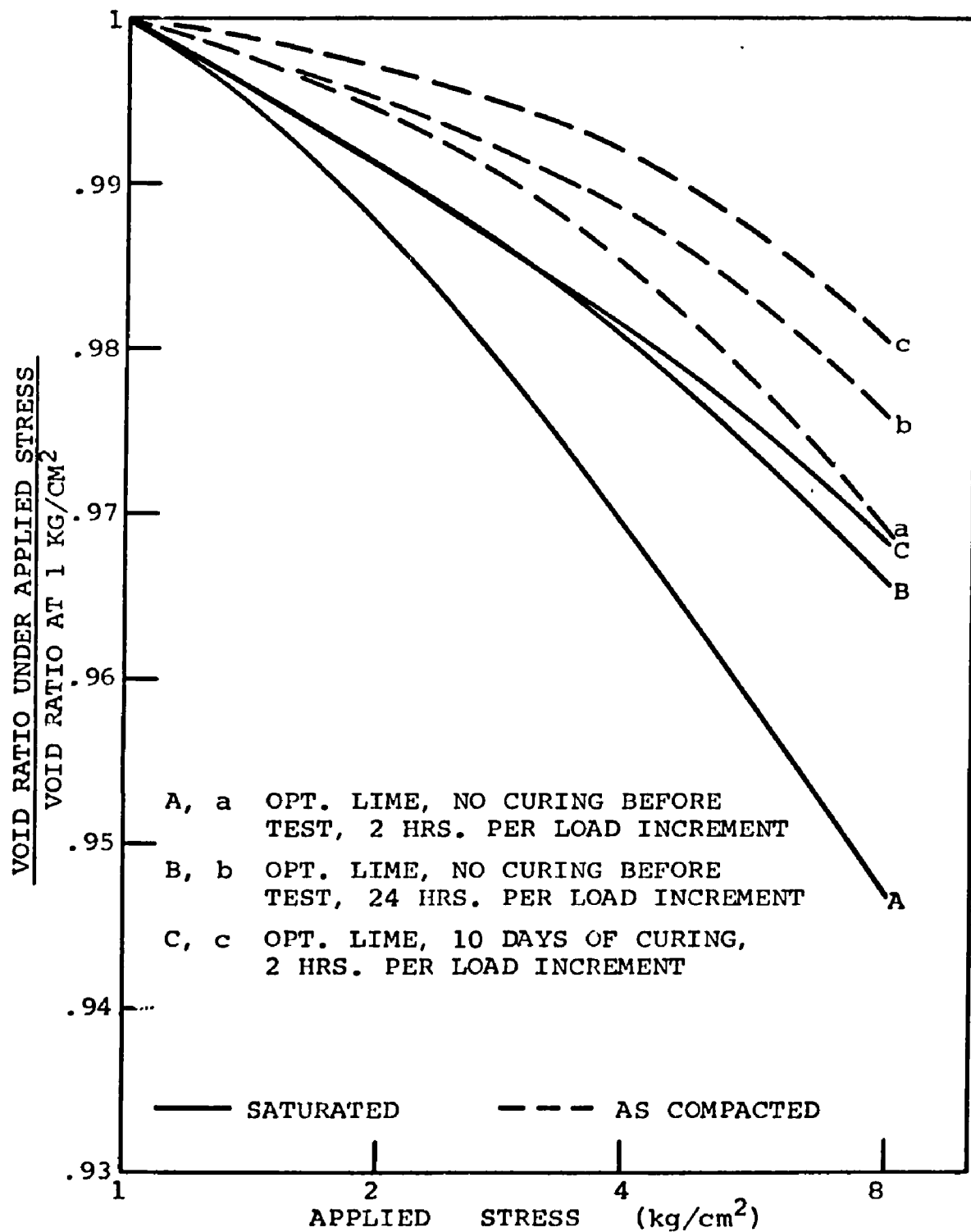


Figure 5.22 Comparison of the compressibility behavior of saturated and partially saturated, compacted fly ashes from Trenton Channel plant. Void ratios have been normalized.

