

Florida Institute of Technology  
150 W. University Blvd.  
Melbourne, Florida 32901-6988  
(407) 768-8000 (ext. 7555)

**Developing Specifications for Waste Glass and  
Waste-to-Energy Bottom Ash as Highway Fill  
Materials  
Volume 1 of 2 (Bottom Ash)**

June 1, 1995

Paul J. Cosentino Ph.D., P.E., Principal Investigator  
Edward H. Kalajian Ph.D., P.E., Co-Principal Investigator  
Howell H. Heck III Ph.D., P.E., Co-Principal Investigator  
Chih-Shin Shieh Ph.D., Co-Principal Investigator

Submitted to  
Robert K. H. Ho, Ph.D., P.E.  
Soils Materials Engineer  
State Materials Office  
Florida Department of Transportation  
2006 N.E. Waldo Road  
Gainesville, Florida 32609  
(904) 372-5304  
SunCom: 642-1206  
Fax: (904) 334-1648

WPA Item Number 0510650  
Contract Number B-7754

1. Report No. FL/DOT/RMC/06650-7754	2. Government Accession No.	3. Recipient's Catalog No.	
4. Title and Subtitle DEVELOPING SPECIFICATIONS FOR WASTE GLASS AND WASTE-TO-ENERGY BOTTOM ASH AS HIGHWAY FILL MATERIALS Volume 1 of 2 (BOTTOM ASH)		5. Report Date June 1995	
		6. Performing Organization Code	
7. Author's P. J. Cosentino, E. H. Kalajian, C-S. Shieh and H. H. Heck		8. Performing Organization Report No.	
9. Performing Organization Name and Address Florida Institute of Technology Civil Engineering Program 150 West University Blvd. Melbourne, FL 32901-6988 407-768-8000 ext 8048		10. Work Unit No. (TRAIS)	
		11. Contract or Grant No. C-7754 WPA # 0510650	
12. Sponsoring Agency Name and Address Florida Department of Transportation 605 Suwannee Street Tallahassee, Florida 32399-0450		13. Type of Report and Period Covered Final Report October 1992 to October 1994	
		14. Sponsoring Agency Code 99700-7587-119	
15. Supplementary Notes			
16. Abstract <p>Municipal waste combustor (MWC) bottom ash from a Mass Burn (MB) and Refuse Derived Fuel ( RDF) Waste-to-Energy facilities was evaluated for potential use as a highway fill material. MWC bottom ash has the physical and geotechnical properties necessary for many highway applications and meets existing environmental acceptability regulations. Bottom ash is classified as A-1a (0). The moisture-density, permeability and unconfined compressive strength of the bottom ash were a function of compaction energy and moisture content with behavior similar to many conventional fill materials. Compacted bottom ash has a negligible shrinkage and swell potential when saturated, however the ash swelled when allowed to air dry. The stress-strain characteristics of both ash types behave similarly to sand. Both ashes develop some cohesion that is attributed to pozzolonic cementing reactions occurring in the bottom ash. The angle of internal friction increases with the density of the compacted bottom ash. Values for these geotechnical properties as well as elastic and resilient moduli, CBR and LBR are presented.</p> <p>Concentrations of leaching of trace metals (Ag, As, Cd, Cr, and Pb) in the leachate from compacted ash columns subjected to DDW and SAR were below the EPA toxicity standard and decreased as a function of time. For Ag, As and Cd, most of the leachate concentrations were below the drinking water standard.</p> <p>Based on the study findings, developmental specifications for incorporation into Florida DOT " Standard Specifications for Road and Bridge Constuction" have been proposed for using bottom ash in highway fill applications.</p>			
17. Key Words MWC Bottom Ash, Highway Applications Geotechnical Properties, Leaching Properties		18. Distribution Statement Document is available to the U.S. public through the National Technical Information Service, Springfield, Virginia 22161	
19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No of Pages 145	22. Price

# METRIC (SI\*) CONVERSION FACTORS

## APPROXIMATE CONVERSIONS TO SI UNITS

Symbol When You Know Multiply By To Find Symbol

### LENGTH

In	Inches	2.54	millimetres	mm
ft	feet	0.3048	metres	m
yd	yards	0.914	metres	m
mi	miles	1.61	kilometres	km

### AREA

In <sup>2</sup>	square inches	645.2	millimetres squared	mm <sup>2</sup>
ft <sup>2</sup>	square feet	0.0929	metres squared	m <sup>2</sup>
yd <sup>2</sup>	square yards	0.836	metres squared	m <sup>2</sup>
mi <sup>2</sup>	square miles	2.59	kilometres squared	km <sup>2</sup>
ac	acres	0.395	hectares	ha

### MASS (weight)

oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg

### VOLUME

fl oz	fluid ounces	29.57	millilitres	mL
gal	gallons	3.785	litres	L
ft <sup>3</sup>	cubic feet	0.0328	metres cubed	m <sup>3</sup>
yd <sup>3</sup>	cubic yards	0.0765	metres cubed	m <sup>3</sup>

NOTE: Volumes greater than 1000 L shall be shown in m<sup>3</sup>.

### TEMPERATURE (exact)

°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C
----	------------------------	----------------------------	---------------------	----

## APPROXIMATE CONVERSIONS TO SI UNITS

Symbol When You Know Multiply By To Find Symbol

### LENGTH

mm	millimetres	0.039	inches	in
m	metres	3.28	feet	ft
m	metres	1.09	yards	yd
km	kilometres	0.621	miles	mi

### AREA

mm <sup>2</sup>	millimetres squared	0.0016	square inches	in <sup>2</sup>
m <sup>2</sup>	metres squared	10.764	square feet	ft <sup>2</sup>
km <sup>2</sup>	kilometres squared	0.39	square miles	mi <sup>2</sup>
ha	hectares (10 000 m <sup>2</sup> )	2.53	acres	ac

### MASS (weight)

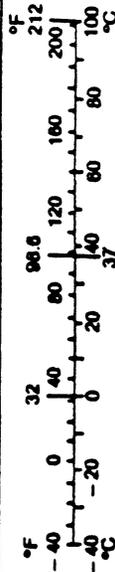
g	grams	0.0353	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams (1 000 kg)	1.103	short tons	T

### VOLUME

mL	millilitres	0.034	fluid ounces	fl oz
L	litres	0.264	gallons	gal
m <sup>3</sup>	metres cubed	35.315	cubic feet	ft <sup>3</sup>
m <sup>3</sup>	metres cubed	1.308	cubic yards	yd <sup>3</sup>

### TEMPERATURE (exact)

°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F
----	---------------------	-------------------	------------------------	----



These factors conform to the requirement of FHWA Order 5190.1A.

\* SI is the symbol for the International System of Measurements

## Table of Contents

ACKNOWLEDGMENTS.....	v
ABSTRACT.....	vii
1.0 INTRODUCTION.....	1
1.1 Ash Management Practices.....	1
1.2 Objectives.....	2
2.0 PREVIOUS MWC BOTTOM ASH STUDIES.....	3
2.1 Metals in WTE Ash.....	3
2.2 Engineering Investigations of Ash.....	4
2.2.1 Highway Applications.....	5
2.2.1.a Federal Highway Administration Ash Study .....	5
2.2.1.b SEMASS Ash Study.....	6
2.2.1.c CRRA and UCONN Ash Study.....	6
2.2.1.d Wheelabrator Ash Study.....	6
2.2.1.e Denmark Ash Study .....	7
2.2.1.f New York Studies .....	7
2.2.2 Building Construction Material.....	8
2.2.2.a Concrete Block Applications.....	8
2.2.2.b Light-Weight and Coarse Aggregate.....	8
2.2.2.c Partial Replacement of Aggregate.....	9
2.3 Geotechnical Properties.....	10
2.3.1 Grain Size Distribution .....	10
2.3.2. Moisture Content.....	10
2.3.3 Specific Gravity.....	11
2.3.4 Absorption.....	11
2.3.5 Bulk Rodded Unit Weight.....	11
2.3.6 Moisture-Density Relationship.....	12
2.3.7 Permeability .....	12
2.3.8 Swelling.....	12
2.3.9 Unconfined Compressive Strength.....	13
2.3.10 Consolidated Drained Triaxial Shear .....	13
2.3.11 Elastic and Resilient Moduli .....	14
2.3.12 California Bearing Ratio (CBR) and Limerock Bearing Ratio (LBR).....	14
2.4 Existing Regulations.....	14
2.4.1 Current MWC Ash Regulations .....	14
2.4.2 Environmental Issues .....	15
3.0 METHODOLOGY AND EXPERIMENTAL PROCEDURE .....	18
3.1 Description of MWC Facilities.....	18
3.1.1 Refuse Derived Fuel (RDF) Facilities.....	18
3.1.2 Mass Burn (MB) Facilities.....	19
3.1.3 WTE Survey Development.....	19
3.2 Ash Sampling.....	19
3.3 Bottom Ash Engineering Properties .....	20
3.3.1 Physical Composition .....	20
3.3.2 Grain Size Distribution .....	20

## Table of Contents (continued)

3.3.3	Moisture Content.....	21
3.3.4	Specific Gravity.....	21
3.3.5	Absorption.....	22
3.3.6	Bulk Rodded Unit Weight.....	22
3.3.7	Moisture-Density Relationship.....	22
3.3.8	Permeability .....	23
3.3.9	Swelling and Shrinkage Potential .....	25
3.3.10	Unconfined Compressive Strength.....	25
3.3.11	Consolidated Drained (CD) Triaxial Shear.....	26
3.3.12	Elastic Modulus .....	27
3.3.13	Resilient Modulus.....	27
3.3.14	California Bearing Ratio (CBR).....	28
3.3.15	Limerock Bearing Ratio (LBR).....	29
3.4	WTE Bottom Ash Environmental Tests .....	29
3.4.1	Loss on Ignition .....	29
3.4.2	Elemental Analysis.....	30
3.4.3	Mineralogy .....	31
3.4.4	Pozzolanic Activity .....	32
3.4.5	Column Leaching Test.....	32
4.0	PHYSICAL, GEOTECHNICAL AND ENVIRONMENTAL PROPERTIES.....	43
4.1	Physical Composition .....	43
4.1.1	Visual Classification.....	43
4.1.2	Specific Gravity.....	43
4.1.3	Moisture Content.....	44
4.1.4	Absorption.....	44
4.1.5	Mineralogy .....	45
4.1.6	Pozzolanic Activity .....	45
4.1.7	Loss on Ignition .....	45
4.2	Engineering Classification.....	45
4.3	Bulk Rodded Unit Weight.....	46
4.4	Moisture Density Relationships .....	46
4.5	Permeability .....	46
4.6	Shrink-Swell Potential .....	47
4.7	Strength-Deformation.....	48
4.7.1	Unconfined Compressive Strength.....	48
4.7.2	Consolidated Drained Triaxial Shear .....	49
4.7.3	Elastic Modulus .....	50
4.7.4	Resilient Modulus.....	51
4.7.5	California Bearing Ratio (CBR).....	51
4.7.6	Limerock Bearing Ratio (LBR).....	52
4.8	Chemical and Environmental Analyses.....	54
4.8.1	Selected Elemental Concentrations of MWC Bottom Ash.....	54
4.8.2	Reproducibility of Column Leaching Test.....	55

## Table of Contents (continued)

4.8.3 Leaching of Calcium .....	56
4.8.3.a MB Ash Results .....	56
4.8.3.b RDF Ash Results .....	57
4.8.4 Leaching of Arsenic .....	57
4.8.4.a MB Ash Results .....	57
4.8.4.b RDF Ash Results .....	59
4.8.5 Leaching of Chromium .....	59
4.8.5.a MB Ash Results .....	59
4.8.5.b RDF Ash Results .....	61
4.8.6 Leaching of Lead .....	61
4.8.6.a MB Ash Results .....	61
4.8.6.b RDF Ash Results .....	63
4.8.7 Leaching of Cadmium and Silver .....	63
4.9 Data Base .....	64
4.10 Availability, Costs and Regulatory Aspects .....	64
5.0 CONCLUSIONS .....	121
6.0 DEVELOPMENTAL SPECIFICATIONS FOR USING BOTTOM ASH IN HIGHWAY APPLICATIONS .....	124
7.0 RECOMMENDATIONS .....	127
8.0 REFERENCES .....	128

## List of Tables

Table 2.1 Previous Investigations of Moisture-Density Relationship .....	17
Table 3.1 Specifications followed to evaluate physical composition .....	35
Table 3.2 CD triaxial shear testing program on bottom ash.....	36
Table 3.3 Conditions for graphite furnace AAS with Zeeman background correction .....	37
Table 3.4 Chemicals used to prepare a primary solution for a 4-liter synthetic acid rain.....	38
Table 4.1 Summary of WTE surveys completed by July 15,1993 .....	65
Table 4.2 Summary of physical properties for MWC bottom ash .....	66
Table 4.3 Moisture-density results for MWC bottom ash.....	67
Table 4.4 CD triaxial shear results for MWC bottom ash .....	68
Table 4.5 Selected elemental concentrations of standard reference material 1633a coal fly ash from NIST and percent recovery using the technique of HF-H3BO3 digestion.....	69
Table 4.6 Comparison of selected elemental concentrations (mg/kg) of bottom ash .....	70
Table 4.7 Data base of MB and RDF bottom ash properties.....	71

## List of Figures

Figure 3.1 Survey of Statewide Waste-to-Energy Plants .....	39
Figure 4.1 Physical composition for mass-burn and refuse-derived-fuel MWC bottom ash.....	74
Figure 4.2 Scanning electron micrograph of MWC bottom ash prior to aging .....	75
Figure 4.3 Scanning electron micrograph of mineral fibers linking ash particles together .....	76
Figure 4.4 Enrichment of Al, Ca, and Si on the material fiber found in the MWC bottom ash at 40°C with 33% saturation (7% water content) for 5- day .....	77
Figure 4.5 Grain size distribution for MB and RDF bottom ash.....	78
Figure 4.6 Moisture density relationship for MB bottom ash .....	79
Figure 4.7 Moisture density relationship for MB bottom ash .....	80
Figure 4.8 Moisture density relationship for RDF bottom ash .....	81
Figure 4.9 Moisture density relationship for RDF bottom ash .....	82
Figure 4.10 Permeability versus moisture content for MB .....	83
Figure 4.11 Permeability versus moisture content for the MB.....	84
Figure 4.12 Range of axial change versus time of compacted MB .....	85
Figure 4.13 Range of axial change versus time of compacted MB bottom ash, passing #4 sieve compacted following ASTM D-698 saturated for 34 days and allowed to air dry .....	86
Figure 4.14 Unconfined compressive strength versus moisture content for standard and modified compaction of mass-burn and refuse-derived-fuel WTE bottom ash passing the #4 sieve .....	87
Figure 4.15 Unconfined compressive strength versus moisture content for standard and modified compaction of mass-burn and refuse-derived-fuel WTE bottom ash passing the #8 sieve .....	88
Figure 4.16 Typical stress strain curves for loose and dense MB bottom ash specimens (1 psi = 6.895 kPa) Dense (#4, 100%, 15 psi), Loose (#8, 95%, 10 psi) .....	89
Figure 4.17 Typical stress strain curves for loose and dense RDF bottom ash specimens (1 psi = 6.895 kPa) Dense (#4, 100%, 15 psi), Loose (#8, 95%, 10 psi) .....	90
Figure 4.18 Angle of internal friction versus dry unit weight for MB and RDF MWC bottom ash (1 pcf = 0.157 kN/m <sup>3</sup> ) .....	91
Figure 4.19 Elastic modulus versus dry unit weight for MB bottom ash over a strain difference of 0.2% (1 psi = 6.895 kPa,.....	92
Figure 4.20 Elastic modulus versus dry unit weight for RDF bottom ash over a strain difference of 0.2% (1 psi = 6.895 kPa,.....	93
Figure 4.21 Unsoaked and soaked CBR and LBR values versus moisture content for MB bottom ash passing the #4 sieve compacted following ASTM D-1557 .....	94

## List of Figures (continued)

Figure 4.22 Unsoaked and soaked CBR and LBR values versus moisture content for RDF bottom ash passing the #4 sieve compacted following ASTM D-1557 .....	95
Figure 4.23 Leaching of calcium using DDW and SAR as leaching fluids.....	96
Figure 4.24 Accumulated percent leaching of calcium using DDW and SAR as leaching fluids .....	96
Figure 4.25 Leaching of calcium using DDW at different flow rate.....	97
Figure 4.26 Accumulated percent leaching of calcium using DDW at different flow rate.....	97
Figure 4.27 Leaching of calcium using SAR at different flow rate.....	98
Figure 4.28 Accumulated percent leaching of calcium using SAR at different flow rate.....	98
Figure 4.29 Variation in calcium leaching from 15.2-cm and 45.7-cm columns subjected to DDW.....	99
Figure 4.30 Accumulated percent of calcium leached from 15.2-cm and 45.7-cm columns subjected to DDW.....	99
Figure 4.31 Variation in calcium leaching from 6-in columns subjected to DDW using Pinellas County MB ash and Palm Beach County RDF ash.....	100
Figure 4.32 Variation in calcium leaching from 18-in columns subjected to DDW using Pinellas County MB ash and Palm Beach County RDF ash.....	100
Figure 4.33 Variation in calcium leaching from 18-in columns subjected to SAR using Pinellas County MB ash and Palm Beach County RDF ash .....	101
Figure 4.34 Leaching of arsenic using DDW and SAR as leaching fluids.....	101
Figure 4.35 Accumulated percent leaching of arsenic using DDW and SAR as leaching fluids .....	102
Figure 4.36 Leaching of arsenic using DDW at different flow rate.....	102
Figure 4.37 Accumulated percent leaching of arsenic using DDW at different flow rate.....	103
Figure 4.38 Leaching of arsenic using SAR at different flow rate .....	103
Figure 4.39 Accumulated percent leaching of arsenic using SAR at different flow rate.....	104
Figure 4.40 Variation in arsenic leaching from 15.2-cm and 45.7-cm columns subjected to DDW.....	104
Figure 4.41 Accumulated percent of arsenic leached from 15.2-cm and 45.7-cm columns subjected to DDW.....	105
Figure 4.42 Variation in arsenic leaching from 15.2-cm and 45.7-cm columns subjected to SAR.....	105
Figure 4.43 Accumulated percent of arsenic leached from 15.2-cm and 45.7-cm columns subjected to SAR .....	106
Figure 4.44 Variation in arsenic leaching from 6-in columns subjected to DDW using Pinellas County MB ash and Palm Beach County RDF ash.....	106
Figure 4.45 Variation in arsenic leaching from 18-in columns subjected to DDW using Pinellas County MB ash and Palm Beach County RDF ash.....	107

## List of Figures (continued)

Figure 4.46 Variation in arsenic leaching from 18-in columns subjected to SAR using Pinellas County MB ash and Palm Beach County RDF ash .....	107
Figure 4.47 Leaching of chromium using DDW and SAR as leaching fluids.....	108
Figure 4.48 Accumulated percent leaching of chromium using DDW and SAR as leaching fluids .....	108
Figure 4.49 Leaching of chromium using DDW at different flow rate.....	109
Figure 4.50 Accumulated percent leaching of chromium using DDW at different flow rate.....	109
Figure 4.51 Leaching of chromium using SAR at different flow rate .....	110
Figure 4.52 Accumulated percent leaching of chromium using SAR at different flow rate.....	110
Figure 4.53 Variation in chromium leaching from 15.2-cm and 45.7-cm columns subjected to DDW.....	111
Figure 4.54 Accumulated percent of chromium leached from 15.2-cm and 45.7-cm columns subjected to DDW.....	111
Figure 4.55 Variation in chromium leaching from 15.2-cm and 45.7-cm columns subjected to SAR.....	112
Figure 4.56 Accumulated percent of chromium leached from 15.2-cm and 45.7-cm columns subjected to SAR .....	112
Figure 4.57 Variation in chromium leaching from 6-in columns subjected to DDW using Pinellas County MB ash and Palm Beach County RDF ash .....	113
Figure 4.58 Variation in chromium leaching from 18-in columns subjected to DDW using Pinellas County MB ash and Palm Beach County RDF ash .....	113
Figure 4.59 Variation in chromium leaching from 18-in columns subjected to SAR using Pinellas County MB ash and Palm Beach County RDF ash.....	114
Figure 4.60 Leaching of lead using DDW and SAR as leaching fluids.....	114
Figure 4.61 Accumulated percent leaching of lead using DDW and SAR as leaching fluids.....	115
Figure 4.62 Leaching of lead using DDW at different flow rate.....	115
Figure 4.63 Accumulated percent leaching of lead using DDW at different flow rate .....	116
Figure 4.64 Leaching of lead using SAR at different flow rate .....	116
Figure 4.65 Accumulated percent leaching of lead using SAR at different flow rate .....	117
Figure 4.66 Variation in lead leaching from 15.2-cm and 45.7-cm columns subjected to DDW.....	117
Figure 4.67 Accumulated percent of lead leached from 15.2-cm and 45.7-cm columns subjected to DDW .....	118
Figure 4.68 Variation in lead leaching from 15.2-cm and 45.7-cm columns subjected to SAR.....	118
Figure 4.69 Accumulated percent of lead leached from 15.2-cm and 45.7-cm columns subjected to SAR.....	119

**List of Figures (continued)**

Figure 4.70 Variation in lead leaching from 6-in columns subjected to DDW using Pinellas County MB ash and Palm Beach County RDF ash..... 119

Figure 4.71 Variation in lead leaching from 18-in columns subjected to DDW using Pinellas County MB ash and Palm Beach County RDF ash..... 120

Figure 4.72. Variation in lead leaching from 18-in columns subjected to SAR using Pinellas County MB ash and Palm Beach County RDF ash ..... 120

## List of Photographs

Photo 3.1. Permeability test set-up.....	40
Photo 3.2. Swell and shrinkage tests, saturated and dry .....	41
Photo 3.3. Unconfined compressive test on the WTE bottom ash specimen.....	42

## ACKNOWLEDGMENTS

The researchers would like to thank the Florida Department of Transportation for funding this research under contract number C-7754. The assistance of Dr. Robert K. H. Ho throughout the study was invaluable. The diligent efforts of the following graduate and undergraduate students made timely completion of this project possible: Mario Chavez, Shakeel Syed, Shih-I Wang, Keith Guthrie, Gudny Palsdottir, Massimo Bosso, Rujroj Waiwudhi, Deborah Pandeline, Jenifer Benaman, Daniel Wintermeyer and Dorothy Rhine.

## ABSTRACT

Municipal waste combustor (MWC) bottom ash from a Mass Burn (MB) and Refuse Derived Fuel (RDF) Waste-to-Energy facilities was evaluated for potential use as a highway fill material. MWC bottom ash has the physical and geotechnical properties necessary for many highway fill applications and meets existing environmental acceptability regulations. Bottom ash is classified as A-1a (0). The moisture-density, permeability and unconfined compressive strength of the bottom ash were a function of compaction energy and moisture content with behavior similar to many conventional fill materials. Compacted bottom ash has a negligible shrinkage and swell potential when saturated, however the ash swelled when allowed to air dry. The stress-strain characteristics of both ash types behave similarly to sand. Both ashes develop some cohesion that is attributed to pozzolanic cementing reactions occurring in the bottom ash. The angle of internal friction increases with the density of the compacted bottom ash. Values for these geotechnical properties as well as elastic and resilient moduli, CBR and LBR are presented.

Concentrations of leaching of trace metals (Ag, As, Cd, Cr, and Pb) in the leachate from compacted ash columns subjected to DDW and SAR were below the EPA toxicity standard and decreased as a function of time. For Ag, As and Cd, most of the leachate concentrations were below the drinking water standard.

Based on the study findings, developmental specifications for incorporation into Florida DOT "Standard Specifications for Road and Bridge Construction" have been proposed for using bottom ash in highway fill applications.

## 1.0 INTRODUCTION

Municipal solid waste (MSW) consists of paper, cardboard, plastics, wood, metals, glass, food-wastes, and other combustible and non-combustible materials. Americans generate more than 4.4 billion tons of MSW each year, resulting in nearly six pounds of garbage being produced by each person per day (Stutzman, 1992). Until 1975, most of this waste was discarded as unusable. However, a study on alternative fuels conducted by The Organization of Petroleum Exporting Countries, turned attention towards the use of MSW as a fossil fuel (Alter and Dunn, 1980). Based on this and other studies, the combustion of MSW has become an economical source of energy for many states (EPA, 1975).

Converting the trash into energy in municipal waste combustor (MWC) facilities, significantly reduces the volume of our garbage by nearly 90%. These facilities also reclaim various metals from the waste stream for reuse. The remaining 10% is composed of fly ash, scrubber ash and bottom ash which can be disposed of in lined landfills or reused in various commercial applications (Wheelabrator, 1991). There are two types of MSW combustion plants: mass burn (MB) facilities and refused-derived-fuel (RDF) facilities. Mass burn facilities combust MSW without pre-combustion separation processing, while RDF facilities separate and recover resources prior to combustion.

At present, Florida burns more MSW than any other state except New York—about 25% of Florida's MSW is combusted in WTE plants. Florida will overtake New York's rate of waste incineration next month when the Lee County incinerator goes on line. This new WTE facility will have the capacity to burn 1,200 tons of waste per day (FDEP, 1993). Also on the horizon is a facility that will have the capacity to burn 1,500 tons of MSW per day—an existing facility is currently in the permitting process to achieve this capacity. By 1995, the estimated production of WTE ash residue in Florida will increase from the present rate of 1.9 million tons per year to 2 million tons per year.

### 1.1 Ash Management Practices

Until 1975, the most common methods of disposal of the ash residues was either land filling or daily cover. However, current and future research indicates that MWC ash has applications as a construction material (Collins, 1977). One important factor which hinders the accepted use of MWC ash as a construction

material is its variability in physical and chemical properties due to different plants' processes. In addition, without consistent use of the ash in engineering applications, it would not be economical to pursue reuse of this material. For these reasons, the long range goals for ash management are: 1) to ensure a consistent and uniform production of ash by controlling the plant processes and the material which goes into the process, 2) to safely increase recycling or reuse of these products, and 3) to develop specific uses for MWC bottom ash as a construction material. In order to reach these goals, a general understanding of the engineering properties of MWC ash should be obtained.

## **1.2 Objectives**

The objective of this investigation was to evaluate the geotechnical engineering properties and the environmental characteristics of MWC bottom ash for use in highway applications. Bottom ash from a mass-burn facility and a refuse-derived-fuel facility was examined to show any variability in the strength and deformation characteristics between the two most prevalent processes used in municipal waste combustion.

The following geotechnical properties of MWC ash were examined: visual classification, grain size analysis, specific gravity, absorption, mineralogy, pozzolanic activity, bulk rodded unit weight, moisture-density, and permeability. Shear and deformation characteristics of the bottom ash were evaluated as follows: unconfined compressive strength, consolidated drained triaxial shear test, determination of elastic modulus and resilient modulus, and California Bearing Ratio and Limerock Bearing Ratio values.

The following environmental characteristics of MWC ash were examined: elemental concentrations of selected metals in MWC bottom ash, leaching characteristics of ash column, and environmental acceptability of MWC bottom ash.

## 2.0 PREVIOUS MWC BOTTOM ASH STUDIES

### 2.1 Metals in WTE Ash

The metals present in WTE ash are either recyclable metal items or elemental metals. Combustion results in the enrichment of the elemental metals in the ash that is produced since metals may be neither created nor destroyed. However, different elemental metals are enriched in different amounts due to two factors: 1) thermal history during combustion and 2) the physical and chemical properties of each metal—such as their boiling point and their reactivity with other elements present in the combustion chamber (Clapp et al., 1988; Hocking, 1975). For example, Fe and Cu are enriched in bottom ash, whereas Hg and Cd are enriched in fly ash.

In addition to the naturally occurring metals present in the raw materials used in commercial products, the primary sources of metals in the combustible fraction of MSW are pigments, inks, stabilizers for plastics, clay filters, whiteners, photosensitizers, and miscellaneous chemical compounds (Campbell, 1976). When waste is combusted as mass burn—i.e., when waste is not separated prior to combustion—the existence of ferrous metals, cans, glass, and wires in the burning materials result in an increase of Fe, Si, Al, and Cu in the ash residues (Law and Gordon, 1979).

The distribution of the elemental metals within the ash matrix is determined by: the operating conditions during combustion, the composition of the municipal waste, the cooling and cleaning of emissions, and the physical-chemical behavior of the individual metals (Brunner and Monch, 1986). The temperature inside combustion chambers is normally above 760 - 980 °C—high enough to melt most metals and their compounds. Metals, such as Al, Fe, Cu, and their compounds, which have high boiling points are enriched in bottom ash after combustion. Cadmium and mercury, which have low boiling points, are vaporized during combustion and either condense and absorb onto the surface of fly ash or condense in the scrubbing system which results in higher concentrations in fly ash and scrubber residue (Brunner and Monch, 1986; Campbell, 1976). Studies have shown that the concentrations of Cr, Mn, Pb, and Sn in fly ash and bottom ash are not significantly different (Brunner and Monch, 1986; Clapp et al., 1988; Lisk, 1988); these metals have more complex physical-chemical properties, and thus, combust differently at each plant.

## 2.2 Engineering Investigations of Ash

Literature was reviewed for information about MWC bottom ash composition, treatment and utilization. Of the sources reviewed, most tend to confirm the belief that selected bottom ash material, especially bottom ash samples from facilities with ferrous metals removal systems, has a reasonable degree of physical uniformity; some studies have suggested that bottom ash has to be aged prior to utilization to modify the physical properties of the material (Hartlen, 1989). Aging bottom ash can reduce the moisture content, organic content, and unreacted lime content of ash; thereby making it more suitable for use (Collins, 1977). This practice has been used in Europe for several years (Hartlen, 1989).

According to Gidley and Sack (1984), there are a wide variety of wastes that are potentially useful in construction. These include wastes from the mining and metallurgical industries, municipal refuse, demolition wastes and ashes from combustion processes. These wastes can be used as embankments materials, highway base courses, landfill liners, concrete aggregates, and bituminous mixtures.

Ash utilization is not a new concept; industrial ashes and slags have been used as the basis of many concrete admixtures. Coal fly ash has been used as a pozzolanic material in concrete for several decades and in many applications such as road fill, as an aggregate in artificial reefs and, when pelletized, as a light weight construction aggregate (HDR Engineering, 1991). Blast furnace slags have been used both as aggregate and as fibers for reinforcement. Foster Wheeler Company (Charleston, S. C.) has been using MSW ash for the last decade as a partial substitute for up to approximately 71% of traditional Portland cement raw material (Darcy, 1991). The Foster Wheeler Company has conducted regular testing on incineration ash and has found the bottom ash to be a non toxic material (Darcy, 1991).

The use of bottom ash as a landfill daily cover or using ash as a bituminous or portland cement matrix for an impermeable cap has been proven to be very successful by the New York State Energy Research and Development Authority (Koppelman, 1990). In addition, Forrester and Goodwin (1990) confirmed that achieving liner-like permeability is attainable and cost effective. The highly compacted densities achieved for use as a landfill liner demonstrate the potential application of bottom ash as a highway fill material.

When considering ash material as an aggregate for construction material applications, four major issues are relevant: 1) economy, 2) compatibility with other materials, 3) concrete properties, and 4) environmental acceptability. The Foster Wheeler Company has conducted regular testing on incineration ash and has found bottom ash to be a non-toxic material (Darcy, 1991). Bottom ash is currently being used or considered for use as construction material in artificial reefs, roads, liners and daily covers for landfills, concrete masonry blocks, lightweight and coarse aggregate, and partial replacement of aggregate.

### 2.2.1 Highway Applications

WTE bottom ash has been used for the last two decades in many road applications around the world; the Netherlands and Germany use as road base and as an aggregate in asphalt applications (Van Der Sloot, 1990). Sweden uses processed bottom ash as structural fill; and tests of structural fill show that the bearing capacity of bottom ash is comparable to natural aggregate (Hartlen and Fallman, 1990). Japan and Taiwan currently use WTE ash as an embankment or as structural fill materials on the coastline. Denmark uses bottom ash in embankment material (Van Der Sloot, 1990).

#### 2.2.1.a Federal Highway Administration Ash Study

The Federal Highway Administration (FHWA) sponsored the first documented field studies of MWC bottom ash use in the United States. Ash from mass-burn facilities was used as an aggregate substitute in asphalt pavements. In addition, lime was added to the ash-asphalt blend as an anti-stripping agent. The ash fraction used ranged from 50% to 100% of the aggregate mix. This ash fraction is significantly higher than the ash fraction used today which, depending on the application, ranges between 10% and 50%. In any case, the FHWA concluded that the bottom ash was acceptable from an engineering perspective as a construction material (Chesner, 1993).

### 2.2.1.b SEMASS Ash Study

A company called, Energy Answers, constructed an access road to the SEMASS refuse-derived-fuel plant located in Rochester, Massachusetts using processed bottom ash. The top and base coarse of the pavement as well as the pavement sub-base consisted of 30% processed bottom ash. After 18 months of daily traffic to the plant, the roadway only showed signs of normal wear and tear. Chemical tests on soil samples adjacent to the road and water samples taken from the monitoring wells showed metal levels to be either non-detectable or well within the normal limits for Massachusetts (Chesner, 1993).

### 2.2.1.c CRRA and UCONN Ash Study

In August of 1992, the Connecticut Resources Recovery Authority (CRRA) and the University of Connecticut (UCONN) constructed an 1800 foot access road to the top of the Shelton landfill. The access road allowed the CRRA and UCONN to demonstrate the use of MWC bottom ash as a structural fill and as an aggregate for bituminous pavement in a controlled and well-monitored site. The laboratory results conducted prior to construction and the actual field results indicate that bottom ash performs well as both structural fill and bituminous concrete. The ash was found to be comparable to standard construction materials in use today. However, the bottom ash could not be recommended for use as a structural fill outside the landfill due to the quantities of leachable salts found in the ash. This issue appeared to be effectively controlled when the bottom ash was used in a bituminous mix (Demars *et al.*, 1994).

### 2.2.1.d Wheelabrator Ash Study

Wheelabrator has patented a Portland cement-based ash aggregate called McKaynite which is used to treat combined ash or bottom ash. Several field tests on McKaynite have been conducted by Wheelabrator in construction applications. These tests utilized ash from various WTE facilities in Florida that were located in Tampa, Pinellas County and Broward County. In 1987, a strip of pavement was installed in Tampa, Florida that utilized McKaynite in the asphalt

mix. In 1991, a fill area was constructed in Ruskin, Florida with McKaynite aggregate fill material. The results from these studies indicated that there were no adverse effects on the groundwater, soil or air quality. The objective of these studies was to generate data needed to acquire state regulatory approval to use McKaynite aggregate in the field. Recently, Wheelabrator received the approval to use the McKay Bay facility's ash in the McKaynite as landfill cover and as road aggregate (Chesner, 1993).

#### 2.2.1.e Denmark Ash Study

Since 1974, Denmark has been utilizing screened and sorted MSW bottom ash as subbase for parking lots, bicycle paths, and roads, both residential and major. Denmark's incentives for utilizing the bottom ash as a subbase are economical as well as derived from a concern for natural resources: as a subbase material, bottom ash can substitute for gravel which is expensive and limited in quantities. The use of bottom ash for civil engineering purposes has been regulated in Denmark since 1983. (Hjelmar, 1990).

#### 2.2.1.f New York Studies

The New York City Sanitation Department (1988) investigated bottom ash as a partial aggregate supplement to determine if it could be used in asphaltic concrete in road paving applications. The testing program consisted of a series of Marshall mix tests that were designed to evaluate the effect that substituting processed residue for natural aggregate would have on wearing course and binder course mixes. This study concluded that 30% of the natural aggregate by weight could be substituted with bottom ash and still meet applicable gradation and Marshall mix design criteria. Asphaltic concrete made from bottom ash looked, handled, and behaved essentially the same as a conventional asphalt paving mix. Using a magnetic separator to remove the ferrous material from the ash did not produce any improvement in asphalt paving mix characteristics. Instead, the air voids of such mixes were considerably higher because some of the fine ash particles were removed during magnetic separation, resulting in a somewhat coarser mix gradation.

Studies performed by the New York State Energy Research and Development Authority (Koppelman, 1988) in Long Island, New York, have found the use of unbound bottom ash in road base material (under a bituminous pavement) to be successful and even suitable in areas that are insensitive to potential salt leaching (i.e., near the marine environment).

## 2.2.2 Building Construction Material

### 2.2.2.a Concrete Block Applications

The first building constructed of concrete masonry blocks made from MSW bottom ash residue was built at the SUNY Stony Brook by Roethel and Breslin (1990). The two-story 8,100 sq. ft. boathouse was made of 350 tons of ash concrete blocks. Bottom ash was used as an aggregate mixed with portland cement. The resulting product has been found to be stronger (1600-2570 psi) than traditional concrete blocks (1200-1800 psi) under similar circumstances.

The results of the SUNY-Stony Brook investigations indicate that incineration bottom ash possesses significant pozzolanic activity and is a suitable substitute for aggregate in the manufacture of concrete blocks for use by the construction industry. On-going research will monitor the boathouse for structural integrity and environmental impact during the next several years. In addition, the air quality within the building will be monitored, and the surrounding soil will be tested for ash constituents.

Chesner (1992) used bottom ash with 15% portland cement and 17 to 19% moisture content to produce blocks at the Besser company in Alpena, Michigan. An average 28 day compressive strength of 1521 psi was achieved for these bottom ash blocks. Control blocks produced using gravel, sand, 11% type II portland cement, and approximately 5.9% moisture achieved a 28 day compressive strength of 3976 psi.

### 2.2.2.b Light-Weight and Coarse Aggregate

A test program was conducted by Plumley and Boley (1990) at ABB Resource Recovery, Windsor, Connecticut, to produce light-weight stabilized ash from the combustion of MSW bottom ash. The incinerator residue was blended with lime and water in a mixer. The "green" mixture was then pelletized on a

spinning-disk pelletizer and dusted with an embedding material to prevent caking. The resulting product was cured in a steam-filled silo. Controlled operation of the pelletizer and screening of the cured pellets resulted in a graded, light-weight aggregate that was found to be environmentally benign using the EPA's TCLP (toxic characteristics leaching procedure) leaching test. The pellets were suitable for utilization in various products or processes that incorporated gravel. These products were about 60% as dense as their natural gravel counterparts.

Lauer (1979) used the nonmagnetic fraction of the residue sized between the 25 mm and 75 mm mesh as a fine and coarse aggregate in concrete. Similarly, Aleshin, and Bortz (1976) used waste glass as coarse aggregate. Neutralysis Industries Inc. of Brisbane, Australia, has been using bottom ash to produce light-weight aggregate by mixing clay or shale to produce pelletized feedstock, which is then pyro-processed in a series of kilns. This process produces light-weight ceramic aggregate for use in structural concrete, masonry block manufacturing, and precast concrete shapes. Light-weight aggregate makes the block or finished concrete shapes one-third lighter than heavy aggregate and provides superior insulating and acoustical characteristics (Merdes, 1990).

#### 2.2.2.c Partial Replacement of Aggregate

Thomas Nevin (1992) investigated the properties of concrete where a percentage of the aggregate (gravel, pearock, and sand) has been partially replaced with bottom ash. The concrete specimens were cured in lime water for 28 days and exposed to sea water for 28 days. The control concrete was designed for a compressive strength of 5000 psi. This study concluded that: 1) the technique of using a constant fineness modulus as a criterion to replace conventional concrete aggregate with bottom ash was an effective means of minimizing changes in the grain size distribution; 2) the workability and unit weight of concrete that contained ash showed a slight decrease as WTE bottom ash was increased from 0 to 16%, while the voids and absorption of the concrete increased; the air content was unaffected by the ash content of the concrete; and 3) WTE bottom ash can be used as a partial replacement of up to 12% of conventional concrete aggregate without causing major changes in the compressive and tensile properties of the concrete when exposed to either lime water or sea water.

## 2.3 Geotechnical Properties

Bottom ash displays a high degree of variability in both physical and engineering properties. In order to identify this variability, a literature review on the physical and engineering properties of WTE bottom ash applicable for use as a highway fill material are describe in the following sections.

### 2.3.1 Grain Size Distribution

According to Forrester and Goodwin (1990), bottom ash reflects a granular material of mostly gravel consistency. In addition, Huang and Lovell (1990), indicated that bottom ash is a relatively well graded, sand-sized material. A well graded material can be readily compacted to a dense condition and will generally develop greater shear strength and lower permeability than a poorly graded material (GAI Consultants, 1979).

### 2.3.2. Moisture Content

Water can also be absorbed into the body of the bottom ash particles or retained on their surfaces as a film of moisture. Thus, the moisture content of bottom ash will affect such engineering properties as compaction behavior and compressive strength. Chesner (1989) found that the moisture content of bottom ash can range from 20 to 70 percent of the dry weight of the ash while natural aggregate has a moisture content between 3 and 10 percent. For construction applications, the moisture content should be within 2 percent of the moisture content at which the aggregate has its maximum density.

Chesner (1989), concluded that bottom ash samples from facilities with ferrous metals removal capabilities, generally display higher moisture content than ash samples from facilities without ferrous metal removal systems. Previous research on bottom ash conducted at Florida Tech reported a moisture content range from 5 to 18 percent during different sampling periods (Wu, 1990, Nevin, 1991 and Jain, 1992).

### 2.3.3 Specific Gravity

The specific gravity is the ratio of the particle mass to the mass of an equal volume of water. The specific gravity of bottom ash reported by Seals, Moulton and Ruth (1972), ranged from 2.35 to 2.78. Huang and Lovell (1990) later stated it ranged from 1.94 to 3.46. Both studies also indicated that the specific gravity of bottom ash is a function of chemical composition. Bottom ash with a high carbon content will typically have a low specific gravity, while bottom ash with a high iron content will typically have a high specific gravity. Other factors that may cause variation in the specific gravity of bottom ash are: the method of incineration, the season and the management procedures employed at the WTE plant (HDR Engineering, 1991).

### 2.3.4 Absorption

The absorption capacity of an aggregate represents the maximum amount of water that can be absorbed into its pore spaces. Natural aggregate absorption typically ranges between 0.5 and 2 percent by weight (Chesner, 1989). Chesner also performed absorption tests on fine (<4.75 mm) and coarse fractions (>4.75 mm) of bottom ash. He reported that absorption values ranged from 3 to 5 percent by weight for the fine material and from 11 to 17 percent by weight for the coarse material. The pores at the surface of the particles affect the adherence among the aggregate particles and might exert some influence on the strength of a construction material (SCS Engineers, 1991).

### 2.3.5 Bulk Rodded Unit Weight

The bulk unit weight of a soil sample is the weight of graded aggregate required to fill a container of specified volume. The term *bulk* is used because the volume is occupied by both aggregate and voids. The bulk unit weight of natural aggregate material generally ranges from 80 to 130 pcf (SCS Engineers, 1991). Chesner (1989), reported that the bulk unit weight for bottom ash samples ranged between 59 and 77 pcf—indicating that bottom ash is classified as a lightweight aggregate.

### 2.3.6 Moisture-Density Relationship

Two compactive methods which have been followed to compact bottom ash are: ASTM D-698 standard proctor compaction and ASTM D-1557 modified proctor compaction. The results of six investigations conducted on bottom ash from various sources are summarized in Table 2.1. Natural soil aggregates have OMC's ranging from 10 to 30 percent and maximum dry densities ranging from 90 to 135 pcf (Chesner, 1989). Bottom ashes have optimum moisture contents that are similar to natural aggregate; however, maximum dry densities are typically at the lower range of natural aggregate material.

### 2.3.7 Permeability

In a literature review, the coefficient of the permeability of bottom ash displayed a high degree of variability. Forrester and Goodwin (1990), conducted several laboratory permeability tests on raw bottom ash achieving a coefficient of permeability of  $1.8 \times 10^{-5}$  cm/sec; however, the moisture content or density used in the laboratory specimens for the permeability tests were not reported. Seals, Moulton and Ruth (1972), indicated that bottom ash permeability, at a relative density of 50 percent, falls into a medium permeability range that has values similar to that of sand. Their research reported that the minimum permeability value of  $5.0 \times 10^{-3}$  cm/sec for bottom ash is due to a high percentage of fine sand size particles (approximately 62%) and fines (approximately 8%). According to Huang and Lovell (1990), the result of permeability tests conducted on the bottom ash compacted to 95 percent of the maximum dry density (109 pcf at 16 % moisture content, with 10 % fines content by weight) was  $2.0 \times 10^{-3}$  cm/sec. Huang and Lovell (1990) verified that the permeability of bottom ash is a function of the percentage of fines.

### 2.3.8 Swelling

Previous investigations made by various researchers (e.g. Roethel, 1986; Forrester, 1988; Chesner, 1989; Forrester and Goodwin, 1990; SCS Engineers, 1991) indicate that MSW ash possesses a potential for pozzolanic behavior due to the presence of high free-lime content and cement-like mineralogy. This pozzolanic behavior or reactivity refers to the tendency for ash to react with

excess lime. Lime reacts with combined ash (bottom ash and fly ash) and water to form cementitious products (calcium silicate hydrates and calcium aluminate hydrates) which cause volume expansion and cracks in the ash product's structure (Chesner, 1989). Since bottom ash has a negligible lime content, it is "expected" that cementitious reactions that could stabilize a bottom ash road base would not be generated (SCS Engineers, 1991).

The swelling of bottom ash results from the reaction of water vapor with salts found in ash residue (Chesner, 1989). The resultant hydrated salts expand the volume of ash. Swell tests on bottom ash resulted in the noticeable expansion of the sample dimensions; however, there is no record of the exact volume increases in the bottom ash samples, because the samples began to crumble after a week of testing (Chesner, 1989). To reduce the chance of swelling during use, bottom ash is typically allowed to age or is "pretreated" prior to use in construction applications. Swelling could be a potential limitation for road base or structural use, because swelling could weaken the road or the structure. Aging allows time for the swelling reactions to occur before the ash is used in construction applications (Hartlen, 1989).

### 2.3.9 Unconfined Compressive Strength

Previous studies on the unconfined compressive strength of bottom ash were not cited in the literature.

### 2.3.10 Consolidated Drained Triaxial Shear

The consolidated drained triaxial shear test is used to determine the shear strength parameters: cohesion ( $c$ ) and angle of internal friction ( $\phi$ ) (Bowles, 1992). Huang and Lovell (1990) reported  $\phi$  values ranging between  $45^\circ$  and  $60^\circ$  for relative densities between 90% and 100% on material passing through a 3/8" sieve. Demars, *et al* (1994) reported  $\phi$  values of  $40^\circ$  to  $45^\circ$  for MSW ash samples passing the #4 sieve. Seals, Moulton and Ruth (1972) reported  $\phi$  values of  $38^\circ$  to approximately  $43^\circ$  for loosely compacted ash samples passing the 3/8" sieve. The friction angles reported by Huang and Lovell (1990), Demars, *et al* (1994) and Seals, Moulton and Ruth (1972) were found using the direct shear test. These friction angles generally increased by  $2^\circ$  over values obtained using the triaxial

test. Seals, Moulton and Ruth (1972) also reported that bottom ashes in dense states would generally yield increases in friction angles of 6° to 8°.

### 2.3.11 Elastic and Resilient Moduli

Typical elastic moduli ranges for various materials are as follows: stabilized base materials:  $5 \times 10^4$  -  $6 \times 10^6$  psi, stiff clay: 7,600 - 17,000 psi, medium clay: 4,700 - 12,300 psi, soft clay: 1,800 - 7,700 psi, and very soft clay: 1,000 - 5,700 psi (Huang, 1993).

The resilient modulus is the elastic modulus based on the recoverable strain under repeated loads.

### 2.3.12 California Bearing Ratio (CBR) and Limerock Bearing Ratio (LBR)

The CBR test measures the shearing resistance of a soil or aggregate material in relation to a standard test material. Typically, natural aggregates have CBR values ranging between 30 and 80 percent. A typical minimum specification for a road base is 100 percent (SCS, 1991). Chesner (1989) reports CBR values between approximately 85 and 155 percent for bottom ash utilizing the compactive energy described in ASTM D-1557.

The LBR test is used for evaluating limerock and other soils for base, stabilized subgrade and subgrade or embankment material found in Florida (FDOT, 1993). The LBR number can be calculated directly from the data from the CBR test where the LBR number is equivalent to the ratio of corrected stress at 0.1" penetration over 800.

## 2.4 Existing Regulations

### 2.4.1 Current MWC Ash Regulations

Florida's regulation on MWC ash management (Florida Administration Code 17-702, 1991) requires that an application for a permit to construct and operate solid waste combustors shall include an ash residue management plan. The plan shall (1) describe the methods, equipment, and structures necessary to control the dispersion of ash residue during handling, processing, storage, loading, transportation, unloading, and disposal; (2) identify disposal sites;

(3) include an estimate of the quantities of bottom and fly ash to be generated; (4) include a quality assurance plan; (5) include procedures to assure that hazardous wastes are not received at or burned in the facility; and (6) be updated as necessary to reflect changing conditions. In addition to the requirement for an ash management plan, the regulation also includes specific requirements for the storage, off-site transportation, disposal, and recycling of ash residue. According to the regulation, WTE ash can be either disposed of in a lined MSW landfill or utilized if the processed ash residue does not cause discharges of pollutants to the environment.

#### 2.4.2 Environmental Issues

Studies have shown MWC ash has promising engineering applications, although heavy metal content has raised a concern that it might cause contamination to groundwater if put into fill material. According to a recent study conducted for the Environmental Protection Agency, all the metals found in the leachates of fly ash and bottom ash combinations were below the EP toxicity maximum allowable limits. In addition, the majority of the metal concentrations found met primary and secondary drinking water standards set by the Safe Drinking Water Act (NUS, 1990).

Another study conducted at the University of Connecticut showed high amounts of chlorides and sulfates leaching out of MB bottom ash. Toxicity Characteristic Leaching Procedure (TCLP) tests were conducted on ash mixes consisting of 50% ash and 50% aggregate. The results indicated that the concentration of metals in the leachate was less than the concentrations of pure ash and the combined ash mixes passed the toxicity tests. The final conclusion was that the leachable elements in the ash were effectively controlled when the ash was mixed with a bituminous material (Demars, *et al.*, 1994).