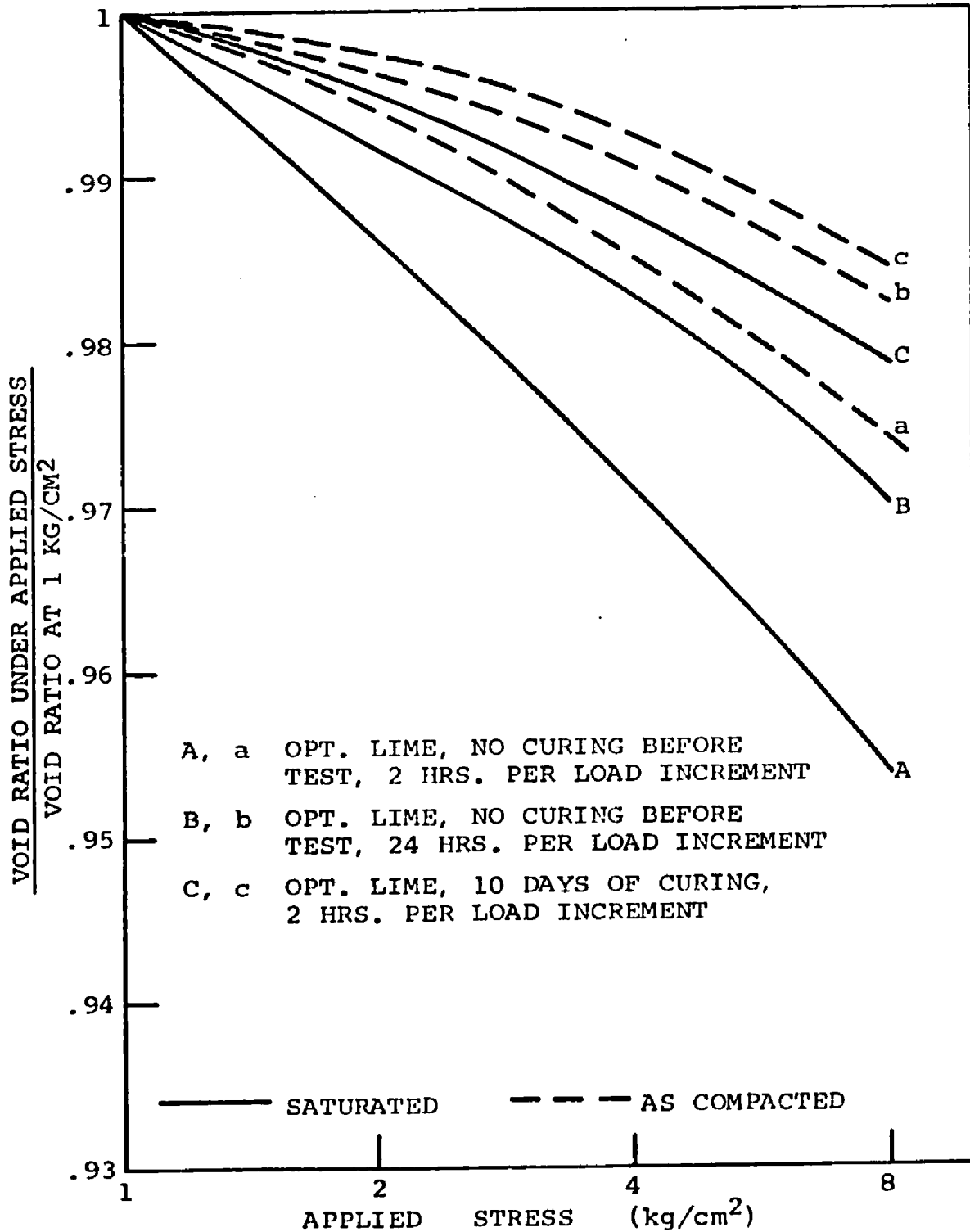


Figure 5.23 Comparison of the compressibility behavior of saturated and partially saturated, compacted fly ashes from Marysville plant. Void ratios have been normalized.

(1965) from the oedometer tests on silts. He suggested that this is due to the increase in the stability of particle contacts in the partially saturated condition. Increases in the contact forces resulting from increments in boundary stress tend to induce particle slippage and reduce the volume. However, increases in the contact forces by suction at the particle contacts will tend to stabilize the contact points. Another factor which may have contributed to the reduction in volume change for partially saturated case is the decrease in the moisture lubrication around particles. Higher energy is required to overcome the friction at the particle contacts before the grains could slip to a more stable position. This test result implies that the compressibility of "as compacted" soil field could be considerably lower than that in the fully saturated condition. Therefore it is important to take into account the field saturation condition when the laboratory test is performed to predict the settlement of the field.

(2) It can be noted from the normalized curves shown in Figure 5.24 that the effects of lime stabilization and material aging on the compressibility of compacted fly ashes are also found in the partially saturated condition. Therefore the correlation of the aging properties of the fly ash and the loading condition in the laboratory to the existing field condition is as essential as it is in the fully saturated condition.

(3) Limited experimental data on the effect of suction on the volume change behavior of partially saturated fly ashes