On May 3, 1994, the Supreme Court ruled that any residue created by MSW incinerators must be tested for hazards using TCLP tests prior to disposal or treatment. The TCLP procedure is cited as the standard test for determining if a material is deemed hazardous. Typically, the leachate concentrations of calcium, cadmium, arsenic, chromium, lead and silver are tested using the TCLP procedure. Wang (1994) showed that typically less than 0.1% of these metals leached out of the bottom ash under simulated real world conditions. The concentrations of Pb (1700 mg/g) and Cd (24 mg/g) in the total bottom ash

leachate were both below the EPA toxicity standard of 5000 µg/L. Therefore, the bottom ash alone would not be considered a hazardous waste and could be disposed of in ordinary landfills if needed.

Table 2.1 Previous investigations of moisture-density relationship for MWC ash

Reference	Ash Source or Type	Compactive Energy - ASTM	Particle Size Passing		OMC (%)	Maximum Dry Density	
			(in)	(mm)		(pcf)	(kN/m <sup>3</sup> )
Seals, Moulton and Ruth (1972)	Mitchell	D-698	3/8	9.5	14.6	116.6	18.3
	Fort Martin	D-698	3/8	9.5	24.8	85.0	13.4
	Kanawha River	D-698	3/8	9.5	26.2	72.6	11.4
Collins (1977)	Well	D-698	NA	NA	12	95.6	15.0
	Burned	D-698	NA	NA	18	110.6	17.4
	Intermediately Burned	D-698	NA	NA	14	90.6	14.2
	Poorly Burned	D-698	NA	NA	20	105.6	16.6
		D-698	NA	NA	16	75.5	11.9
		D-698	NA	NA	24	90.6	14.2
Huang and Lovell (1990)	Gallagher	D-698	NA	NA	16	109.0	17.2
	Gibson	D-698	NA	NA	18	101.7	16.0
	Schafer	D-698	NA	NA	22	95.4	15.0
Demars, et al (1994)	Bridgeport	D-1557	0.75	19.0	13	125.8	19.8
		D-1557	0.75	19.0	10	114.4	18.0
Chesner (1989)	Bottom Ash	D-1557	NA	NA	11	80.0	12.6
		D-1557	NA	NA	22	100.0	15.7
Forrester and Goodwin (1990)	Bottom Ash	D-1557	NA	NA	15	116.4	18.3

OMC - Optimum Moisture Content

NA - Not Available

## 3.0 METHODOLOGY AND EXPERIMENTAL PROCEDURE

### 3.1 Description of MWC Facilities

The geotechnical properties of MWC bottom ash were examined in order to evaluate its performance in highway applications. The MWC bottom ash used in this study was provided by a mass-burn (MB) combustion facility and a refuse-derived-fuel (RDF) combustion facility. Descriptions of the operation of these facilities and the production of ash are provided in the following sections.

#### 3.1.1 Refuse Derived Fuel (RDF) Facilities

The RDF process starts when collection vehicles and transfer trailers deposit solid waste onto the tipping room floor. White goods, (i.e., refrigerators, water heaters, etc.) are separated from the solid waste stream and processed on the oversize-bulky-waste (OBW) line while tires are processed on a separate line. Front end loaders are then used to transfer the remaining waste from the tipping room floor to conveyors that feed the flail shredder. The flail shredder breaks the waste down into a manageable size. A grapple crane is then used to extract items which are non-processible such as rugs, mattresses and cables. Once removed, these items are sent directly to the landfill. The shredded waste is conveyed beneath a large magnet for ferrous metal removal. The shredded waste is then screened in a trommel where glass and grit are removed. Further along the trammel, a picking conveyor recovers the aluminum for recycling. The remaining waste is transported to a storage bin for use as fuel (Project Profile, 1990). The fuel is now ready for combustion and is fed uniformly onto the boiler's traveling-grate stoker (Keith *et al.*, 1990). Here, the heat generated by the combustion of the waste is converted into steam to power the turbines to produce electricity which is then sold back to the local power industry for distribution to the power grid (Project Profile, 1990). The bottom ash from the combustion chamber is discharged through a diverter valve into a water-bath deasher. The bottom ash then moves up the inclined end of the deasher for dewatering and discharges onto the bottom ash transfer conveyor. The conveyor transports the bottom ash to the ash building where fly ash is combined with it. The combined ash is then hauled to the landfill as daily cover (Keith *et al.*, 1990).

### 3.1.2 Mass Burn (MB) Facilities

At MB facilities, sorting takes place after combustion. The ash that falls through the grate as well as that which remains on it is quenched in water. The fly ash which is collected with precipitators is then mixed with the bottom ash to form a combined ash. This combined ash undergoes metal separation to remove ferrous and non-ferrous materials for recycling (Wheelabrator, 1991). The ferrous materials are extracted using magnets while the non-ferrous materials are sorted by size and weight using trommel screens. The aluminum is separated along an inclined conveyor belt by means of eddy currents. (Pfeffer, 1992). Once the valuable metals have been recovered and shipped for recycling, the remaining combined ash is deposited in a landfill (Wheelabrator, 1991).

### 3.1.3 WTE Survey Development

To determine the location, contact person(s), availability, processes and costs of WTE the survey form shown in Figure 3.1 was developed. Copies of this form were sent to the WTE facility plant managers in Florida.

## 3.2 Ash Sampling

The MB bottom ash used for this research was provided by the Pinellas County Resource Recovery Facility, located in Pinellas County, FL. This WTE facility consumes an average of 3150 tons of MSW per day. The plant is publicly owned but privately operated by Wheelabrator Pinellas, Inc. and currently delivers approximately 75 megawatts of power per day to the Florida Power Corporation.

The RDF bottom ash used for this research was provided by Palm Beach Solid Waste Authority, located in Palm Beach County, Florida. This WTE facility consumes an average of 2250 tons of MSW per day. The plant is operated through a 20-year contractual agreement with Babcock & Wilcox (a McDermott Company) and currently delivers approximately 61.3 megawatts of power per day to Florida Power & Light Corporation (Project Profile, 1990).

### 3.3 Bottom Ash Engineering Properties

The physical properties of the bottom ash were determined from several different samples in order to minimize variations in measured characteristics between the test sample selected and the 55 gallons drums received at Florida Tech. The test samples were taken in accordance with ASTM Standard C 702-87, Method A. This method consists of reducing the sample using a mechanical splitter. Table 3-2 presents a summary of the test methods and procedures used to evaluate the engineering properties of the MB and RDF ashes. These procedures are briefly presented in the following sections.

#### 3.3.1 Physical Composition

A visual classification was performed on the MWC bottom ash samples that were received from the MB and RDF facilities. The visual classification was conducted to determine the percentage of recoverable materials such as metals and glass contained in the ash. A representative sample retained on the #8 sieve and passing the #4 sieve was air dried, weighed, and then washed. Once washed, the material was air dried and weighed again. The material that was lost due to washing was classified as fines. The remaining bottom ash was separated into three categories: glass, metals and clinker. The glass and metals are recyclable materials while the clinker consisted of any objects that were unidentifiable or miscellaneous, such as; rocks, ceramics, wood and paper. The ferrous materials were separated from the sample using a magnet while the non-ferrous materials and glass were removed by visual inspection. The remaining unidentifiable material was grouped into the clinker category. Once the sample was completely divided into categories, a percentage by weight of each category was recorded.

#### 3.3.2 Grain Size Distribution

The grain size distribution of the MB and RDF ashes was conducted in accordance with ASTM C 136-84a, "Standard Method for Sieve Analysis of Fine and Coarse Aggregate" (ASTM, 1991). The bottom ash was oven dried at  $110 \pm 5^\circ$  C for 24 hours before being sieved in a mechanical sieve shaker for 15 minutes. A series of US standard sieves were used: 1 in.(25.40 mm), 3/4 in., 3/8 in., #4

(4.75 mm), #10 (2.00 mm), #20 (0.85 mm), #50 (0.30 mm), #100 (0.15 mm), #200 (0.08 mm).

The parameters to be determined from the grain-size distribution of the WTE bottom ash are: the uniformity coefficient ( $C_u$ ) and the coefficient of gradation, or coefficient of curvature ( $C_c$ ), and the effective diameter ( $D_{10}$ ), (Unified Soil Classification System, USCS). These coefficients are defined as follows:

$$C_u = \frac{D_{60}}{D_{10}} \quad (3.1)$$

$$C_c = \frac{D_{30}^2}{(D_{60})(D_{10})} \quad (3.2)$$

where:

$D_{10}$  = diameter corresponding to 10 percent passing by weight, in mm.,

$D_{30}$  = diameter corresponding to 30 percent passing by weight, in mm.,

and

$D_{60}$  = diameter corresponding to 60 percent passing by weight, in mm.

### 3.3.3 Moisture Content

The moisture content of MB and RDF bottom ashes was conducted in accordance with ASTM C 566-89 "Standard Test Method for Total Moisture Content of Aggregate by Drying" (ASTM, 1991). A ventilated oven was used to dry the sample at a temperature of  $110 \pm 5^\circ \text{C}$ . Two samples were used to determine the average moisture content. Moisture content tests were also conducted throughout the testing program.

### 3.3.4 Specific Gravity

Specific gravity tests were performed on two different MWC bottom ash grain size samples; one consisted of material passing the #4 sieve and the other consisted of material passing the #8 sieve. These tests were conducted in accordance with ASTM D-854-92, "Standard Test Method for Specific Gravity of Soils."

### 3.3.5 Absorption

The absorption of MB and RDF bottom ashes was conducted in accordance with ASTM C 128-88 (ASTM, 1991) "Specific Gravity and Absorption of Fine Aggregate". The absorption capacity of the aggregate represents the maximum amount of water that the aggregate can absorb. The bottom ash was oven dried at  $110 \pm 5^\circ \text{C}$  for 24 hours and then submerged in water for another 24 hours before testing and divided into two size fractions using the # 4 (4.75 mm) sieve. For purpose of comparison with previous investigations conducted at Florida Tech, only the smaller size fraction passing the # 4 (4.75 mm) sieve was used.

### 3.3.6 Bulk Rodded Unit Weight

The bulk unit weight of MB and RDF bottom ashes was determined in accordance with ASTM C29/C29M-90 "Standard Test Method for Unit Weight and Voids in Aggregate" (ASTM, 1991). The sample was oven dried at  $110 \pm 5^\circ \text{C}$  for about 24 hours before the test and then placed in a Proctor mold of  $1/30 \text{ ft}^3$ . The sample of bottom ash was compacted in three equal layers with a  $5/8 \text{ in.}$  diameter steel rod with a hemispherical tip, each layer was rodded 25 times with the rod. Once the mold was filled, the weight of the material was determined.

### 3.3.7 Moisture-Density Relationship

Moisture-density relationships were performed on MWC bottom ash from MB and RDF facilities. Two different grain size samples of each ash were used; one consisted of material passing the #4 sieve and the other consisted of material passing the #8 sieve. These tests were conducted in accordance with ASTM D-698-91, "Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort" and ASTM D1557-91, "Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort." A total of 42 specimens were fabricated and tested as shown in Table 3.1 for comparison to data reported by Chavez (1993).

Prior to sample fabrication, the sieved ash was air dried for approximately 72 hours. In order to achieve the desired moisture content range of 10 % to 24%, water was added to the ash. The samples were then mixed thoroughly in a

mechanical mixer for approximately ten minutes and stored in air tight plastic bags for 48 hours to produce a uniform moisture distribution and to allow for any absorption to occur. After 24 hours, the moisture content of each sample was determined. If the moisture content revealed that the sample was on the dry side of the desired moisture content, water was added to the sample to obtain the desired moisture content. Samples which were found to be on the wet side of the desired moisture content were discarded and new samples were prepared. In either case, the samples would not be tested for 48 hours from the time of their preparation.

The samples of bottom ash were compacted to the desired energy level using either a manually operated or mechanical compactor. The compacted bottom ash was trimmed even with the top of the mold. From the excess material of each sample, approximately three 50 gram samples were collected for moisture content determination.

### 3.3.8 Permeability

To develop relationships between permeability and molding water content, multiple samples of MB and RDF bottom ashes were tested for permeability. The tests were conducted in accordance with AASHTO T-215-70. Falling head tests were conducted using compaction permeameters model K-610A designed by Soiltest. This unit consists of a standard steel compaction mold of  $1/30 \text{ ft}^3$  which was replaced with a polyvinyl chloride plastic PVC mold of  $1/30 \text{ ft}^3$ , a cast aluminum base with a built-in porous stone that had an average pore size of 225 microns, and a cast aluminum top with overflow valve assembly. The samples were compacted following ASTM D-698. The two reasons for using the PVC mold were: 1) avoidance of corrosion of the steel mold due to extended exposure to water; and 2) prevention of chemical reactions which may occur between the ions in the ash and the metal. The apparatus shown in Photo 3.1, depicts five molds with specimens for falling head testing.

Factors which may influence the reliability of the permeability test in the laboratory, include: (a) air bubbles trapped in the test specimen (b) the degree of saturation of the test specimen and (c) temperature variation during the test. To minimize errors, the water temperature was recorded before and after each test, and the water was allowed to flow for about two hours until no air bubbles were observed before data collection.

For the specimens compacted wet of optimum, minimum flows were observed during the two hours of the deairing phase. A vacuum pump was used to remove the air bubbles for about 45 minutes before the specimen was saturated from the bottom upward under vacuum to free any remaining air bubbles in the specimens.

After the specimens had been saturated and the permeameters were full of water, the vacuum was disconnected. After saturation each specimen was subjected to steady state flow. Falling head tests were then conducted on each specimen. Five to ten trials were conducted on each specimen to determine an average permeability.

The following falling head formula was used to calculate the coefficient of permeability ( $k$ ) of the test specimens.

$$k = 2.303 \frac{aL}{At} \log \frac{h_1}{h_2} \quad (3.3)$$

where:

$k$  = coefficient of permeability in cm/sec,

$L$  = length of specimen in cm,

$h_1$  = initial head in cm,

$h_2$  = final head in cm,

$t$  = total time elapsed during measurements in seconds,

$A$  = cross-sectional area of specimen in  $\text{cm}^2$ , and

$a$  = cross-sectional area of the tube used as a burette in  $\text{cm}^2$ .

The permeability was corrected using the equation:

$$k_{20^\circ\text{C}} = k_{T^\circ\text{C}} \frac{\eta_T}{\eta_{20}} \quad (3.4)$$

where:

$k_{20^\circ\text{C}}$  = coefficient of permeability at 20 °Celsius (68°F) in cm/sec,

$k_{T^\circ\text{C}}$  = coefficient of permeability at test temperature in cm/sec,

$20$  = viscosity ratio of the water at 20 °Celsius (68° F), and

$T$  = viscosity ratio of the water at test temperature.

### 3.3.9 Swelling and Shrinkage Potential

Swelling potential is either the percent swell, or the swelling pressures generated by the expansion of soils. A method to determine swell pressures has not been standardized or accepted universally at this time (Sridharan *et al.*, 1986). The experiments conducted in this investigation are based on the percent swell of soils as suggested by Seed and Chan (1959). To evaluate the swelling and shrinkage potential of MB and RDF bottom ashes, compacted specimens at five different moisture contents were evaluated in both saturated and air dry conditions. Five of the remaining 10 specimens that were produced at the same time as the specimens used in the permeability test of WTE bottom ash were placed in plastic containers. A perforated 4 inch diameter clear acrylic plate with a centered 1/4 inch diameter 2 inch high clear acrylic rod extension was located on the upper surface of each specimen. A dial indicator of one full inch overall range with 0.001 inch continuous graduation divisions was seated on the upper portion of each stand (Photo 3.2). Water was added to the plastic containers and the axial expansion was recorded daily for data collection.

An additional five specimens were set up the same manner as described above with the exception that these five specimens were not submerged in water, and therefore allowed to air dry. This second set-up enabled the cementitious reactivity on soaked specimens to be compared with air dried specimens. After the submerged specimens reached an apparent equilibrium, the water was removed and the volume reduction or expansion measured while the specimens were drying.

### 3.3.10 Unconfined Compressive Strength

Unconfined compression tests were performed on MWC bottom ash from MB and RDF facilities. Two different grain size samples of each ash were used; one consisted of material passing the #4 sieve and the other consisted of material passing the #8 sieve. These tests were conducted in accordance with ASTM D-2166-91, "Standard Test Method for Unconfined Compressive Strength of Cohesive Soil." Samples were prepared both wet and dry of optimum. A total of six to eight samples were prepared for each bottom ash grain size ranging in moisture contents from 6% to 26%. The samples were then compacted to a relative density of 100% of either standard or modified proctor maximum.

The samples were extracted from the compaction molds using a Carver hydraulic press model C. Approximately 40 psi of pressure was applied to extract the specimens. Once the specimens were extracted from the molds, each one, in turn, was placed between the loading plates of a compression test machine. A metal cap and a rubber seat were used at each sample end to provide a uniform distribution of pressure on both surfaces. The unconfined compressive strength test was conducted using a GEO Brainard-Kilman model S-610 CBR/UCC compression test machine. A compressive load was applied to the surface of the sample and steadily increased at a loading rate of 0.005 inches per minute, until failure occurred. The compressive strength was calculated by dividing the ultimate load at failure by the cross-sectional area of the specimen.

### 3.3.11 Consolidated Drained (CD) Triaxial Shear

The testing program consisted of 72 CD triaxial shear tests. The tests were conducted with MB and RDF bottom ashes. Two different grain size samples of each ash were used; one consisted of material passing the #4 sieve and the other consisted of material passing the #8 sieve. The samples were prepared at relative densities varying from 90% to 100% of either standard or modified proctor maximum. The samples were then tested at various confining pressures from 5 psi to 15 psi. Table 3.2 presents a summary of the testing program in more detail.

The CD triaxial shear testing procedure outlined by Bishop and Henkel (1964) was used in this study. The CD triaxial shear test was conducted as an axial compression test in which the sample was consolidated under isotropic conditions, and subjected to shear stress by increasing the axial load with drainage. A very slow rate of loading is used to prevent excess pore pressures from being introduced into the system during the application of the axial load. For this study, a loading rate of 0.005 inches per minute was applied to the sample, until failure occurred. Since the pore pressures are negligible, the principal stresses become the effective stresses (Bishop and Henkel, 1964). The majority of the triaxial samples were prepared using a Harvard Miniature compaction procedure outlined by Wilson (1964). These samples were compacted to the desired density in a split mold by varying the number of layers, the number of tamps per layer and the spring used. The number of layers ranged from 3 to 6, the number of tamps per layer ranged from 10 to 30 and either a 20 lb or 40 lb spring was used. In addition, the porous stones were

covered with filter paper to prevent clogging and strips of filter paper were draped across each sample to aid in the saturation process. However, some of the samples exhibiting low relative densities crumbled while being removed from the mold. These low relative density samples were prepared following the procedure outlined by Bishop and Henkel (1964). A brief summary of this procedure follows: 1) samples are constructed by placing a split mold around the membrane; 2) the prepared ash is weighed to get an initial weight; 3) the ash is placed inside the membrane and gently compacted using a wooden tamper until a height of 2.8" is attained; 4) the remainder of the prepared ash is weighed again to determine the weight of the sample; and 5) the density of the constructed sample is also calculated, since an approximate moisture content of the prepared sample is known.

### 3.3.12 Elastic Modulus

The elastic modulus was calculated from the stress-strain data for each CD triaxial shear test. This was accomplished by calculating the slope of the elastic portion of the stress-strain curve. The elastic modulus equals the stress divided by the strain as shown in equation 3.5.

$$E = \frac{\sigma}{\epsilon} \quad (3.5)$$

where:      E = modulus of elasticity  
                   σ = stress  
                   ε = strain

### 3.3.13 Resilient Modulus

The resilient modulus is defined as the change in stress divided by the change in strain as shown in equation 3.6.

$$M_r = \frac{\Delta\sigma}{\Delta\epsilon} \quad (3.6)$$

where:      M<sub>r</sub> = resilient modulus  
                   Δσ = change in stress @ given strain  
                   Δε = 0.001 in/in

The resilient modulus was determined for the majority of the CD triaxial tests. This was accomplished by unloading and reloading the specimen. The specimen was unloaded at 0.030" displacement (1.1% strain) and reloaded at 0.027" displacement (1.0% strain). This allowed a resilient modulus at 1% strain to be calculated. In addition, the specimen was also unloaded at 0.050" displacement (1.8% strain) and reloaded at 0.047" displacement (1.7% strain).

#### 3.3.14 California Bearing Ratio (CBR)

California Bearing Ratio (CBR) tests were performed on MB and RDF bottom ash passing the #4 sieve. These tests were conducted in accordance with ASTM D-1883-92, "Standard Test Method for CBR (California Bearing Ratio) of Laboratory Compacted Soils" using the compactive energy as described in ASTM D-1557-91. A surcharge of 15 lbs. was applied to simulate the overburden pressure. Four MB bottom ash samples were prepared with moisture contents ranging from approximately 11% to 16% and four RDF bottom ash samples were also prepared with moisture contents ranging from 14% to 19%. Each set of four ash samples were tested unsoaked.

In addition to the unsoaked CBR test, one MB bottom ash sample and one RDF bottom ash sample were tested soaked. These samples were at their maximum dry density before being immersed in water for 96 hours prior to testing. This was done: 1) to provide the engineer with both expansion and strength loss information on the ash when it becomes saturated in the field 2) and for comparison to the unsoaked values (Bowles, 1992).

The ASTM specification defines the CBR number as, "the ratio of the unit stress required to effect a certain depth of penetration of the standard into a compacted specimen of soil at some water content and density to the standard unit stress required to obtain the same depth of penetration on a standard sample of crushed stone." The CBR number is usually based on the ratio for a penetration of 0.1". However, if the CBR number at a penetration of 0.2" is greater than that at 0.1", the test must be redone. If the second test yields the same results, the CBR number at 0.2" penetration is used.

These relationships are shown in equations 3.7 and 3.8.

$$\text{CBR @ 0.1"} = \frac{\sigma_{0.1"}}{1000 \text{ psi}} * 100 \quad (3.7)$$

$$\text{CBR @ 0.2"} = \frac{\sigma_{0.2"}}{1500 \text{ psi}} * 100 \quad (3.8)$$

where:        CBR = California Bearing Ratio in percent  
                $\sigma_{0.1}$  = Corrected stress @ piston penetration

### 3.3.15 Limerock Bearing Ratio (LBR)

The Limerock Bearing Ratio (LBR) values were calculated for MB and RDF bottom ash samples passing the #4 sieve using the data collected from the CBR tests. The previously collected data was used since the procedure for the CBR tests (ASTM D-1883-92) is in accordance with the procedure for the LBR tests (FDOT FM-5-515). The LBR was determined to evaluate the MWC bottom ash in various highway applications encountered in Florida.

The LBR number equals the stress at 0.1" penetration divided by 800 as shown in equation 3.9.

$$\text{LBR} = \frac{\sigma_{0.1}}{800 \text{ psi}} * 100 \quad (3.9)$$

where:        LBR = Limerock Bearing Ratio in percent  
                $\sigma_{0.1}$  = Corrected stress @ 0.1"

## 3.4 WTE Bottom Ash Environmental Tests

### 3.4.1 Loss on Ignition

The procedure outlined by ASTM C311 "Sampling and Testing Fly Ash or Natural Pozzolans for Use as a Mineral Admixture in Portland-Cement Concrete" (ASTM, 1990) was used to determine the loss-on-ignition of WTE bottom ash. Samples were oven dried at 105°C for 24 hours and then combusted within a furnace at 550° C for at least 2 hours to a consistent weight.

### 3.4.2 Elemental Analysis

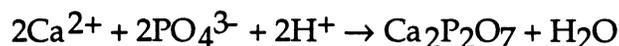
Ash samples for the study were sieved through a No. 4 sieve (4.76 mm in opening size) and stored in plastic containers until used. Two batches of bottom ash were retrieved at different times. Six samples that were 1,000 g each were taken for an analysis of Ag, As, Ca, Cd, Cr, and Pb. Ash samples were oven-dried at  $105\pm 5^\circ$  C for 4 hours and then crushed until a minimum of 500 g could be collected through a 500- $\mu$ m sieve. The crushed samples were weighed and dried in 24 hour increments at  $105\pm 5^\circ$  C until the change in the weight of the sample was less than 0.5%. All oven-dried samples were cooled in a desiccator prior to digestion. The plasticware used for the study was cleaned and soaked in a 10% HNO<sub>3</sub>-10% HCl acid bath for at least 24 hours and then rinsed at least three times with distilled-deionized water (DDW). All acid-cleaned apparatus was stored in plastic bags until used.

The Silberman and Fisher (1979) technique, which uses hydrofluoric-boric acids (HF-H<sub>3</sub>BO<sub>3</sub>), was utilized in this study. Approximately 0.5 g of each dried sample was weighed and transferred to a 125-ml polyethylene bottle. Next, 5 ml of DDW and 5 ml of HF were added to each sample. The samples were then shaken in a reciprocating shaker for 24 hours. Afterwards, 90 ml of 3% H<sub>3</sub>BO<sub>3</sub> was added to the samples, and the samples were shaken again for another 24 hours. Samples with undissolved residue were vacuum-filtered through a 0.40  $\mu$ m Millipore<sup>®</sup> membrane filter. The filtrate was then diluted to 100 ml of final solution.

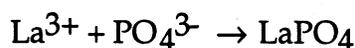
The filtrates were analyzed for concentrations of selected metals— which included Ag, As, Ca, Cd, Cr, and Pb—using a Perkin-Elmer Zeeman 5100 Atomic Absorption Spectrophotometer (AAS) equipped with a graphite furnace. Calcium was analyzed using the flame AAS, and Ag, As, Cd, Cr, and Pb were analyzed using the graphite furnace AAS. A well-characterized ash sample— Standard Reference Material fly ash (SRM-1633a) from the National Institute of Standards and Technology (NIST)—was used as a reference material. This was acid-digested using exactly the same procedure as that used for the ash samples to determine the completeness of the digestion of the ash and to provide the accuracy and quality assurance of analytical methods.

A 1% La<sup>3+</sup> (w/v) solution was used for the analysis of calcium in the AAS operation using an air-acetylene flame to suppress the interference caused by the presence of the stable oxysalts: aluminum, beryllium, phosphorus, silicon,

titanium, vanadium, and zirconium. The stable oxysalt phosphorus, for example, was formed by the reaction below:



With the addition of  $\text{La}^{3+}$  (w/v) solution,  $\text{Ca}^{2+}$  was able to be analyzed more effectively by minimizing the phosphate effect according to following equation:



The  $\text{La}^{3+}$  solution was prepared by adding 11.728 g of lanthanum oxide ( $\text{La}_2\text{O}_3$ ) to 10 ml of DDW and by next adding 50 ml of concentrated HCl to dissolve the  $\text{La}_2\text{O}_3$ . The solution was then diluted to 1 liter with DDW.

During the time the AAS graphite furnace was used, selected modifier solutions were utilized to improve the performance of metal analysis. The optimum parameters for furnace operation are presented in Table 3.3.

A standard calibration curve using three standards was carried out in each AAS operation. A solution of NIST 3171 or 1633a coal fly ash with a known concentration was analyzed to ensure the accuracy of the standard calibration curve.

### 3.4.3 Mineralogy

Samples were examined for the morphology of individual particles and for mineral formation within the ash matrix using a scanning electron microscope (SEM) equipped with an energy dispersive x-ray spectrometer (EDS). Prior to SEM/EDS analysis, samples were dried and sieved manually to collect particles in the range smaller than 0.5 mm or in the range smaller than 1 mm but greater than 0.5 mm. Both fraction sizes of ash samples were examined for mineral formation. Selectively, based on SEM observation, some samples were applied to EDS analysis to determine the predominant elements of ash particles or identified mineral.

### 3.4.4 Pozzolanic Activity

The pozzolanic activity index of MWC ash was determined according to ASTM C311-90 "Standard Test Methods for Sampling and Testing Fly Ash or Natural Pozzolans for Use as a Mineral Admixture in Portland-Cement Concrete" (ASTM, 1991). Cubes 5 x 5 x 5 cm were used as molds for curing specimens up to 28 days in a water-lime bath. This was followed by testing for compressive strength. The control mixture for the test was a combination of graded sand, type I portland cement, and water. The test mixture was prepared by replacing 20% of the cement with ground ash and passed through a #100 US standard sieve. The index was determined by the ratio of the strength of the test samples over the strength of the control samples multiplied by 100. An index of 100 indicates that ash has pozzolanic qualities equal to that of portland cement. An index greater than 100 indicates that ash has better pozzolanic qualities than portland cement, while an index less than 100 indicates that ash has poor pozzolanic qualities when compared with portland cement.

### 3.4.5 Column Leaching Test

The column leaching test, modified from the ASTM D4874 "Standard Test Method for Leaching Solid Waste in a Column Apparatus" (ASTM, 1989), was conducted to investigate the leaching characteristics of compacted MWC bottom ash. A segment of PVC pipe, with an inside diameter of 10.2 cm (4 in) and a height of 45.7 cm (18 in), was used in the study as a column apparatus. The PVC cylinder was mounted on a perforated clear acrylic plate and a rubber gasket was placed between the plate and the bottom of the cylinder. Before preparing a sample column, all parts of the test apparatus that would contact the ash material, leaching fluid, or product leachate were cleaned by submersion in an acid bath for 24 hours, rinsed with DDW at least three times and dried in a ventilating hood.

A previous study reports that the compactive effort of the modified Proctor test is suitable to simulate heavy field equipment used in construction (Forrester and Goodwin, 1990). Ash columns for leaching tests in the study were fabricated using the compaction technique according to the ASTM D698, "The Standard Methods of Test for Moisture-Density Relations of Soils Using 2.5-kg Hammer and 304.8-mm Drop" (ASTM, 1989). The PVC cylinder and a metal

collar were fastened firmly to a detachable base plate to ensure that the compaction energy was not absorbed by the vibration or shifting of the column cylinder during compacting. The compaction mold was weighed before and after compaction. The weight difference was used to determine the density of the compacted ash column.

To fabricate the ash column, ash material (less than 4.76 mm) was compacted into the cleaned PVC cylinder to a height of 15.2 cm or 45.7 cm in 4 and 12 layers, respectively, with 25 uniformly distributed blows per layer. The compaction was performed using a 24.5 Newton compaction hammer which dropped 0.305 m onto the ash material.

After compaction the whole apparatus, with the ash material inside it, was weighed again to determine the compacted weight of the MWC bottom ash. Approximately 300 g of ash from the same container was dried in the oven at  $105 \pm 5^\circ \text{C}$  until the weight difference after two successive periods of drying was less than 0.5% (ASTM D2216, 1989). The weight was then used to determine the moisture content of the wet sample—obtained by dividing the weight of the water in the ash by the dry weight of the ash.

The compacted ash column was carefully assembled in the laboratory. To avoid the potential error of presaturating an ash column before the leaching test was begun, a sprinkler system was developed in the study to simulate the mean annual maximum rainfall of 5.1 cm/hour (U.S. Geological Survey, 1970), which was converted to the flow rate of 7 ml/min. by multiplying 5.1 cm/hour by the surface area of the ash column (i.e.,  $81 \text{ cm}^2$ ).

To determine the effects of flow rate on the leaching characteristics of MWC bottom ash, leaching fluids at a rate of 4, 7, and 10 ml/min. were used, respectively, in the study. Leaching fluid was pumped to the top of the column and through the sprinkler system by an electrically driven peristaltic pump to simulate the rainfall event. The flow rate of the leaching fluid was determined before and after the leaching test by measuring the volume of the leaching fluid collected over a specific period of time. The sprinkler system was assembled by fitting 17 tubes with an inside diameter of 0.58-mm into a larger tube with a 3.18-mm inside diameter. The space between the 0.58-mm tubes was sealed with polyurethane glue (3M Marine Adhesive/Sealant, Part No. 05203). Leachate was collected from the bottom of the column.

For tests at a flow rate of 4 ml/min., leachate samples were collected every 20 minutes for 6 hours. At flow rates of 7 and 10 ml/min., leachate samples were

collected every 10 minutes for the first 2 hours, and every 20 minutes for the next 4 hours. The volume of individual leachate samples taken at each interval was measured and recorded. An aliquot of 10 ml of leachate was taken from each sample for pH determination before filtration. The rest of the sample was filtered to remove suspended solids. The filtered samples were then treated with 0.5-ml ultra pure HNO<sub>3</sub> to reduce pH to less than 2. The filtrates were then analyzed for concentrations of Ag, As, Ca, Cd, Cr, and Pb.

In addition to the change of flow rate, the effects of the contact time of leaching fluid with the ash column were determined by changing the height of the compacted MWC ash column. There was no criteria for selecting the height of the ash column to meet the objective of the study. Generally, a 30.5- cm column is used for column leaching study. To create a significant range of difference in height and to cover the general height of 30.5 cm, the column heights of 15.2 and 45.7 cm were investigated.

The leaching fluids used in the study were a pH 4.5 distilled-deionized water (DDW) and a pH 4.2 synthetic acid rain (SAR) solution. The effect of acid rain was determined by using an SAR solution as the leaching fluid. The SAR was prepared according to the National Atmosphere Deposition Program (NADP) quality reference to simulate acid rain common to the Northeastern United States (U.S. EPA, 1990). A primary solution was prepared by the addition of the chemicals shown in Table 3.4 to four liters of DDW. The primary solution was then diluted ten fold. A 1 N NaOH solution was used, when necessary, to adjust the pH to approximately 4.2.

Table 3.1 Specifications followed to evaluate physical composition

- Grain Size Distribution (ASTM C 136-84a)
- Moisture Content (ASTM C-566-89)
- Specific Gravity (ASTM D-854-92)
- Moisture-Density Relationship (ASTM D-698-91 & ASTM D-1557-91)
- Permeability (AASHTO T-215-70)
- Swell Potential (Seed and Chan, 1959)
- Shrinkage Potential (Seed and Chan, 1959)
- Unconfined Compressive Strength (ASTM D-2166-91)
- Consolidated Drained Triaxial Shear Test
- Determination of Elastic Modulus
- Determination of Resilient Modulus
- California Bearing Ratio (CBR) Test (ASTM D-1883-92)
- Limerock Bearing Ratio (CBR) Test

Table 3.2 CD triaxial shear testing program on bottom ash

ASTM D-698-91				ASTM D-1557-91			
MB < #4		RDF < #4		MB < #4		RDF < #4	
Dr	$\sigma_3$	Dr	$\sigma_3$	Dr	$\sigma_3$	Dr	$\sigma_3$
100%	5 psi	100%	5 psi	100%	5 psi	100%	5 psi
	10 psi		10 psi		10 psi		10 psi
	15 psi		15 psi		15 psi		15 psi
95%	5 psi	95%	5 psi	95%	5 psi	95%	5 psi
	10 psi		10 psi		10 psi		10 psi
	15 psi		15 psi		15 psi		15 psi
90%	5 psi	90%	5 psi	90%	5 psi	90%	5 psi
	10 psi		10 psi		10 psi		10 psi
	15 psi		15 psi		15 psi		15 psi
MB < #8		RDF < #8		MB < #8		RDF < #8	
Dr	$\sigma_3$	Dr	$\sigma_3$	Dr	$\sigma_3$	Dr	$\sigma_3$
100%	5 psi	100%	5 psi	100%	5 psi	100%	5 psi
	10 psi		10 psi		10 psi		10 psi
	15 psi		15 psi		15 psi		15 psi
95%	5 psi	95%	5 psi	95%	5 psi	95%	5 psi
	10 psi		10 psi		10 psi		10 psi
	15 psi		15 psi		15 psi		15 psi
90%	5 psi	90%	5 psi	90%	5 psi	90%	5 psi
	10 psi		10 psi		10 psi		10 psi
	15 psi		15 psi		15 psi		15 psi

Dr = Relative Density       $\sigma_3$  = confining pressure      1 psi = 6.895 kPa

Table 3.3. Conditions for graphite furnace AAS with Zeeman background correction

Element	Wavelength (nm)	Slit (nm)	Pretreatment temp./time <sup>a</sup>	Atomize temp./time	Modifier Solution
Ag	328.1	0.7	800/20	1800/5	$1/2 (\text{NH}_4)_2\text{HPO}_4$ <sup>b</sup> + $1/2 \text{NH}_4\text{NO}_3$ <sup>c</sup>
As	193.7	0.7	1200/35	2300/5	$\text{Ni}(\text{NO}_3)_2$ <sup>d</sup>
Cd	228.8	0.7	600/35	1600/5	$\text{NH}_4\text{NO}_3$
Cr	357.9	0.7	1650/30	2500/5	$1/2 (\text{NH}_4)_2\text{HPO}_4$ + $1/2 \text{NH}_4\text{NO}_3$
Pb	283.3	0.7	700/35	1800/5	$1/2 (\text{NH}_4)_2\text{HPO}_4$ + $1/2 \text{NH}_4\text{NO}_3$

<sup>a</sup>Temperature was given in °C and time in second.

<sup>b</sup> 3 %  $(\text{NH}_4)_2\text{HPO}_4$

<sup>c</sup> 10 %  $\text{NH}_4\text{NO}_3$

<sup>d</sup> 2 %  $\text{Ni}(\text{NO}_3)_2$

Table 3.4 Chemicals used to prepare a primary solution for a 4-liter synthetic acid rain

Chemical	Weight (g)
NaNO <sub>3</sub>	0.1150
KNO <sub>3</sub>	0.2196
NH <sub>4</sub> NO <sub>3</sub>	0.6480
MgCl <sub>2</sub>	0.0821
H <sub>2</sub> SO <sub>4</sub>	0.1755 <sup>a</sup>
CaSO <sub>4</sub>	0.1057

<sup>a</sup> The H<sub>2</sub>SO<sub>4</sub> had a concentration of 98 percent by weight.

**Figure 3.1 Survey of Statewide Waste-to-Energy Plants  
A Study for the Florida Department of Transportation  
to Enable the Development of FDOT Specifications for WTE Bottom Ash**

1. Facility name, address, phone number and contact person(s) \_\_\_\_\_  
\_\_\_\_\_
2. Name and phone number of the contractor (or plant operator) that operates the plant \_\_\_\_\_  
\_\_\_\_\_
3. Is the facility a mass-burn(    ), or a refuse derived fuel plant (    )?
4. Plant operation information:
  - a. MSW burn rate \_\_\_\_\_
  - b. Furnace temperature \_\_\_\_\_
  - c. Production rate of MSW ashes:
    - fly ash \_\_\_\_\_
    - bottom ash \_\_\_\_\_
    - scrubber ash \_\_\_\_\_
5. Are the incinerator ashes combined (    ) or kept separate (    ) before landfilling?
  - If combined what are the approximate percentages of fly ash (    ), scrubber residue (    ), and bottom ash(    )?
6. Is there any additional treatment performed on the incinerator ashes before landfilling? Yes (    ) No (    ).  
If yes please check the appropriate items below.
  - a. removal of metals (    ) \_\_\_\_\_
  - b. removal of certain metals (    ) \_\_\_\_\_
  - c. removal of other materials (    ) \_\_\_\_\_
7. Who is responsible for your resource recovery (name, address, phone number)? \_\_\_\_\_  
\_\_\_\_\_
8. How are you currently using the bottom ash? \_\_\_\_\_
  - a. percent landfilled \_\_\_\_\_
  - b. percent daily cover \_\_\_\_\_
  - c. percent other, please specify \_\_\_\_\_
9. Please include any additional information or reports on your facilities and bottom ash which may be available.
10. If available, please provide printed materials (i.e. brochures) about your facility to:  
Paul J. Cosentino, Ph.D., P.E.  
Civil Engineering Department  
Florida Institute of Technology  
Melbourne Florida, 32901-6988  
(407) 768-8000 (ext. 7555)

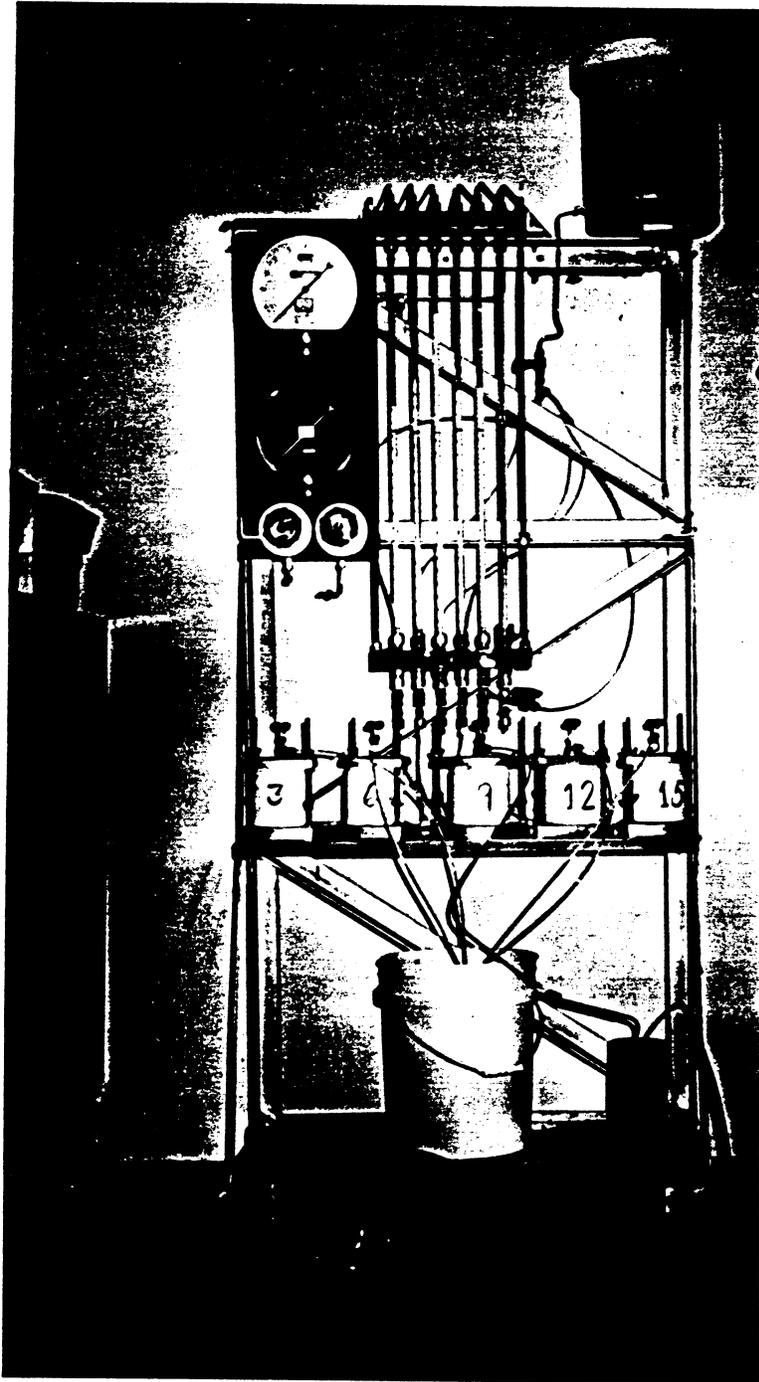


Photo 3.1. Permeability test set-up

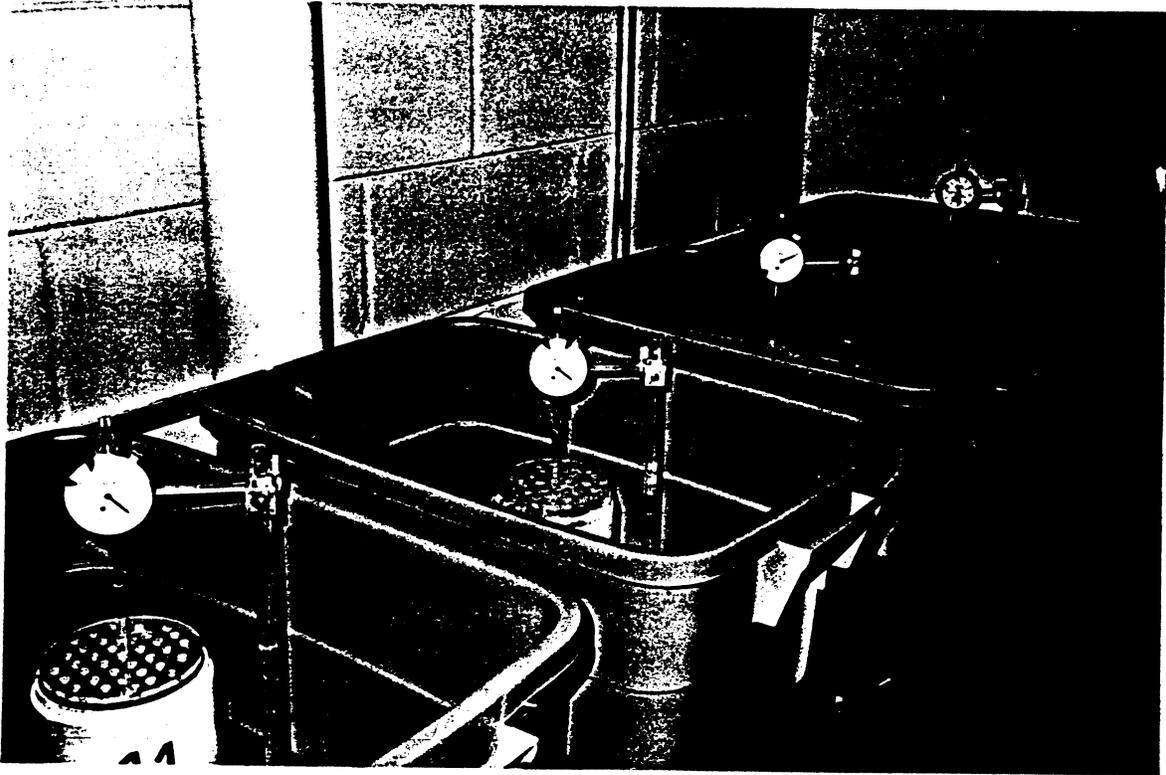
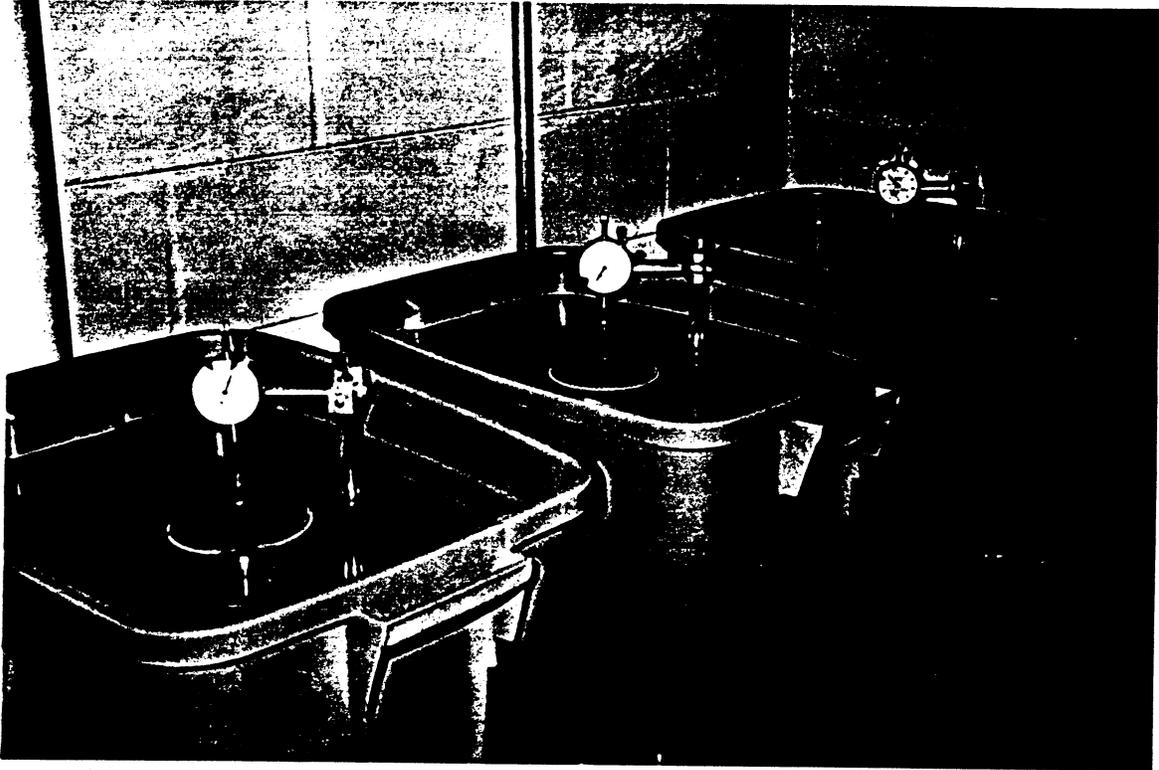
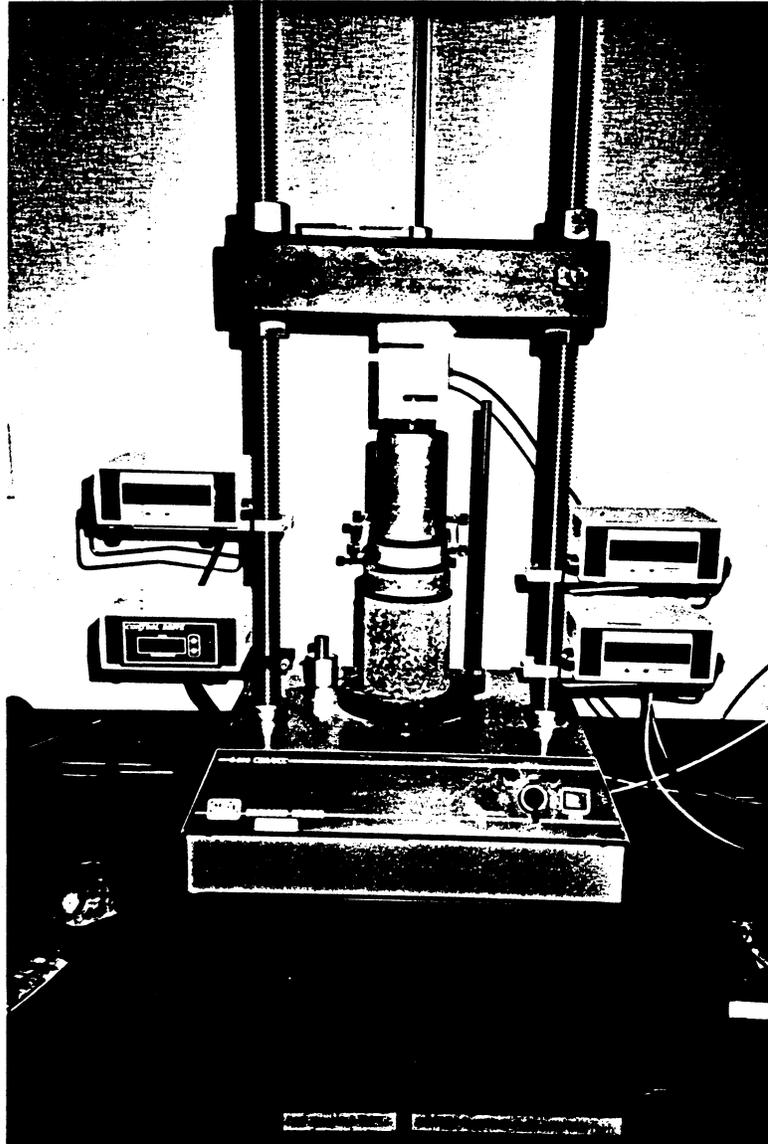


Photo 3.2. Swell and shrinkage tests, saturated and dry



**Photo 3.3. Unconfined compressive test on the WTE bottom ash specimen**

## 4.0 PHYSICAL, GEOTECHNICAL AND ENVIRONMENTAL PROPERTIES

MWC bottom ash was collected from the facilities described in Section 3.2. Normally bottom ash is combined with fly ash at most WTE facilities. The MWC bottom ash from the MB facility was collected in two plastic 55 gallon drums and shipped to Florida Tech in Melbourne, on October 30, 1992 and January 22, 1993. The MWC bottom ash from the RDF facility was also collected in two plastic 55 gallon drums and shipped to Florida Tech in Melbourne, on January 21, 1993. The bottom ash was stored indoors in sealed 55 gallon plastic drums. The MB ash could be described as a coarse grained material that is dark gray to black in color while the RDF ash could be described as a medium grained material that is light gray to gray in color

### 4.1 Physical Composition

#### 4.1.1 Visual Classification

The physical composition of the bottom ash evaluated at Florida Tech is shown in Figure 4.1. The MB bottom ash sample consisted of: 48% clinker and miscellaneous, 32% metals, 16% glass and 4% fines. The RDF bottom ash sample consisted of 60% clinker and miscellaneous, 16% metals, 20% glass and 4% fines. These analyses represent the composition of the ash sample that were used for this study and should not be considered as representative of the ash produced at the two facilities. The RDF bottom ash had a lower percentage of metals than the MB bottom ash. This lower percentage of metals in the RDF bottom ash can be attributed to the processing of the MSW prior to combustion. Thus, it could be inferred that the RDF facility was recovering twice as many of recyclable metals as the MB facility. The composition of bottom ash may be a significant factor in determining the suitability of different bottom ash sources for highway applications.