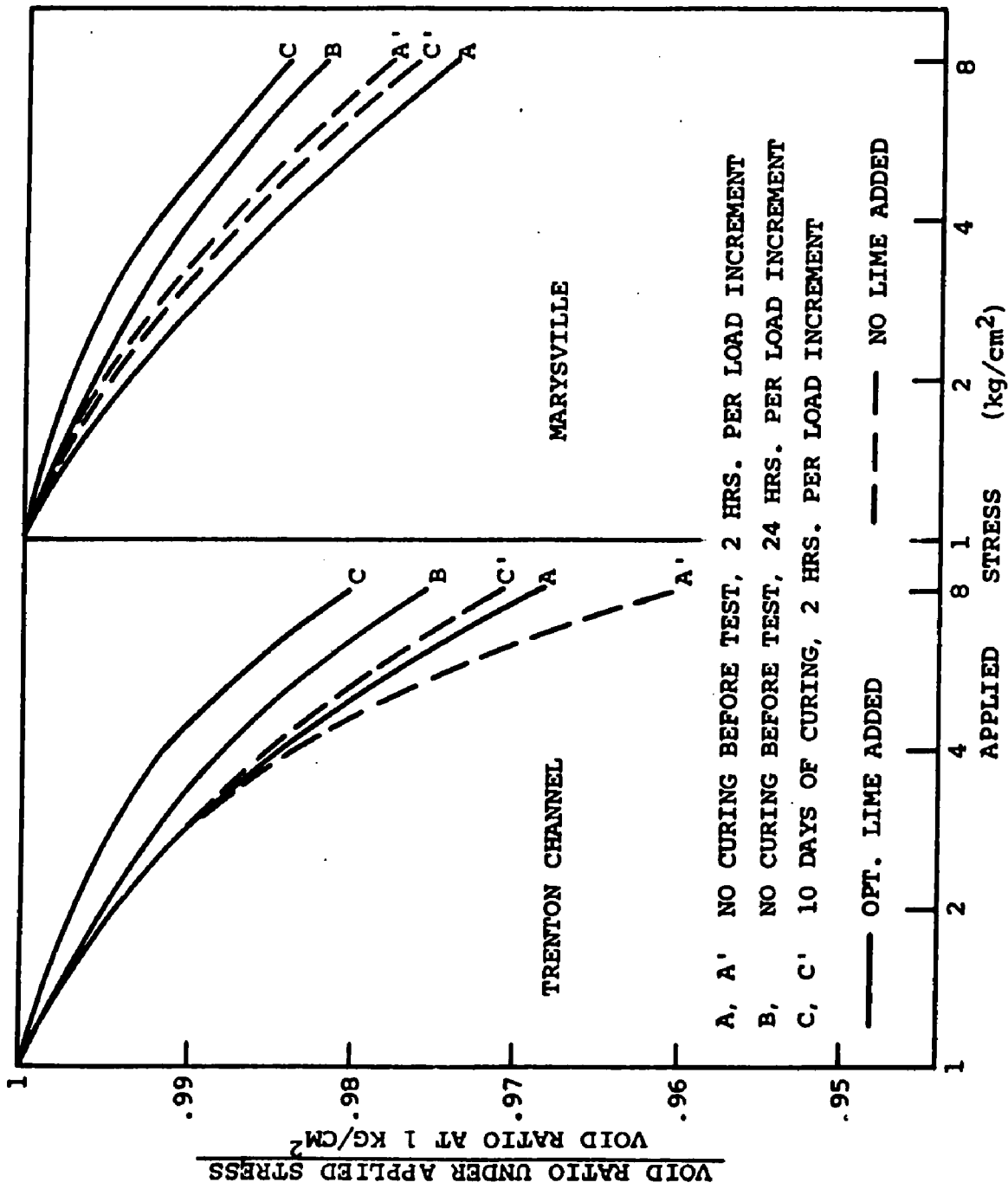


Figure 5.24 Effects of lime treatment and loading-time interval on the compressibility of compacted fly ashes. Void ratios have been normalized.

are presented in Figures 5.25 and 5.26. Some curves have been shifted along the axis of ordinates for the convenience of comparisons. It seems evident that the ashes with higher suction possess lower compressibility. This phenomenon can be also accounted for with Burland's particle-stability hypothesis as mentioned previously. This emphasizes the fact that not only the effective stress but also pore pressure and the state of the soil have to be taken into account in evaluating the compression properties of the partially saturated soils. On the other hand, these figures also show that this phenomenon becomes less significant as the aging effect becomes more substantial. The pozzolanic reaction produces the bonding between stabilizing agent and ash particles and increases the friction between particles which after reaction with lime have a rougher surface texture. As a result a rather stable soil structure is formed. The stability of soil structure caused by this aging process outweighs the particle stability contributed by the presence of suction. Therefore, the effect of suction on the volume change behavior is overshadowed by the effect of aging.

In summary, partially saturated, compacted fly ashes appear less compressible than the fully saturated ashes. The effect of aging on the compression behavior of partially saturated fly ashes is equally important as it is in the fully saturated condition. In the partially saturated fly ashes, not only effective stress but also the state of pore pressure component and the change in soil condition during a test can influence