#### 4.1.2 Specific Gravity

The average specific gravity for the MB bottom ash samples passing the #4 and #8 sieves were found to be 2.79 and 2.72, respectively. The average specific gravity for the RDF bottom ash samples passing the #4 and #8 sieves

were found to be 2.55 and 2.45, respectively. These specific gravities fall within the range of values reported by: Chavez (1993) [2.72], Huang and Lovell (1990) [1.94 to 3.23] and Seals, Moulton and Ruth (1972) [2.28 to 2.78].

The specific gravity of bottom ash is a function its chemical constituents (Seals, *et al.*, 1972). Huang and Lovell (1990) found that bottom ash with high carbon content resulted in low specific gravity, whereas bottom ash with high iron content resulted in high specific gravity. The differences in specific gravity reported in this investigation can be attributed to the different compositions of the two bottom ashes. The MB bottom ash has a larger percentage of metals which tends to increase the specific gravity, while the RDF bottom ash has a higher percentage of glass which decreases the specific gravity. The specific gravity of the bottom ash decreased with a decrease in grain size which may be attributed to the mass being concentrated in the larger particles.

#### 4.1.3 Moisture Content

The moisture content of the MB and RDF bottom ash when it was delivered ranged from 11% to 19%, and from 16% to 26%, respectively. These moisture contents were taken during the two year investigation period. Previous studies utilizing MB bottom ash have reported similar moisture contents at Florida Tech Wu (1990) 10.6% to 18%, Nevin (1992) 4.4% to 7.5% and Jain (1992) 12% to 18%. Chesner (1989) found that bottom ash samples from facilities which have ferrous metal removal systems tend to have a higher moisture content than ash samples from facilities without these systems. This was not found to be the case in this investigation where processing of ash for metals removal is conducted after combustion.

#### 4.1.4 Absorption

The absorption of the MB and RDF bottom ash ranged from 9% to 12%, and 11%, respectively. The water absorption rates of the ash are higher than conventional fill materials.

#### 4.1.5 Mineralogy

Results of mineral examination showed the presence of mineral fibers on the surface of ash particles ( Figure 4.2). The fiber is embedded in the ash particle and is found to link ash particles together ( Figure 4.3). Identification of the mineral fiber using an energy dispersive x-ray spectrophotometer has shown the predominance of aluminum(Al), silica (Si), and Calcium(Ca) on the fiber ( Figure 4-4).

#### 4.1.6 Pozzolanic Activity

The pozzolanic activity index of MB bottom ash was found to be 135– indicating that MB bottom ash possesses better pozzolanic properties than cement. The index value was very similar to that reported by Shieh and Kalajian (1994). The index of RDF bottom ash, however, was much less than 100,–*i.e.*, 65– indicating that RDF bottom ash possesses poor pozzolanic properties when compared to cement.

#### 4.1.7 Loss on Ignition

The loss on ignition of MB and RDF bottom ash was found to range between 4.4% to 5% for both bottom ash sources. These values are at the low end of the range of coal fly ash (2% to 20%) used as a concrete admixture.

### 4.2 Engineering Classification

Grain size distribution curves for the bottom ash from both facilities are shown in Figure 4.5. Table 4.1 presents classification parameters and a brief summary of the physical properties of the bottom ash. Both ashes display physical characteristics similar to a well-graded, granular soil. Although the effective particle sizes, uniformity coefficients and coefficients of curvature differ between the two ashes, both materials would be classified under the Unified Soil Classification System as well graded sands with little or no fines (SW). Under the American Association of Transportation and Highway Officials, the ash is classified as a type A-1-a (0), which is characterized as granular material with a small amount of fines (Holtz and Kovacs, 1981). These classification systems are used only to classify the ash as a soil. The MB and RDF bottom ash would be classified as #89 using the ASTM aggregate classification.

### 4.3 Bulk Rodded Unit Weight

The bulk rodded unit weight of the MB and RDF bottom ash ranged from 77 pcf to 81 pcf, and 69 pcf, respectively. The bulk rodded unit weight of the ash are at the lower range associated with conventional fill materials.

### 4.4 Moisture Density Relationships

The moisture-density relationship results for the WTE bottom ash are summarized in Table 4.2 and are shown in Figures 4.6 to 4.9. The moisture-density relationships for the MB bottom ash samples have higher maximum dry densities and lower optimum moisture contents as compared to the RDF bottom ash samples. These results compare well with previous investigations summarized in Table 2.1. The shape of the compaction curves shown in Figures 4.6 to 4.9 resemble the parabolic Type A compaction curve found in typical laboratory investigations by Winterkorn and Fang (1975).

For a given ash source, the maximum dry density decreased with a decrease in grain size while the optimum moisture content increased. Increasing the compaction energy tends to increase the maximum dry density but also decreases the optimum moisture content. The moisture density relationships for ash behave very similar to conventional soils, as shown by Holtz and Kovacs (1981).

### 4.5 Permeability

The variation in permeability with respect to moisture content is shown in Figure 4.10. The RDF and MB bottom ash when compacted dry of optimum moisture content; displayed permeability with magnitudes less than  $1 \times 10^{-2}$  cm/sec. However, once optimum moisture content was reached, the permeability reduced by 3 orders of magnitude to  $1 \times 10^{-5}$  cm/sec in both bottom ashes. This behavior is similar to that of a sandy clay, where the coefficient of permeability decreases as the compaction moisture content increases (Holtz and Kovacs, 1981). Demars *et al.* (1994) report similar results for MB ash at optimum with a magnitude of  $1 \times 10^{-4}$  cm/sec. Huang and Lovell (1990) report permeability coefficients of  $5.0 \times 10^{-3}$  and  $2.0 \times 10^{-3}$  cm/sec for bottom ashes having 6 and 10 percent fines and compacted at 95 percent of the maximum dry

density (109 pcf). They concluded that the percentage of fines has a predominant effect on the permeability of bottom ash. The percentage of fines for the MB bottom ash used in this investigation was 3 percent.

The effects that age after compaction has on permeability of bottom was determined by examining the results of tests conducted on compacted MB bottom ash at various compaction moisture contents (10% to 16%) and at time periods of 0, 7, 14, 21 and 120 days. Permeability tests conducted at 0, 7, 14 and 21 days were kept saturated while tests conducted at 120 days were allowed to air dry after compaction. The results are presented in Figure 4.11 and show very small variations of permeability with age for ash that is below or near optimum moisture content.

Permeability in bottom ash decreases significantly as the compaction moisture content increases past the optimum moisture content. Age has a minimal effect on the permeability of bottom ash compacted below optimum moisture content.

#### **4.6 Shrink-Swell Potential**

To evaluate the free shrink-swell characteristics of compacted WTE bottom ash, ten (10) specimens of MB bottom ash were compacted following ASTM D 698 at moisture contents ranging from 10 % to 16%. The moisture contents were selected so that the prepared specimens would bracket the optimum moisture content. Half of the specimens were allowed to air dry to determine volume change over a period of 120 days. These specimens will be referred as Dry Condition

Figure 4.12 represents the maximum and minimum of the percent axial change versus time of the compacted bottom ash specimens. The compacted bottom ash did not show any significant volume change over the initial ten to fourteen days. Approximately nine days after beginning the test, the bottom ash exhibited an increase in volume and white air bubbles were observed on the upper surface of the specimens. The rate of axial expansion in the compacted MWC bottom ash that was allowed to air dry decreased after fifty days, though axial expansion continued to the completion of the testing period. Figure 4.12 shows that after 120 days the axial expansion ranged from 2.3 % to 3.6 %.

The remaining specimens of compacted bottom ash were saturated in water. Figure 4.13 represents the percent axial expansion versus time of the five

specimens that were submerged in water. During the initial 34 days of submersion, the compacted bottom ash had a negligible volume change. After day 34, it was decided that the specimens should be removed from the water. The bottom ash compacted wet of optimum shrunk about 0.5 percent, while the remaining specimens exhibited little volume change.

Fifteen days later (50 days after initial specimen fabrication) all the specimens began to increase in volume with axial expansion of 0.8 to 1.6 percent at 120 days. At the same time as the specimens increased in volume more air bubbles appeared on the upper surface of the specimens. These white air bubbles, observed in all the bottom ash specimens, may be an indication that a chemical reaction was inducing the change in volume.

When Figure 4.12 is compared with Figure 4.13, it is evident that bottom ash specimens that had been submerged for 34 days (Saturated Condition) had less of a tendency to swell than the specimens that had been allowed to air dry (Dry Condition). While the axial expansion of the bottom ash is less than that of expansive clay, it may be possible to reduce its magnitude by submerging in water immediately and allowing air drying for a period of time (aging). This aging procedure allows time for the swelling process to take place before the bottom ash is used in any construction application (Hjelmar, 1990).

## **4.7 Strength-Deformation**

The strength-deformation properties of the bottom ash that were evaluated, were unconfined compressive strength, consolidated drained triaxial shear strength, the elastic and resilient moduli and the California and limerock bearing ratios.

### **4.7.1 Unconfined Compressive Strength**

The unconfined compressive strength test results for the MB and RDF bottom ash passing the #4 and #8 sieves are shown in Figures 4.14 and 4.15, respectively. Both figures show significantly higher unconfined compressive strengths for bottom ash occurring at higher compaction energy levels and independent of ash source.

From Figures 4.14 and 4.15, both the MB and RDF bottom ash samples compacted utilizing ASTM D-1557 exhibited higher compressive strengths on the

dry side of optimum and lower compressive strengths on the wet side of optimum. The MB and RDF ash samples passing the #8 sieve and compacted utilizing ASTM D-1557 or D-698 also displayed higher compressive strengths on the dry side of optimum. The only exception to this behavior was the bottom ash passing the #4 sieve and compacted utilizing D-698 which displayed maximum compressive strengths at or near the optimum moisture content.

Figure 4.14 also includes the results of the MB ash samples tested by Chavez (1993) for comparison to MB and RDF ash samples passing the #4. The bottom ash samples in this study were tested immediately after compaction while the MB ash samples tested by Chavez (1993) were compacted utilizing ASTM D-698 and then tested 120 days later. Chavez (1993) aged MB ash samples attained a compressive strength of nearly 100 psi while the MB ash samples that were tested immediately after compaction had a compressive strength of approximately 25 psi. From these results, it is evident that pozzolanic reactions which increase unconfined compressive strength have occurred as the compacted ash ages.

The unconfined compressive strength of compacted bottom ash is quite similar to compacted soils in that the unconfined compressive strength is higher either dry or near optimum moisture content and increases with compaction energy. Thus compaction moisture content and compaction energy are controlling factors for unconfined compressive strength. Allowing compacted bottom ash to age increases its strength due to pozzolanic activity of the bottom ash.

#### 4.7.2 Consolidated Drained Triaxial Shear

The CD triaxial shear results are summarized in Table 4.3 for densities at relative compaction of 90%, 95%, and 100% of ASTM D-698 and D-1557. Every attempt was made to prepare the samples at moisture contents which were within 1% dry of optimum. The MB bottom ash samples passing the #4 sieve could not be compacted to the relative compaction of 95% and 100%. The number of layers and the number of blows were continually increased, however the required densities could not be achieved in the laboratory.

Typical stress-strain curves and the volumetric strain versus axial strain curves for MB and RDF ash passing the #4 and #8 sieves in loose and dense conditions are presented in Figures 4.16 and 4.17, respectively. The concepts

regarding dilatancy and interlocking as presented by Lambe and Whitman (1979) were found to hold true during triaxial compression. The densely compacted ash samples displayed a well defined peak and the principal stress difference decreased following this peak. The loosely compacted ash samples did not display a well defined peak and the principal stress difference remained fairly constant once the compressive strength had been reached.

The densely compacted ash samples significantly expanded in volume as the strain increased while the loosely compacted ash samples first decreased in volume and then either expanded to almost the initial volume or continually decreased. Failure occurred between 2.3% and 6.6% axial strain for confining pressure ranging between 5 psi ( 35 kPa) and 15 psi ( 105 kPa), respectively.

The cohesion of both MB and RDF bottom ash ranges from 2 psi to 5 psi (14 kPa to 35 kPa). A cohesion of zero would have been expected since the bottom ash is a granular material. This relatively small amount of cohesion can be attributed to pozzolanic reactions occurring in the bottom ash. The friction angles of both MB and RDF bottom ash ranges from 24° to 50° and were found to be a function of relative compaction and grain size. When the compaction energy and relative compaction were kept constant, the bottom ash passing the #4 sieve tends to have slightly larger values of cohesion and larger friction angles than the bottom ash passing the #8 sieve. Both the MB and RDF bottom ash exhibited an increase in friction angle from 2° to 7° as the relative compaction increased. The MB bottom ash had slightly larger friction angles than the RDF bottom ash.

Figure 4.18 shows the relationship between the friction angle and the density of the bottom ash. This graph shows that the friction angle increases with an increase in density for both ash types. However, the MB bottom ash has a slightly greater increase in friction angle with respect to density than the RDF bottom ash. The RDF bottom ash exhibits higher friction angles with respect to density than the MB bottom ash.

#### 4.7.3 Elastic Modulus

The elastic modulus results for the MB and RDF bottom ash are shown in Figures 4.19 and 4.20, respectively. The elastic moduli values were calculated over a strain difference of 0.2% in the elastic portion of the stress strain curve. Both figures show an increase in elastic modulus as the dry unit weight and

confining pressures increase. Both ash types exhibit elastic modulus values in the range of 500 to 5,000 psi (3,500 kPa to 35,000 kPa). For a given density, the RDF bottom ash yields an elastic modulus approximately twice as large as the MB bottom ash, i.e. the RDF bottom ash is twice as stiff as the MB bottom ash.

Typical elastic moduli ranges for various granular materials are as follows: silty sand: 9,800 - 19,600 kN/m<sup>2</sup>, loose sand: 9,800 - 30,000 kN/m<sup>2</sup>, and dense sand: 39,000 - 78,500 kN/m<sup>2</sup> (Jumikis, 1984). The MB and RDF bottom ash samples exhibit elastic moduli within the range of loose sands.

#### 4.7.4 Resilient Modulus

A strain increment of 0.1% was used to calculate the resilient moduli. The resilient modulus values for both the MB and RDF bottom ash are combined in these figures. The resilient modulus for the MB bottom ash resulted in the following equation:  $M_r = 1.362q^{0.634}$  while the resilient modulus for the RDF bottom ash resulted in the following equation:  $M_r = 1.336q^{0.645}$ . Since there was little difference between these two equations, the MB and RDF resilient modulus values were combined to obtain the following correlation between the resilient modulus and the first stress invariant:  $M_r = 1.349q^{0.640}$ . The correlation between the resilient modulus and the deviator stress for the MB and RDF bottom ash is as follows:  $M_r = 8.437 + 0.432\sigma_d$ . These equations only give approximate relationships for the resilient modulus of bottom ash.

The resilient modulus equation was developed as follows:  $M_r = K_1 q^{K_2}$  where  $K_1 = 1.349$  and  $K_2 = 0.640$ . These values of  $K_1$  and  $K_2$  are much smaller than those reported by Sneddon (1988) for fine sands. Sneddon (1988) used a faster loading rate and a much smaller strain as compared to the loading rate and strain used in this study.

#### 4.7.5 California Bearing Ratio (CBR)

The soaked and unsoaked CBR results for the MB and RDF bottom ash passing the #4 sieve are shown in Figures 4.21 and 4.22, respectively. The MB bottom ash exhibits CBR values approximately twice as large as the RDF bottom ash. The MB bottom ash also exhibits higher CBR values on the wet side of optimum with the peak occurring at a moisture content of 13.5% and a CBR of 194. The RDF bottom ash exhibits higher CBR values on the dry side of optimum

with the peak occurring at a moisture content of 15.6% and a CBR of 80. The CBR values of both MB and RDF ash are very sensitive to the compaction moisture content of the sample.

The above results are based on the first trial run and were retested because the CBR values at 0.2" were greater than those at 0.1" (ASTM D-1883-87). However, only one sample was retested for each ash type. The retested CBR value for the MB bottom ash sample showed a decrease for the given moisture content while the retested CBR value for the RDF bottom ash sample showed an increase for the given moisture content. These results verify the inconsistency in the CBR test as discovered by Rodriguez *et al.*, (1988). Therefore, the remaining points along each curve were not retested and only one point for each ash type was tested soaked.

The CBR values obtained after soaking for both ash types were less than the unsoaked CBR values with respect to the compaction moisture content. Since both ash types are very sensitive to the compaction moisture content, a slight increase in the moisture content would have resulted in a very small difference between the soaked and unsoaked CBR values while a slight decrease in moisture content would have resulted in a large difference between the soaked and unsoaked CBR values. From the soaked samples, the recorded percent swell for both ash types over the four day period was found to be less than 0.02% and is considered negligible. Therefore, a loss in strength occurs when the samples are saturated.

From the above results, MB bottom ash can be utilized in road base since its CBR values exceed 100. However, RDF bottom ash exhibits CBR values below 100 and is not recommended for use in road base based on this criteria. Both ashes have CBR values greater than ten and can be classified as very good subgrade materials according to Baker (1982). Also, both ash types exhibit little to no swell and should not cause any field problems during saturated conditions.

#### 4.7.6 Limerock Bearing Ratio (LBR)

The soaked and unsoaked LBR results for the MB and RDF bottom ash passing the #4 sieve are shown in Figures 4.21 and 4.22, respectively. According to these figures, the MB bottom ash exhibits LBR values approximately twice as large as the RDF bottom ash. The MB bottom ash also exhibits higher LBR values on the wet side of optimum with the peak occurring at a moisture content of

13.5% and an LBR of 183. The RDF bottom ash exhibits higher LBR values on the dry side of optimum with the peak occurring at a moisture content of 15.6% and an LBR of 92. The LBR values of both MB and RDF ash are very sensitive to the compaction moisture content of the sample.

The above results are based on the data obtained from the first CBR trial run and were retested since the CBR values at 0.2" were greater than those at 0.1" (ASTM D-1883-87). However, only one sample was retested for each ash type. The retested LBR value for the MB bottom ash sample showed a decrease for the given moisture content while the retested LBR value for the RDF bottom ash sample showed an increase for the given moisture content. These results also verify that there is an inconsistency in the LBR test, as well as in the CBR test. Therefore, the remaining points along each curve were not retested and only one point for each ash type was tested in the soaked condition.

The LBR values obtained after soaking for both ash types were less than the unsoaked LBR values with respect to the compaction moisture content. Since both ash types are very sensitive to the compaction moisture content, a slight increase in the moisture content would have resulted in a very small difference between soaked and unsoaked LBR values while a slight decrease in moisture content would have resulted in a large difference between soaked and unsoaked LBR values. From the soaked samples, the recorded percent swell for both ash types over the four day period was found to be less than 0.02% and is considered negligible. Therefore, a loss in strength occurs when the samples are saturated.

The above results suggest that MB bottom ash can be utilized in road base since its LBR values exceed 100. However, the RDF bottom ash exhibits LBR values below 100 and should not be recommended for use in road base. Also, both ash types exhibit little to no swell and should not be a problem in the field during saturated conditions.

## 4.8 Chemical and Environmental Analyses

The results of the column leaching study were presented both in leached concentration of selected metals and in accumulated leaching percentage (ALP) with respect to time. The ALP was expressed as the percentage of the selected metal that was leached from the column.

$$ALP = \sum_{k=1}^{k=24} \frac{C_{L_k} \times V_{L_k}}{C_T \times W_T} \times 100$$

Where

$k$  = sampling of leachate,

$C_{L_k}$  = metal concentration of leachate collected at the  $k_{th}$  sampling (g/L),

$V_{L_k}$  = volume of leachate collected in the  $k_{th}$  sampling (L),

$C_T$  = metal concentration of MWC bottom ash (or g/kg), and

$W_T$  = net weight of bottom ash compacted into the PVC cylinder (g).

### 4.8.1 Selected Elemental Concentrations of MWC Bottom Ash

Table 4.5 shows the results of the elemental analysis of Standard Reference Material 1633a coal fly ash from NIST. The technique of HF-H<sub>3</sub>BO<sub>3</sub> digestion recovered 96 % of Ca, 88 % of As, 100 % of Cd, 94 % of Cr, and 99 % of Pb. The 88 % recovery of As was acceptable since the coefficient of variation of the certified value for As was about 11 %.

Table 4.6 shows the results of elemental analysis of this study and the reported values from another study (EPA, 1990) as well as the abundance of the selected metals in soil (Brooks, 1978). Metal concentrations in MWC bottom ash varied largely in the EPA's study. The MWC bottom ash provided by the Pinellas County Facility had Ag, Cd, and Ca concentrations at the low end of the EPA's reported values. Concentrations of As, Cr, and Pb in the study were found in the middle range of the EPA's reported values. When compared to the natural abundance in soil, As and Cd concentrations in MWC bottom ash were relative high, while Pb was much higher. Concentrations of Ag and Cr in MWC bottom ash were similar to those found in soil. Concentration of calcium, a major binding element in MWC bottom ash, was slightly lower in the MWC bottom ash than that in average soil.

## 4.8.2 Reproducibility of Column Leaching Test

All the compacted ash columns in the study had a density of  $16.7 \pm 0.5$   $\text{kN/cm}^3$  under the compaction energy of  $605 \text{ kJ/m}^3$ . In each of columns 4, 5, and 6, two replicate tests were conducted to determine the reproducibility of the column leaching test. To estimate the degree of reproducibility of the two replicates, the p-value test was applied to the analytical data (Goldman and Weinberg, 1986). A hypothesis ( $H_0$ ) assuming replicate tests 1 and 2 were the same and its alternative hypothesis ( $H_1$ ) assuming replicate tests 1 and 2 were different were made. A "z" value was obtained according to the following equation, which expressed the difference between  $R_1$  and  $R_2$  in units of standard deviation of the replicate test 2.

$$z = \frac{R_2 - R_1}{\frac{s}{\sqrt{n}}}$$

where

$R_1$  = mean concentration of metals in the leachate from replicate test 1,

$R_2$  = mean concentration of metals in the leachate from replicate test 2,

$n$  = number of leachate samples, and

$s$  = standard deviation of metal concentrations in the leachate from  $R_2$ .

The p value for this test was the areas under the normal distribution curve to the right of  $z$  and to the left of  $-z$ . For convenience, a computer program of Microsoft Excel 4.0 was used to calculate the p-values. A significance level of 0.05 was used as an indicator to measure the extent to which the data do support  $H_0$ . If the p value  $< 0.05$ , the reproducibility of two replicate tests is not good at the 0.05 level of significance, and vice versa. The p-values of two replicate tests from columns 4, 5, and 6 are listed in Table 4.6. All the columns, except the leaching of Pb from column 4, had very good reproducibility, which was attributed to the invariant conditions under which the ash column was compacted. At pH 12.5, Pb in MWC bottom ash is highly leachable (Van Der Sloot *et al.*, 1989). The observed variations in the leaching of Pb from column 4 might be due to the presence of more leachable Pb in one of the column 4 tests. However, the leaching of Pb from columns 5 and 6 was reproducible when DDW at pH 4.5 was used as the leaching fluid.

### 4.8.3 Leaching of Calcium

#### 4.8.3.a MB Ash Results

##### Effect of pH

Figures 4.23 and 4.24 show the leaching of Ca from columns 9 and 11 subjected to DDW and SAR, respectively. No significant difference in the leaching of Ca was found between columns 9 and 11 (Figure 4.23). Both columns exhibited the same release pattern of Ca over time; the leaching of Ca decreased at the beginning of test and increased afterwards. The percentage of the total Ca leached from both columns during the 8-hour test was about 0.16 % (Figure 4.24). The initial pH for SAR and DDW was 4.2 and 4.5, respectively. The major difference between SAR and DDW was that SAR contained various cations and anions. The results indicated that leaching of Ca from ash columns was not affected by other ionic species that were present in the SAR solution.

##### Effect of Flow Rate

Figures 4.25 and 4.26 show the leaching of Ca from columns 7, 8, and 9 subjected to DDW at a flow rate of 4, 7, and 10 ml/min., respectively. The measured pH of the leachate for the three columns declined steadily from 11.9 in the beginning to 10.7 at the end. Concentrations of Ca in the leachate from columns 7, 8, and 9 decreased within the first hour of the test and increased slowly in the rest of testing period (Figure 4.25). After an 8-hour test, approximately 0.04 %, 0.10 %, and 0.16 % of the total Ca were leached from columns 7, 8, and 9, respectively (Figure 4.26). The heights of columns 7, 8, and 9 were fixed at 15.2 cm. Under the same compaction technique, it could be assumed that same amount of ash was compacted into each of the three columns for testing. Therefore, the observed difference in the leaching of Ca (Figure 4.25) was from the difference in flow rate. That is, for ash columns with the same height, a higher flow rate of leaching fluid results in a higher leaching of Ca.

Figures 4.27 and 4.28 show the leaching of Ca from columns 10 and 11 subjected to SAR at a flow rate of 4 and 10 ml/min., respectively. Similarly, the higher flow rate of leaching fluid resulted in higher leaching of Ca from ash column (Figure 4.27). The percentages of the total Ca leached from columns 10 and 11 were 0.03 % and 0.17 %, respectively (Figure 4.28).

### Effect of Column Height

Figures 4.29 and 4.30 show the leaching of Ca from columns 1, 2, 5, and 6 at different heights subjected to DDW at a flow rate of 7 ml/min. Concentrations of Ca in the leachate from columns 1 and 5 (15.2 cm) decreased in the beginning of the test to a minimum, and subsequently increased steadily as a function of time (Figure 4.29). Concentrations of Ca in the leachate from columns 2 and 6 (45.7 cm) leveled off within 3 hours. As shown in Figure 4.30, the total percentages of Ca leached over time from columns 1, 2, 5, and 6 were 0.11 %, 0.01 %, 0.09 %, and 0.02 %, respectively, during the 8-hour test. Over the 8-hour test, the release of Ca from 15.2-cm ash columns increased steadily.

### Summary

It was found that SAR and DDW had similar effects on the leaching of Ca from ash columns. For ash columns with the same height, a higher flow rate resulted in higher leaching of Ca. For ash columns with different heights at the same flow rate of leaching fluid, 15.2-cm columns resulted in higher leaching of Ca than 45.7-cm columns.

#### 4.8.3.b RDF Ash Results

Figures (4.31), (4.32) and (4.33) show the leaching of Ca from 6- and 18-inch RDF and MB ash columns when they were subjected to DDW or SAR. The trend in Ca leaching for both MB and RDF ashes was very similar, however, RDF ash had more leachable Ca than MB ash. No significant difference in the leaching of Ca was found between RDF and MB ash when SAR was used in the leaching test.

#### 4.8.4 Leaching of Arsenic

##### 4.8.4.a MB Ash Results

### Effect of pH

Figures 4.34 and 4.35 show the leaching of As from columns 9 and 11 subjected to DDW and SAR, respectively. Concentrations of As in the leachate from columns 9 and 11 were below the EPA toxicity standard of 5,000 g/L and decreased as a function of time—leveled off within 2 hours (Figure 4.34). The release pattern of As from columns 9 and 11 was very similar, suggesting that an

SAR solution had no significant effect on the leaching of As, when compared with DDW. Approximately the same amount of the total As (i.e., 0.12 %) was leached from columns 9 and 11 (Figure 4.35).

#### Effect of Flow Rate

Figures 4.36 and 4.37 show the leaching of As from columns 7, 8, and 9 at a flow rate of 4, 7, and 10 ml/min., respectively. Concentrations of As in the leachate from columns 7, 8, and 9 were below the EPA toxicity standard of 5,000 g/L and decreased as a function of time (Figure 4.36). Most of the leachate had As concentrations below the drinking water standard of 50 g/L. Concentrations of As in the leachate from columns 8 and 9 leveled off within 2 hours. Column 7 had higher concentrations of As in the leachate than columns 8 and 9 at the same sampling period. However, approximately the same amount of the total As (i.e., 0.12 %) was leached from columns 7, 8, and 9 regardless of the flow rate of the leaching fluids (Figure 4.37).

Figures 4.38 and 4.39 show the leaching of As from columns 10 and 11 after they were subjected to SAR at a flow rate of 4 and 10 ml/min., respectively. The results observed in columns 10 and 11 were similar to those observed in columns 7 and 9 (Figures 4.36 and 4.37). Approximately 0.12 % of the total As was leached from each of columns 10 and 11 (Figure 4.39). This suggested that the increase of flow rate using SAR resulted in no significant difference in the leaching of Cr.

#### Effect of Column Height

Figures 4.40 and 4.41 show the leaching of As from columns 1, 2, 5, and 6 at different heights when they were subjected to DDW at a flow rate of 7 ml/min. Concentrations of As in the leachate from columns 1, 2, 5, and 6 were below the EPA toxicity standard of 5,000 g/L and decreased as a function of time (Figure 4.40). With the exception of some initial samples collected within 30 minutes, most of the leachate had As concentrations below the drinking water standard of 50 g/L. The leaching of As from 15.2-cm columns and 45.7-cm columns leveled off within 2 and 4 hours, respectively. The 45.7-cm columns had higher concentrations of As in the leachate than the 15.2-cm columns. Approximately 0.06 %, 0.04 %, 0.08 %, and 0.06 % of the total As were leached from columns 1, 2, 5, and 6, respectively (Figure 4.41).

Figures 4.42 and 4.43 show the leaching of As from columns 10 and 12 at different heights when subjected to SAR at a flow rate of 4 ml/min. Concentrations of As in the leachate from columns 10 and 12 were below the

EPA toxicity standard of 5,000 g/L and decreased as a function of time (Figure 4.42). The percentages of the total As leached from columns 10 and 12 were 0.12 % and 0.08 %, respectively (Figure 4.43).

#### Summary

Concentrations of As in the leachate from ash columns were below the EPA toxicity standard of 5,000 g/L and decreased as a function of time. Other than some samples collected within the initial 30 minutes of testing, most of the leachate had As concentrations below the drinking water standard of 50 g/L. The SAR solution had no significant effect on the leaching of As when compared with DDW. Approximately the same percentage of As was leached from the columns regardless of the flow rate of leaching fluids and the column height.

#### 4.8.4.b RDF Ash Results

Figures (4.44), (4.45) and (4.46) show the leaching of As from 6- and 18-inch RDF and MB ash columns when they were subjected to DDW or SAR. The trend in As leaching for both MB and RDF ashes was very similar; however, unlike Ca, RDF ash had less leachable As than MB ash. No significant difference in the leaching of As was found between RDF and MB ash when SAR was used in the leaching test.

#### 4.8.5 Leaching of Chromium

##### 4.8.5.a MB Ash Results

#### Effect of pH

Figures 4.47 and 4.48 show the leaching of Cr from columns 9 and 11 when they were subjected to DDW and SAR, respectively. Concentrations of Cr in the leachate from columns 9 and 11 were below the EPA toxicity standard of 5,000 g/L and decreased as a function of time. They both leveled off to minimum values within 3 hours (Figure 4.47). The release patterns of Cr from columns 9 and 11 were very similar, suggesting that an SAR solution has little more significant effect on the leaching of Cr than DDW. The total percentages of Cr leached from columns 9 and 11 during the 8-hour test were 0.05 % and 0.07 %, respectively (Figure 4.48).

### Effect of Flow Rate

Figures 4.49 and 4.50 show the leaching of Cr from columns 7, 8, and 9 when they were subjected to DDW at a flow rate of 4, 7, and 10 ml/min., respectively. Concentrations of Cr in the leachate from columns 7, 8, and 9 were below the EPA toxicity standard of 5,000 g/L and decreased as a function of time. All three leveled off to minimum values after 2 hours (Figure 4.49). Approximately the same amount of the total Cr (i.e., 0.04 %) was leached from each of the three columns regardless of the flow rate of leaching fluids (Figures 4.50).

Figures 4.51 and 4.52 show the leaching of Cr from columns 10 and 11 when subjected to SAR at a flow rate of 4 and 10 ml/min., respectively. The results observed in columns 10 and 11 were similar to those observed in columns 7, 8, and 9 (Figures 4.49 and 4.50). This once again evidenced that SAR and DDW resulted in no significant difference in the leaching of Cr. The percentages of the total Cr leached from columns 10 and 11 were 0.06 % and 0.07 %, respectively (Figure 4.52).

### Effect of Column Height

Figures 4.53 and 4.54 show the leaching of Cr from different heights. Columns 1, 2, 5, and 6 were subjected to DDW at a flow rate of 7 ml/min. Concentrations of Cr in the leachate were below the EPA toxicity standard of 5,000 g/L and decreased as a function of time. The leaching of Cr from columns 1 and 2 leveled off within 2 and 5 hours, respectively (Figures 4.53). Concentrations of Cr in the leachate from columns 5 and 6 remained at a lower level than those from columns 1 and 2, respectively. Approximately 0.03 %, 0.03 %, 0.02 %, and 0.02 % of the total Cr were leached from columns 1, 2, 5, and 6, respectively, after the 8-hour tests (Figure 4.54). This suggested that a change of column height resulted in no significant difference in the leaching of Cr.

Figures 4.55 and 4.56 show the leaching of Cr from columns 10 and 12 at different heights subjected to SAR at a flow rate of 4 ml/min. Concentrations of Cr in the leachate from columns 10 and 12 were below the EPA toxicity standard of 5,000 g/L and decreased as a function of time (Figure 4.55). The percentages of the total Cr leached from columns 10 and 12 were 0.06 % and 0.05 %, respectively (Figure 4.56).

The observed difference in the leaching of Cr between columns 1 and 5 and between columns 2 and 6 might be the result of sampling from different

batches of ash with different initial water content. No attempt was made to compare a difference, if there was one, in the batches or water content in the study. The water content of the ash for columns 1 and 2 was 10 %, and 16 % for columns 5 and 6. MWC bottom ash aged under different water content for a period of time could result in the formation of different minerals (Shieh, 1994), and thus might affect the leaching characteristics of the ash.

### Summary

Concentrations of Cr in the leachate from ash columns were below the EPA toxicity standard of 5,000 g/L. The SAR and DDW solution had no different effects on the leaching of Cr. Regardless of the flow rate of the leaching fluid and the height of the ash column, approximately the same amount of the total Cr would leach from each ash column during the testing period.

#### 4.8.5.b RDF Ash Results

Figures (4.57), (4.58) and (4.59) show the leaching of Cr from 6- and 18-inch RDF and MB ash columns when they were subjected to DDW or SAR. The trend of Cr leaching was very similar for both MB and RDF ashes. When SAR was used, more leachable Cr was released though the amount of leaching decreased with time.

#### 4.8.6 Leaching of Lead

##### 4.8.6.a MB Ash Results

### Effect of pH

Figures 4.60 and 4.61 show the leaching of Pb from columns 9 and 11 when subjected to DDW and SAR, respectively. Concentrations of Pb in the leachate from columns 9 and 11 were below the EPA toxicity standard of 5,000 g/L and decreased as a function of time (Figure 4.60). Columns 9 and 11 had a similar release pattern of Pb with respect to time, suggesting that an SAR solution had no significant effect on the leaching of Pb, when compared with DDW. Approximately 0.006 % and 0.004 % were leached from columns 9 and 11, respectively, during the 8-hour testing period (Figure 4.61).

### Effect of Flow Rate

Figures 4.62 and 4.63 show the leaching of Pb from columns 7, 8, and 9 at a flow rate of 4, 7, and 10 ml/min., respectively. Concentrations of Pb in the leachate from columns 7, 8, and 9 were all below the EPA toxicity standard of 5,000 g/L and decreased as a function of time (Figure 4.62). Regardless of the flow rate of the three columns, approximately the same percentage of the total Pb (i.e., 0.005 %) was leached from each of the three columns (Figure 4.63). This suggested that the leaching of Pb was not affected by the flow rate of DDW.

Figures 4.64 and 4.65 show the leaching of Pb from columns 10 and 11 when they were subjected to SAR at a flow rate of 4 and 10 ml/min., respectively. The results observed in columns 10 and 11 were similar to those observed in columns 7 and 9 (Figures 4.62 and 4.63). Concentrations of Pb in the leachate from columns 10 and 11 were below the EPA toxicity standard of 5,000 g/L and decreased as a function of time (Figure 4.64). Approximately 0.003 %, and 0.004 % of the total Pb were leached from columns 10 and 11, respectively (Figure 4.65). This suggested that the leaching of Pb was not affected by the flow rate of SAR.

### Effect of Column Height

Figures 4.66 and 4.67 show the leaching of Pb from different heights. Columns 1, 2, 5, and 6 were subjected to DDW at a flow rate of 7 ml/min. Concentrations of Pb in the leachate from columns 1, 2, 5, and 6 were all below the EPA toxicity standard of 5,000 g/L. Leachate concentrations from columns 1, 5, and 6 decreased as a function of time, and those from column 2 tended to increase at the beginning of leaching to a maximum and then decreased as a function of time (Figure 4.66). The leaching of Pb from columns 1 and 2 leveled off to minimum values within 2 and 6 hours, respectively. Concentrations of Pb in the leachate in columns 5 and 6 remained at lower levels than those in columns 1 and 2. Approximately 0.016 %, 0.014 %, 0.008 %, and 0.005 % of the total Pb were leached from columns 1, 2, 5, and 6 respectively after the 8-hour test (Figure 4.67).

Figures 4.68 and 4.69 show the leaching of Pb from different heights. Columns 10 and 12 were subjected to SAR at a flow rate of 4 ml/min. Concentrations of Pb in the leachate in columns 10 and 12 were below the EPA toxicity standard (Figure 4.68). Approximately 0.003 % and 0.002 % of the total Pb were leached from columns 10 and 12, respectively (Figure 4.69).

The observed difference in the leaching of Pb between columns 1 and 5 and between columns 2 and 6 might be the result of sampling from different batches of ash with different initial water content. No attempt was made to compare a difference, if there was one, in the batches or water content in the study. The water content of the ash for columns 1 and 2 was 10 %, and 16 % for columns 5 and 6. MWC bottom ash, if aged under different water content for a period of time, could result in the formation of different minerals (Shieh, 1994), and thus affect the leaching characteristics of the ash.

#### Summary

Concentrations of Pb in the leachate from ash columns subjected to DDW and SAR were below the EPA toxicity standard of 5,000 g/L and decreased as a function of time. The SAR solution and DDW had no different effects on the leaching of Pb. Approximately the same amount of Pb was leached from the ash columns regardless of the flow rate of the leaching fluid and the height of ash column.

#### 4.8.6.b RDF Ash Results

Figures (4.70), (4.71) and (4.72) show the leaching of Pb from 6- and 18-inch RDF and MB ash columns when they were subjected to DDW or SAR. The results showed that MB ash had more leachable Pb than RDF ash. The leaching of Pb from both ashes was found to decrease with time. DDW and SAR caused no significant difference in Pb leaching.

#### 4.8.7 Leaching of Cadmium and Silver

Most of the leachate had Cd concentrations that were below the detection limit (1 g/L) of the analytical methods. All the leachate had Ag concentrations below the detection limit (1 g/L) of the analytical methods, and hence comparisons of the results are not presented (i.e., the effect of various controlling factors, pH, flow rate, and column height, on the leaching of Cd and Ag remained unidentified since most of the leachate had concentrations below the detection limit). Since very little Ag and Cd were detectable in the leachate, it is suggested that the Ag and Cd in the ash column did not pose a toxicity threat to the environment.

## **4.9 Data Base**

A data base of MB and RDF bottom ash properties is presented in Table 4.7.

## **4.10 Availability, Costs and Regulatory Aspects**

This investigation was predicated on the availability of ash and regulatory issues in place at the time of the study. At the onset of the study, bottom ash was provided and available as a fill at no charge. The only costs of using ash as a source of fill would be the trommel costs to insure correct sizing and the hauling costs to the specific site. MWC facilities would also realize a potential savings of at least \$20/ ton by not landfilling the ash.

Changes to ash management practices used by MWC facility operators have occurred as a result of US Supreme Court ruling 92-1639 of May 1994. This ruling does not categorize MWC ash as hazardous waste, however it instructs municipalities to test MWC ash following TCLP for compliance with Federal EPA standards. As a result of this ruling, MWC facility operators have been formulating operational practices and are reluctant to separate ash into bottom and fly ash components. They have taken the viewpoint that combined ash currently meets the specified EPA standards. If the ash is separated, then the fly ash may not meet EPA standards and would require treatment as a hazardous waste. As a result of the various changes in ash management practice, it is currently impossible to make a reliable estimate of the availability and costs of bottom ash.

The State of Florida has enacted legislation limiting the amount of metals in consumer goods and is conducting trial household battery recycling programs. These policy changes will reduce the amounts of metals available to the MSW ash stream and it may be speculated that MWC ash will become "cleaner". This statement suggests that ash produced by MWC facilities could be considered for use as a highway fill material in the future.

Table 4.1 Summary of WTE surveys completed by July 15, 1993

FACILITY NAME	LOCATION	PLANT OPERATOR	TYPE [1]	BURN RATE (T/day)	PRODUCTION RATE OF ASH (T/day)			Additional Treatment	Current use of Bottom ash
					Fly ash	Bottom ash	Scrubber ash		
Bay Co. RRF	Bay Co.	Westinghouse En. Sys.	MB	510	SNR	SNR	SNR	SNR	SNR
N. Broward Co. RRF	Broward Co.	Wheelabrator	MB	2250	SNR	SNR	SNR	SNR	SNR
S. Broward Co. RRF	Broward Co.	Wheelabrator	MB	2250	SNR	SNR	SNR	SNR	SNR
Hillsborough Co. ERF	Hillsborough Co.	Ogden Martin Systems	MB	1200	SNR	SNR	SNR	SNR	SNR
Metro Key West Co. RRF	Monroe Co.	Montenay	MB	150	SNR	SNR	SNR	SNR	SNR
Lake Co. RRF	Lake Co.	Ogden Martin Systems	MB	528	SNR	SNR	SNR	SNR	SNR
Mcintosh Power Plant RRF	Polk Co.	City of Lakeland	RDF	300	SNR	SNR	SNR	SNR	SNR
McKay Bay REP	Hillsborough Co.	Wheelabrator	MB	1000	SNR	SNR	SNR	SNR	SNR
Miami International Airp. I.	Dade Co.	Dade Co.	MB	60	SNR	SNR	SNR	SNR	SNR
Pasco Co. RRF	Pasco Co.	Ogden Martin Systems	MB	750	177.6	combined	N/A	A	Landfill
Ogden Martin of Lake	Lake Co.	Ogden Martin Systems	MB	528	6.5	97.5	26	none	N/A
Dade Co. RRF	Dade Co.	Montenay Power Corp.	RDF	2100	60	318	0	none	Landfill
North Co. RRF	Palm Beach Co.	Babcock & Wilcox	RDF	2000	100	200	0	none	none
Mayport Naval St.	Duval Co.	Global Associates	RDF	36	0.125	5	0	A	Landfill
Pinellas Co. RRF	Pinellas Co.	Wheelabrator	MB	3000	75	675	0	***	Landfill
				TOTAL	16662				

Waste-to-energy facilities under construction

FACILITY NAME	LOCATION	PLANT OPERATOR	TYPE [1]	BURN RATE (T/day)	PRODUCTION RATE OF ASH (T/day)			Additional Treatment	Current use of Bottom ash
					Fly ash	Bottom ash	Scrubber ash		
Lee County Solid Waste RRF	Lee Co.	Ogden Martin Systems	MB	1200	SNR	SNR	SNR	SNR	SNR
Ridge Generating Station	Polk Co.	Wheelabrator	TDF	1000	SNR	SNR	SNR	SNR	SNR
				TOTAL	2200				

N/A = Not Available

SNR = Survey not Returned

[1] MB = Mass Burn RDF = Resource Derived Fuel

A = Removal of Metals

Table 4.2 Summary of physical properties for MWC bottom ash

Property	Mass Burn	Refuse-Derived-Fuel
Moisture Content Range as Delivered %	11-19	16-26
Grain Diameter at 10% passing (mm)	0.33	0.17
Grain Diameter at 60% passing (mm)	4.1	1.8
Grain Diameter at 30% passing (mm)	1.5	0.55
Uniformity Coefficient, Cu	12.4	10.6
Coefficient of Curvature, Cc	1.66	0.99
Specific Gravity (Passing #4 sieve)	2.79	2.55
Specific Gravity (Passing #8 sieve)	2.72	2.45
AASHTO Classification	A-1-a	A-1-a
USCS Classification	SW	SW
ASTM Aggregate Classification	# 89	# 89

Table 4.3 Moisture-density results for MWC bottom ash

Bottom Ash Source	Compactive Energy ASTM	Passing #4 sieve		
		OMC (%)	Maximum Dry Density	
			(pcf)	(kN/m <sup>3</sup> )
MB	D-698	15.5	114.8	18.1
	D-1557	12.6	121.7	19.1
RDF	D-698	18.3	98.0	15.4
	D-1557	17.6	103.8	16.3
Bottom Ash Source	Compactive Energy ASTM	Passing #8 sieve		
		OMC (%)	Maximum Dry Density	
			(pcf)	(kN/m <sup>3</sup> )
MB	D-698	17.8	104.9	16.5
	D-1557	16.4	112.9	17.8
RDF	D-698	20.8	82.0	12.9
	D-1557	19.3	98.0	15.4

OMC = Optimum Moisture Content

Table 4.4 CD triaxial shear results for MWC bottom ash

Ash Type	Relative Compaction (%)	Dry Unit Weight as Tested		Compactive Energy ASTM	Friction Angle (°)	Cohesion	
		(pcf)	(kN/m <sup>3</sup> )			(psi)	(kPa)
MB < #4	100	115.0	18.1	D-698	50	4	27.6
	95	108.4	17.0	D-698	48	4	27.6
	90	105.2	16.5	D-698	44	4	27.6
MB < #4	100	NA	NA	D-1557	NA	NA	NA
	95	NA	NA	D-1557	NA	NA	NA
	90	108.7	17.1	D-1557	47	2	13.8
RDF < #4	100	97.8	15.4	D-698	45	4	27.6
	95	94.0	14.8	D-698	42	4	27.6
	90	87.7	13.8	D-698	38	2	13.8
RDF < #4	100	102.7	16.1	D-1557	47	5	34.5
	95	97.3	15.3	D-1557	45	3	20.7
	90	93.3	14.6	D-1557	41	3	20.7
MB < #8	100	106.3	16.7	D-698	42	2	13.8
	95	101.5	15.9	D-698	39	2	13.8
	90	93.3	14.6	D-698	30	2	13.8
MB < #8	100	112.5	17.7	D-1557	47	3	20.7
	95	107.9	16.9	D-1557	43	3	20.7
	90	102.3	16.1	D-1557	40	2	13.8
RDF < #8	100	82.6	13.0	D-698	41	2	13.8
	95	78.1	12.3	D-698	35	2	13.8
	90	74.3	11.7	D-698	24	2	13.8
RDF < #8	100	99.5	15.6	D-1557	46	3	20.7
	95	93.4	14.7	D-1557	42	3	20.7
	90	89.2	14.0	D-1557	39	2	13.8

NA - Not Attainable

**Table 4.5 Selected elemental concentrations of standard reference material 1633a coal fly ash from NIST and percent recovery using the technique of HF-H<sub>3</sub>BO<sub>3</sub> digestion**

Element	Certified Value	Measured Value	Recovery (%) <sup>a</sup>
Ca (%)	1.11 ± 0.01	1.07 ± 0.02	96
Ag (g/g)	NA <sup>b</sup>	NA	NA
As (g/g)	145 ± 15	128 ± 19	88
Cd (g/g)	1.00 ± 0.15	1.03 ± 0.10	100
Cr (g/g)	196 ± 6	184 ± 4	94
Pb (g/g)	72.4 ± 0.4	71.4 ± 1.1	99

<sup>a</sup> Percent recovery was considered as 100 % if the measured mean value was within the range of the certified mean ± standard deviation. Where measured mean value was beyond the range of certified value, percent recovery was calculated by dividing the measured mean value by the certified mean value.

<sup>b</sup> No certified value available.