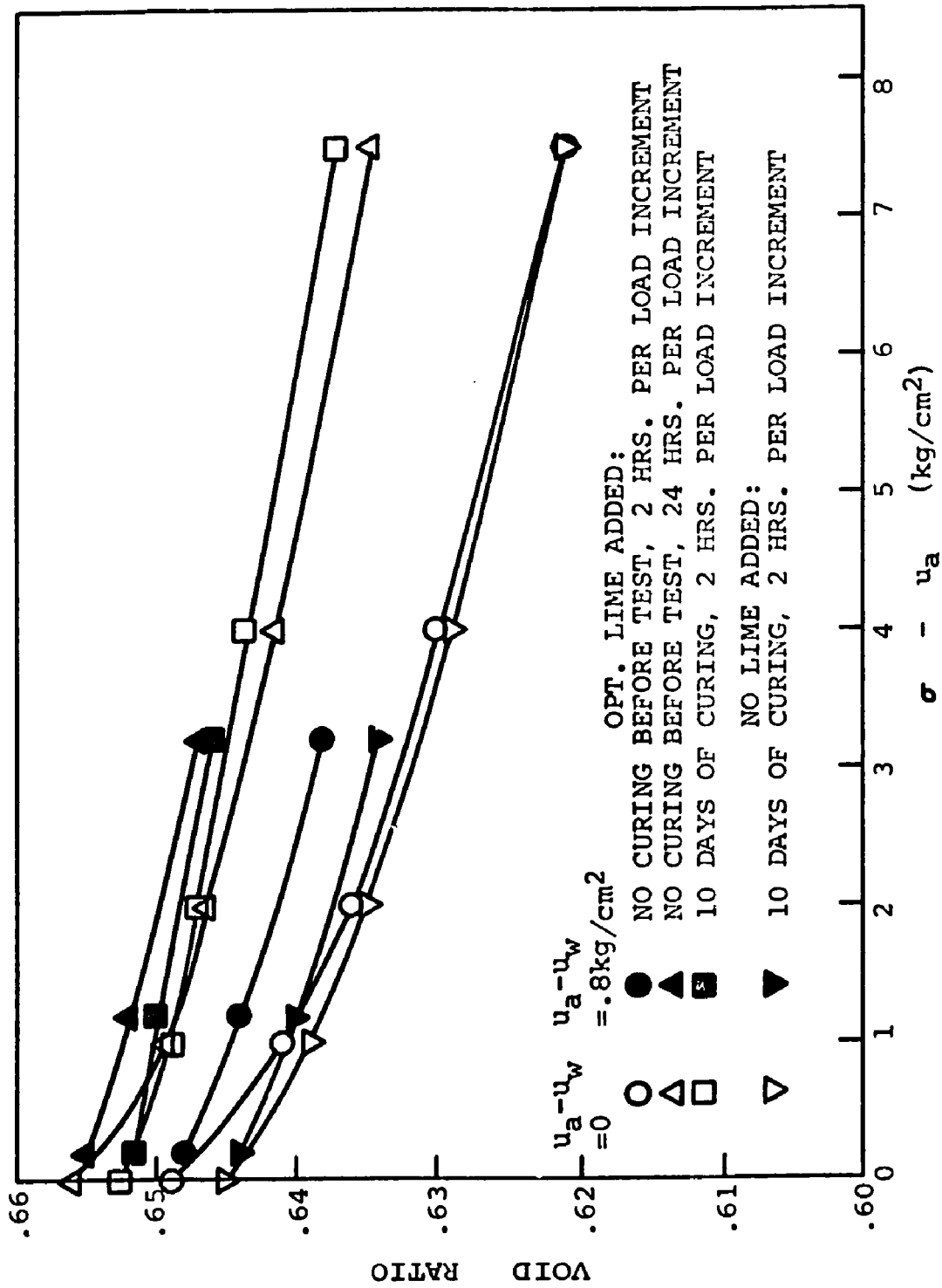


Figure 5.25 Effect of change in suction on the volume change behavior of compacted fly ash from Trenton Channel plant.

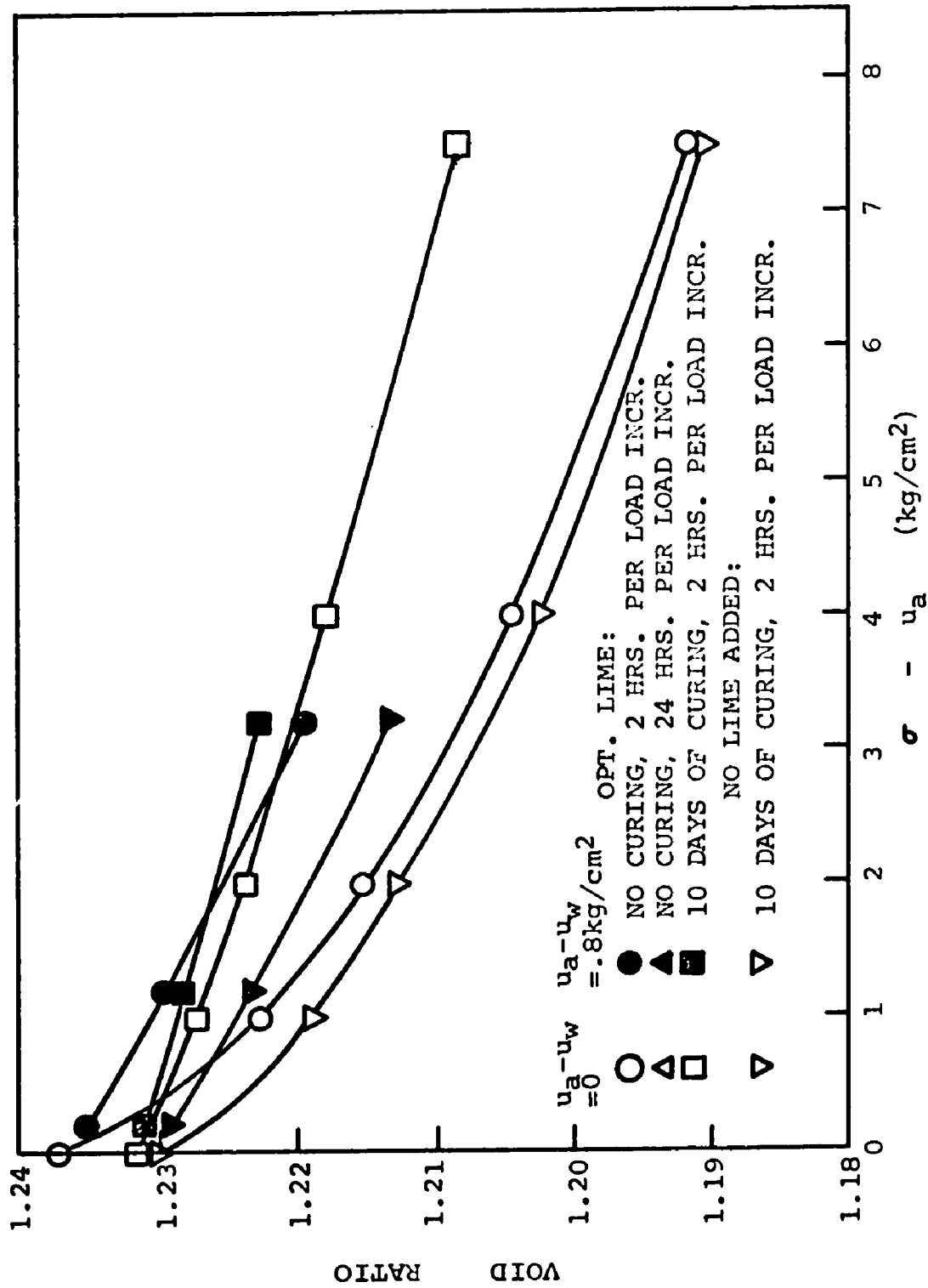


Figure 5.26 Effect of change in suction on the volume change behavior of compacted fly ash from Marysville plant.

the volume change properties. However, suction becomes less important when the aging effect comes into play because the structural stability of soil contributed by aging is greater than the influence of suction and soil saturation conditions.

CHAPTER 6

SUMMARY AND CONCLUSIONS

This dissertation is a comprehensive study of the engineering properties of some typical Michigan fly ashes. The ultimate objective of this study was to determine the suitability of fly ash as a construction material in load-bearing fills and subbases. This study includes the investigation of index properties, strength, frost susceptibility, and compressibility of compacted ash. The experiments were limited to laboratory investigations. Detailed discussions of possible variables that affect specific engineering properties of fly ash are presented in the previous chapters. Effects of material aging on these engineering properties are emphasized throughout in recognition of the pozzolanic behavior of fly ashes containing small amounts of lime.

Chapter 3 presents the summary of index properties of some typical Michigan fly ashes, and their compaction characteristics and strength. Effects of lime stabilization and curing temperature on the strength of these fly ashes are also included.

Chapter 4 contains a review of frost heave phenomena in fine grained materials. Experimental results of cement

and lime stabilization on frost heave of two Michigan ashes are also presented in this chapter.

Chapter 5 deals with the compressibility behavior of two Michigan fly ashes. Experimental results in regard to the effects of aging and partial saturation are presented. The use of a creep function obtained from viscoelastic theory for aging material is proposed for purposes of predicting pressure vs. void ratio relationships in compacted ash.

From results presented in these chapters, a number of useful conclusions can be drawn:

Strength

(1) The unconfined compressive strength of compacted, untreated Michigan fly ashes ranged from only 1.2 to 3.9 kg/cm² when samples were compacted to maximum dry density (modified AASHTO test). Moist curing and higher curing temperature did not improve their strengths. This indicates that none of these ashes exhibit significant self-hardening properties. Compacted ash with this low a strength would not be acceptable in structural fills or subbases.

(2) A substantial improvement in strength as a result of lime stabilization was observed. The addition of a few percent by weight of hydrated lime increased the compressive strength more than 10-fold after four weeks of moist curing. This strength is still slightly lower than the strength required for base courses beneath road pavements.

(3) The amount of additional lime required in fly ash to reach the highest strength varies from one ash to the other.

It ranged from 7 to 13 percent by weight for the Michigan fly ashes under study.

(4) From the test results of two Michigan fly ashes, it appears that not only the rate but also the ultimate magnitude of strength of lime treated, compacted fly ash increases with higher curing temperatures. All samples treated with optimum amounts of lime and cured at 60°C exhibited strengths approximately four times higher than those cured at 20°C after one month of curing. Strengths of samples cured at 40°C also increased more than three-fold. This implies that the favorable time for road base, fill, and embankment constructions using stabilized fly ashes is during the warmer seasons of the year.

(5) Samples of untreated, compacted ash tended to disintegrate as a result of imbibition of water during moist curing at 100% relative humidity. This suggests that partial saturation should be maintained and good drainage should be provided for a fill constructed from untreated ashes, or from stabilized ashes which have not yet reached an advanced stage of chemical hardening.

Frost Susceptibility

(1) All the untreated samples of both Trenton Channel and Marysville fly ashes exhibited excessive frost heave. The strengths resulting from self-hardening apparently were not sufficient to significantly limit frost heave in this case.