Table 4.6 Comparison of selected elemental concentrations (mg/kg) of bottom ash

Element	This Study	EPA (1990)	Soil (Brooks, 1978)
Ag	1.3 ± 0.2	ND <sup>a</sup> - 38	1
As	18.8 ± 1.9	1.3 - 24.6	5
Ca	53,000 ± 3,000	5,900 - 69,500	63,000
Cd	7.2 ± 1.2	1.1 - 46	0.5
Cr	254 ± 56	13 - 520	200
Pb	2590 ± 950	110 - 5,000	10

<sup>a</sup> Not detectable.

Table 4.7 Data base of MB and RDF bottom ash properties

Property	Mass Burn	Comments	Refuse Derived Fuel	Comments
Moisture Content (mc)	11%-19 %	as delivered	16%-26%	as delivered
Physical Composition		higher % of metals		lower % of metals
Specific Gravity	2.72-2.79		2.45-2.55	
Classification	A-1-a (0) SW # 89	well-graded granular material	A-1-a (0) SW # 89	same as MB
Absorption	9%-12%	greater than conventional fill	11%	greater than conventional fill
Mineralogy		predominance of Al, Si & Ca		same as MB
Pozzolanic Activity Index	135	more active than coal fly ash	65	less active than coal fly ash
Loss on Ignition	4.4%-5%	similar to coal fly ash	5%	similar to coal fly ash
Bulk Rodded Unit Weight	77 pcf -81 pcf	lower than conventional fill	69 pcf	lower than conventional fill
Moisture-Density	(pcf) @ (mc %)	higher unit weight	(pcf) @ (mc %)	lower unit weight
ASTM D-698 < #4 sieve	114.8 @ 15.5	lower OMC	98.0 @ 18.3	higher OMC
ASTM D-698 < #8 sieve	104.9 @ 17.8	increase of compaction energy and grain size will increase dry density	82.0 @ 20.8	
ASTM D-1557 < #4 sieve	121.7 @ 12.6		103.8 @ 17.6	
ASTM D-1557 < #8 sieve	112.9 @ 16.4	(.0157 kN/m <sup>3</sup> = 1 pcf)	98 @ 19.3	

Table 4.7 Data base of MB and RDF bottom ash properties,continued

Property	Mass Burn	Comments	Refuse Derived Fuel	Comments
Permeability wet of optimum dry of optimum	cm/sec 1.00E-02 1.00E-05	very sensitive to compaction moisture content	cm/sec 1.00E-02 1.00E-05	same as MB
Free Shrink Swell soaked dry soaked then dry	<.1% 2.3%-3.6% .50%-1.50%	negligible volume change axial expansion axial expansion reduced		not evaluated
Unconf. Compressive Strength	10-70 psi 70-490 kPa	decrease with increase in moisture content increase with compaction energy increase with age	10-50 psi 70-350 kPa	decrease with increase in moisture content increase with compaction energy
Friction Angle	degrees 30-50	increase with density increase with grain size	degrees 24-46	increase with density increase with grain size
Elastic Modulus	500-5000 psi 3,500-35,000 kPa		500-5000 psi 3,500-35,000 kPa	
CBR LBR	100-200 100-185	very sensitive to compaction moisture content	25-85 25-100	same as MB same as MB

Table 4.7 Data base of MB and RDF bottom ash properties, continued

Property	Mass Burn	Comments	Refuse Derived Fuel	Comments
Leaching: Ag, Cd	BDL	below detection limits	BDL	same as MB
Leaching: Cr, Pb	< EPA TS	EPA Toxicity Std (5000µg/l)	< EPA TS	same as MB
Leaching: Ca		No available standard		
Leaching: As	< EPA TS < DWS	EPA Toxicity Std (5000µg/l) Drinking Water Std (50 µg/l)	< EPA TS < DWS	same as MB same as MB

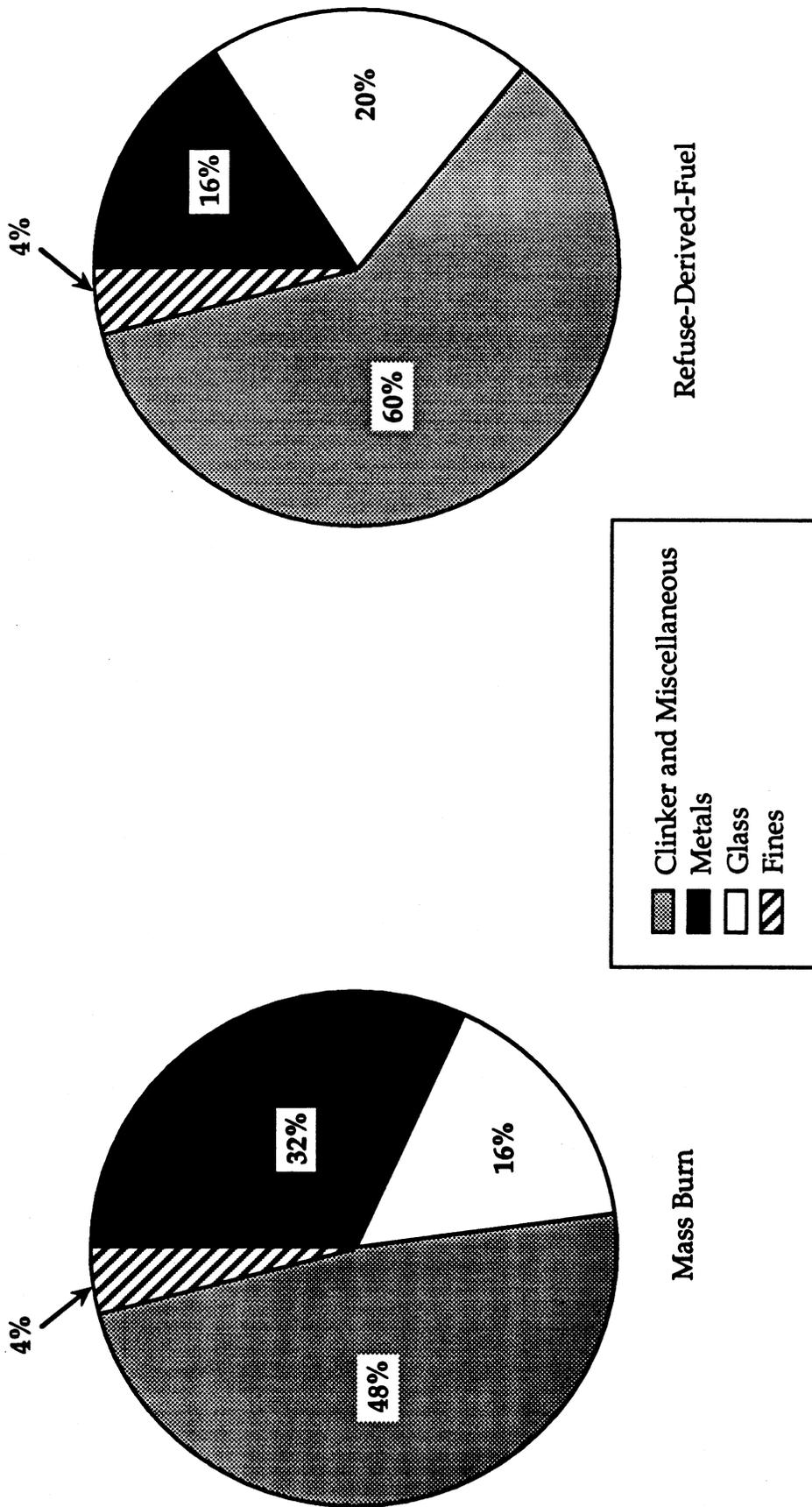


Figure 4.1 Physical composition for mass-burn and refuse-derived-fuel MWC bottom ash

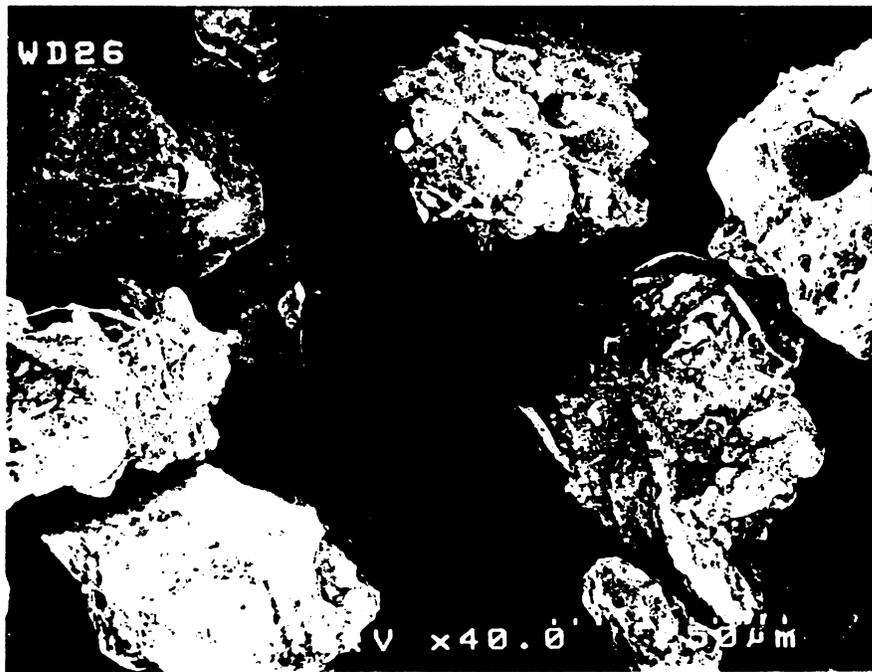


Figure 4.2 Scanning electron micrograph of MWC bottom ash prior to aging

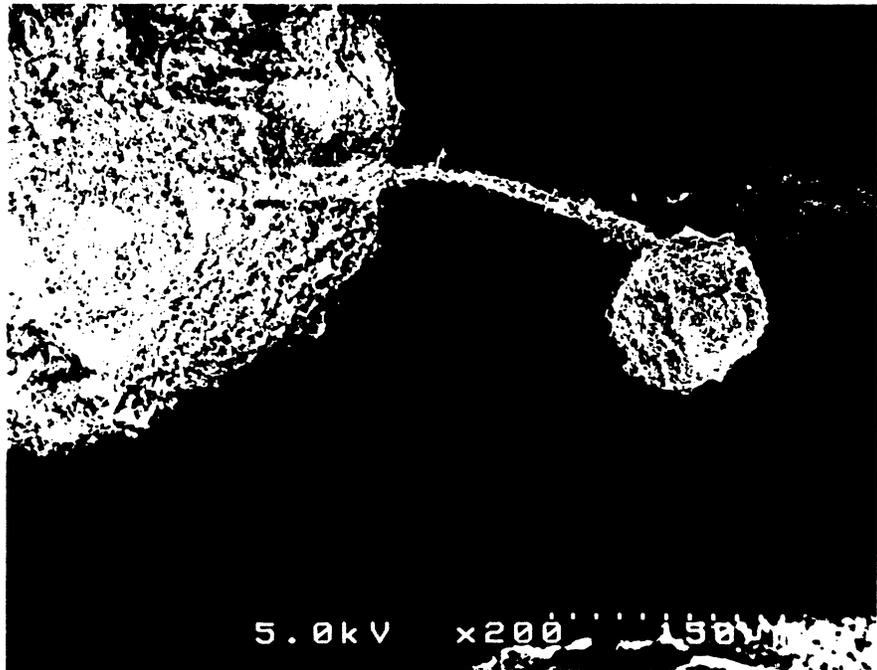


Figure 4.3 Scanning electron micrograph of mineral fibers linking ash particles together

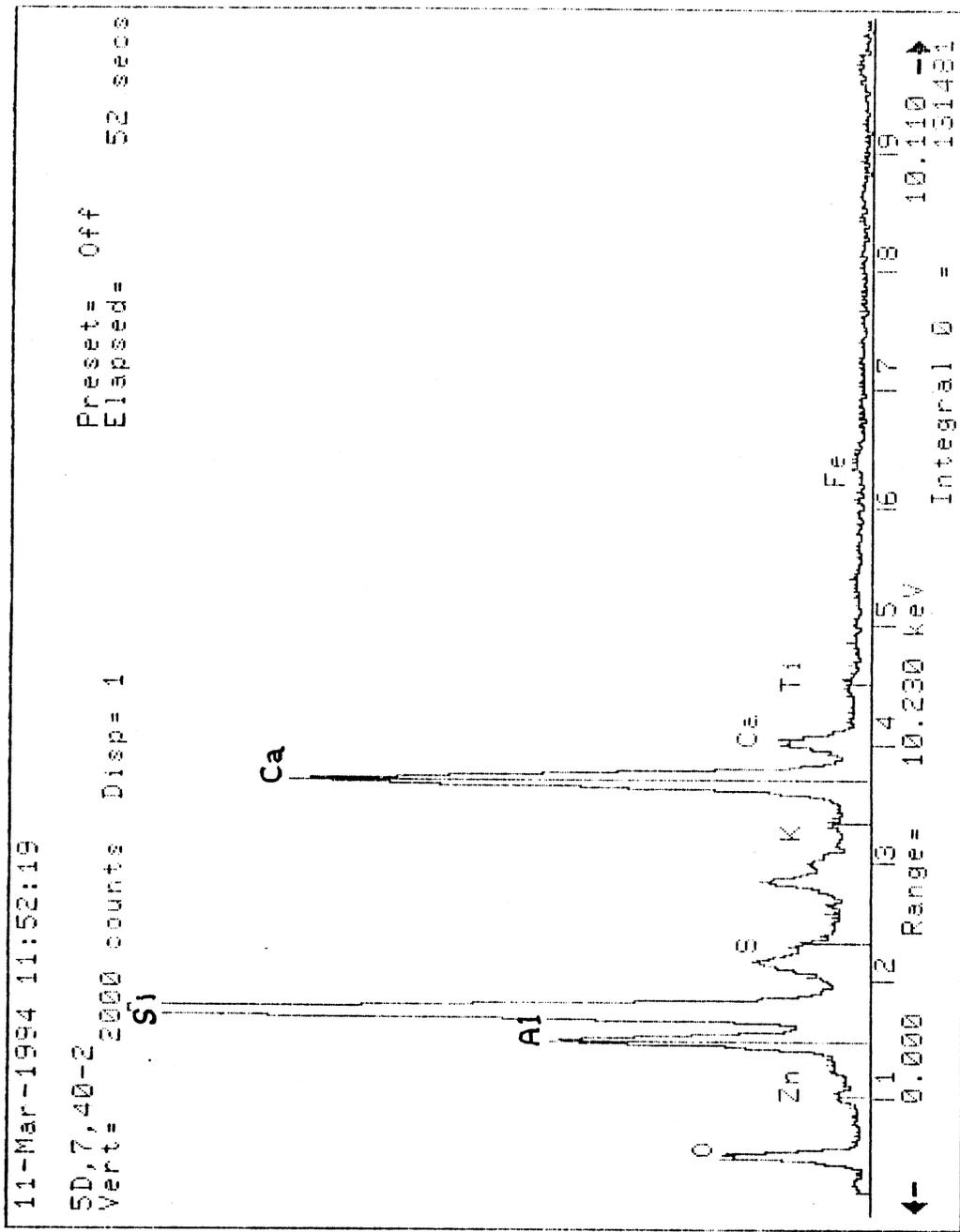


Figure 4.4 Enrichment of Al, Ca, and Si on the material fiber found in the MWC bottom ash at 40°C with 33% saturation (7% water content) for 5-day

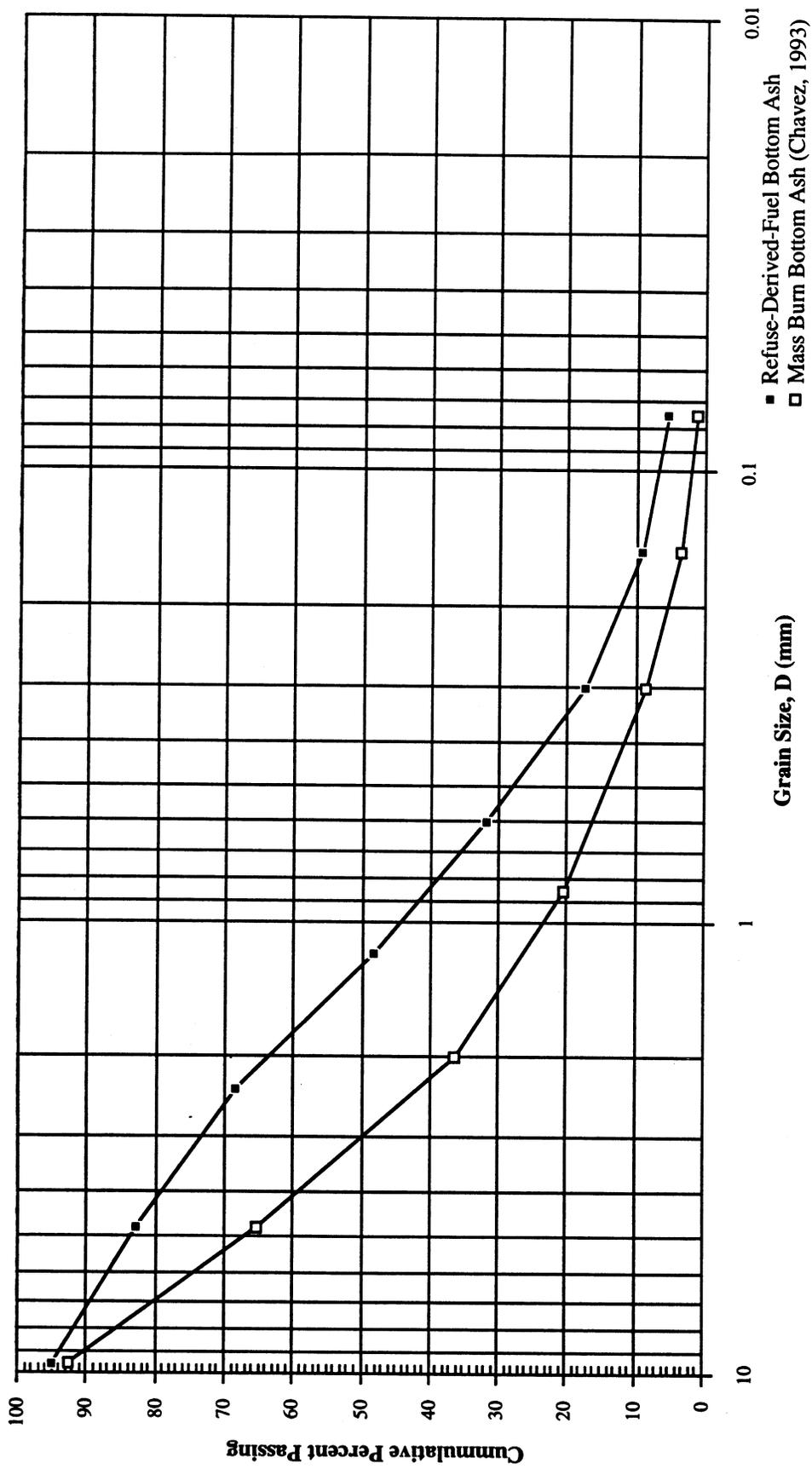


Figure 4.5 Grain size distribution for MB and RDF bottom ash

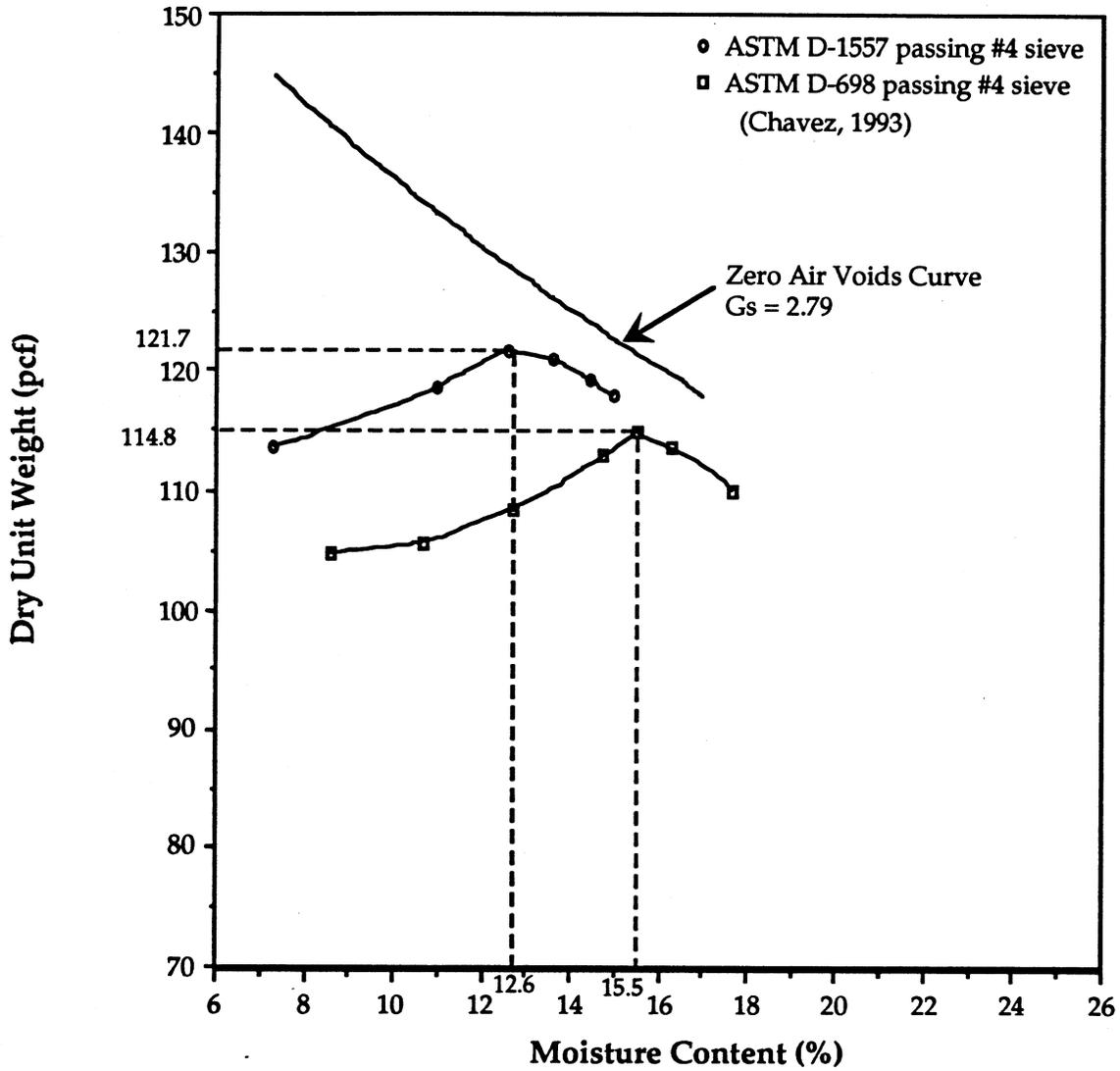


Figure 4.6 Moisture density relationship for MB bottom ash passing the #4 sieve (1 pcf = 0.157 kN/m<sup>3</sup>)

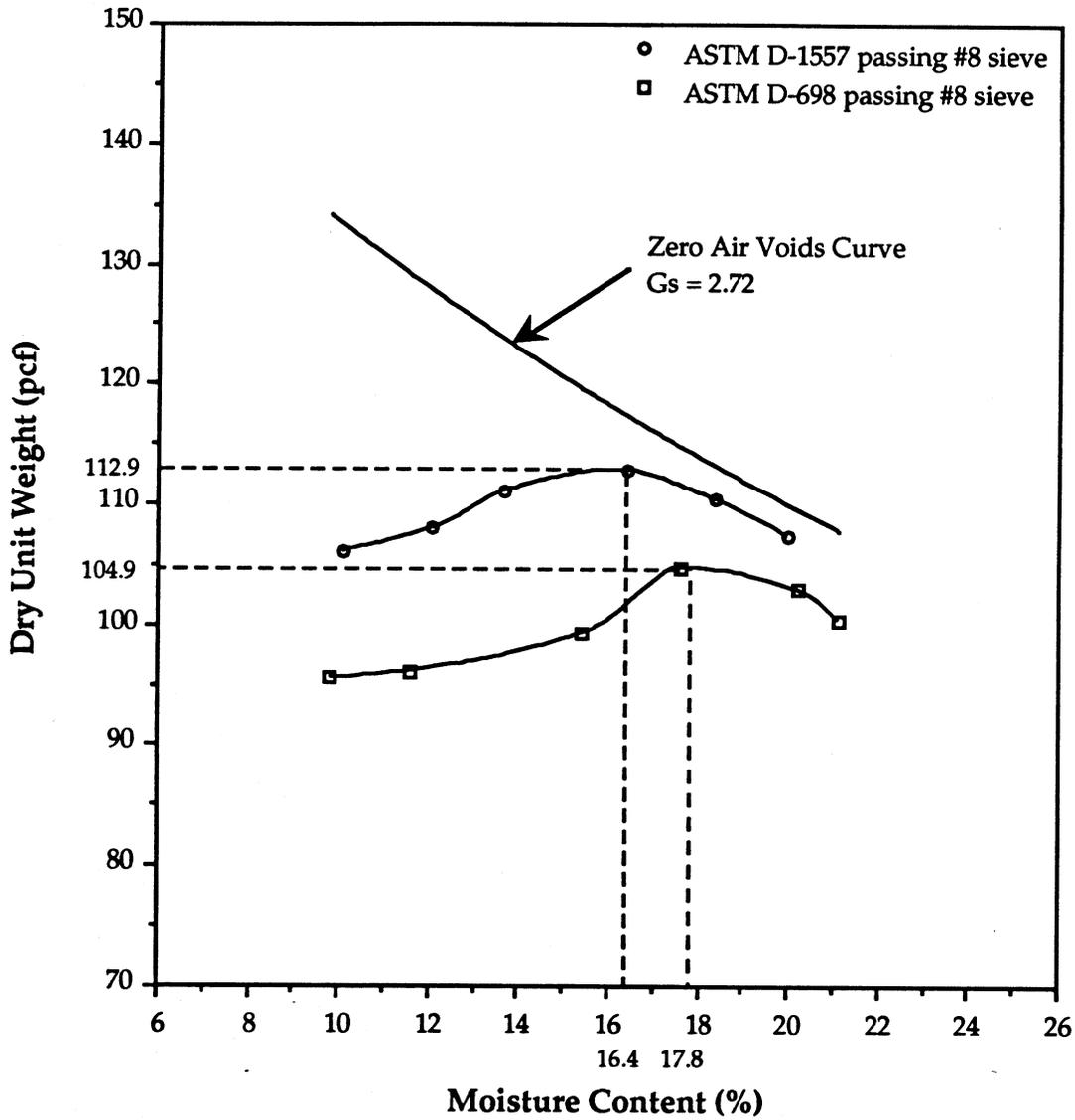


Figure 4.7 Moisture density relationship for MB bottom ash passing the #8 sieve (1 pcf = 0.157 kN/m<sup>3</sup>)

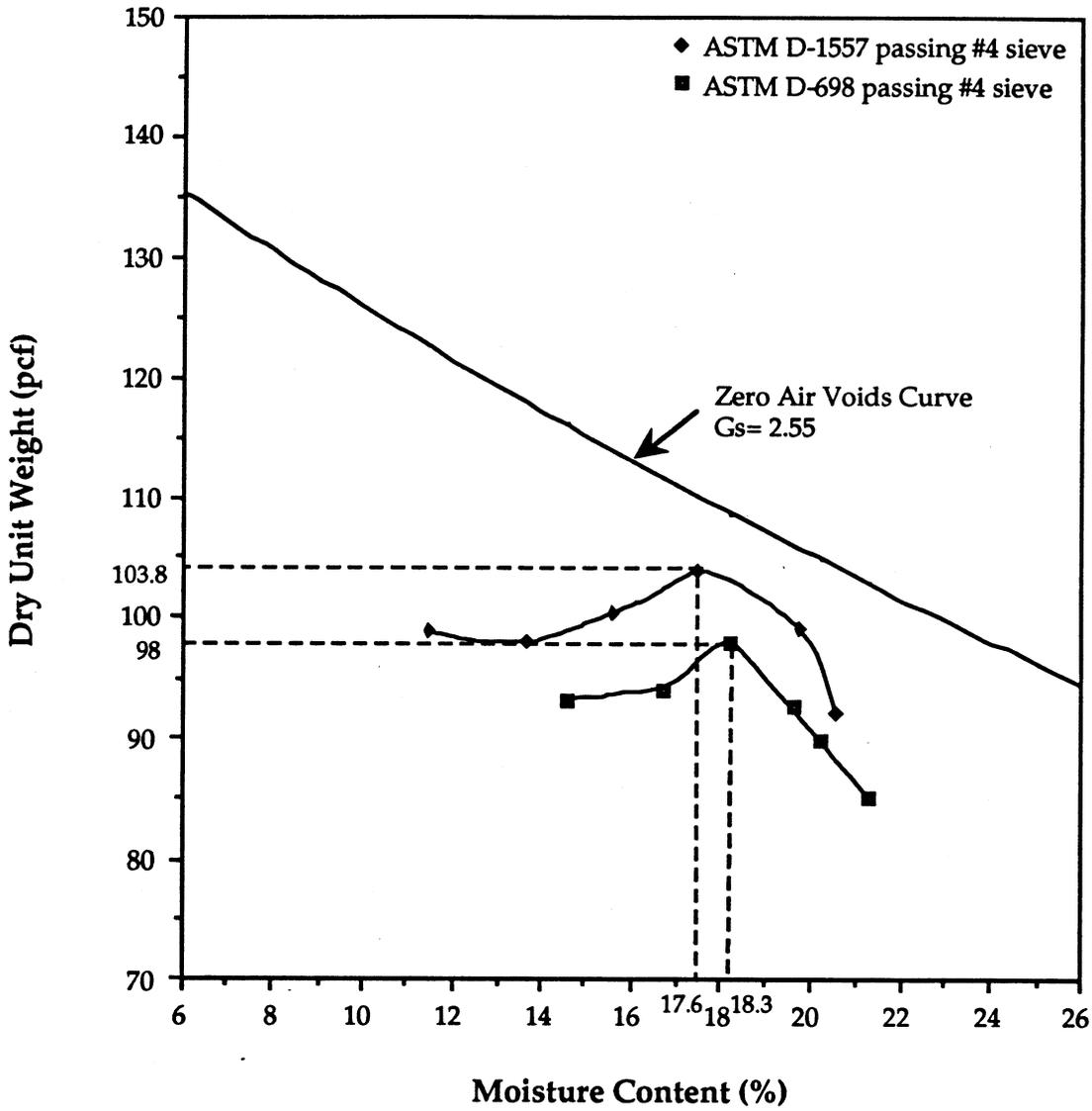


Figure 4.8 Moisture density relationship for RDF bottom ash passing the #4 sieve (1 pcf = 0.157 kN/m<sup>3</sup>)

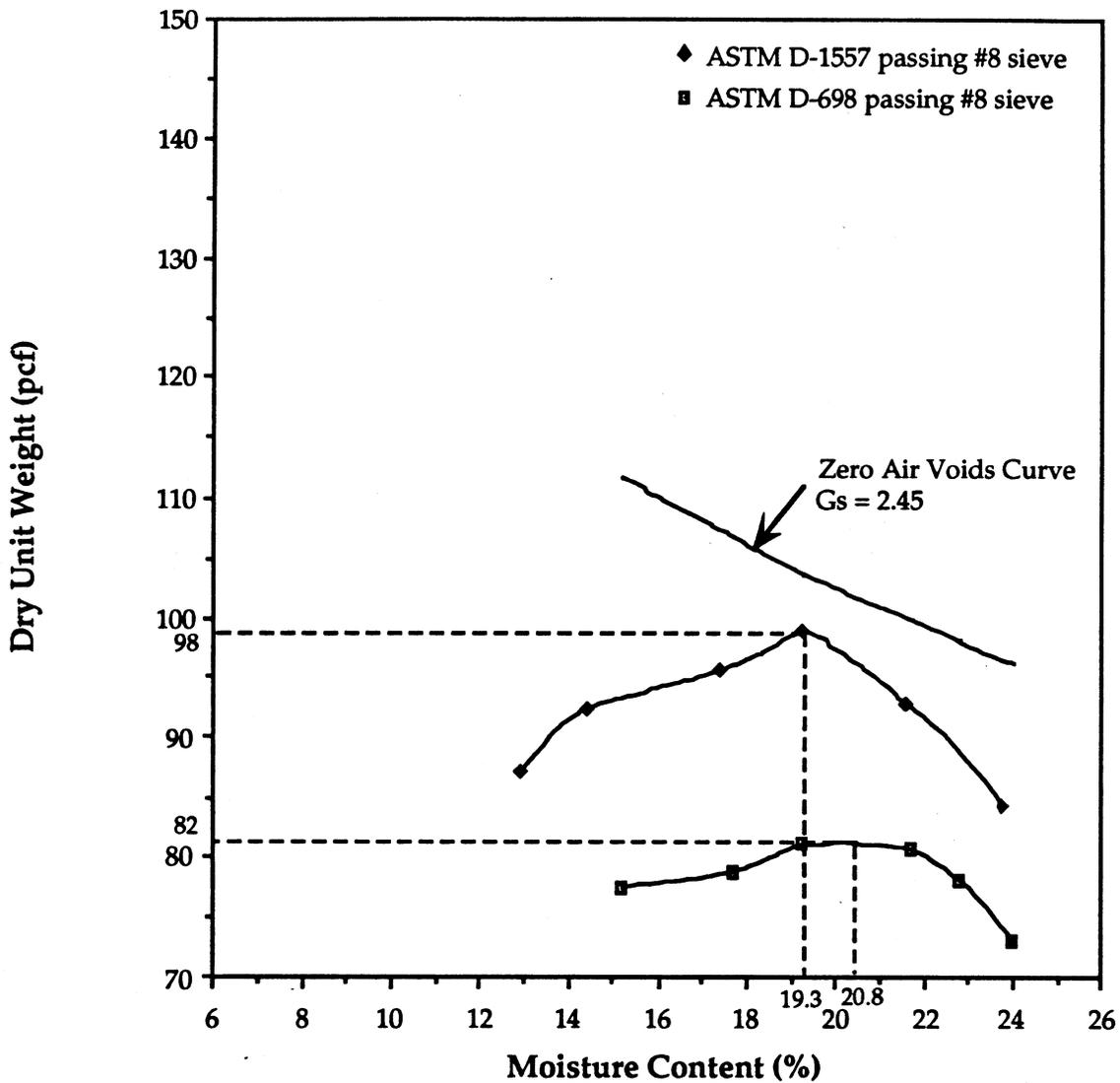


Figure 4.9 Moisture density relationship for RDF bottom ash passing #8 sieve (1 pcf = 0.157 kN/m<sup>3</sup>)

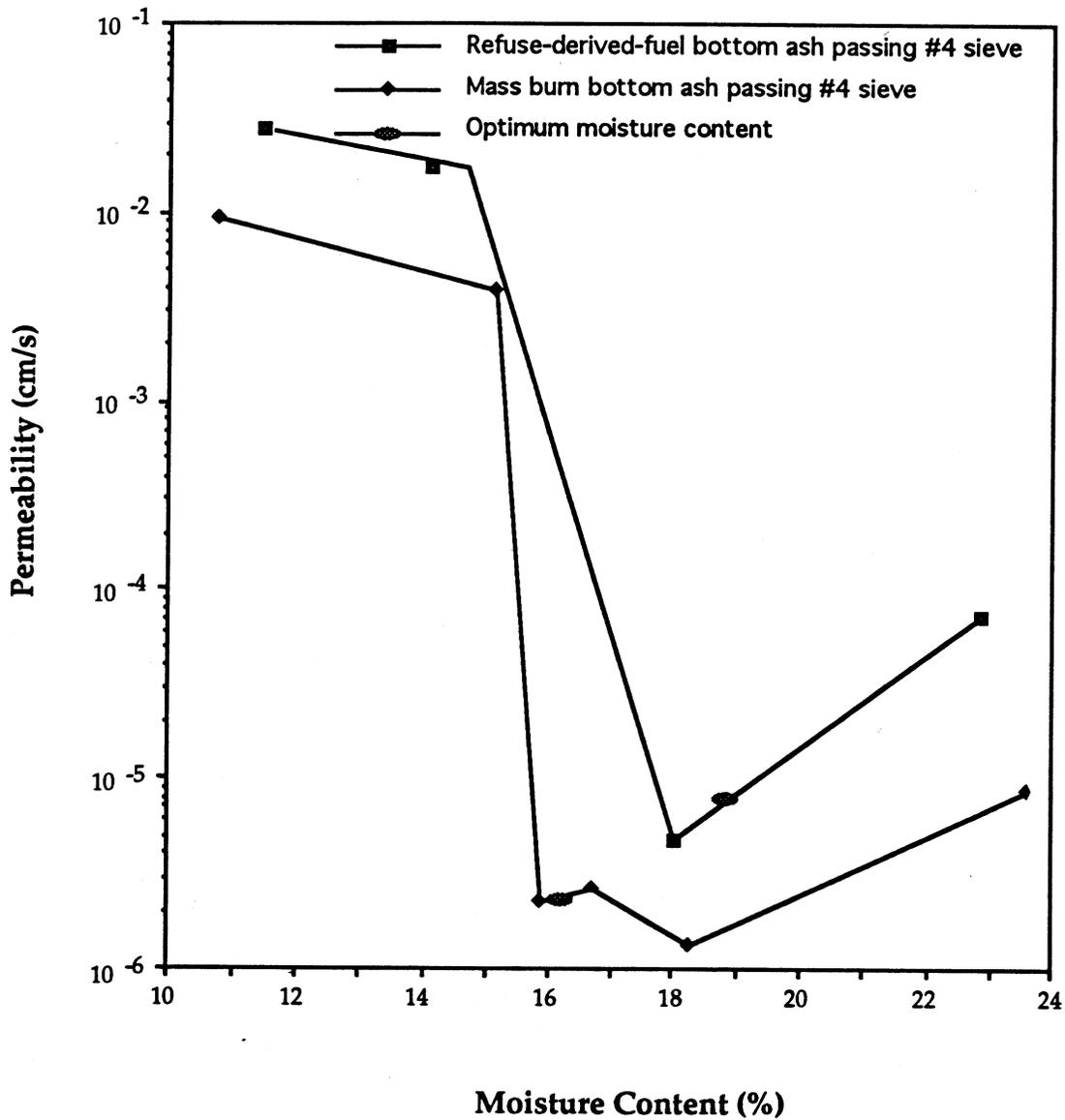


Figure 4.10 Permeability versus moisture content for MB bottom ash passing #4 sieve compacted following ASTM D-698

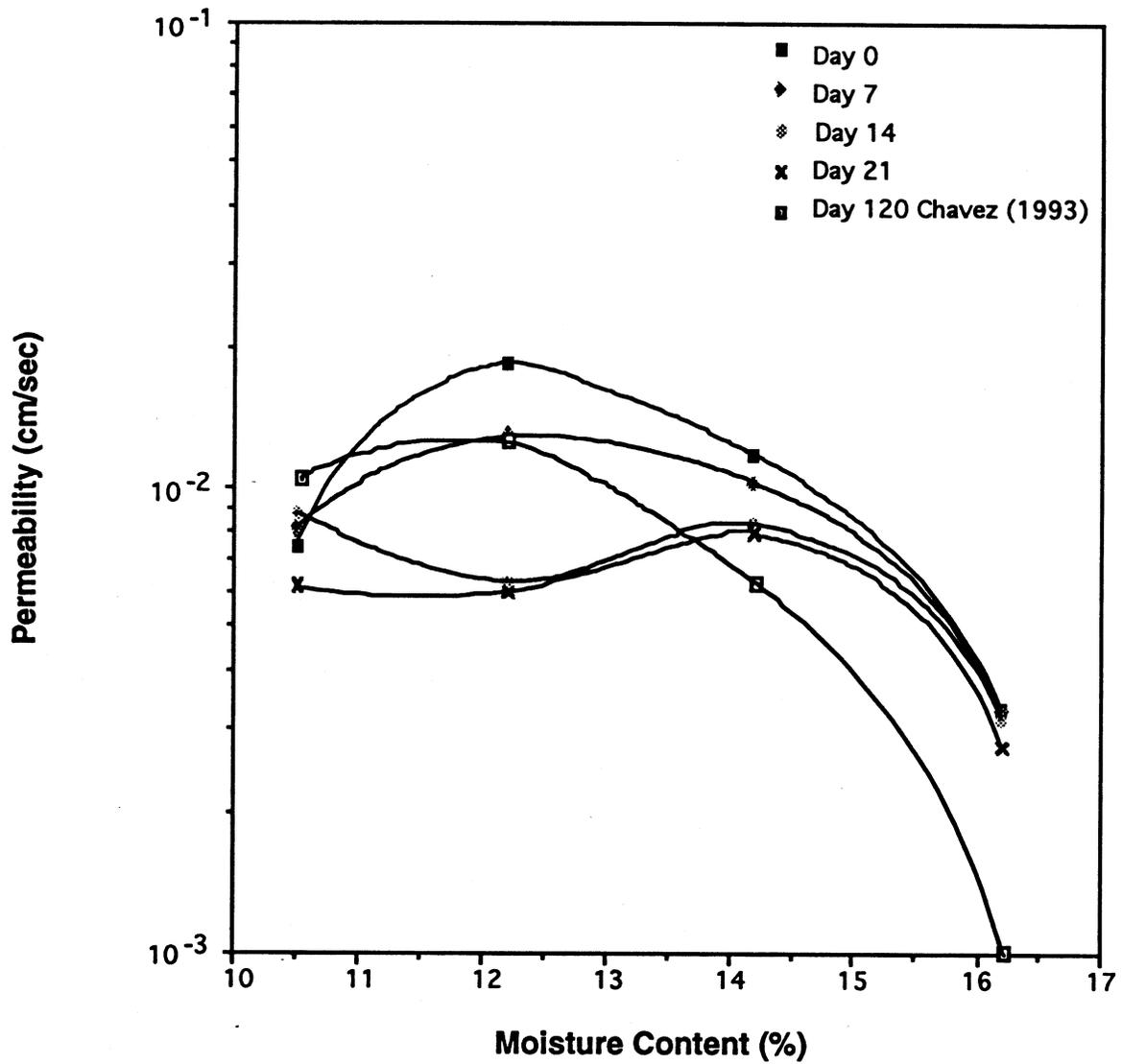


Figure 4.11 Permeability versus moisture content for the MB bottom ash passing #4 sieve compacted following ASTM D-698

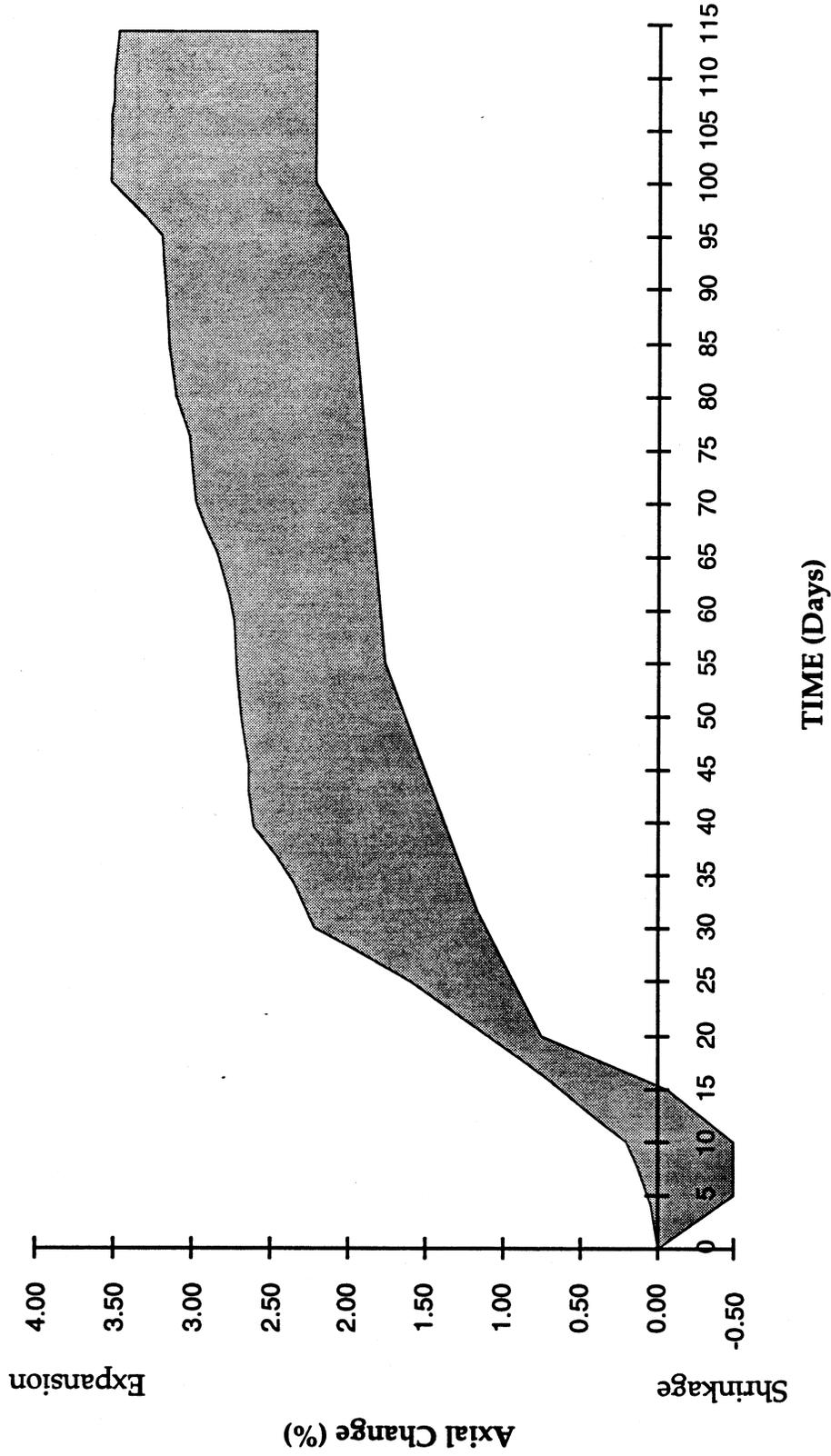


Figure 4.12 Axial change versus time of compacted WTE bottom ash, dry

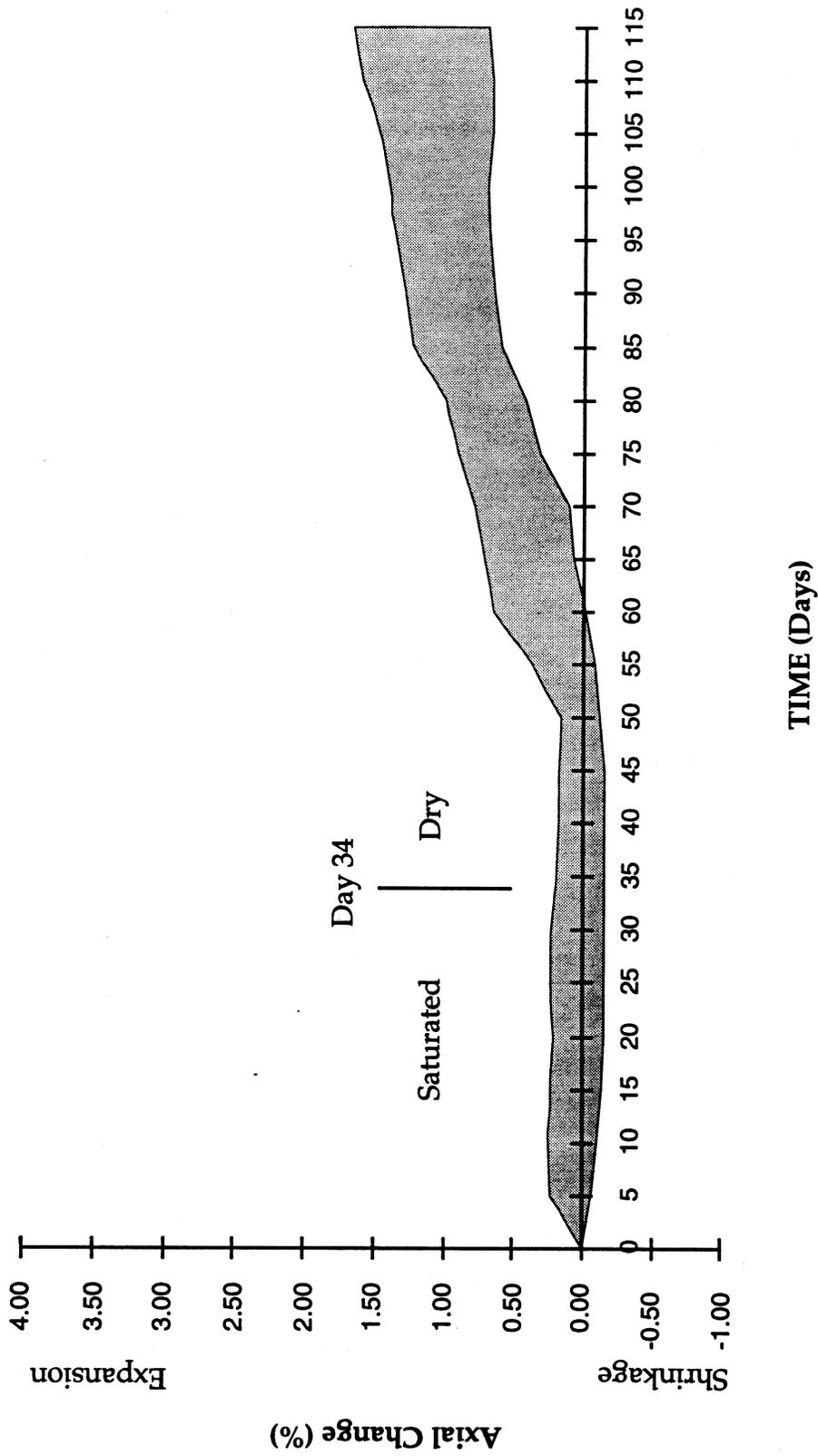


Figure 4.13 Axial change versus time of compacted MB bottom ash, passing #4 sieve compacted following ASTM D-698 saturated for 34 days and allowed to air dry