(2) Samples treated with lime failed to reduce heaving within acceptable limits. This can be attributed to the lower

rate of pozzolanic reaction in a lime-treated as opposed to a cement-treated ash. Longer curing periods, or higher curing temperatures than those used for cement treated ashes before subjecting them to freezing temperatures may be necessary in the case of lime-treated ashes.

(3) A 5-percent by weight addition of Portland cement reduced the amount of heave of the Trenton Channel fly ash to less than 7% of the original sample height, and completely eliminated heave in the Marysville fly ash. It appears that 5 percent cement is sufficient to reduce frost heave of Michigan fly ashes within tolerable limits. However, each ash which is being considered for use in the field should be investigated separately.

Compressibility

(1) Freshly compacted, saturated fly ashes exhibited consolidation characteristics similar to those of ordinary soils. In the case of lime-treated samples with longer curing periods, cementing forces between particles developed to such an extent that the internal structure was stiff enough to carry part of the initial load that would normally be carried by the pore fluid. Consequently, the primary consolidation phase cannot be distinguished readily from secondary compression during consolidation of the samples. The magnitude of deformation in lime-treated ashes was also considerably smaller than that in untreated samples.

(2) The portions of the consolidation curves which represent secondary compression are not straight lines on a logarithmic time scale (as for most ordinary soils). This most

likely occurs because compression is proceeding at a much slower rate in a material which is also becoming stiffer (age hardening) with time.

The above two findings have some important practical implications. These phenomena indicate that the primary consolidation is less significant than it would be in ordinary soils, hence the rate of settlement would tend to be independent of the thickness of the soil layer. The lower magnitude of deformation for ash cured for longer time, and the decreasing rate of secondary compression imply that the magnitude of settlement would be less than that of ordinary soils and would depend upon the aging properties of the individual material and the age of the material at which the load is applied.

(3) The compressibilities of lime treated samples either tested in long term loading or cured before testing are considerably lower than samples tested in short term loading. This emphasizes the importance of good correlation between the laboratory tests and existing field loading conditions.

(4) The four untreated, Michigan fly ashes did not show any effect of aging on their compressibility behavior. In this case, fly ash can be treated as an ordinary soil in conducting a consolidation test.

(5) The compressibility of partially saturated fly ash is lower than that in a fully saturated condition. This may be attributed to the increase in the stability of particle contacts in the partially saturated condition.

(6) Analogous to ordinary soils, the volume change properties of partially saturated, compacted fly ashes can be influenced by the pore pressure component and the change in ash saturation. However, with increase in age of a compacted ash, these effects can be overshadowed by the effect of material aging which tends to increase the stiffness of soil skeleton due to the cementitious binding between particles. An ash with this less compressible internal structure will no longer be so sensitive to changes in suction and saturation condition in ash.

(7) To reduce the laboratory effort that is required to take into account the aging effect of fly ash, a creep function derived from viscoelastic theory can be used to predict the compressibility of fly ash at different stages of aging. However, it can only provide an approximate estimate of settlement and its accuracy depends on the sensitivity of a fly ash to its previous stress history. Nevertheless, this procedure represents a considerable improvement over conventional settlement predictions which are often grossly in error when applied to a material like compacted, stabilized fly ash.

REFERENCES

- Anonymous. (1969), "Ancient Concrete Mix Makes Jet-Age Runway," Engineering News - Record, May 22, 1969, pp. 20-24.
- Barden, L. (1965), "Consolidation of Compacted and Unsaturated Clays," Geotechnique, Vol. 15, pp. 267-286.
- Barden, L. (1969), "Time Dependent Deformation of Normally Consolidated Clays and Peats," Journ. Soil Mech. and Found. Div., Proc. ASCE, Vol. 95, No. SM1, pp. 1-30.
- Barden, L., Madedor, A.O., and Sides, G.R. (1969), "Volume Change Characteristics of Unsaturated Clay," Journ. Soil Mech. Div., Proc. ASCE, Vol. 95, No. SM1, pp. 33-51.
- Beskow, G. (1935), "Soil Freezing and Frost Heaving with Special Application to Roads and Railroads," Swedish Geological Society, Series C, No. 375. Translated by J.O. Osterberg, Northwestern University, November, 1947, 145 pp.
- Bishop, A.W. (1960), "The Measurement of Pore Pressure in the Triaxial Test," Pore Pressure and Suction in Soils, Butterworths, London, pp. 38-46.
- Bishop, A.W., and Blight, G.E. (1963), "Some Aspects of Effective Stress in Saturated and Partly Saturated Soils," Geotechnique, Vol. 13, No. 3, pp. 177-197.
- Brackett, C.E. (1970), "Production and Utilization of Ash in the United States," Proc. Second Symposium on Ash Utilization, Pittsburg, pp. 11-16.
- Burgers, J.M., and Scott Blair, G.W. (1948), "Report on the Principles of Rheological Nomenclature," Proc. International Rheological Congress, Amsterdam.
- Burland, J.B. (1965), "Some Aspects of the Mechanical Behavior of Partly Saturated Soils," Proc. Symposium on Moisture Equilibria and Moisture Changes in Soils beneath Covered Areas. Australia, pp. 270-278.

- Croney, D., and Jacobs, J.C. (1967), "The Frost Susceptibility of Soils and Road Materials," Road Research Laboratory, RRL Report, LR 90 (England), 68 pp.
- Davidson, D.T., Sheeler, J.B., and Delbridge, N.C., Jr. (1958), "Reactivity of Four Types of Fly Ash with Lime," Highway Research Board, Bull. No. 193, pp. 24-31.
- DiGioia, A.M., and Nuzzo, W.L. (1970), "Fly Ash as a Structural Fill," ASCE Meeting Preprint, No. JPG-70-9, ASME-IEEE joint Power Generation Conference, Pittsburgh, 29 pp.
- Freden, S. (1965), "Mechanism of Frost Heave and Its Relation to Heat Flow," Proc. 6 th. International Conference on Soil Mech. and Found. Engrg., Vol. 1, pp. 41-45.
- Gray, D.H. (1970), "Properties of Compacted Sewage Ash," Journ. Soil Mech. and Found. Div., Proc. ASCE, Vol. 96, No. SM2, pp. 439-451.
- Gray, D.H., and Lin, Y.K. (1971), "Engineering Properties of Compacted Fly Ash," Meeting Preprint, No. 1311, ASCE National Water Resources Engineering Meeting, Phoenix, Arizona, 30 pp.
- Hester, J.A. (1967), "Fly Ash in Roadway Construction," Proc., Symposium on Fly Ash Utilization, Pittsburgh, pp. 87-100.
- Hoekstra, P. (1969), "The Physics and Chemistry of Frozen Soils," Highway Research Board, Special Report, No. 103, pp. 78-83.
- Jennings, J.E., and Burland, J.B., (1962), "Limitations to the Use of Effective Stresses in Partially Saturated Soils," Geotechnique, Vol. 12, No. 2, pp. 125-144.
- Lee, E.H. and Rogers, T.G. (1963), "Solution of Viscoelastic Stress Analysis Problem Using Measured Creep or Relaxation Functions," Journal of Applied Mechanics, Vol. 30, Translated ASME, Vol. 85, Series E.
- Leonard, R.J., and Davidson, D.T. (1959), "Pozzolanic Reactivity Study of Fly Ash," Highway Research Board, Bull. No. 231, pp. 1-17.
- Lo, K.Y. (1961), "Secondary Compression of Clays," Journ. Soil Mech. and Found. Div., Proc. Vol. 87, No. SM4, pp. 61-87.
- Lubliner, J., and Sackman, J.L. (1966), "On Aging Viscoelastic Materials," Journ. Mech. Phys. Solids, Vol. 14, pp. 25-32.

- Margason, G., and Cross, J.E. (1966), "Settlement behind Bridge Abutments," Road Research Laboratory, RRI Report, No. 48 (England), 27 pp.
- Mateos, M. (1964), "Heat Curing of Sand-Lime-Fly Ash Mixtures," Material Research and Standards, May, 1964, pp. 212-217.
- Matyas, E.L., and Radhakrishna, H.S. (1968), "Volume Change Characteristics of Partially Saturated Soils," Geotechnique, Vol. 18, No. 4, pp. 432-448.
- Minnick L.J. (1967), "Reaction of Hydrated Lime with Pulverized Coal Fly Ash," Proc. Symposium on Fly Ash Utilization, Pittsburgh, pp. 287-315.
- Minnick, L.J. (1970), "The New Fly Ash," Proc. Second Symposium on Ash Utilization, Pittsburgh, pp. 269-281.
- Mitchell, J.K. (1960), "Fundamental Aspects of Thixotropy in Soils," Journ. Soil Mech. and Found. Div., Proc. ASCE, Vol. 86, No. SM3, pp. 19-52.
- Murayama, S., and Shibata, T. (1961), "Rheological Properties of Clays," Proc. 5 th. International Conference Soil Mech. Found. Engrg., Paris, Vol. 1, pp. 269-273.
- Osler, J.C. (1967), "The Influence of Depth of Frost Penetration on the Frost Susceptibility of Soils," Canadian Geotechnical Journal, Vol. IV, No. 3, pp. 335-346.
- Penner, E. (1959), "The Mechanism of Frost Heaving in Soils," Highway Research Board, Bull. No. 225, pp. 1-22.
- Raask, E. (1968), "Cenospheres in Pulverized Fuel Ash," Journ. of the Institute of Fuel (British), pp. 339-344.
- Raymond, S. (1961), "Pulverized Fuel Ash as Embankment Material," Proc. Inst. of Civil Engrg. (London), Vol. 19, pp. 515-536.
- Raymond, S. (1965), "The Engineering Properties of P.F. Ash," Highways and Bridges (London), Vol. 33, No. 1623/4.
- Raymond, S., and Smith, P.H. (1964), "The Use of Stabilized Fly Ash in Road Construction," Civil Engrg. and Public Works Review (London), Vol. 59, Nos. 691, 692, 693, pp. 70-72, 238-240, 361-362.
- Raymond, S., and Smith, P.H. (1966), "Shear Strength, Settlement and Compaction Characteristics of Pulverized Fuel Ash," Civil Engrg. and Public Works Review, Sept., pp. 1107-1113, Oct., pp. 1265-1269.