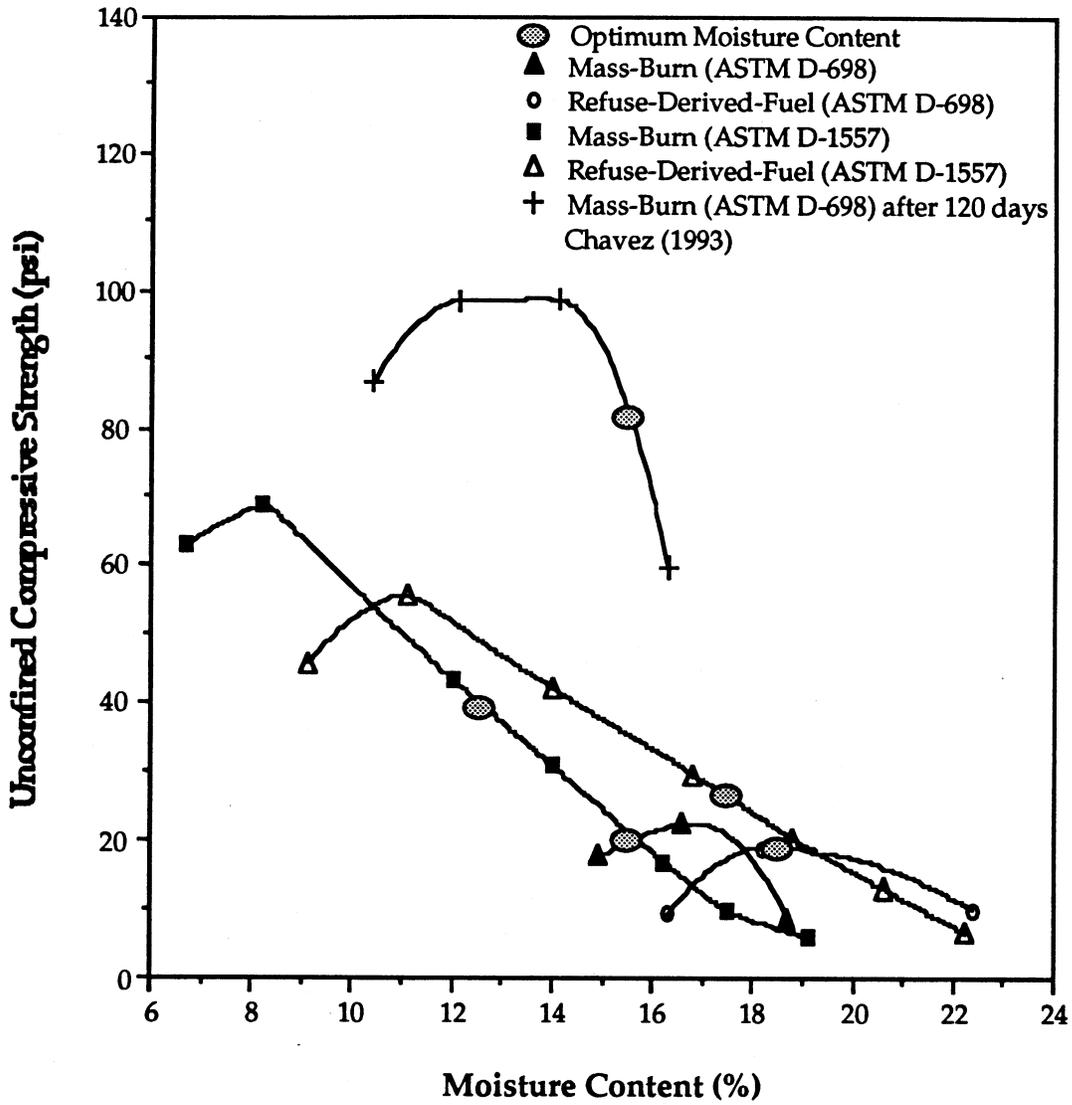


Figure 4.14 Unconfined compressive strength versus moisture content for standard and modified compaction of mass-burn and refuse-derived-fuel WTE bottom ash passing the #4 sieve (1 psi = 6.895 kpa)

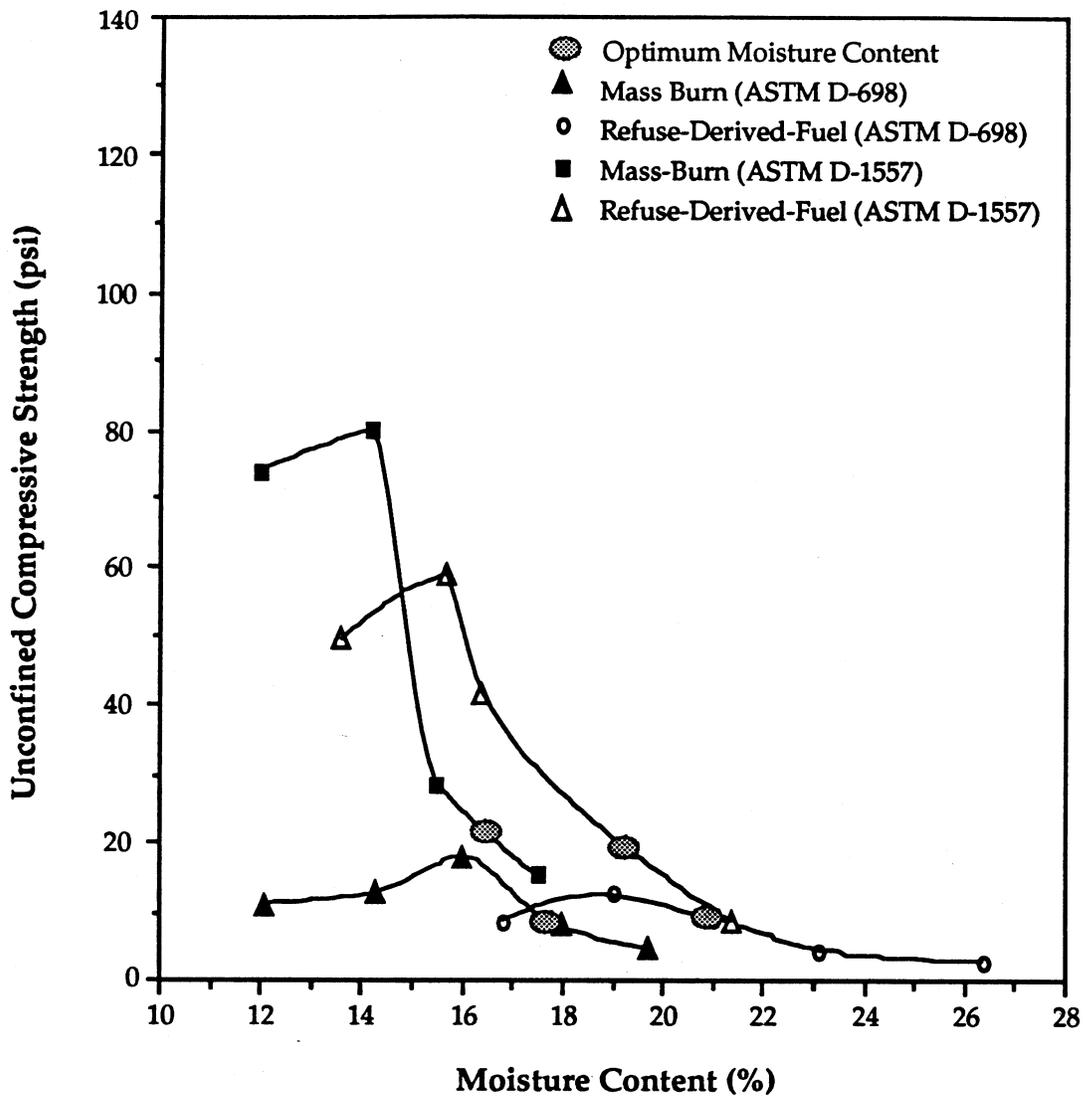


Figure 4.15 Unconfined compressive strength versus moisture content for standard and modified compaction of mass-burn and refuse-derived-fuel WTE bottom ash passing the #8 sieve (1 psi = 6.895 kpa)

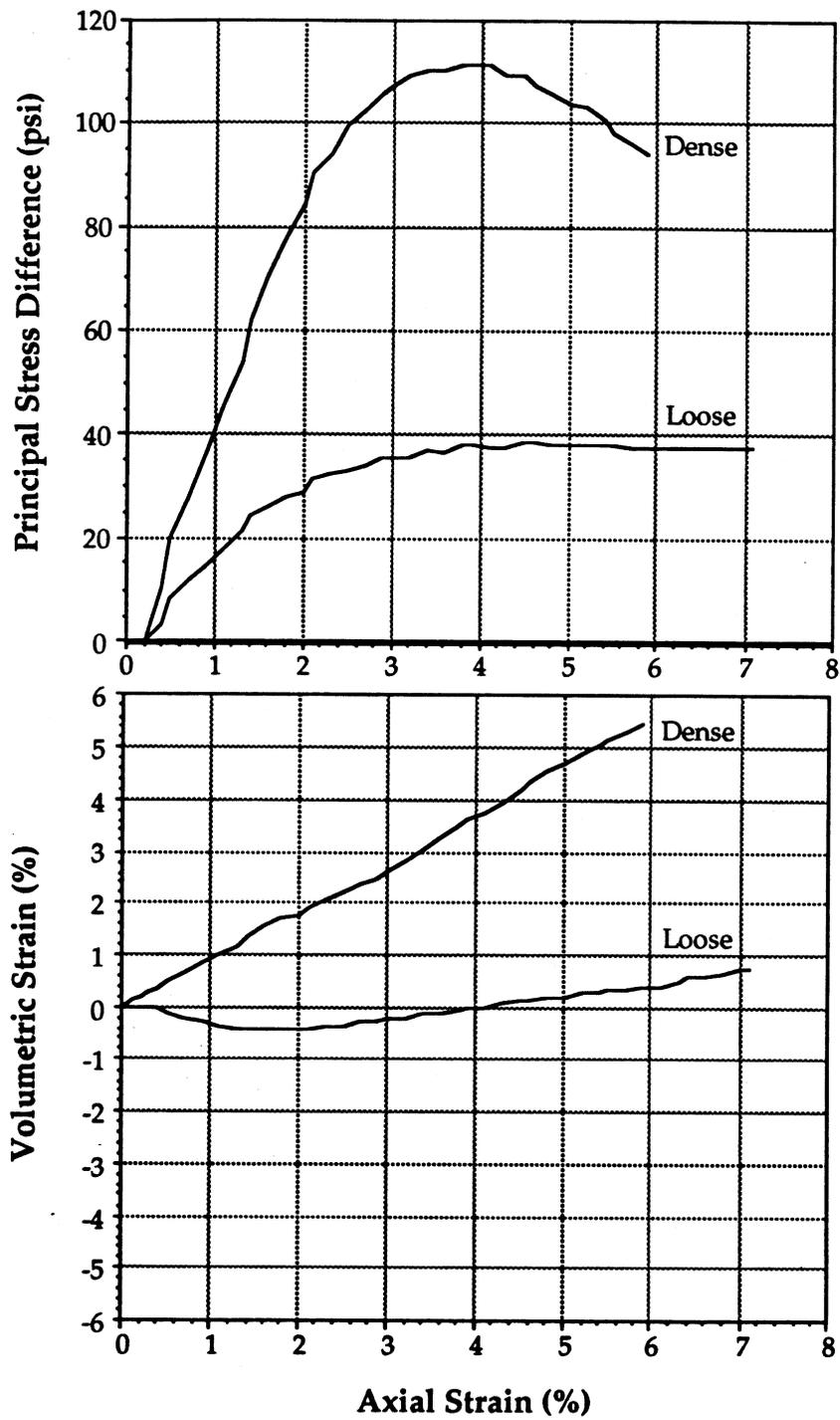


Figure 4.16 Typical stress strain curves for loose and dense MB bottom ash specimens (1 psi = 6.895 kPa) Dense (#4, 100%, 15 psi), Loose (#8, 95%, 10 psi)

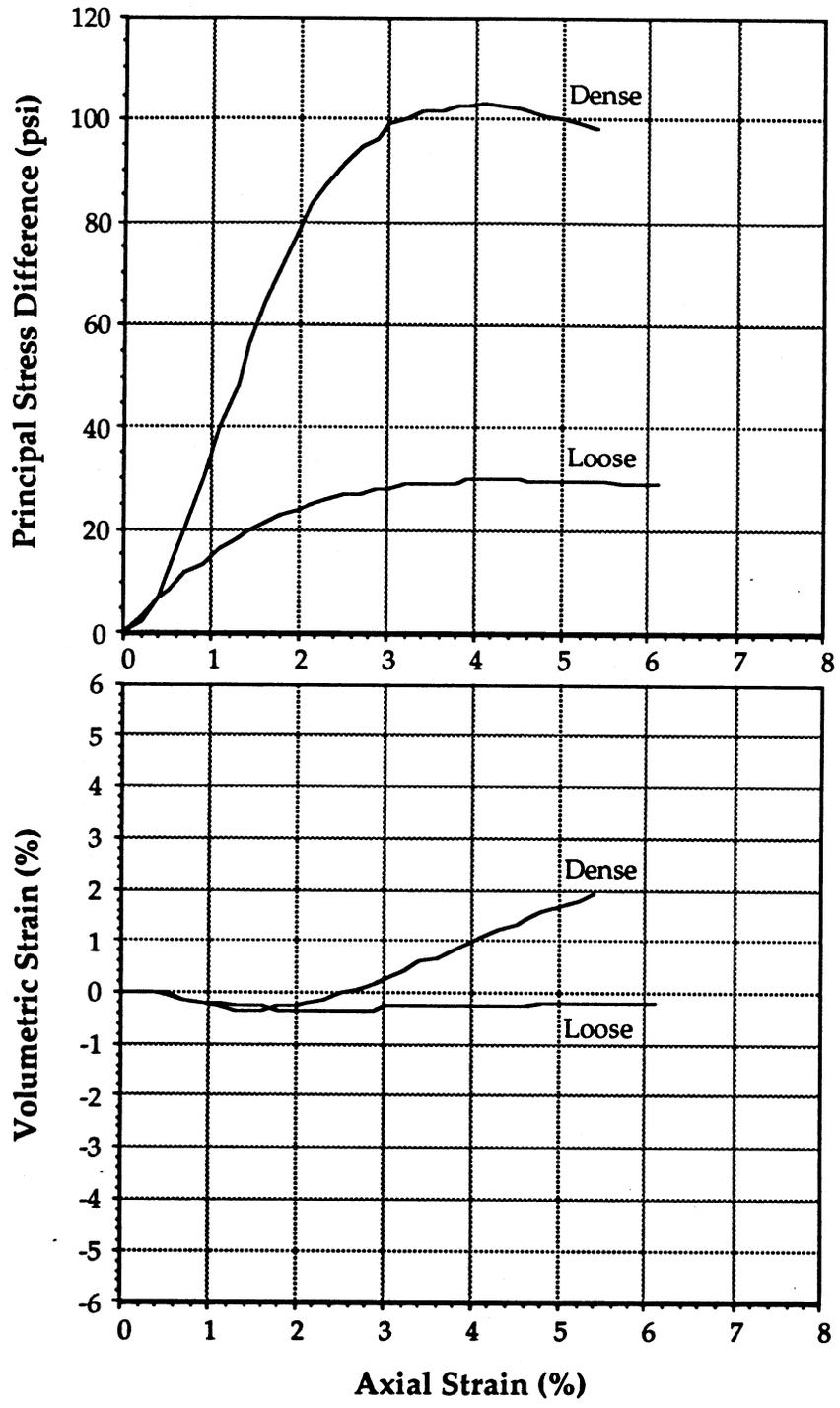


Figure 4.17 Typical stress strain curves for loose and dense RDF bottom ash specimens (1 psi = 6.895 kPa) Dense (#4, 100%, 15 psi), Loose (#8, 95%, 10 psi)

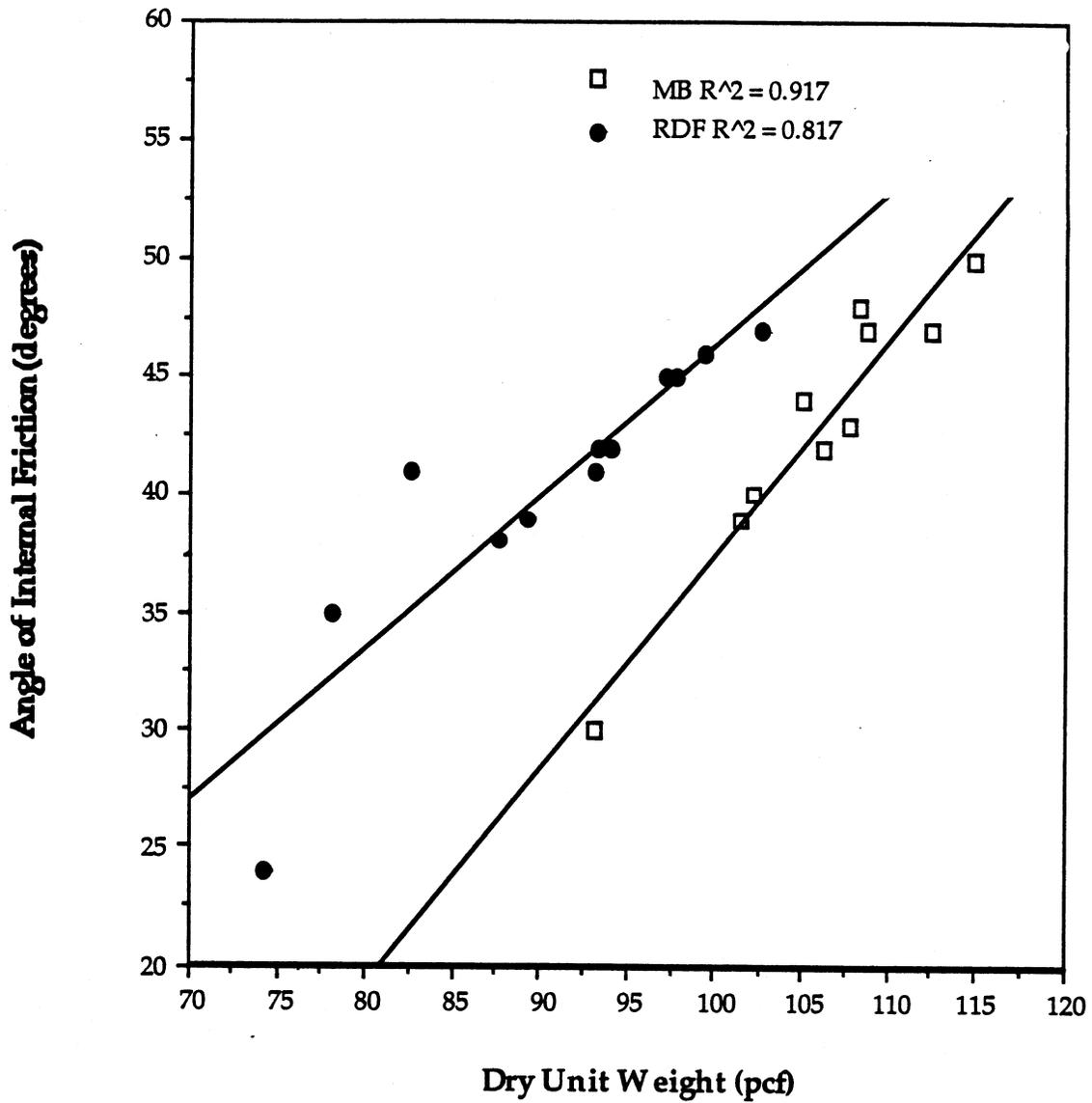


Figure 4.18 Angle of internal friction versus dry unit weight for MB and RDF MWC bottom ash (1 pcf = 0.157 kN/m<sup>3</sup>)

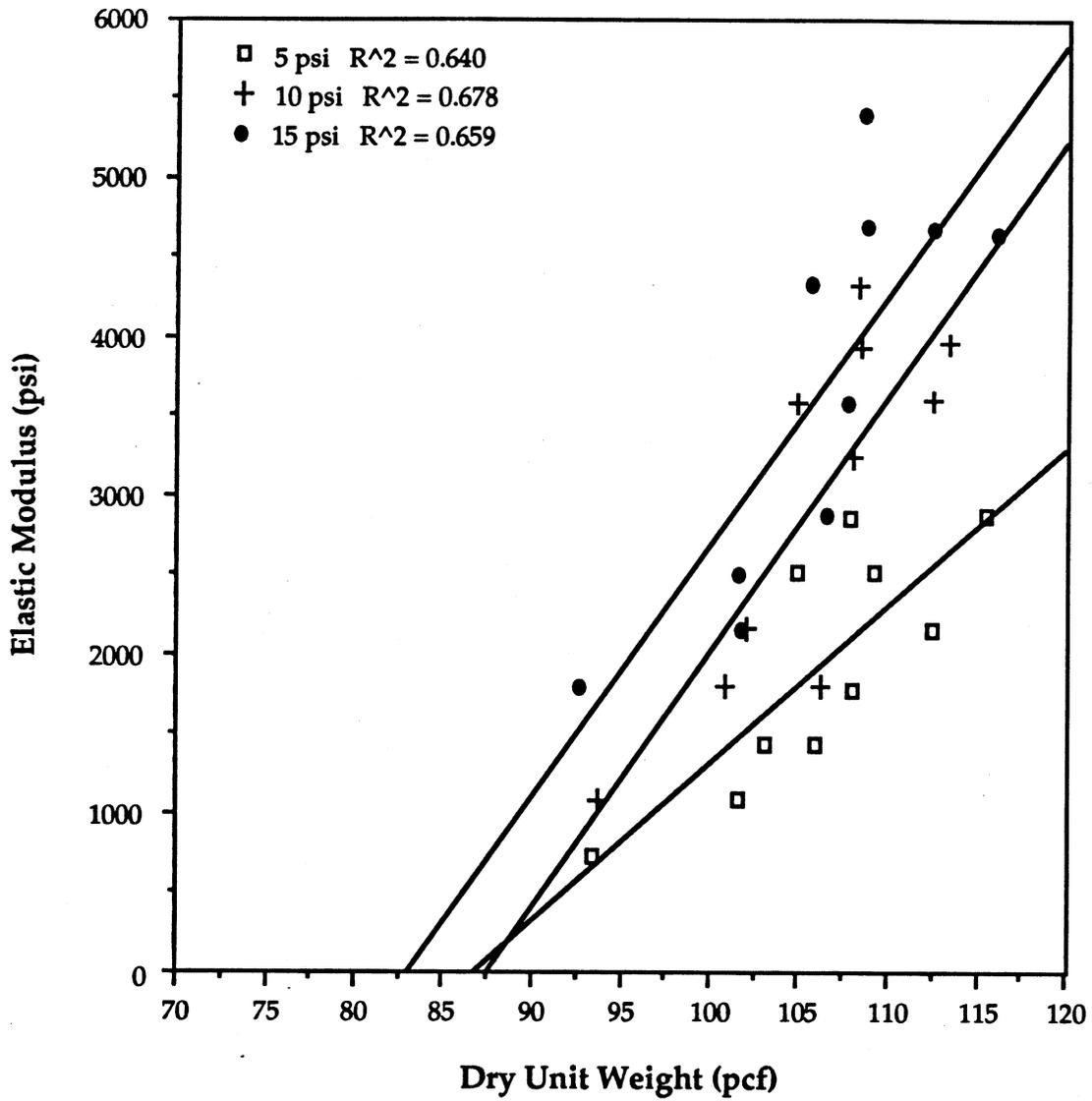


Figure 4.19 Elastic modulus versus dry unit weight for MB bottom ash over a strain difference of 0.2% (1 psi = 6.895 kPa, 1 pcf = 0.157 kN/m<sup>3</sup>)

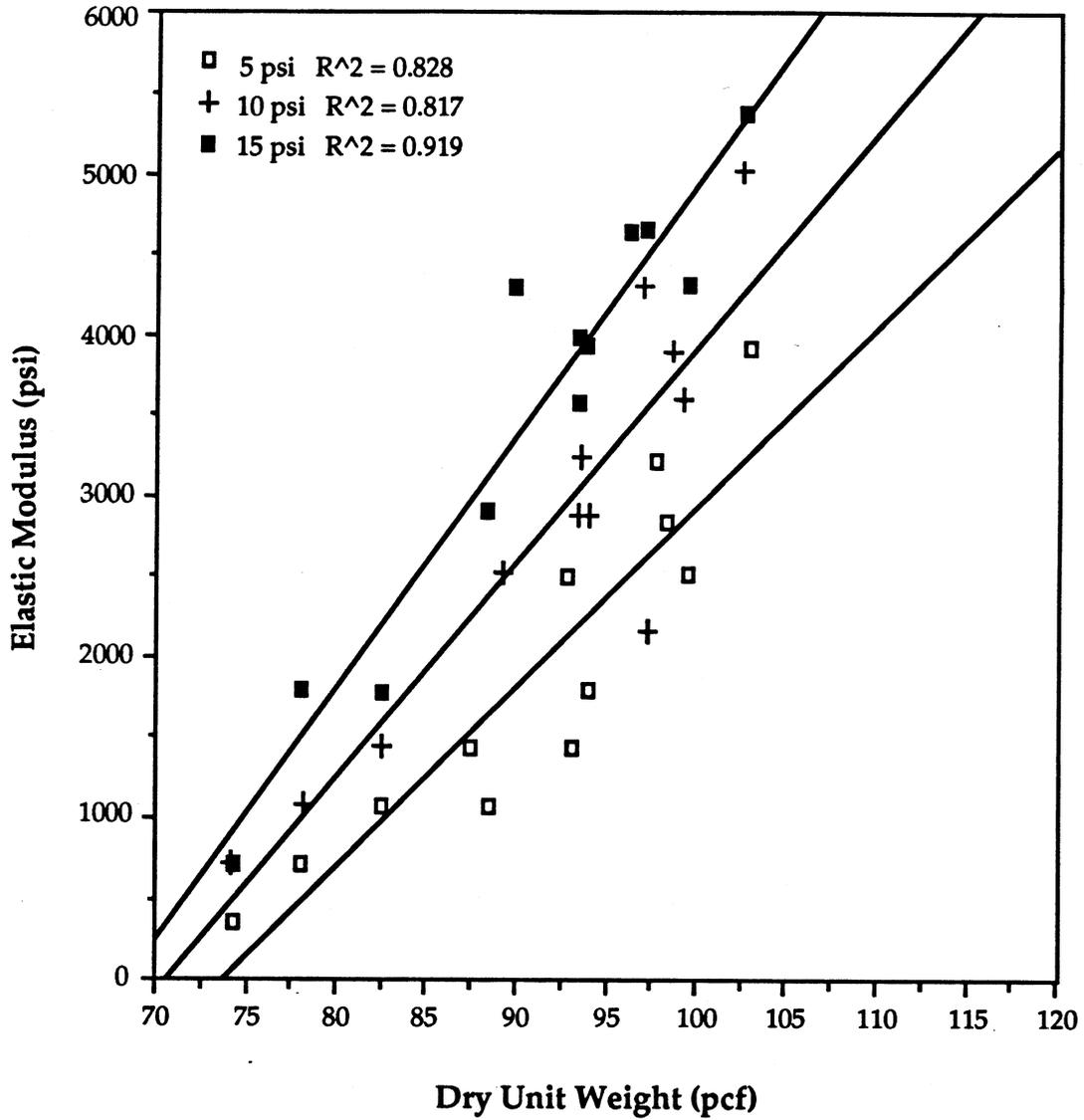


Figure 4.20 Elastic modulus versus dry unit weight for RDF bottom ash over a strain difference of 0.2% (1 psi = 6.895 kPa, 1 pcf = 0.157 kN/m<sup>3</sup>)

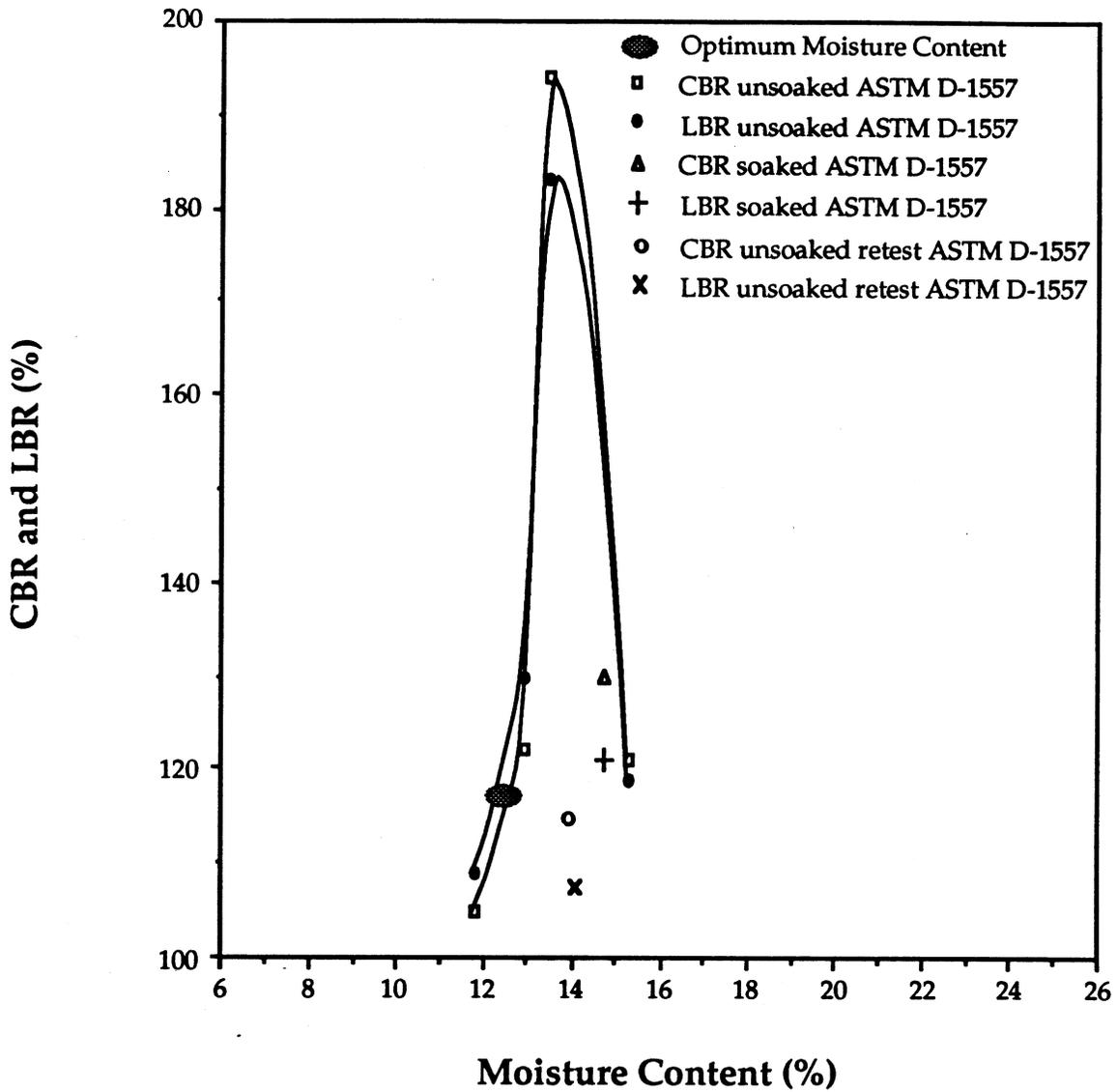


Figure 4.21 Unsoaked and soaked CBR and LBR values versus moisture content for MB bottom ash passing the #4 sieve compacted following ASTM D-1557

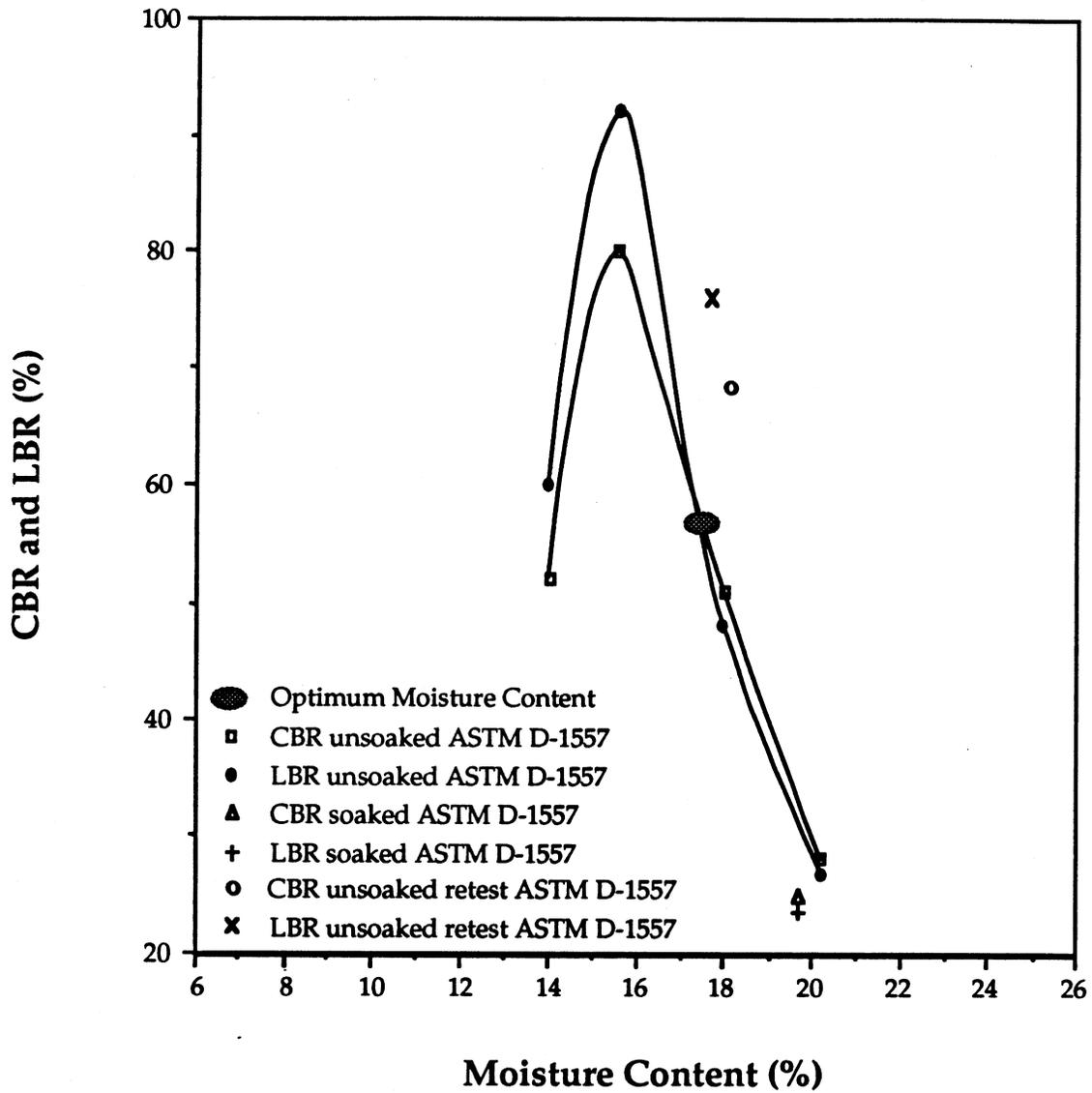


Figure 4.22 Unsoaked and soaked CBR and LBR values versus moisture content for RDF bottom ash passing the #4 sieve compacted following ASTM D-1557

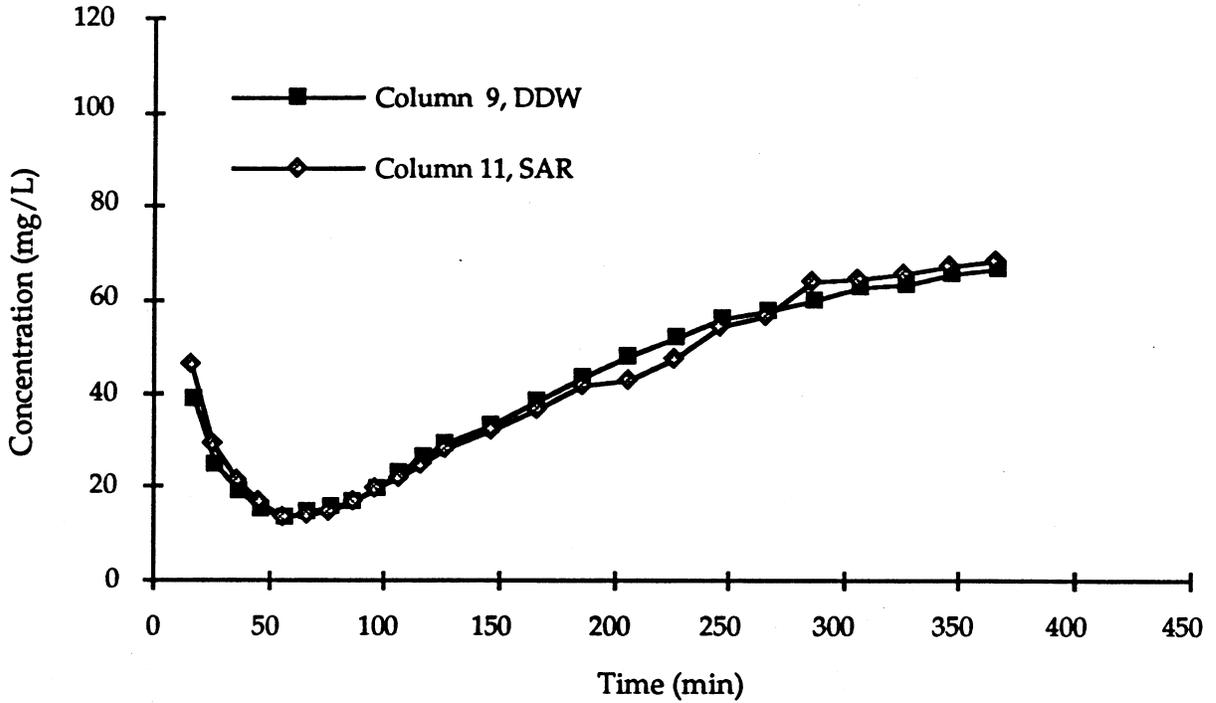


Figure 4.23 Leaching of calcium using DDW and SAR as leaching fluids.

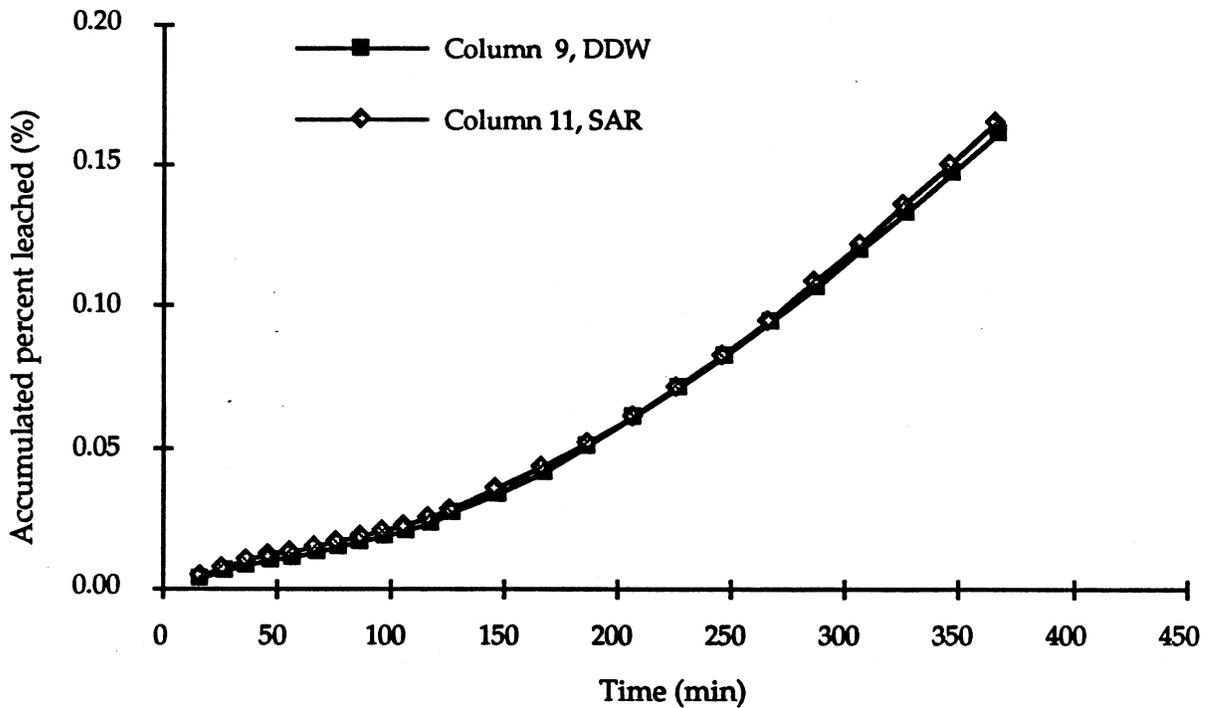


Figure 4.24 Accumulated percent leaching of calcium using DDW and SAR as leaching fluids

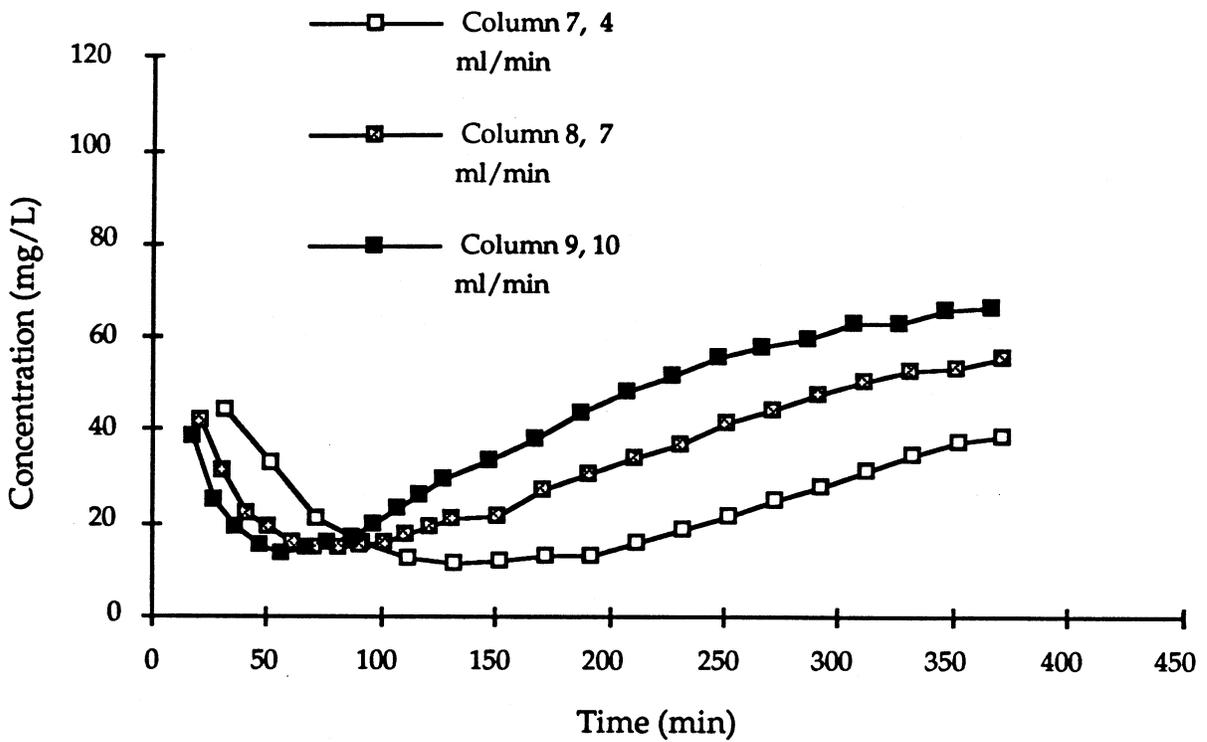


Figure 4.25 Leaching of calcium using DDW at different flow rate

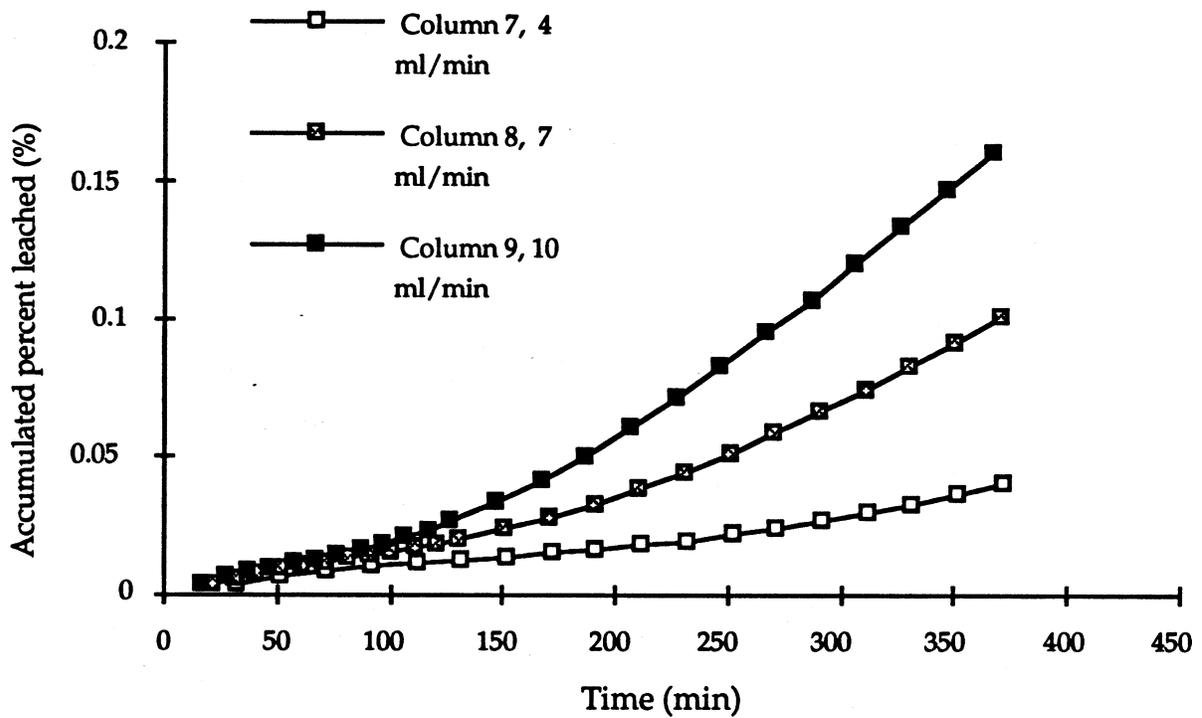


Figure 4.26 Accumulated percent leaching of calcium using DDW at different flow rate

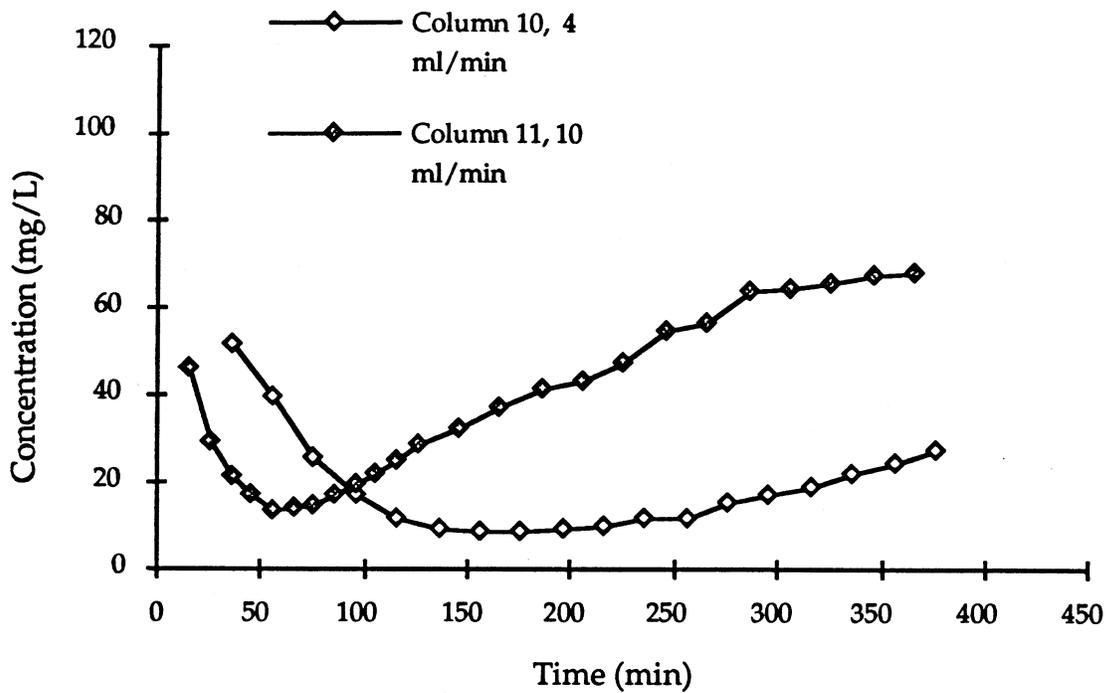


Figure 4.27 Leaching of calcium using SAR at different flow rate

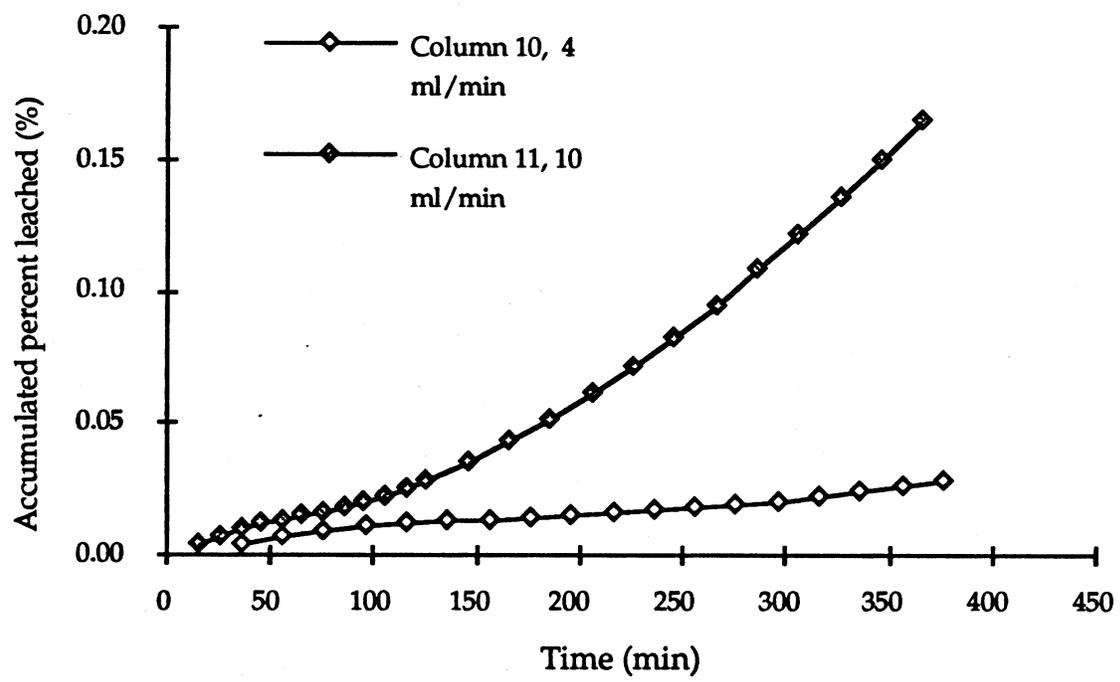


Figure 4.28 Accumulated percent leaching of calcium using SAR at different flow rate