- Savage, F.M. (1970), "Establishing a Market for Lime-Fly Ash Base," Proc. Second Symposium on Ash Utilization, Pittsburgh, pp. 165-170.
- Schiffman, R.L. (1959), "The Use of Visco-Elastic Stress-Strain Laws in Soil Testing," American Society of Testing and Materials, Special Technical Publication, No. 254, pp. 131-155.
- Second Symposium on Ash Utilization, Pittsburgh (1970): Proc., Compiled by J.H. Faber et al., U.S. Dept. of Interior, Bureau of Mines, Info. Circular No. 8488, 351 pp.
- Sherwood, P.T., and Ryley, M.D. (1966), "The Use of Stabilized Pulverized Fuel Ash in Road Construction," Road Research Laboratory, RRL Report, No. 49 (England), 44 pp.
- Snyder, M.J. (1967), "Fly Ash: Specification, Limitations, and Restrictions," Proc. Symposium on Fly Ash Utilization, Pittsburgh, pp. 37-45.
- Stouffer, D.C. (1963), "On Linear Viscoelastic Materials with Aging or Environmental-Dependent Properties," Chapter 3, Ph.D. Thesis, University of Michigan, 1968.
- Šuklje, L. (1969), "Rheological Aspects of Soil Mechanics," Wiley-Interscience, 571 pp.
- Sutherland, H.B., and Finlay, T.W. (1967), "Split Cylinder Tensile Tests on Unstabilised and Stabilised P.F. Ashes of Low Compressive Strength," Civil Engrg. and Public Works Review (London), Pp. 539-542.
- Sutherland, H.B., Finlay, T.W., and Cram, I.A. (1968), "Engineering and Related Properties of Pulverized Fuel Ash," Journ. the Institution of Hwy. Engrs., (London), Vol. 15, No. 6, pp. 19-27.
- Sutherland, H.B., and Gaskin, P.N. (1970), "Factors Affecting the Frost Susceptibility Characteristics of Pulverized Fuel Ash," Canadian Geotechnical Journal, Vol. 7, pp. 69-78.
- Symposium on Fly Ash Utilization, Pittsburgh (1967); Proc. Compiled by J.H. Faber et al., U.S. Dept. of Interior, Bureau of Mines, Info. Circular No. 8348, 345 pp.
- Taber, S. (1916), "The Growth of Crystals under External Pressure," Amer. Journ. of Science, Vol. 41, pp. 544-545.

- Tan, T.K. (1964), "Determination of the Rheological Parameters and the Hardening Coefficients of Clays," Symposium, International Union of Theoretical and Applied Mechanics, Rheology and Soil Mechanics, Grenoble, pp. 256-272.
- The Central Electricity Generating Board (1967), PFA Data Books. Part I. Load-Bearing Fill. Part II. Stabilization. Available from Central Electricity House, Manchester, England.
- Thorne, D.J., and Watt, J.D. (1965), "Composition and Pozzolanic Properties of P.F. Ashes. II. Pozzolanic Properties of Fly Ashes, as Determined by Crushing Strength Tests on Lime Mortars," Journ. Appl. Chem. (London), Vol. 15, pp. 595-604.
- Tsytoovich, N.A. (1957), "The Fundamentals of Frozen Ground Mechanics (New Investigations), 4 th. International Conf. Soil Mech. and Found. Engg., Vol. 1, pp. 116-119.
- Vincent, R.D., Mateos, M., and Davidson, D.T. (1961), "Variation in Pozzolanic Behavior of Fly Ashes," Proc., Amer. Society for Testing and Materials, Vol. 61, p. 1094.
- Watt, J.D., and Thorne, D.J. (1965), "Composition and Pozzolanic Properties of Pulverized Fuel Ashes. I. Composition of Fly Ashes from Some British Power Stations and Properties of their Component Particles," Journ. Appl. Chem. (London), Vol. 15, pp. 585-604.
- Williams, P.J. (1964), "Unfrozen Water Content of Frozen Soils and Soil Moisture Suction," Geotechnique, Vol. 14, pp. 231-246.
- Williams, P.J. (1966), "Pore Pressure at a Penetrating Frost Line and Their Prediction," Geotechnique, Vol. 16, pp. 187-208.
- Wilson, A., and Barber, E.G. (1970), "PFA Utilization in the United Kingdom," Proc., Second Symposium on Ash Utilization, Pittsburgh, pp. 144-150.
- Woollard, J.H. (1967), "Report on the Use of P.F. Ash at Elmdon Airport, Birmingham," Civil Engng. and Public Works Review (London), pp. 542-544.
- Wu, T.H., Resendiz, D., and Neukrichner, R.J. (1966), "Analysis of Consolidation by Rate Process Theory," Journ. Soil Mech. and Found. Div., Proc., ASCE, Vol. 92, No. SM6, pp. 229-248.

- Yoshimi, Y., and Osterberg, J.O. (1963), "Compression of Partially Saturated Cohesive Soils," Journ. Soil Mech. and Found. Div., Proc. ASCE, Vol. 89, No. SM4 pp. 1-24.
- Zimmer, F.V. (1967), "Problems in Fly Ash Marketing," Proc., Symposium on Fly Ash Utilization, Pittsburgh, pp. 58-68.