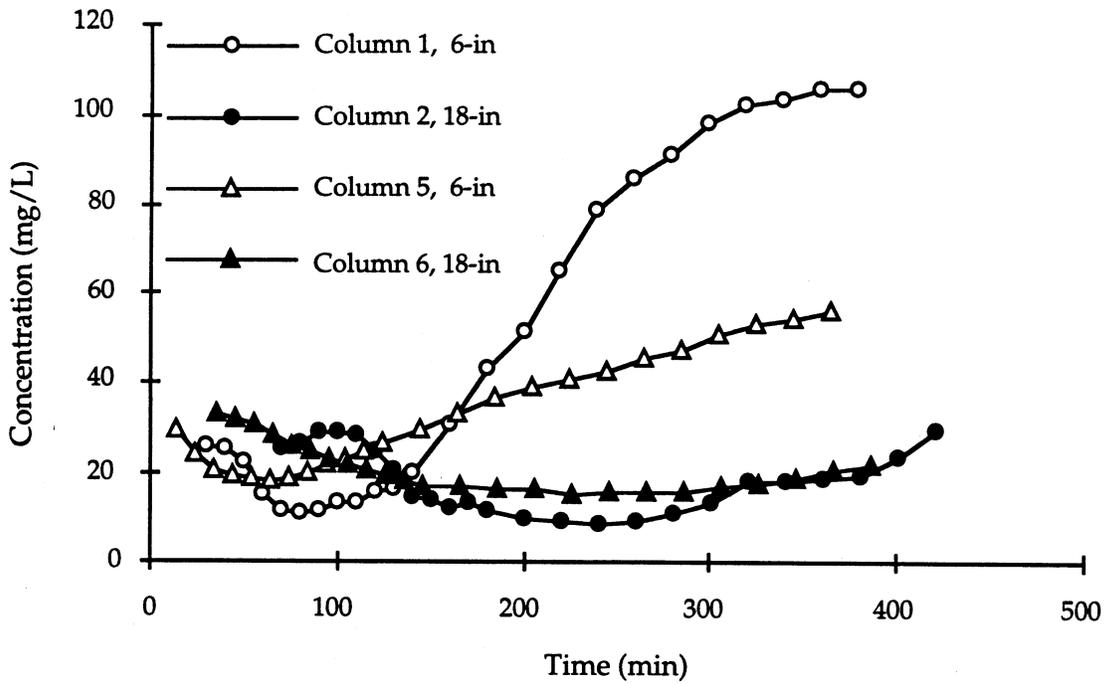


Figure 4.29 Variation in calcium leaching from 15.2-cm and 45.7-cm columns subjected to DDW

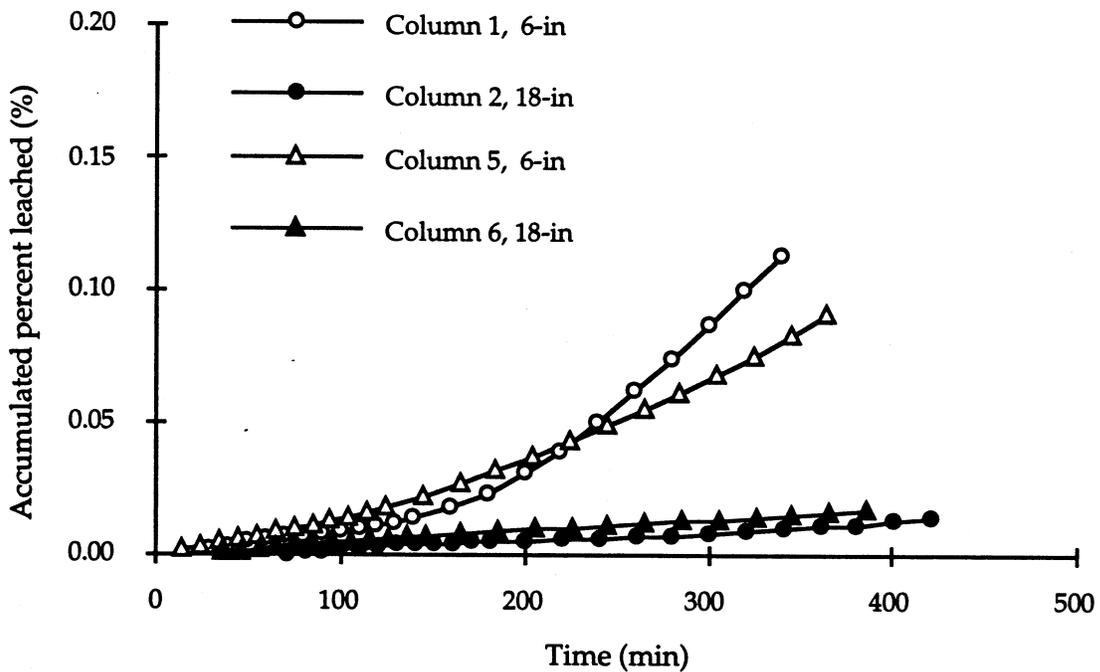


Figure 4.30 Accumulated percent of calcium leached from 15.2-cm and 45.7-cm columns subjected to DDW

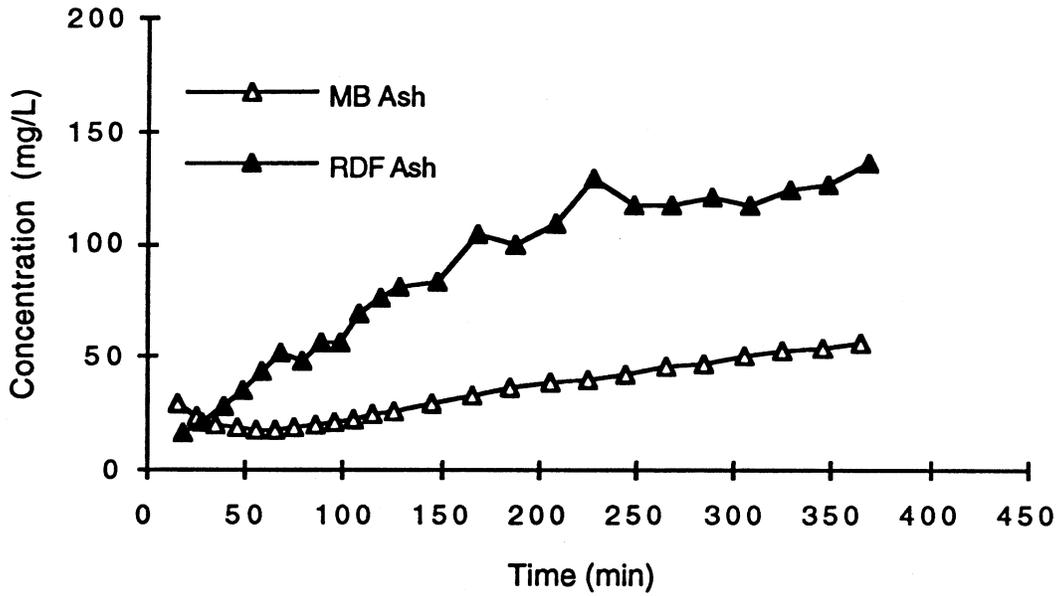


Figure 4.31 Variation in calcium leaching from 6-in columns subjected to DDW using Pinellas County MB ash and Palm Beach County RDF ash

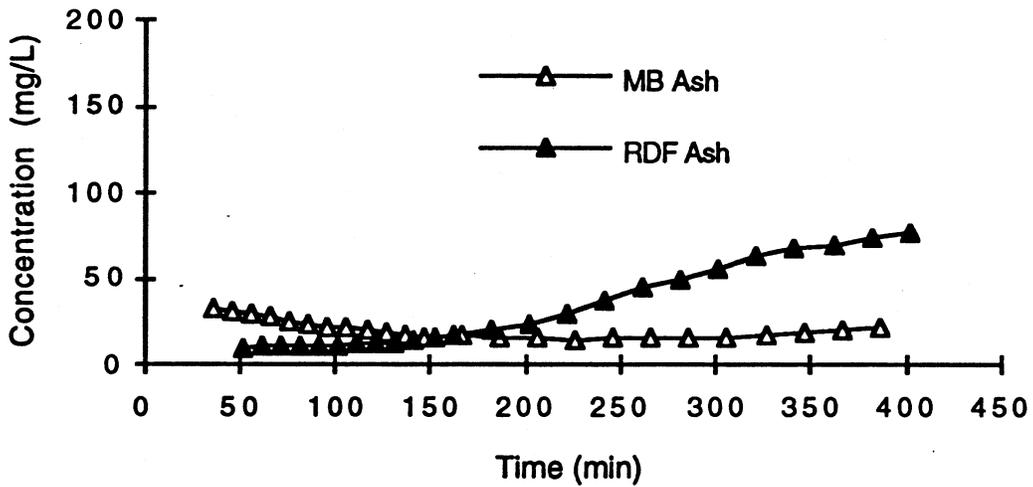


Figure 4.32 Variation in calcium leaching from 18-in columns subjected to DDW using Pinellas County MB ash and Palm Beach County RDF ash

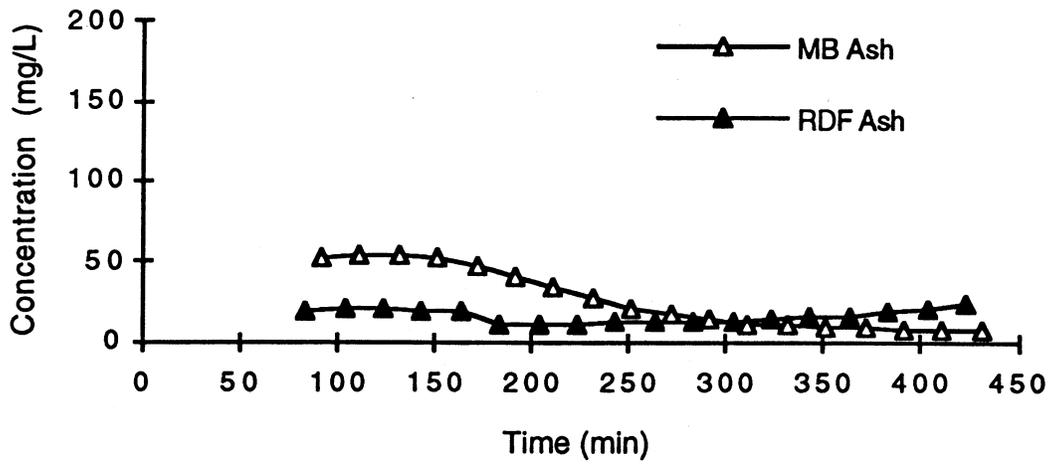


Figure 4.33 Variation in calcium leaching from 18-in columns subjected to SAR using Pinellas County MB ash and Palm Beach County RDF ash

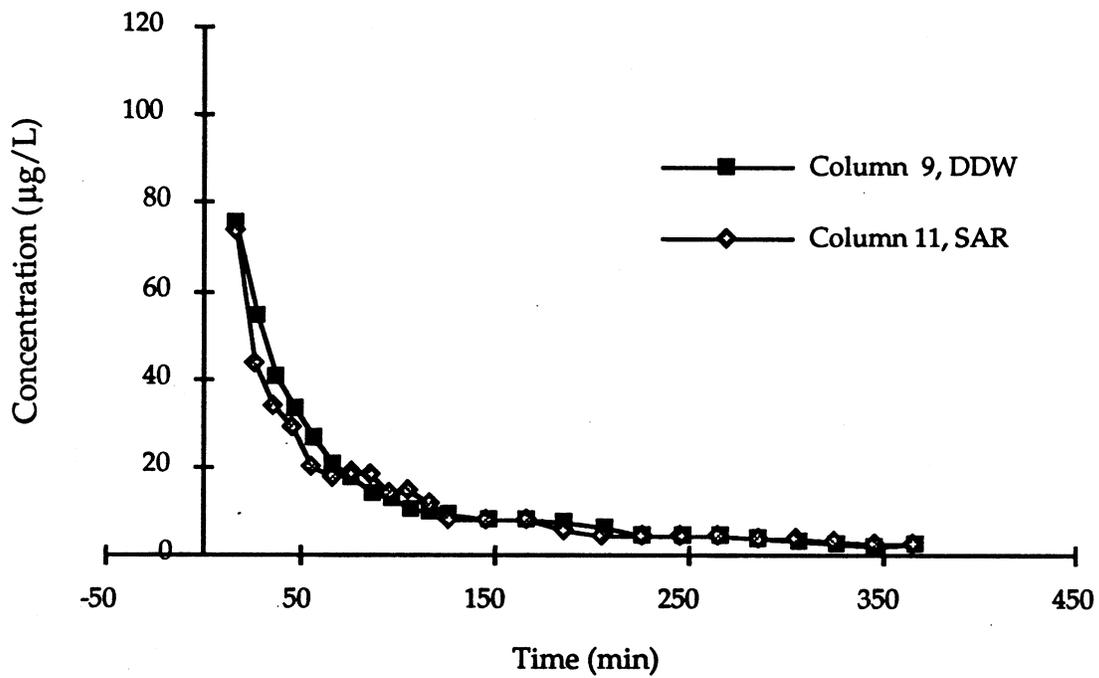


Figure 4.34 Leaching of arsenic using DDW and SAR as leaching fluids.

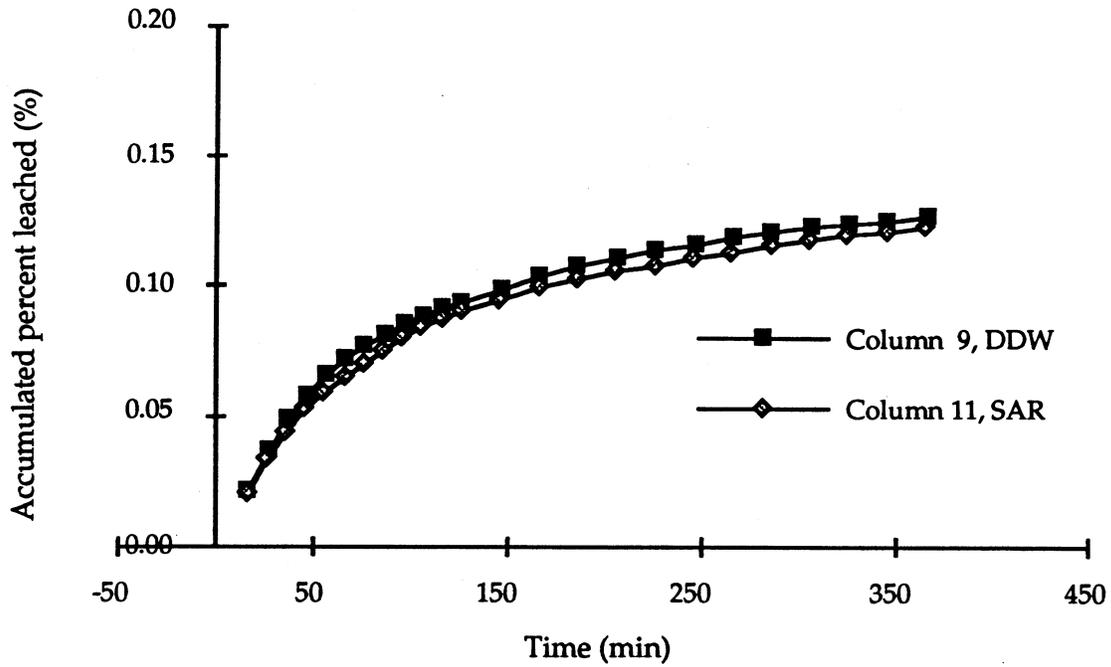


Figure 4.35 Accumulated percent leaching of arsenic using DDW and SAR as leaching fluids

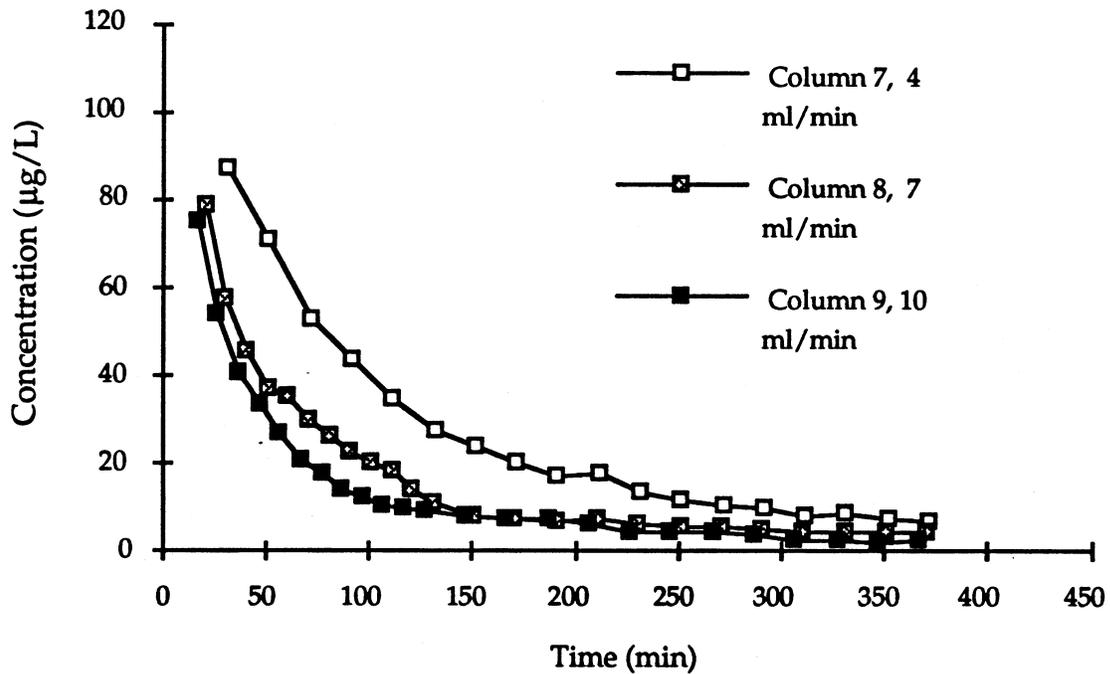


Figure 4.36 Leaching of arsenic using DDW at different flow rate

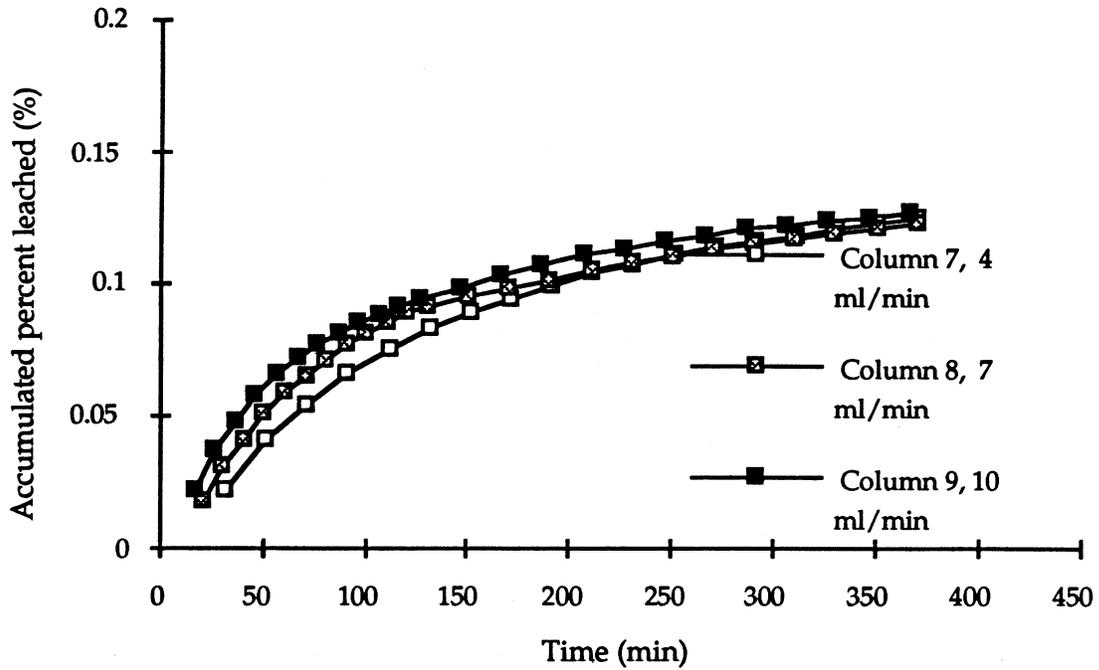


Figure 4.37 Accumulated percent leaching of arsenic using DDW at different flow rate

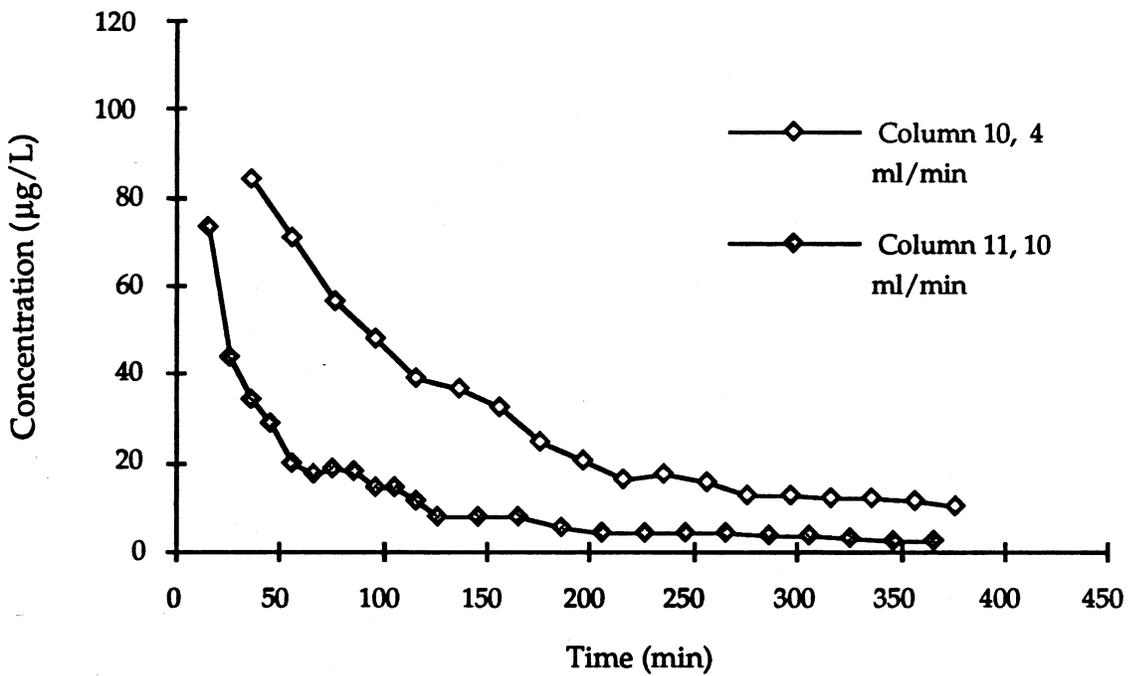


Figure 4.38 Leaching of arsenic using SAR at different flow rate.

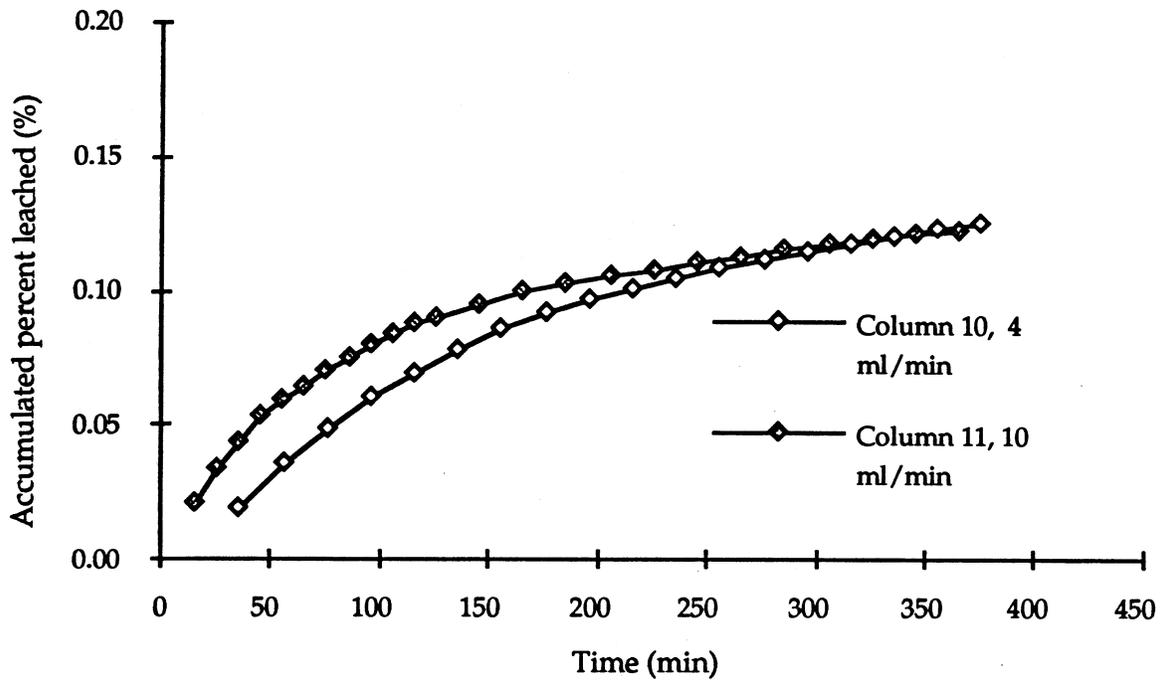


Figure 4.39 Accumulated percent leaching of arsenic using SAR at different flow rate

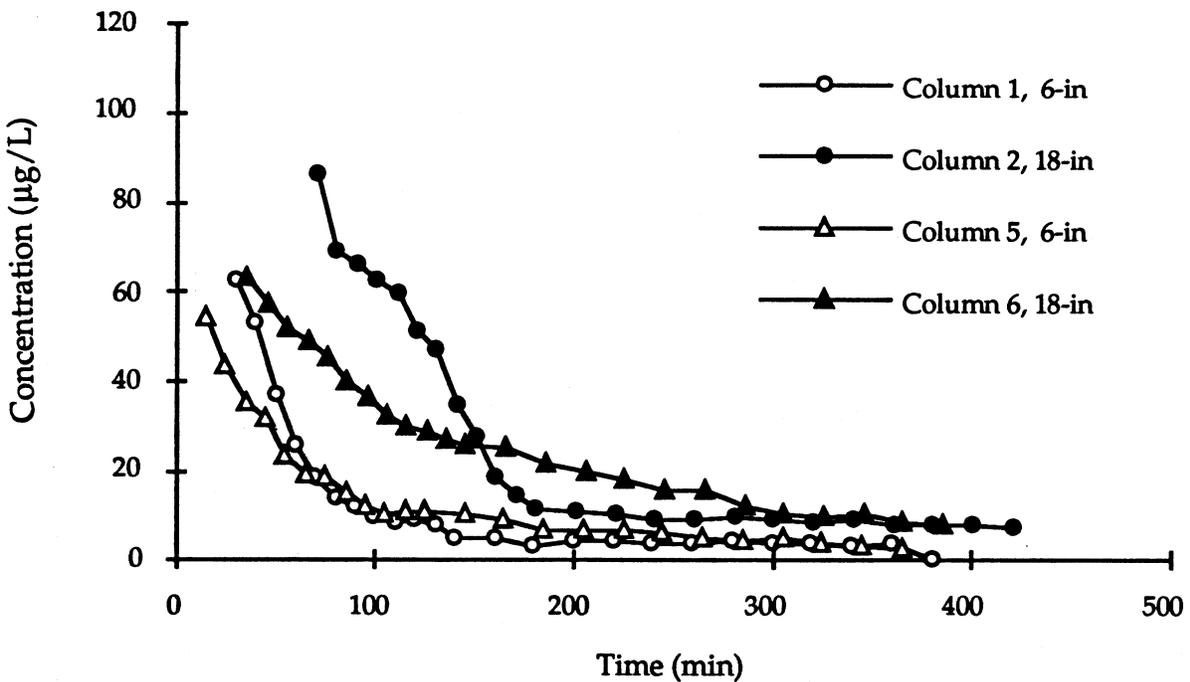


Figure 4.40 Variation in arsenic leaching from 15.2-cm and 45.7-cm columns subjected to DDW

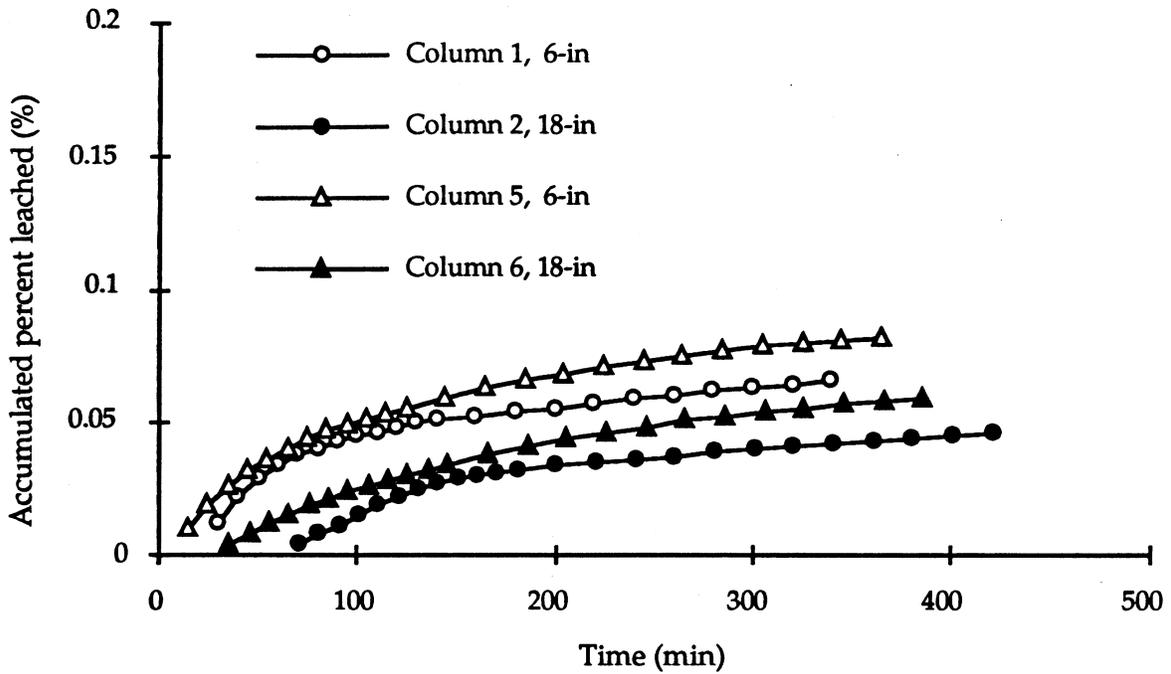


Figure 4.41 Accumulated percent of arsenic leached from 15.2-cm and 45.7-cm columns subjected to DDW.

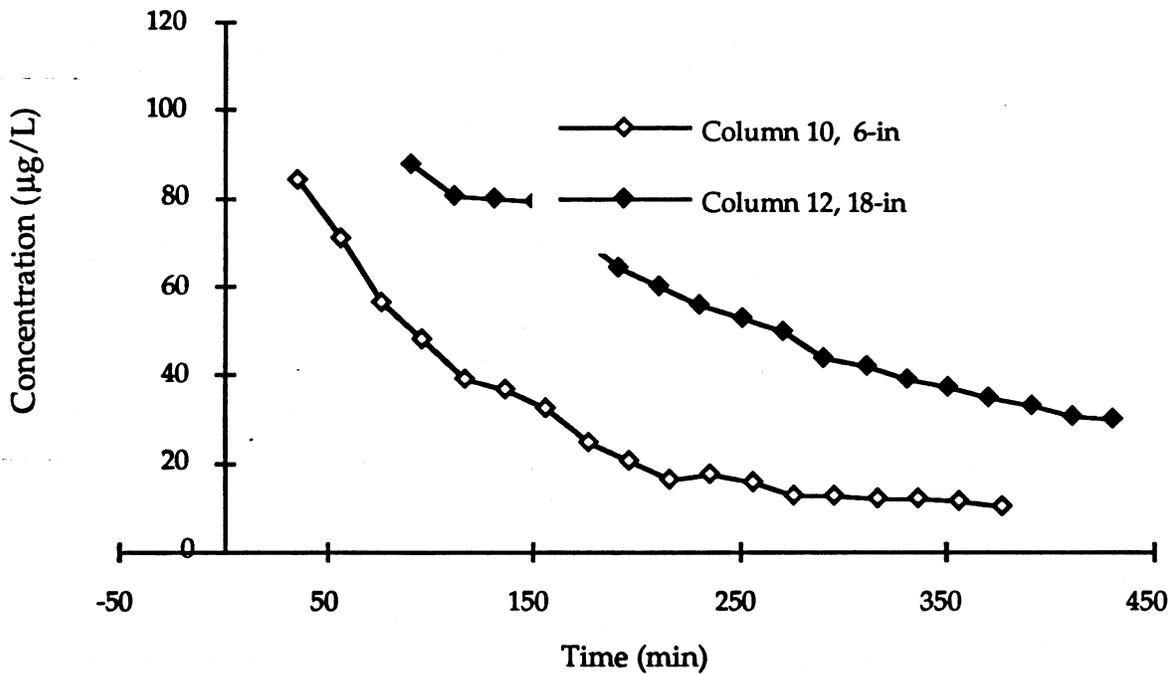


Figure 4.42 Variation in arsenic leaching from 15.2-cm and 45.7-cm columns subjected to SAR.

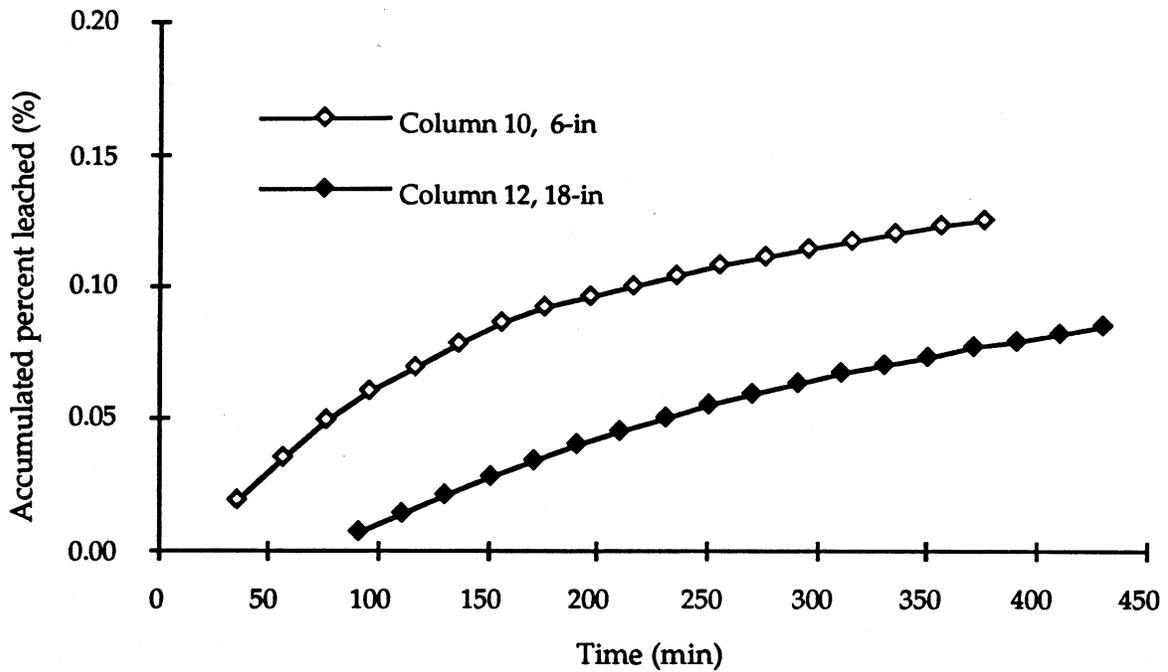


Figure 4.43 Accumulated percent of arsenic leached from 15.2-cm and 45.7-cm columns subjected to SAR

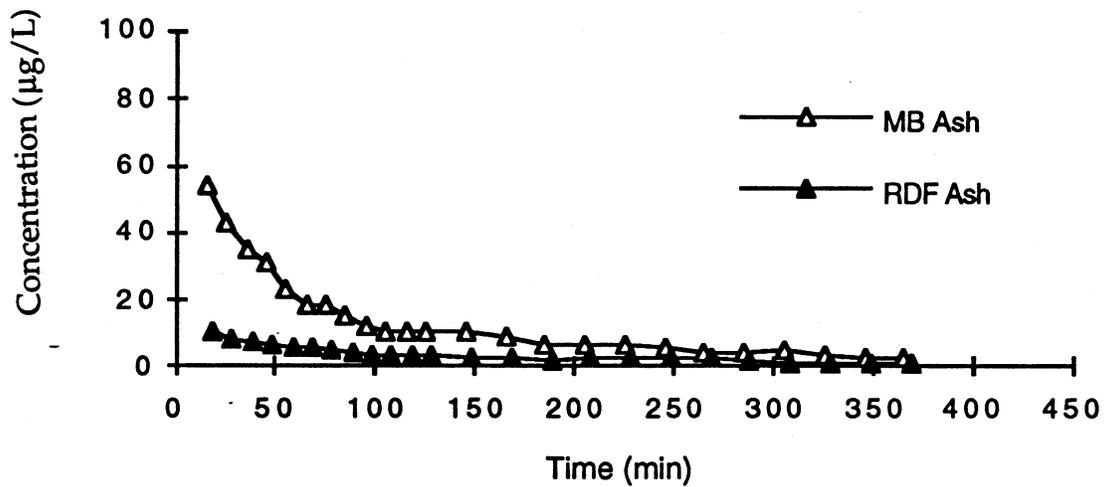


Figure 4.44 Variation in arsenic leaching from 6-in columns subjected to DDW using Pinellas County MB ash and Palm Beach County RDF ash

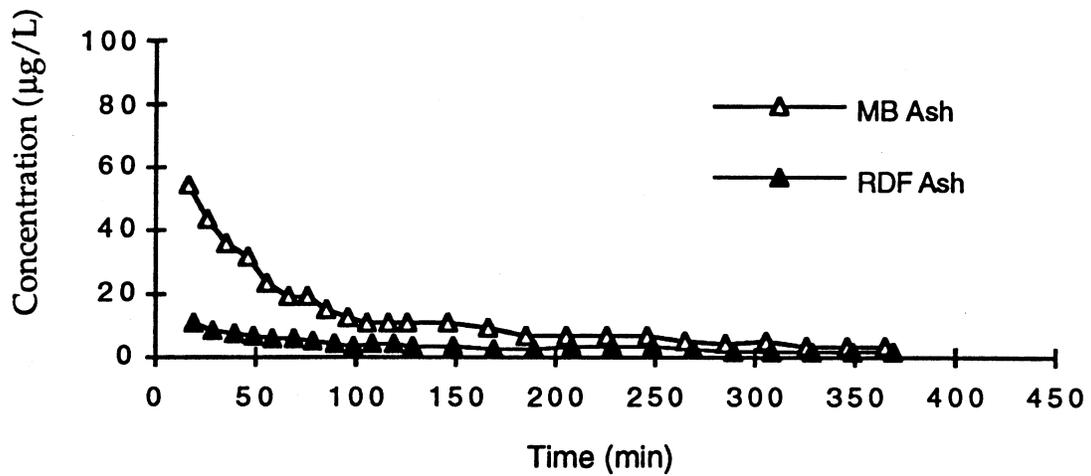


Figure 4.45 Variation in arsenic leaching from 18-in columns subjected to DDW using Pinellas County MB ash and Palm Beach County RDF ash

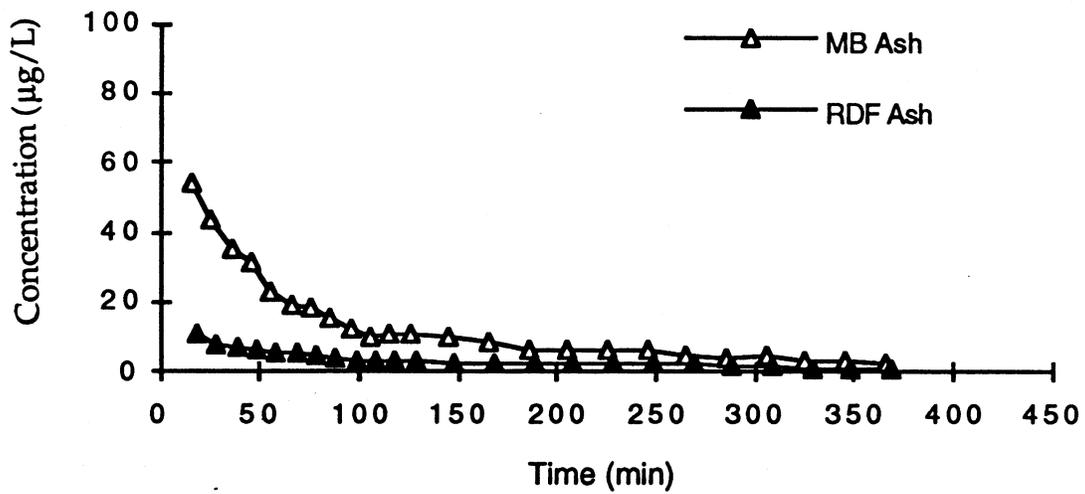


Figure 4.46 Variation in arsenic leaching from 18-in columns subjected to SAR using Pinellas County MB ash and Palm Beach County RDF ash

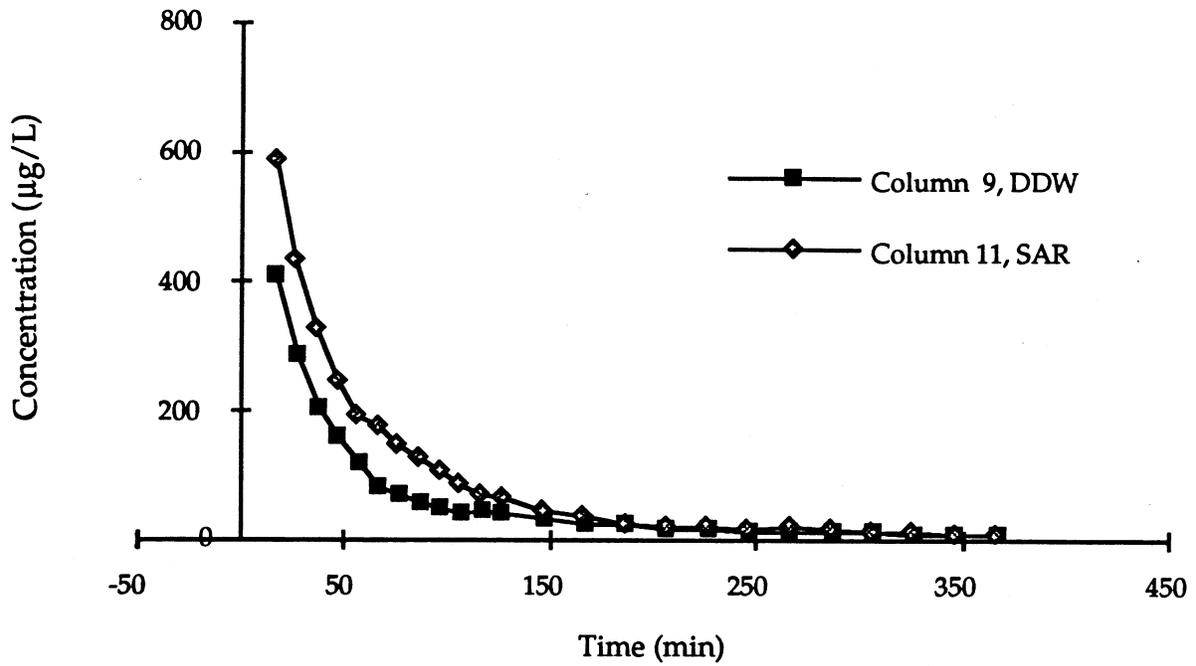


Figure 4.47 Leaching of chromium using DDW and SAR as leaching fluids

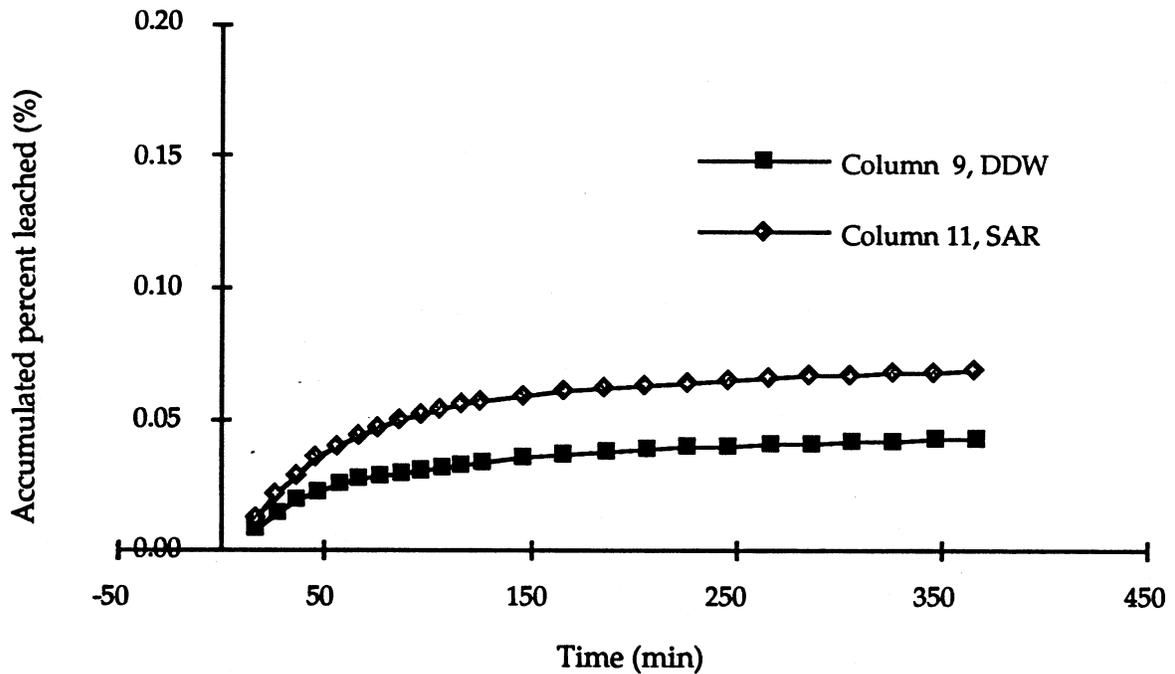


Figure 4.48 Accumulated percent leaching of chromium using DDW and SAR as leaching fluids

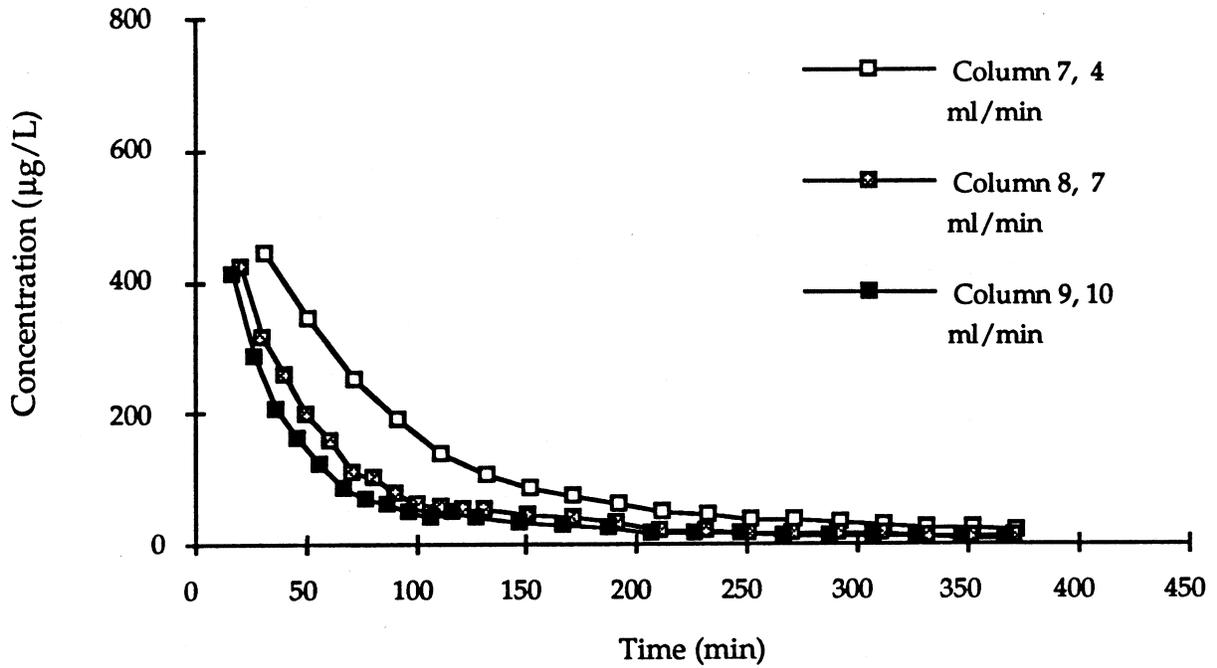


Figure 4.49 Leaching of chromium using DDW at different flow rate

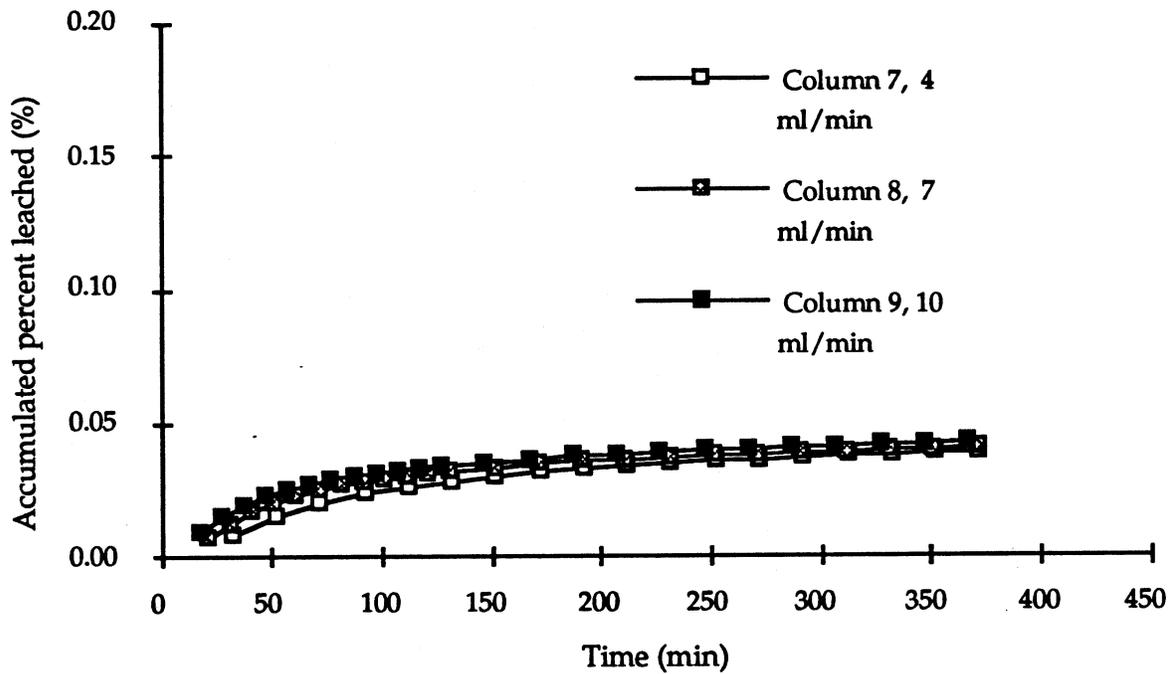


Figure 4.50 Accumulated percent leaching of chromium using DDW at different flow rate

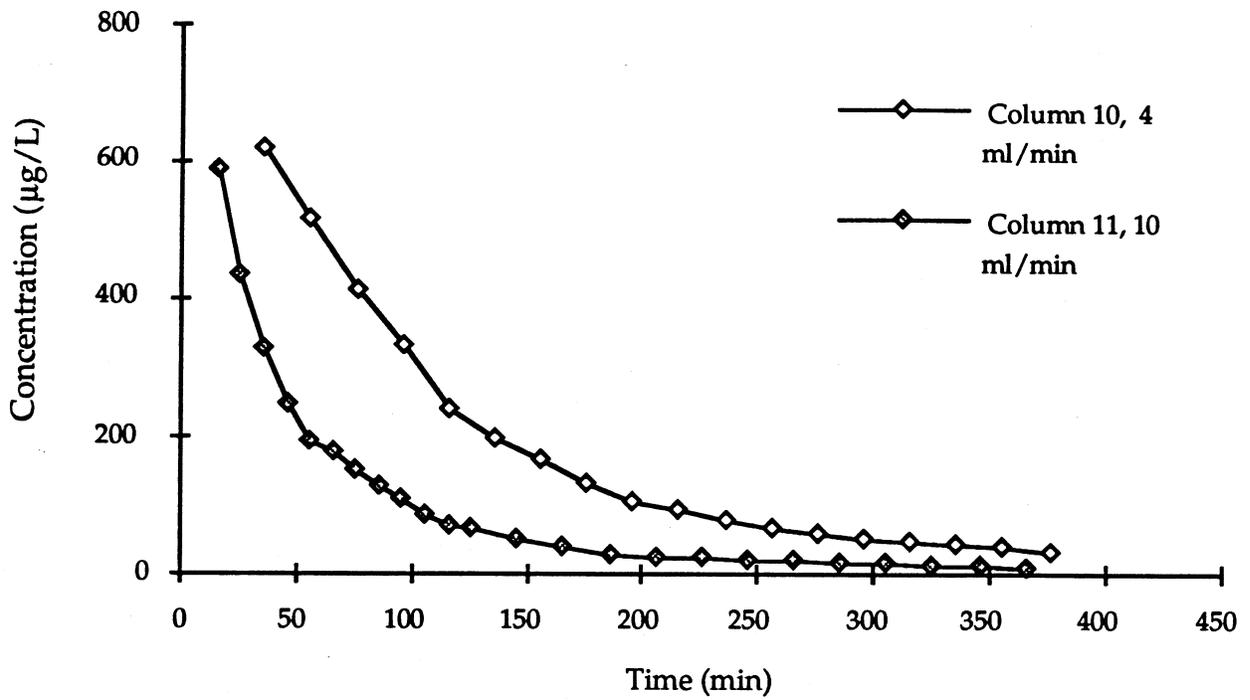


Figure 4.51 Leaching of chromium using SAR at different flow rate

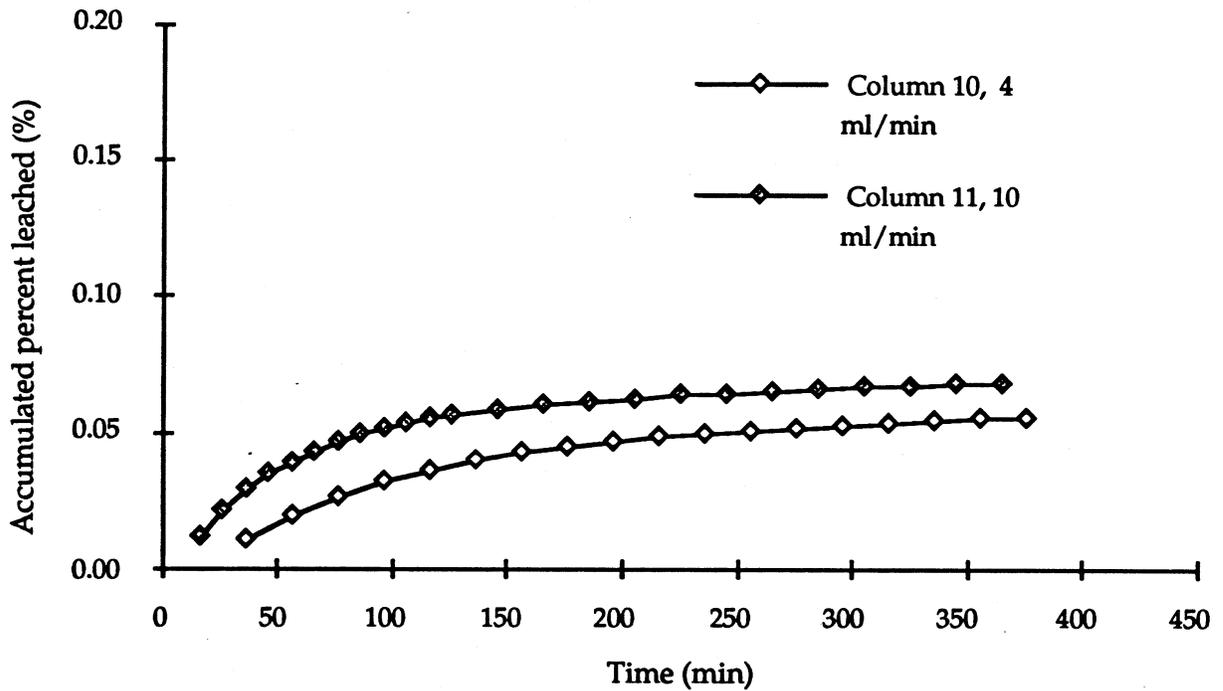


Figure 4.52 Accumulated percent leaching of chromium using SAR at different flow rate

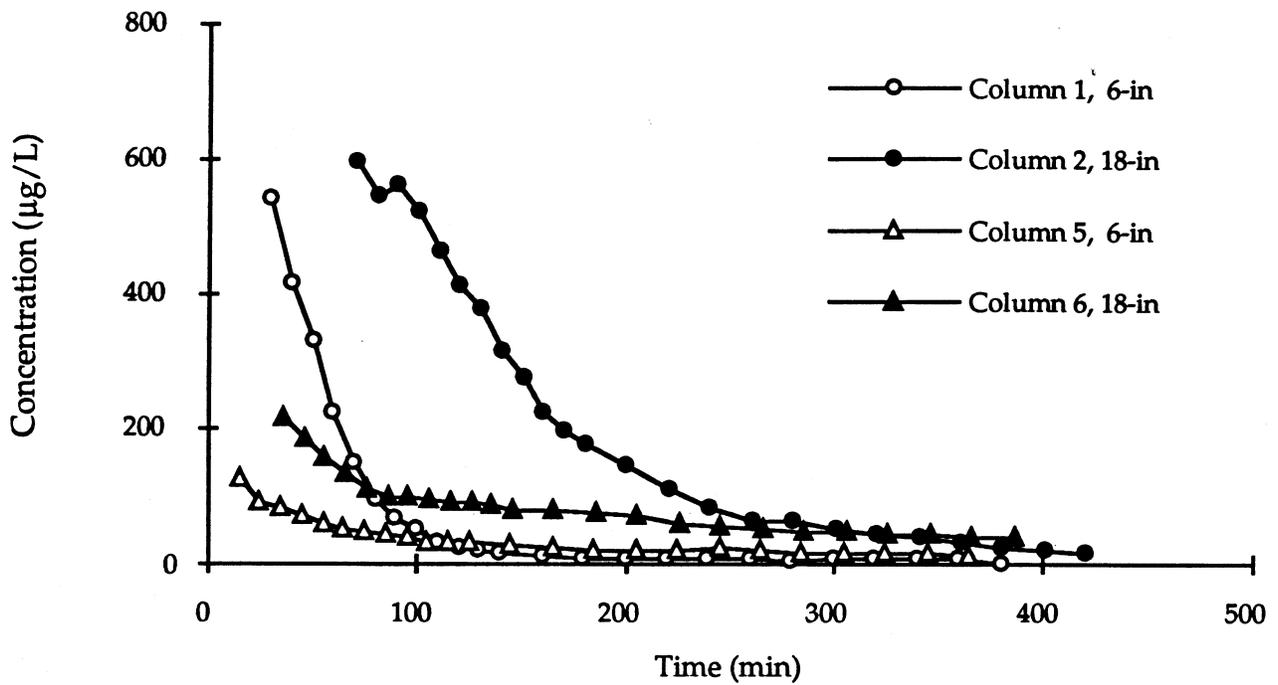


Figure 4.53 Variation in chromium leaching from 15.2-cm and 45.7-cm columns subjected to DDW

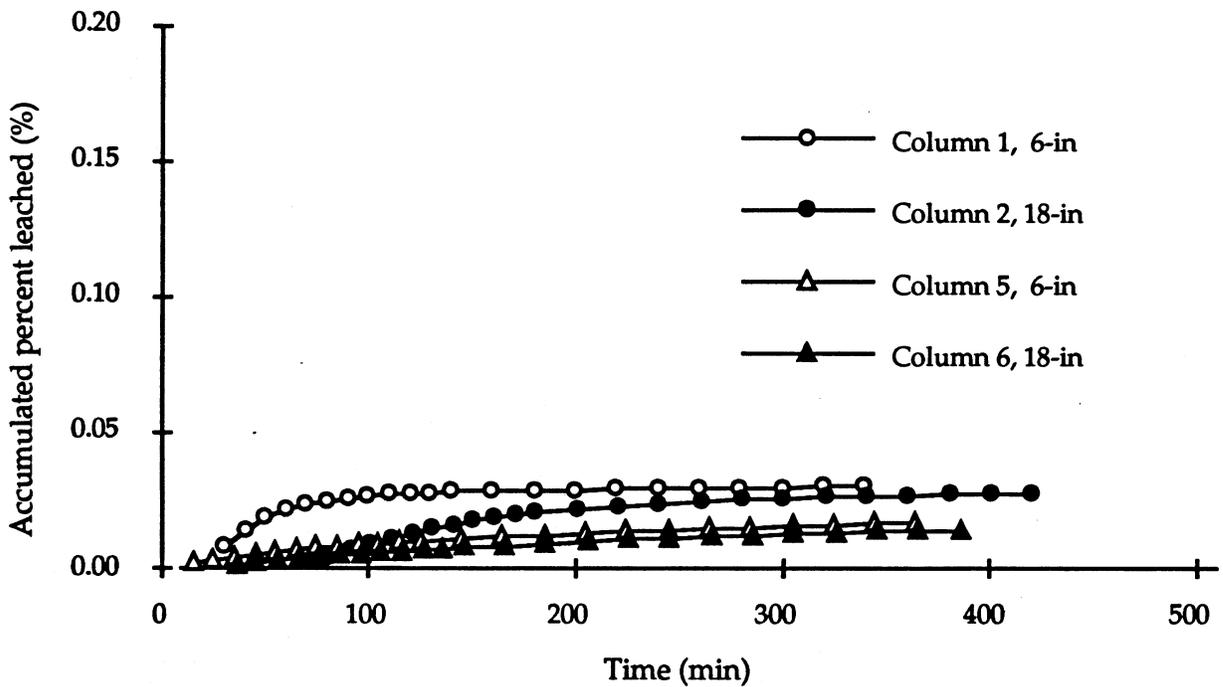


Figure 4.54 Accumulated percent of chromium leached from 15.2-cm and 45.7-cm columns subjected to DDW

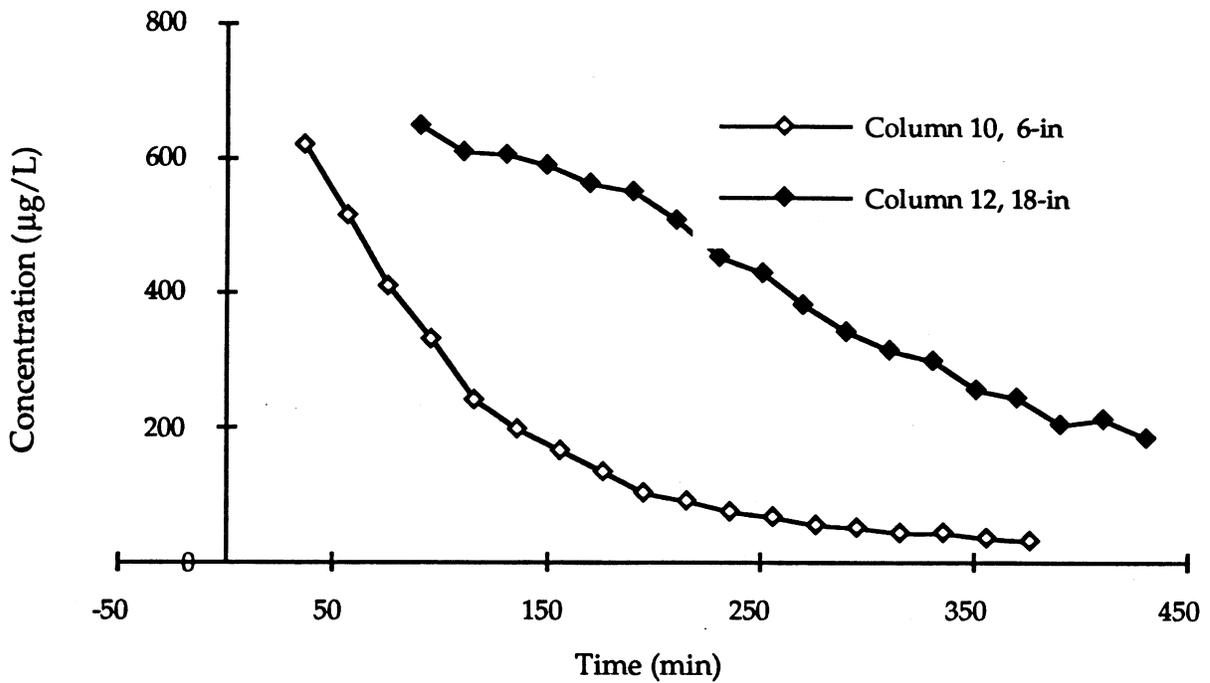


Figure 4.55 Variation in chromium leaching from 15.2-cm and 45.7-cm columns subjected to SAR

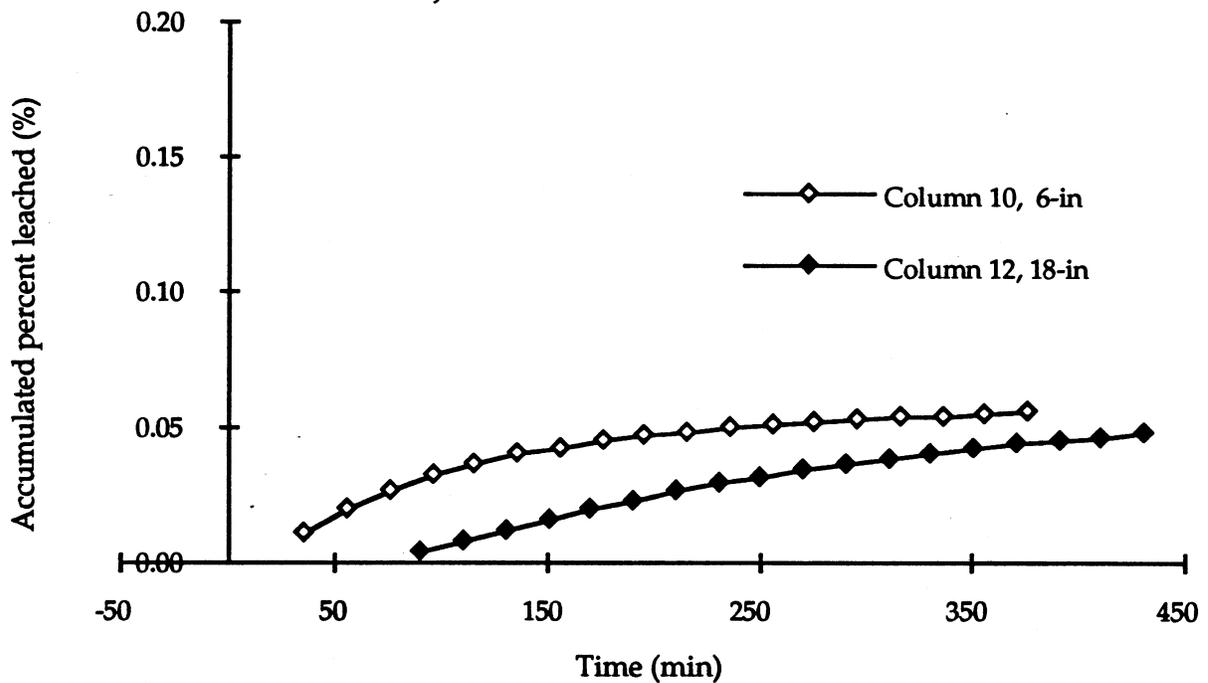


Figure 4.56 Accumulated percent of chromium leached from 15.2-cm and 45.7-cm columns subjected to SAR

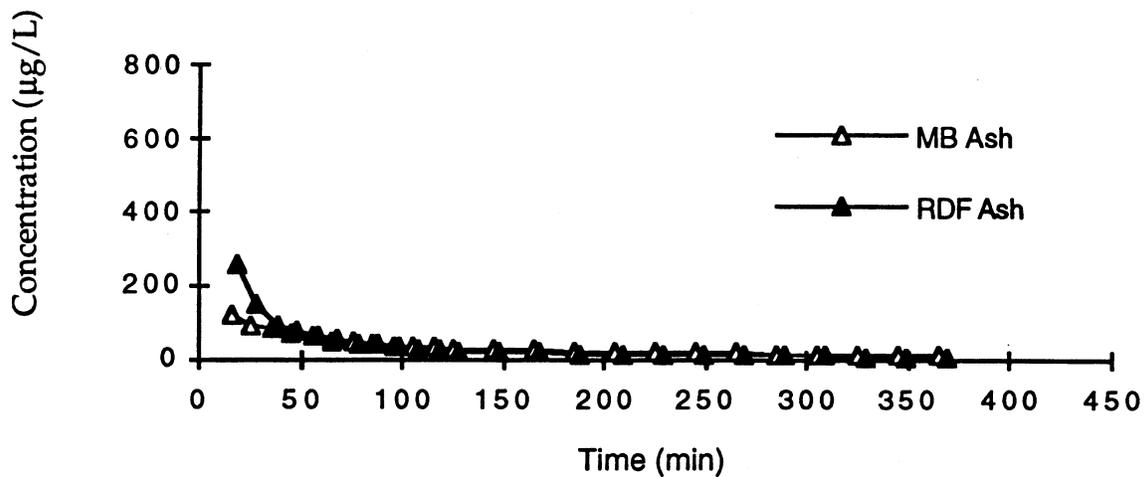


Figure 4.57 Variation in chromium leaching from 6-in columns subjected to DDW using Pinellas County MB ash and Palm Beach County RDF ash

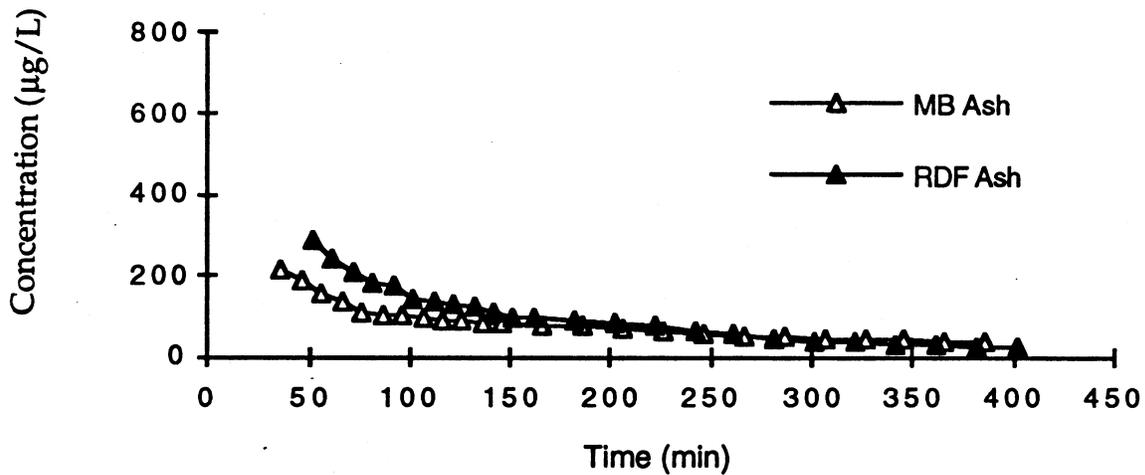


Figure 4.58 Variation in chromium leaching from 18-in columns subjected to DDW using Pinellas County MB ash and Palm Beach County RDF ash

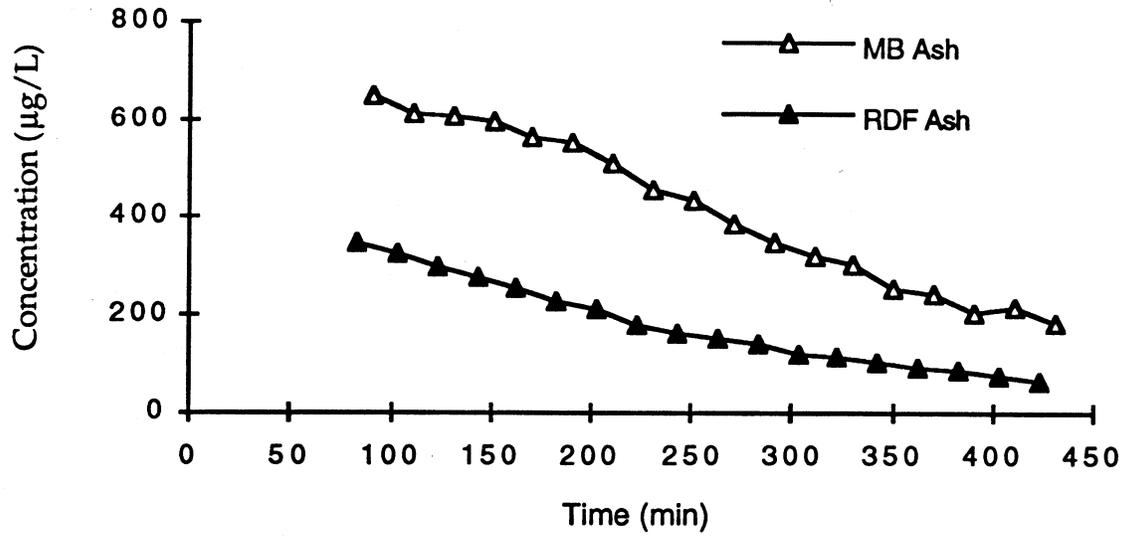


Figure 4.59 Variation in chromium leaching from 18-in columns subjected to SAR using Pinellas County MB ash and Palm Beach County RDF ash

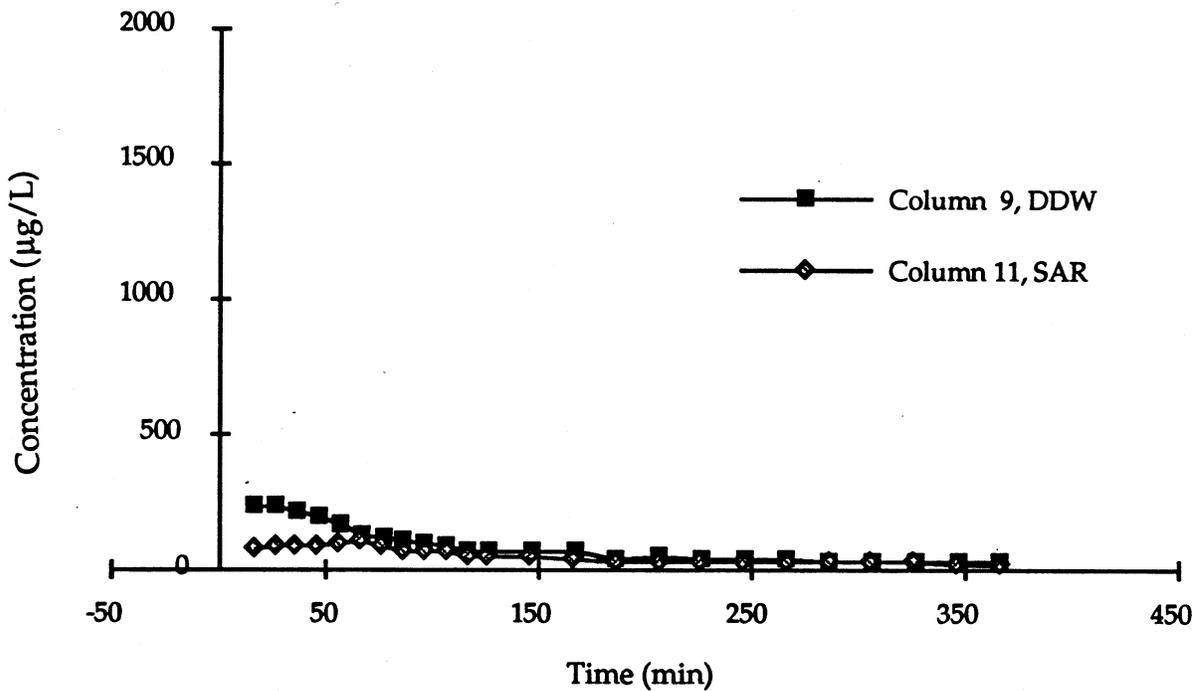


Figure 4.60 Leaching of lead using DDW and SAR as leaching fluids

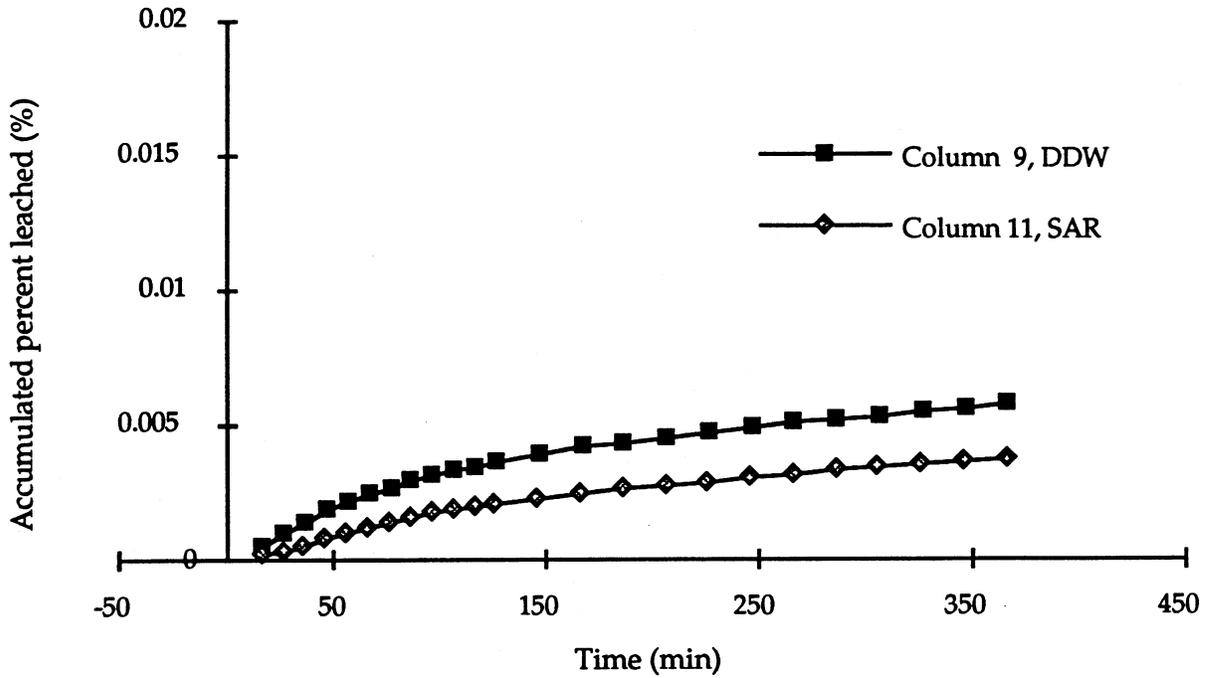


Figure 4.61 Accumulated percent leaching of lead using DDW and SAR as leaching fluids

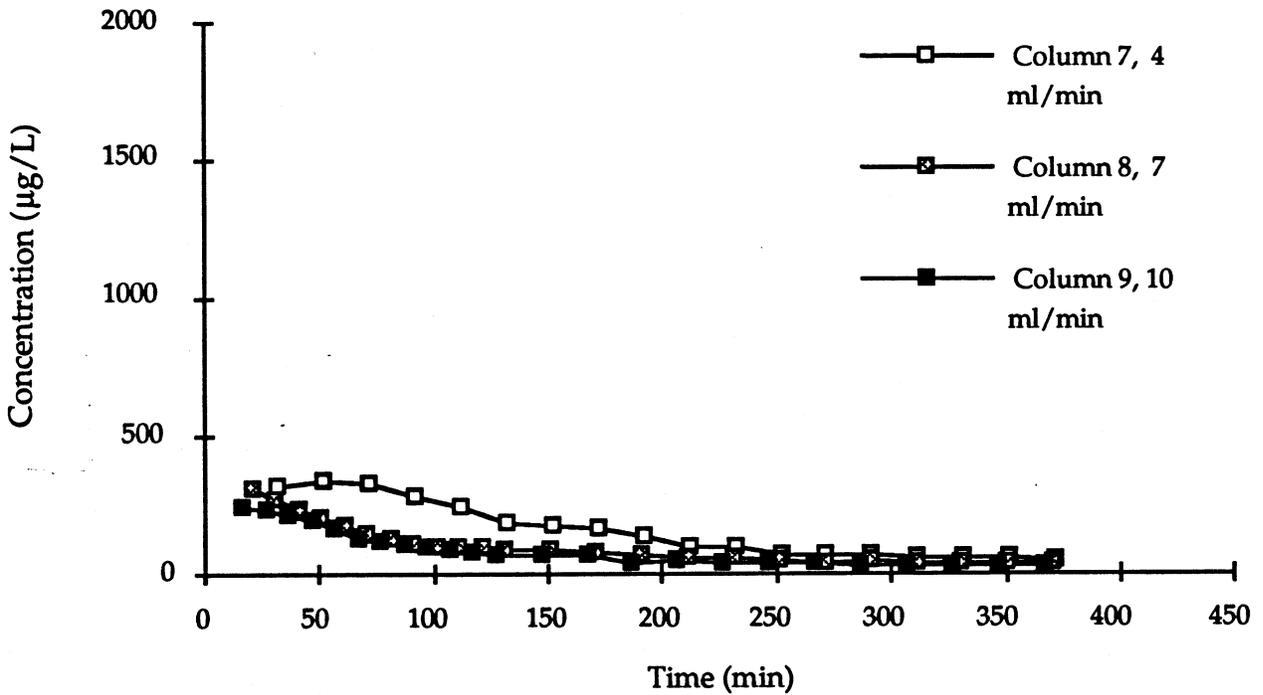


Figure 4.62 Leaching of lead using DDW at different flow rate

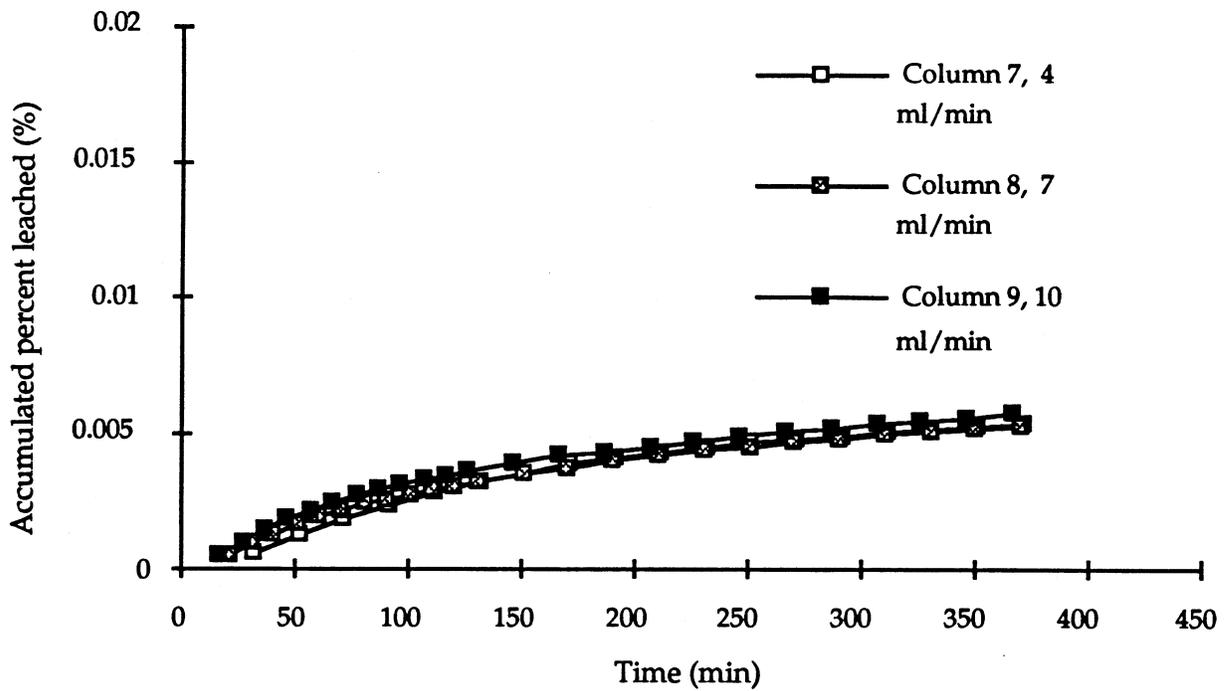


Figure 4.63 Accumulated percent leaching of lead using DDW at different flow rate

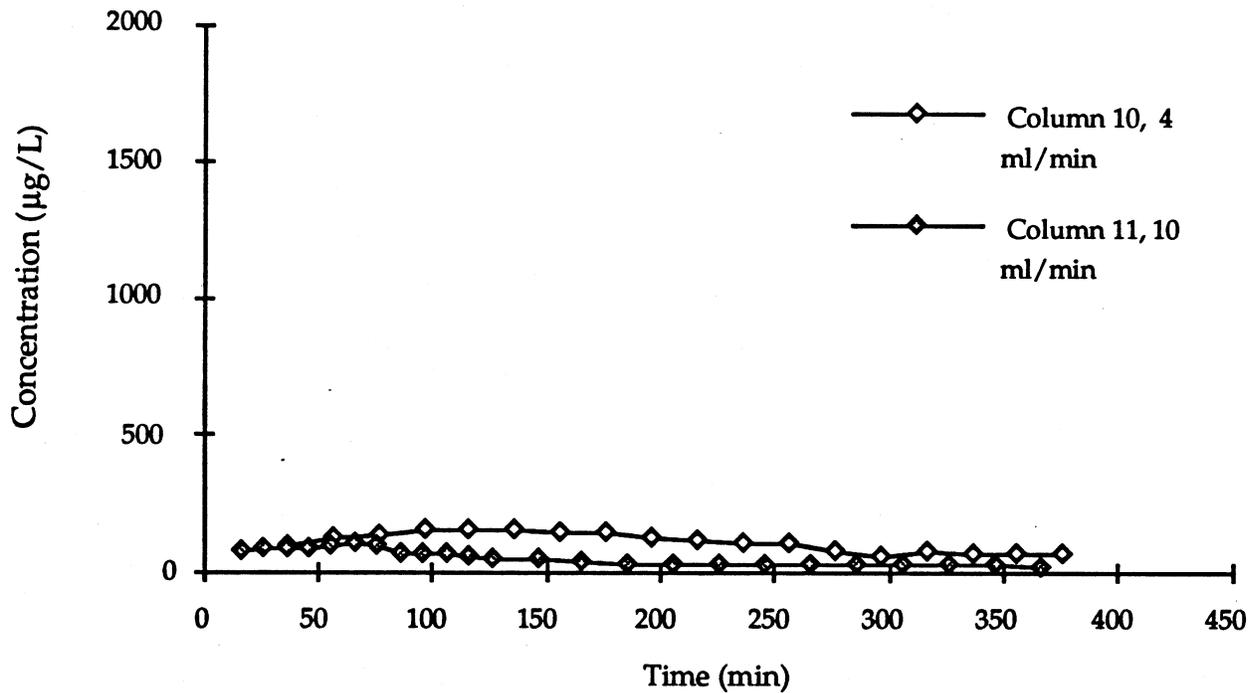


Figure 4.64 Leaching of lead using SAR at different flow rate

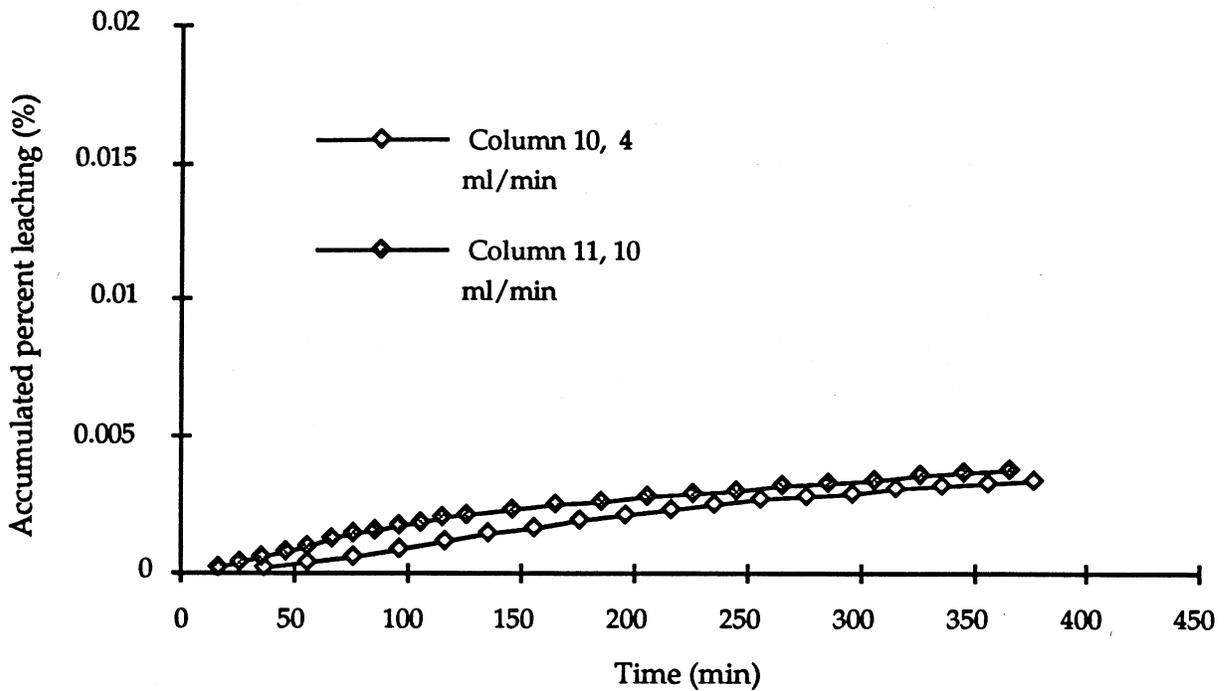


Figure 4.65 Accumulated percent leaching of lead using SAR at different flow rate

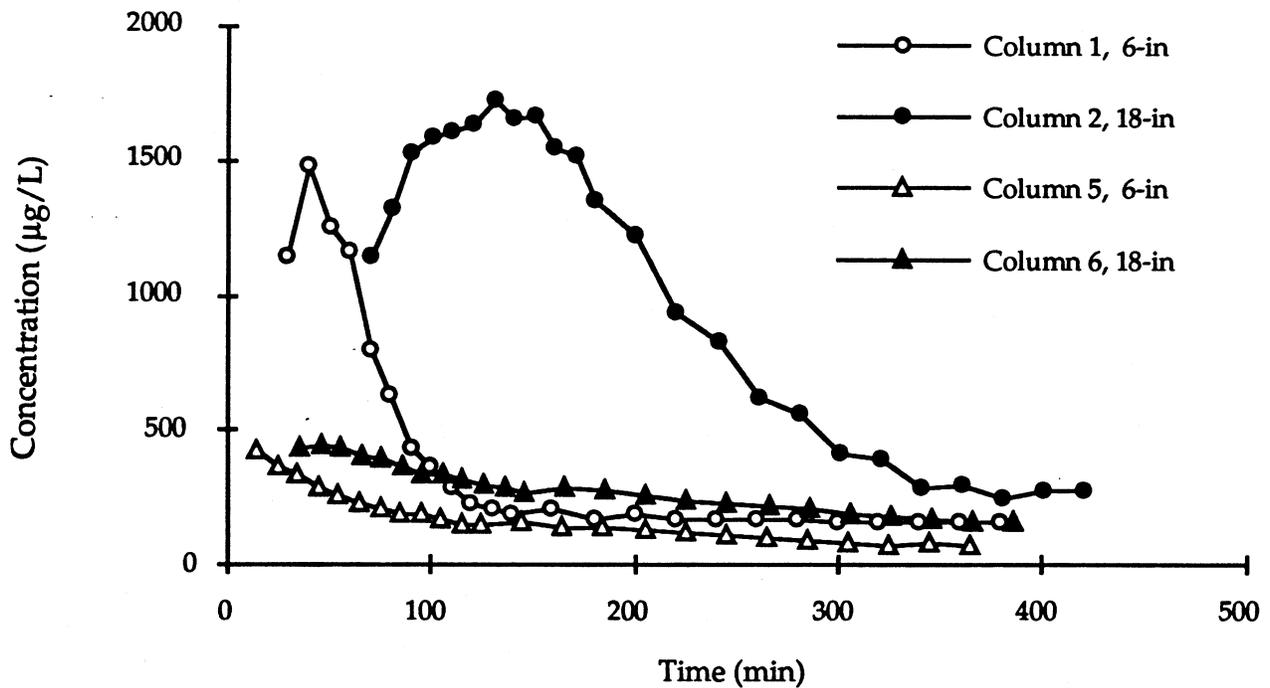


Figure 4.66 Variation in lead leaching from 15.2-cm and 45.7-cm columns subjected to DDW

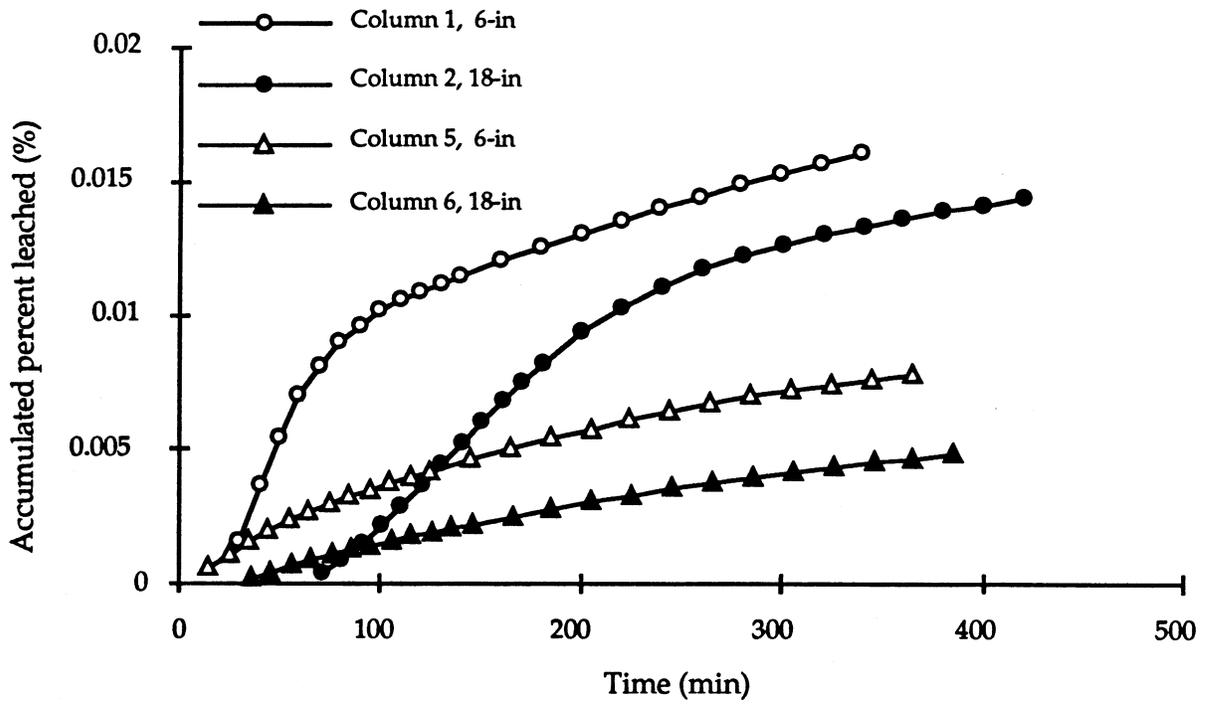


Figure 4.67 Accumulated percent of lead leached from 15.2-cm and 45.7-cm columns subjected to DDW

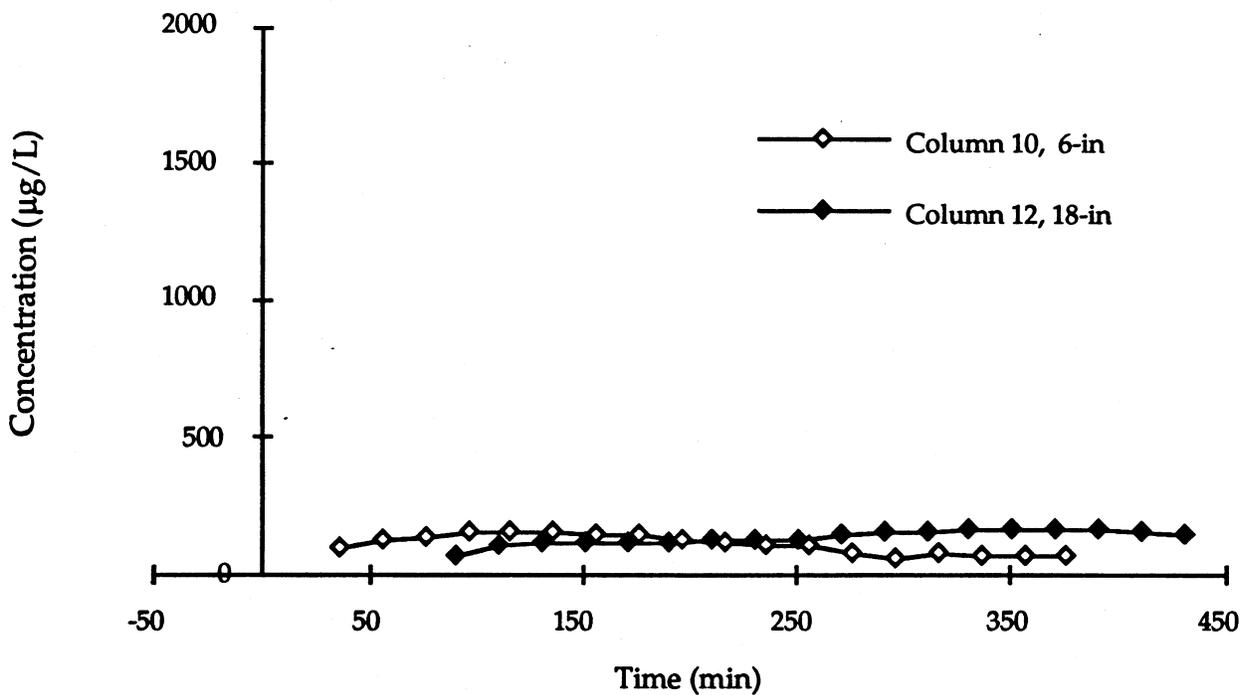


Figure 4.68 Variation in lead leaching from 15.2-cm and 45.7-cm columns subjected to SAR

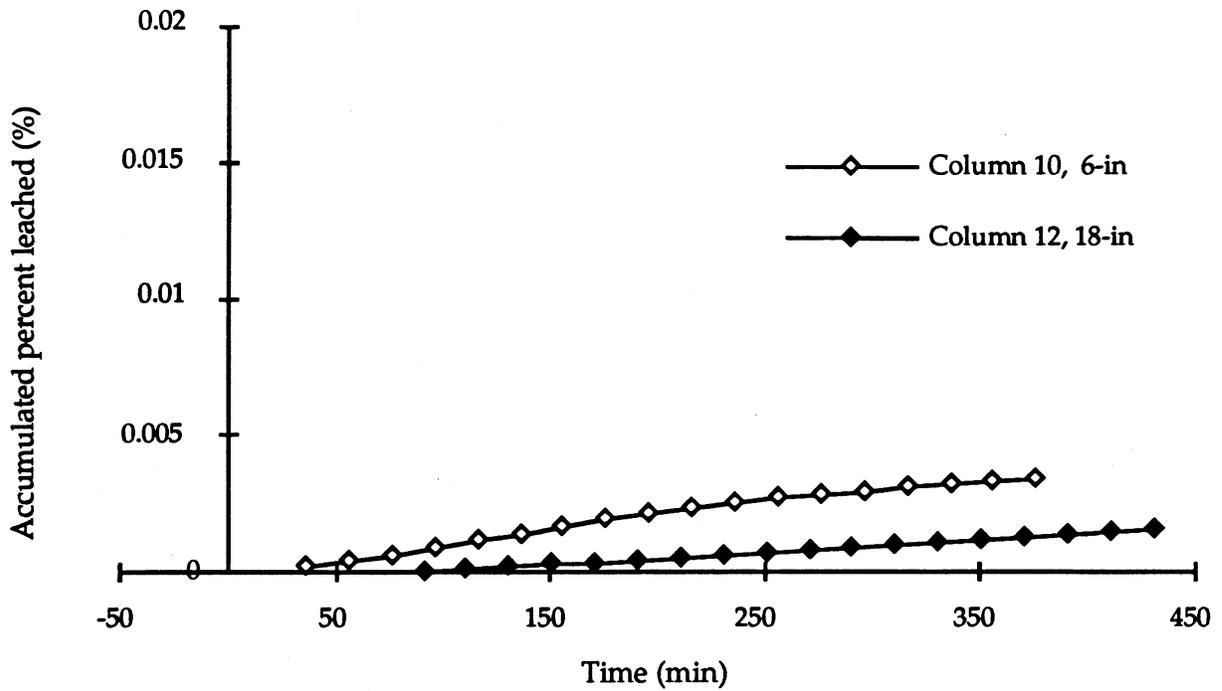


Figure 4.69 Accumulated percent of lead leached from 15.2-cm and 45.7-cm columns subjected to SAR

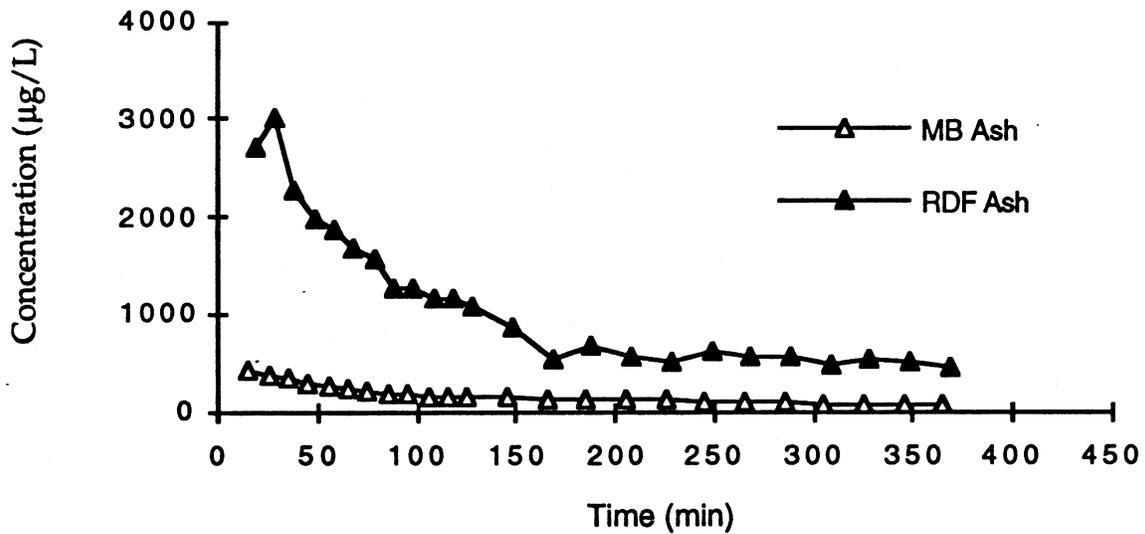


Figure 4.70 Variation in lead leaching from 6-in columns subjected to DDW using Pinellas County MB ash and Palm Beach County RDF ash

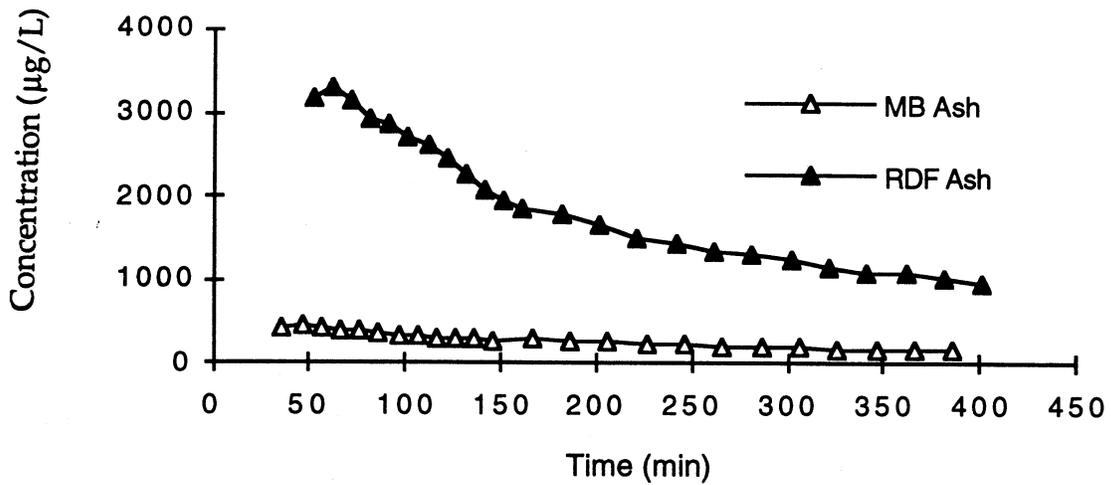


Figure 4.71 Variation in lead leaching from 18-in columns subjected to DDW using Pinellas County MB ash and Palm Beach County RDF ash

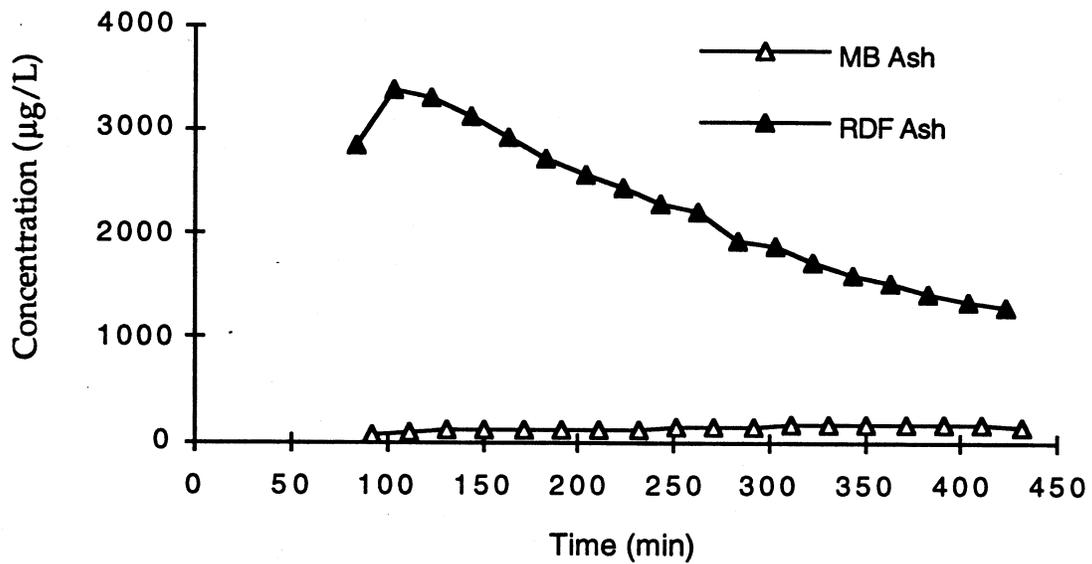


Figure 4.72. Variation in lead leaching from 18-in columns subjected to SAR using Pinellas County MB ash and Palm Beach County RDF ash

## 5.0 CONCLUSIONS

The results of this study show that MWC bottom ash has the physical and geotechnical properties necessary for application as a highway fill material and meets existing environmental acceptability regulations. As a result of various changes in ash management practice, a reliable cost estimate for the use of ash can not be developed at this time.

*The following specific findings support this conclusion:*

- Bottom ash meets the grain size classification criteria for use as a highway fill material. It is classified as A-1a with a group index of zero (predominately stone fragments and gravel size particles, with a well-graded binder of fine material) using the AASHTO classification. The grain size distribution of bottom ash samples showed little variability with time; suggesting that bottom ash can be obtained for use as a fill material at different times.
- MB bottom ash contained a higher percentage of metals than RDF bottom ash. The lower percentage of metals in the RDF bottom ash can be attributed to the processing of the MSW prior to combustion. The specific gravity of the bottom ash was found to be a function of metal content. MB bottom ash exhibited higher specific gravity than RDF bottom ash. A decrease in grain size resulted in a decrease in specific gravity for both ash types.
- The moisture-density relationships in bottom ash behave very similarly to conventional soils. MB bottom ash samples exhibited higher maximum dry densities and lower optimum moisture contents as compared to RDF bottom ash samples. A decrease in bottom ash grain size resulted in a decrease in maximum dry density and an increase in optimum moisture content. Increasing the compaction energy increased the maximum dry density but also decreased the optimum moisture content. The high absorption of water by the ash affects the moisture-density relationship, especially on the wet side of the optimum moisture content.

- The coefficient of permeability of freshly compacted bottom ash was found to be a function of the initial molding water content—with the highest permeability on the dry side of the optimum moisture content. The coefficient of permeability decreased significantly on the wet side of the optimum moisture content.
- Compacted bottom ash has a negligible shrinkage and swell potential when saturated. The ash swelled when allowed to air dry.
- The unconfined compressive strength of the compacted bottom ash behaves similarly to compacted soils in that the unconfined compressive strength is higher either dry or near optimum moisture content and increases with compaction energy. Thus, compaction moisture content and compaction energy are controlling factors for unconfined compressive strength. Allowing compacted bottom ash to age increases the compressive strength.
- The stress-strain curves of both ash types in loose and dense conditions behave similar to sand. Both ashes develop some cohesion which is attributed to pozzolanic cementing reactions occurring in the bottom ash. The angle of internal friction increases with the density of the compacted bottom ash.
- The elastic moduli for both ash types increase as the dry unit weight and confining pressures increase. The RDF bottom ash was found to be twice as stiff as the MB bottom ash. Both the MB and RDF bottom ashes exhibit elastic moduli within the range of loose sands.
- The CBR/LBR values of both ash types are very sensitive to the compaction moisture content of the sample. MB bottom ash exhibits unsoaked CBR/LBR values that are twice as large as the RDF bottom ash. MB bottom ash also exhibits higher unsoaked CBR/LBR values on the wet side of optimum while RDF bottom ash exhibits higher unsoaked CBR/LBR values on the dry side of optimum. A loss in strength occurs for both MB and RDF bottom ash after soaking.

- Concentrations of leaching of trace metals (Ag, As, Cd, Cr, and Pb) in the leachate from compacted ash columns subjected to DDW and SAR were below the EPA toxicity standard and decreased as a function of time. For Ag, As and Cd, most of the leachate concentrations were below the drinking water standard.
- The leaching of Ca increased when the rate of rainfall increased. However, the rate of rainfall had no effect on the release of Ag, As, Cd, Cr and Pb from the compacted ash columns. It is concluded that MWC bottom ash does not pose a threat to the environment even under extremely stormy conditions (i.e., 5 cm/hour).
- No special concern would be required if compacted MWC bottom ash were exposed to acid rain at pH around 4.

## 6.0 DEVELOPMENTAL SPECIFICATIONS FOR USING BOTTOM ASH IN HIGHWAY APPLICATIONS

*Based on the results from this study, the following developmental specifications are proposed. These specifications have been formatted to fit into the general section on Earthwork and Related Operations in " Standard Specifications for Road and Bridge Construction" (1991) from the Florida Department of Transportation. Section number 1890 was developed such that any new specification fuse of waste materials could be added at the end of the section as they were approved. For completeness of this report the description (Section 180-1) associated with this new section has been presented in both volumes of this final report. Section 180-3 Waste Glass is presented in Volume 2 of 2.*

### DEVELOPMENTAL SPECIFICATION SECTION 180

#### REUSE OF DISCARDED MATERIALS AND BYPRODUCTS

##### 180-1 Description

Discarded materials and byproducts shall consist, in general of municipal waste combustor bottom ash and waste glass generated from state mandated recycling quotas. The specification requirements for various discarded materials as contained in this Section are to govern their use only when these materials are used as a source of borrow material.

Sources of supply shall be approved by the Department.

##### 180-2 Municipal Waste Combustor Bottom Ash.

**180-2.1 Composition:** Bottom ash shall consist of the solid material remaining after combustion of municipal solid waste which is discharged from the grates or stoker of a solid waste combustor at a facility designed to combust waste for electric power generation. The facility shall exclude other combustion residues from being mixed with the bottom ash. The facility shall process the solid waste for metals recovery (using the best available technology) before combustion or process the bottom ash for metals recovery after combustion.

**180-2.2 Gradation:** Materials classified as bottom ash shall meet the following gradation requirements:

Passing the 3/4-inch sieve	Minimum 97%( max. dimension < 1-inch)
Passing the 3/8-inch sieve	Maximum 80 % (by weight)
Passing the No. 200 sieve	Maximum 5 % (by weight)

**180-2.3 Organic Content:** Bottom ash shall have a maximum loss on ignition of 6 %.

**180-2.4 Furnishing and Stockpiling:** All bottom ash shall be furnished for a specific project from one facility. The bottom ash shall be trammed through a 3/8-inch trammel screen and aged for a minimum of 60 days prior to use to allow aging reactions to occur.

**180-2.5 Physical Properties:** The dry rodded bulk unit weight (FM 1-T 019) for the bottom ash shall be greater than 65 pounds per cubic foot.

**180-2.6 Chemical Properties:** Concentrations of silver, arsenic, barium, selenium, cadmium, chromium, mercury and lead in the leachate from bottom ash shall be below the toxicity limits specified by the Environmental Protection Agency. In addition the contractor shall comply with regulatory issues of other environmental regulatory agencies.

**180-2.7 Construction Methods:** The contractor shall comply with construction methods specified in the DOT Standard Specifications for backfilling.

**180-2.7.1 Support of Vegetation:** Areas to be covered with grass shall be covered with a minimum thickness of twelve inches of topsoil over the bottom ash. For trees and shrubs, the depth of the topsoil shall be adjusted to accommodate the root system.

**180-2.7.2 Use with Metallic Construction:** Buried metallic materials such as culverts shall be coated with a bitumen or rubberized compound or separated with an inert borrow.

**180-2.7.3 Use with Concrete Construction:** Concrete structures constructed using Class I or Class III concrete having contact with bottom ash shall be coated with a bitumen or rubberized compound or separated with an inert borrow.

180-2.7.4 *Watertable*: Bottom ash shall be placed at a minimum of twelve inches above the top of the capillary zone.

**180-2.8 Safety and Health:** The contractor shall comply with the requirements of Section 7-1.4 of the Florida DOT Standard Specifications.

## 7.0 RECOMMENDATIONS

This research has yielded valuable information on the use of bottom ash in highway applications. The following continuation studies are suggested:

- Stockpile RDF and MB bottom ash and evaluate their *in-situ* environmental and geotechnical properties to ensure they are acceptable.
- Evaluate the variation in engineering and physical properties when RDF and MB bottom ash are combined with FDOT conventional base courses.
- Construct and monitor a field demonstration project using bottom ash as embankment, base and subbase material. The construction should be performed within a landfill to reduce permitting problems. Emphasis should be placed on construction methods and on how *in-situ* densities will be achieved. Data should be collected on engineering performance of the project and on leachate collected from the constructed site.
- Develop an *FDOT Users Manual* explaining when, where, and how waste materials can be used. This document will provide FDOT with needed information on current environmental concerns.
- Expand the current data base on the engineering and environmental characteristics of waste materials in highway applications.

## 8.0 REFERENCES

- Aleshin, E., and Bortz, S. A. (1976) "Aggregate Manufactured from Waste Materials." *Living with Marginal Aggregates, American Society for Testing Materials*, 85-96, Philadelphia, PA.
- Alter, H. and Dunn, J. J. (1980) *Solid Waste Conversion to Energy.*, Marcel Decker, Inc., New York, NY.
- ASTM, (1990) "American Society for Testing and Materials", *Annual Book of ASTM Standards* Vol. 4.02, C330-89, Standard Specification for Light Weight Aggregate for Structural Concrete, Philadelphia, Pa.
- ASTM, (1991) "American Society for Testing and Materials", *Annual Book of ASTM Standards*, Vol. 04.08, Philadelphia, PA.
- ASTM Standard Method (1989) "American Society for Testing and Materials", *Annual Book of ASTM Standards*, Vol. 04.08, American Society for Testing and Materials, Philadelphia.
- ASTM Standard Method (1989) "American Society for Testing and Materials", *Annual Book of ASTM Standards*, Vol. 11.04, American Society for Testing and Materials, Philadelphia.
- Baker, R. F. (1982) *Handbook of Highway Engineering*, Robert E. Krieger Publishing Company, Malabar, FL.
- Bishop, A. W. and Henkel, D. J. (1964) *Soil Properties in the Triaxial Test.*, Edward Arnold Publishers LTD, London.
- Bowles, J. E. (1992) *Engineering Properties of Soils and Their Measurement.*, 4th Edition, McGraw Hill, New York, NY.
- Brooks, R. R. (1978) "Pollution through Trace Elements", in *Environmental Chemistry*, edited by J. O. Bockris, Plenum Press, New York and London.
- Brunner, P. H. and Monch, H. (1986) "The Flux of Metals through Municipal Solid Waste Incinerators." *Waste Management and Research* , 4, 105-109.
- Campbell, W. J. (1976) "Metal in the Wastes We Burn?" *Environmental Science and Technology*, 10 436-439.

- Chavez, M. F. (1993) "Engineering Properties of Compacted Waste-To-Energy Ash", *M.S. Thesis in Civil Engineering.*, Florida Tech, Melbourne, FL.
- Chesner, W. H. (1993) "Working Towards Beneficial Use of Waste Combustor Ash" *Solid Waste & Power.*, September/October.
- Chesner, W. H. (1992) Personal communication on presto brick and block fabrication from WTE bottom ash.
- Chesner, W. H. (1989) "Aggregate-Related Characteristics of Municipal Solid Waste Combustion Residues", *Proceedings of the Second International Conference on Municipal Solid Waste Combustor Ash Utilization*, Arlington, VA.
- Clapp, T. L., Magee II, J. F., Ahlert, R. C. and D. S. Kosson (1988) "Municipal Solid Waste Composition and the Behavior of Metals in Incinerator Ashes", *Environmental Progress*, 7: 22-29.
- Collins, R. J. (1977) "Technology for Use of Incinerator Residue as Highway Material", *Report prepared for US DOT Federal Highway Administration*, September.
- Darcy, S. (1991) "Resource Recovery Firms Find Market Niches", *World Waste*, January, pp 34-41.
- Demars, K. R., Garrick, N. W., Long, R. P., Lentz, D., Maddali, R., Stephens, J. E. and C. Recchia (1994) "Municipal Waste Combustor Bottom Ash Road Paving and Structural Fill Demonstration Project." *Presented at 73rd Annual Transportation Research Board Meeting*. Washington, DC.
- Federal Register, (1986) Part 261-*Identification and Listing of Hazardous Waste*, Vol. 51, No. 114, June 13.
- Florida Administration Code (FAC) #17-702, (1991) Solid Waste Combustor Ash Management, Florida Department of Environmental Regulation.
- Florida Department of Environmental Regulation, (1992) *Solid Waste Management in Florida*, Bureau of Solid Waste and Hazardous Waste, pp. 69 + appendices.
- Florida Department of Transportation (FDOT), (1993) Florida Method of Test for Limerock Bearing Ratio. Designation: FM-5-515. September.

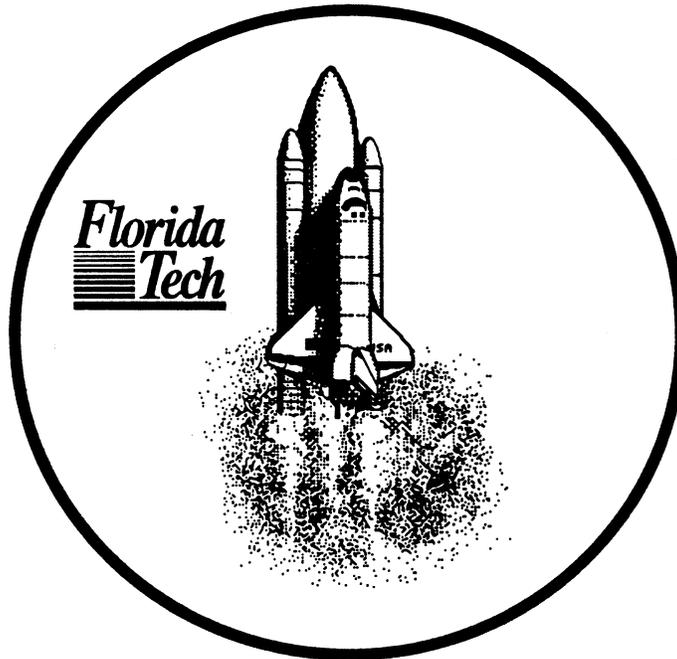
- Forrester, K. E. and Goodwin, R. W. (1990) MSW-Ash Field Study: Achieving Optimal Disposal Characteristics. *Journal of Environmental Engineering*, ASCE, Volume 116, No. 5 pp 880-889.
- GAI Consultants, Inc. (1979) "Fly Ash Structural Fill Handbook", *Research project prepared for Electric Power Research Institute*, December, Palo Alto, CA.
- Gidley, J. S. and Sack, W. A. (1984) "Environmental Aspects of Waste Utilization in Construction", *Journal of Environmental Engineering*, ASCE Vol. 110, No. 6, pp 1117-1133.
- Glenn, J. (1990) "The State of Garbage in America", *Biocycle*, March.
- Goldman, R. N. and Weinberg, J. S. (1986) *Statistics: An Introduction*, Prentice Hall Inc., New York.
- Hartlen, Jan. (1989) Swedish Geotechnical Institute, "Regulatory Aspects of Utilization of Ash in Sweden", *The Second International Conference on MSW Combustor Ash Utilization and Stabilization*, November 8-9, Washington, D. C.
- Hartlen, J. and Fallman, A.M. (1990) "Sorted Bottom Ash from MSW Incineration in Road Embankments", *Proceedings of the Third International Conference on MSW Combustion Ash Utilization*, November, Arlington, VA.
- HDR Engineering Inc. (1991) "Research, Development, and Application for Permit", *Report prepared for The Florida Department of Environmental Regulation, Tampa, Florida*, prepared by HDR Engineering, Inc., Tampa, Florida, and Florida Institute of Technology, Melbourne, FL.
- Hjelmar, O. (1990) "Regulatory and Environmental Aspects of MSWI Ash Utilization in Denmark", *The Third International Conference on Ash Utilization and Stabilization*, November, Arlington, VA.
- Hocking, M. B. (1975) "A Chemical Input-output Analysis of Municipal Solid Waste Incineration." *Journal of Environmental Systems*, 5.
- Holtz, R. D. and Kovacs, W. D. (1981) *An Introduction to Geotechnical Engineering*. Prentice Hall, New Jersey.
- Huang, W. H. and Lovell, C. W. (1990) "Bottom Ash As Embankment Material", *Geotechnics of Waste Fills - Theory and Practice*, ASTM STP 1070, pp 71 - 85.

- Huang, Y. H. 1993 "*Pavement Analysis and Design* ", Prentice Hall.
- Jain, M. (1992) "Waste-To-Energy Ash Concrete for Use as Building Construction Material", *M.S. Thesis in Environmental Engineering*, Florida Tech, Melbourne, FL.
- Jumikis, A. R. (1984) *Soil Mechanics*. Robert E. Krieger Publishing Company, Inc., Malabar, FL.
- Keith, T. R., Bielawski, G. T., Mazur, K. S. and R. Herrmann. (1990) "RDF-Fired Plant Aims for High Efficiency, Low Emissions." *Power*. April.
- Koppelman, L. E. (1990) "The Potential for Beneficial Use of Waste-to-Energy Facility Ash." Long Island Regional Planning Board, New York, NY.
- Lambe, T. W. (1969) "*Soil Mechanics*", John Wiley & Sons, New York.
- Lambe, T. W, and Whitman, R. V. (1979) "*Soil Mechanics*", John Wiley & Sons, New York, NY.
- Lauer, K. R. (1979) "Potential Use of Incineration Residue as Aggregate for Portland Cement Concrete." *Transportation Research Record*, April, pp 44-46.
- Law, S. L. and Gordon, G. E. "Sources of Metals on Municipal Incinerator Emissions." *Environmental Science and Technology*, 13, 432-438.
- Lisk, D. J. (1988) "Environmental Implications of Incineration of Municipal Solid Waste and Ash Disposal." *The Science of the Total Environment*, 74, 39-66.
- Merdes, R. S. (1990) "The Neutralysis System." *Proceedings of the Third International Conference on MSW Combustion Ash Utilization*, November, Arlington, VA.
- Nevin, Thomas J. (1992) "Effects on Properties by Partial Replacement of the Aggregate with Waste-To-Energy Ash", *M.S. Thesis in Civil Engineering*, Florida Tech, Melbourne, FL.
- New York City Department of Sanitation. (1988) "Assessment of the Potential Suitability of Southwest Brooklyn Incinerator Residue in Asphaltic Concrete Mixes", Report 90-15.

- NUS Corp., March (1990) "Characterization of Municipal Combustion Ash, Ash Extracts, and Leachates. Coalition on Resource Recovery and the Environment." *Special report prepared for the Environmental Protection Agency.*
- Pfeffer, John T. (1992) *Solid Waste Management Engineering*. Prentice Hall, Englewood Cliffs, NJ.
- Plumley, A. L. and Boley, G. L. (1990) *Proceedings of the Third International Conference on MSW Combustor Ash Utilization*, November, Arlington, VA.
- "Project Profile. 1990. " North County Resource Recovery and Solid Waste Disposal Facility." *Solid Waste Authority of Palm Beach County*. 3rd Edition. Palm Beach County, FL.
- Rodriguez, Alfonso Rico, Hermillo del Castillo, and George F. Sowers. *Soil Mechanics in Highway Engineering*. Germany: Trans Tech Publications, 1988.
- Roethel, F. J., Schaeperkoetter, V., Gregg, R. and Park, K. (1986) "The Fixation of Incinerator Residues", *Marine Sciences Research Center at State University of New York, Stony Brook, NY*.
- Roethel, F. J., and Breslin, V. 1990. "Incineration Residue Used to Build Boathouse", *Waste Management Research Report*, Summer.
- Sawhney, B. L. and Frink, C. R. (1991) "Heavy Metals and Their Leachability in Incinerator Ash", *Water, Air, and Soil Pollution*, 57-58: 289-296.
- SCS Engineers (1991) "Utilization Technologies for Municipal Waste Combustor Ash", *Reston Virginia and Science Applications International Corporation Report prepared for US EPA*, Washington, DC.
- Seals, R. K., Moulton, L. K. and Ruth, B. E. (1972) "Bottom Ash: An Engineering Material", *Journal of Geotechnical Engineering*, ASCE Vol. 98, pp 311 - 325.
- Seed, H. B., and Chan, C. K. (1959) "Structure and Strength Characteristics of Compacted Clays", *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 85, No. SM 5, pp 87-128, October .

- Shieh, C-S. (1994) *Effect of Aging on the Suitability of Municipal Waste Combustion Ash of Stabilization and Utilization*, Submitted to the Florida Center for Solid and Hazardous Waste Management, University of Florida, Gainesville, FL.
- Shively, W. , Bishop, P., Gress, D. and T. Brown (1986) "Leaching Tests of Heavy Metals Stabilized with Portland Cement", *Journal of Water Pollution Conference Federation*, 58: 234-241.
- Silberman, D. and Fisher, G. L. (1979) "Room-Temperature Dissolution of Coal Fly Ash for Trace Metal Analysis by Atomic Absorption Spectrometry", *Analytica Chimica Acta*, 106:299-307.
- Sneedon, Dr. Roy V. (1988) "Resilient Modulus Testing of 14 Nebraska Soils." *Prepared for Nebraska Department of Roads.*
- Snoeyink, V. L. and Jenkins, D. (1979) *Water Chemistry*, John Wiley & Sons, Inc., New York.
- Sridharan, A., Rao, A. S., and Sivapullaiah, P. V. (1986) "Swelling Pressures of Clays", *Geotechnical Testing Journal, GTJODJ*, Vol. 9, No. 1, pp 24-33.
- Stutzman, T. (1992) "Environmental Almanac." *World Resource Institute*. Boston MA.
- U. S. EPA, (1975) "Characterization and Utilization of Municipal and Utility Sludges and Ashes." *Vol. IV Municipal Incinerator Residues*. Government National Environmental Research Center, Cincinnati, OH.
- U.S. EPA, (1990) *Characterization of Municipal Waste Combustion Ash, Ash Extracts, and Leachates*, EPA 530-SW-90-029A, Washington D. C.
- U.S. Geological Survey, (1970) *The National Atlas of the United States of America*, Department of the Interior, Washington, D. C.
- Van Der Sloot, H. A., De Groot, G. J. and J. Wijkstra, (1989) "Leaching Characteristics of Incinerator Residues and Potential for Modification of Leaching", in *Proceedings of the International Conference on Municipal Waste Combustion*, April 11-14, Hollywood, Florida, 2B:1-19.

- Van Der Sloot, H. A. and De Groot, G. J. (1989) "Characterization of Municipal Solid Waste Incineration Residues for Utilization: Leaching Properties", *Proceedings of Ash Utilization and Stabilization Conference (Ash II)*, Washington, D. C.
- Van Der Sloot, Hans A. (1990) Soil and Waste Research, Netherlands Energy Research Foundation, "Personal Letter to Edward H. Kalajian and Iver W. Duedall,".
- Wheelabrator Environmental Systems Inc. (1991) *Q & A Issues Trash to Energy*. Wheelabrator Environmental Systems Inc., NH.
- Wilson, Stanley D. (1964) "Suggested Method of Test for Moisture-Density Relations of Soils using Harvard Miniature Compaction Apparatus." *ASTM Procedures for Testing Soils*. Fourth Edition.
- Wang, S. -I. (1994) "Environmental Acceptability of Using Municipal Waste Combuster Bottom Ash as Highway Fill Material." *M.S. Thesis in Environmental Science*. Florida Tech, Melbourne, FL.
- Winterkorn, Hans F. and Hsai-Yang Fang. *Foundation Engineering Handbook*. New York: Van Nostrand Reinhold Company, 1975.
- Wu Hsio-Chung (1990) "Engineering Properties of Waste-To-Energy Ash Concrete for Artificial Reef Construction", *M.S. Thesis in Civil Engineering*, Florida Tech, Melbourne, FL.
- Yoo, H. -Y. (1991) "Chemical Characterization of Waste-to-Energy Ashes and Stabilized Ash-Concrete", *M. S. Thesis*, Florida Institute of Technology, Melbourne, pp. 84.



Florida Institute of Technology  
150 W. University Blvd.  
Melbourne, Florida 32901-6988  
(407) 768-8000 (ext. 7555)

**Developing Specifications for Waste Glass and  
Waste-to-Energy Bottom Ash as Highway Fill  
Materials  
Volume 2 of 2 (Waste Glass)**

June 1, 1995

**Paul J. Cosentino Ph.D., P.E., Principal Investigator  
Edward H. Kalajian Ph.D., P.E., Co-Principal Investigator  
Howell H. Heck III Ph.D., P.E., Co-Principal Investigator  
Chih-Shin Shieh Ph.D., Co-Principal Investigator**

**Submitted to  
Robert K. H. Ho, Ph.D., P.E.  
Soils Materials Engineer  
State Materials Office  
Florida Department of Transportation  
2006 N.E. Waldo Road  
Gainesville, Florida 32609  
(904) 372-5304  
SunCom: 642-1206  
Fax: (904) 334-1648**

**WPA Item Number 0510650  
Contract Number B-7754**

1. Report No. FL/DOT/RMC/06650-7754		2. Government Accession No.		3. Recipient's Catalog No.	
4. Title and Subtitle DEVELOPING SPECIFICATIONS FOR WASTE GLASS AND WASTE-TO-ENERGY BOTTOM ASH AS HIGHWAY FILL MATERIALS Volume 2 of 2 (WASTE GLASS)			5. Report Date June 1995		
			6. Performing Organization Code		
			8. Performing Organization Report No.		
7. Author's P. J. Cosentino, E. H. Kalajian, C-S. Shieh and H. H. Heck			10. Work Unit No. (TRAIS)		
9. Performing Organization Name and Address Florida Institute of Technology (407) 768-8000 (ext. 7555) Civil Engineering Program 150 West University Blvd. Melbourne, FL 32901-6988			11. Contract or Grant No. C-7754 WPA # 0510650		
			13. Type of Report and Period Covered Final Report October 1992 to October 1994		
12. Sponsoring Agency Name and Address Florida Department of Transportation 605 Suwannee Street Tallahassee, Florida 32399-0450			14. Sponsoring Agency Code 99700-7587-119		
			15. Supplementary Notes		
16. Abstract  <p>Waste glass (WG), consisting of the non-recyclable portion of disposed glass, from typical sources in Florida was evaluated to determine potential applications for highway fill material. WG has the physical and engineering properties necessary for use in many highway applications. It can be safely handled when sized to meet ASTM D 448 number 8 or finer material, it is suitable for use as a drainage material, it has excellent frictional characteristics and its grains do not break down when subjected to high confined compression. WG has relatively low bearing ratios (i.e., CBR and LBR) and is therefore, not currently recommended for use in base/subbase applications. The geotechnical engineering properties of WG have been evaluated and are summarized.</p> <p>WG used as a fill material is considered clean debris and requires no special permits or regulatory involvement. However, WG is contaminated with soluble organics that must be washed from the material before use. The shake extraction procedure typically used to determine organic pollutant levels must be modified by using a 1:1 volumetric ratio of glass to water to achieve applicable results.</p> <p>Based on the study findings developmental specifications have been proposed for using WG in highway applications. They will be incorporated into Standard Specifications for Road and Bridge Construction, Florida Department of Transportation, 1991.</p>					
17. Key Words Waste Glass, Highway Applications Geotechnical Properties, Leaching Properties			18. Distribution Statement Document is available to the U.S. public through the National Technical Information Service, Springfield, Virginia 22161		
19. Security Classif. (of this report) Unclassified		20. Security Classif. (of this page) Unclassified		21. No of Pages 149	22. Price

# METRIC (SI\*) CONVERSION FACTORS

## APPROXIMATE CONVERSIONS TO SI UNITS

Symbol When You Know Multiply By To Find Symbol

### LENGTH

in	inches	2.54	millimetres	mm
ft	feet	0.3048	metres	m
yd	yards	0.914	metres	m
mi	miles	1.61	kilometres	km

### AREA

in <sup>2</sup>	square inches	645.2	millimetres squared	mm <sup>2</sup>
ft <sup>2</sup>	square feet	0.0929	metres squared	m <sup>2</sup>
yd <sup>2</sup>	square yards	0.836	metres squared	m <sup>2</sup>
mi <sup>2</sup>	square miles	2.59	kilometres squared	km <sup>2</sup>
ac	acres	0.395	hectares	ha

### MASS (weight)

oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg

### VOLUME

fl oz	fluid ounces	29.57	millilitres	mL
gal	gallons	3.785	litres	L
ft <sup>3</sup>	cubic feet	0.0328	metres cubed	m <sup>3</sup>
yd <sup>3</sup>	cubic yards	0.0765	metres cubed	m <sup>3</sup>

NOTE: Volumes greater than 1000 L shall be shown in m<sup>3</sup>.

### TEMPERATURE (exact)

°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C
----	------------------------	----------------------------	---------------------	----

\* SI is the symbol for the International System of Measurements

## APPROXIMATE CONVERSIONS TO SI UNITS

Symbol When You Know Multiply By To Find Symbol

### LENGTH

mm	millimetres	0.039	inches	in
m	metres	3.28	feet	ft
m	metres	1.09	yards	yd
km	kilometres	0.621	miles	mi

### AREA

mm <sup>2</sup>	millimetres squared	0.0016	square inches	in <sup>2</sup>
m <sup>2</sup>	metres squared	10.764	square feet	ft <sup>2</sup>
km <sup>2</sup>	kilometres squared	0.39	square miles	mi <sup>2</sup>
ha	hectares (10 000 m <sup>2</sup> )	2.53	acres	ac

### MASS (weight)

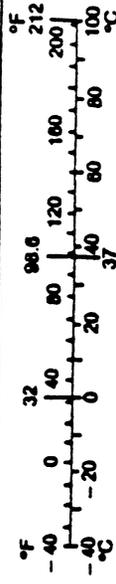
g	grams	0.0353	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams (1 000 kg)	1.103	short tons	T

### VOLUME

mL	millilitres	0.034	fluid ounces	fl oz
L	litres	0.264	gallons	gal
m <sup>3</sup>	metres cubed	35.315	cubic feet	ft <sup>3</sup>
m <sup>3</sup>	metres cubed	1.308	cubic yards	yd <sup>3</sup>

### TEMPERATURE (exact)

°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F
----	---------------------	-------------------	------------------------	----



These factors conform to the requirement of FHWA Order 5180.1A.

## Table of Contents

1.0 INTRODUCTION .....	1
1.1 Waste Glass (WG) Quantities .....	1
1.2 WG Production .....	2
1.3 WG Disposal Techniques .....	3
1.4 Secondary WG Uses.....	3
1.5 Objectives .....	4
2.0 PREVIOUS WG INVESTIGATIONS .....	5
2.1 Environmental/chemical investigations .....	5
2.1.1 Clean Washington Center .....	5
2.1.1.a Contaminant Leaching .....	5
2.1.1.b Health Evaluation .....	6
2.2 Environmental Regulations .....	6
2.3 Engineering Investigations .....	7
2.3.1 WG Properties .....	7
2.3.2 Permeability .....	7
2.3.3 Grain Size Distribution .....	10
2.3.4 Specific Yield .....	10
2.3.5 Relative Density.....	11
2.3.6 Specific Gravity .....	12
2.3.7 Strength and Deformation Characteristics of Aggregates ...	12
2.3.7.a Friction Angle .....	13
2.3.7.b Modulus of Elasticity.....	14
2.3.7.c California Bearing Ratio (CBR).....	15
2.3.7.d Limerock Bearing Ratio (LBR) .....	16
2.3.8 Using WG in Bound Pavement Layers .....	16
2.4 Existing Highway Applications And Specifications .....	17
3.0 METHODOLOGY AND EXPERIMENTAL PROCEDURES .....	26
3.1 WG Sources .....	26
3.1.1 Southeast Recycling WG .....	27
3.1.2 West Palm Beach WG .....	27
3.1.3 WG Survey Development .....	28
3.2 Visual Classification.....	28
3.3 Grain Size Distribution Analysis .....	29
3.4 Sample Preparation .....	29
3.5 Engineering Testing .....	30
3.5.1 Density .....	30
3.5.1.a Minimum Density Testing Procedure .....	31
3.5.1.b Maximum Density Testing Procedure .....	31
3.5.1.c Densities for Drainage Testing .....	32
3.5.1.d Densities for Shear Strength and Deformation Testing.....	33
3.5.2 Permeability .....	33
3.5.2.a Standard Constant Head Permeability Testing .....	33

## Table of Contents (cont.)

3.5.2.b Preliminary Testing Procedure .....	34
3.5.2.c Validation of Standard Permeability Testing .....	34
3.5.2.d Modification of the Permeameter .....	35
3.5.2.e Final Testing Procedure.....	35
3.5.3 Specific Yield .....	36
3.5.3.a Description of Specific Yield Apparatus.....	36
3.5.3.b Testing Procedure for Specific Yield .....	37
3.5.4 Specific Gravity .....	37
3.5.5 Confined Compression Testing.....	38
3.5.6 Direct Shear Testing .....	39
3.5.7 Triaxial Shear Testing .....	40
3.5.8 California Bearing Ratio (CBR) Testing .....	41
3.5.9 Limerock Bearing Ratio (LBR) Testing.....	43
3.6 Environmental Testing .....	43
3.6.1 Column Leaching .....	43
3.6.2 Biochemical Oxygen Demand (BOD) .....	44
3.6.3 Dissolved Oxygen (DO) .....	44
3.6.4 Total Phosphorous (TP).....	45
3.6.5 Solids .....	45
3.6.5.a Total Dissolved Solids (TDS).....	45
3.6.5.b Total Suspended Solids (TSS).....	45
3.6.5.c Fixed and Volatile Suspended Solids .....	46
3.6.6 Total Kjeldahl Nitrogen (TKN) .....	46
4.0 PHYSICAL, GEOTECHNICAL AND ENVIRONMENTAL PROPERTIES ..	57
4.1 Visual Classification.....	57
4.2 Grain Size Distribution .....	58
4.3 Specific Gravity .....	59
4.4 Density .....	59
4.5 Permeability .....	61
4.5.1 Relationship between Permeability and Density .....	61
4.5.2 Relationship between Permeability and Gradation .....	62
4.5.3 Relationship between Permeability Coefficient and Effective Grain Size (D <sub>10</sub> ).....	63
4.5.4 Relationship between Permeability and Void Ratio (e) .....	63
4.6 Specific Yield .....	64
4.6.1 Relationship between Specific Yield and Gradations .....	65
4.6.2 Relationship between Specific Yield and Porosity .....	65
4.6.3 Relationship between Specific Yield and Permeability .....	66
4.7 Drainage Analysis .....	66
4.8 Confined Compression.....	68
4.9 Direct Shear .....	69
4.10 Triaxial Shear .....	70
4.11 CBR/LBR .....	71

**Table of Contents (cont.)**

4.12 Environmental WG Analysis .....	71
4.12.1 Availability of WG .....	71
4.12.2 Method of Leachate Generation .....	71
4.12.3 Biochemical Oxygen Demand (BOD) .....	73
4.12.4 Total Kjeldahl Nitrogen (TKN) .....	74
4.12.5 Total Phosphorous (TP) .....	75
4.12.6 Solids .....	75
4.12.7 Rate Constants .....	75
4.12.8 Mass of Pollutants Released .....	76
4.12.9 Comparison to Shake Extraction .....	76
4.13 WG Data Base .....	77
4.14 WG Materials Cost .....	78
5.0 CONCLUSIONS FOR WG HIGHWAY APPLICATIONS .....	128
6.0 DEVELOPMENTAL SPECIFICATIONS FOR WG HIGHWAY APPLICATIONS .....	131
7.0 RECOMMENDATIONS FOR WG HIGHWAY APPLICATIONS .....	133
8.0 REFERENCES .....	136

## List of Tables

Table 2.1 Chemical characterization-sequential batch extraction glass feedstock samples (from Dames and Moore 1993) .....	20
Table 2.2 Crystalline silica and dust results (after Dames and Moore, 1993) .....	21
Table 2.3 WG Permeability and dry density (after Dames and Moore, 1993) ...	22
Table 2.4 Typical values of $\phi$ for aggregate (from Das, 1983) .....	23
Table 2.5 Young's modulus (E) for initial loading from zero to one-half peak deviator stress (from Das, 1983) .....	24
Table 2.6 CBR values of compacted soils (National Highway Institute, 1990) ..	25
Table 3.1 Standard sizes of selected road aggregates (ASTM D 448) .....	47
Table 4.1 Thicknesses of recycled glass bottle fragments .....	79
Table 4.2 WG highway classifications .....	80
Table 4.3 Maximum and minimum densities for WG meeting ASTM #8, #9, #10 and WPBMRF gradations .....	81
Table 4.4 Range of permeabilities for WG meeting ASTM D 448 #8, #9, #10 and WPBMRF gradations .....	82
Table 4.5 Range of specific yield for WG meeting ASTM D 448 gradations .....	83
Table 4.6 Results of WG drainage analysis.....	84
Table 4.7 Direct shear friction angles for various WG gradations and densities	85
Table 4.8 Summary of CBR/LBR testing .....	86
Table 4.9 Summary of WG surveys .....	87
Table 4.9 Summary of WG surveys (continued) .....	88
Table 4.9 Summary of WG surveys (continued) .....	89
Table 4.10 2 Ft. Leaching column results for BSMG.....	90
Table 4.11 4 Ft. Leaching column results for BSMG.....	91
Table 4.12 6 Ft. Leaching column results for BSMG.....	92
Table 4.13 2 Ft. Leaching column results for WPBMRF WG .....	93
Table 4.14 4 Ft. Leaching column results for WPBMRF WG .....	94
Table 4.15 6 Ft. Leaching column results for WPBMRF WG .....	95
Table 4.16 Regression analysis results for BOD5 and TP .....	96
Table 4.17 Total mass of BOD and TP released during column leaching .....	97
Table 4.18 Shake extraction results .....	98
Table 4.19 Data base of physical and engineering WG properties .....	99

## List of Figures

Figure 3.1 WG survey form .....	48
Figure 3.1 WG survey form (cont.) .....	49
Figure 3.2 Schematic of the standard permeability test apparatus (1 in = 25.4 mm) .....	50
Figure 3.3 Schematic of the modified permeability test apparatus (1 in = 25.4 mm) .....	51
Figure 3.4 Schematic of piezometer inlet within permeameter mold(1 in = 25.4 mm) .....	52
Figure 3.5 Schematic of specific yield test apparatus (1 in = 25.4 mm) .....	53
Figure 3.6 Mohr circles for preliminary WPBMRF gradation triaxial testing (1 psi = 6.895 kPa) .....	54
Figure 3.7 Schematic of column leaching apparatus (from Wang, 1993) .....	55
Figure 4.1 WG grain size distributions for WPBMRF and BSMG.....	100
Figure 4.2 WG grain size distribution for ASTM D 448 coarse aggregate Designations #8, #9, #10, and WPBMRF .....	101
Figure 4.3 Relationship between density and permeability for WG meeting ASTM D 448 #8, #9, #10, and WPBMRF gradations .....	102
Figure 4.4 WG permeability and D10 (mm) relationship compared to Hazens' empirical formula with $C_u < 5$ and $0.1 < D_{10} \text{ (mm)} < 3$ .....	103
Figure 4.5 Relationship between $e_2$ and coefficient of permeability for WG meeting ASTM #8 gradation .....	104
Figure 4.6 Relationship between $e_2$ and coefficient of permeability for WG meeting ASTM D 448 #9 gradation .....	105
Figure 4.7 Relationship between $e_2$ and coefficient of permeability for WG meeting ASTM D 448 #10 lower and WPBMRF gradations .....	106
Figure 4.8 Relationship between $e_2$ and coefficient of permeability for WG meeting ASTM D 448 #10 average and upper limit gradations .....	107
Figure 4.9 Relationship between density and specific yield for WG meeting ASTM D 448 #8, #9, #10, and WPBMRF gradations .....	108
Figure 4.10 Relationship between porosity and specific yield for WG meeting ASTM D 448 #8, #9, #10, and WPBMRF gradations .....	109
Figure 4.11 Relationship between permeability and specific yield for WG meeting ASTM D 448 #8, #9, #10, and WPBMRF gradations .....	110
Figure 4.12 Typical confined compression results for mixed cullet meeting WPBMRF .....	111
Figure 4.13 Nonlinear Mohr-Coulomb failure typical of WG direct shear tests. ....	112
Figure 4.14 Typical vertical stress versus vertical strain and volumetric strain versus vertical strain plots for WG triaxial shear testing. ....	113
Figure 4.15a Variation of friction angle with density and strain level during CD triaxial testing on mixed cullet meeting WPBMRF gradation (1 psi = 7 kPa) .....	114
Figure 4.15b Variation of friction angle with density and strain level during CD triaxial testing on mixed cullet meeting ASTM # 8 average gradation (1 psi = 7 kPa) .....	115

## List of Figures (cont.)

Figure 4.15c Variation of friction angle with density and strain level during CD triaxial testing on mixed cullet meeting ASTM # 9 average gradation (1 psi = 7 kPa) .....	116
Figure 4.15d Variation of friction angle with density and strain level during CD triaxial testing on mixed cullet meeting ASTM # 10 average gradation (1 psi = 7 kPa) .....	117
Figure 4.16a Variation of secant modulus with density and strain level during CD triaxial testing on mixed cullet meeting WPBMRF gradation (1 psi = 7 kPa) .....	118
Figure 4.16b Variation of secant modulus with density and strain level during CD triaxial testing on mixed cullet meeting ASTM # 8 average gradation (1 psi = 7 kPa) .....	119
Figure 4.16c Variation of secant modulus with density and strain level during CD triaxial testing on mixed cullet meeting ASTM # 9 average gradation (1 psi = 7 kPa) .....	120
Figure 4.16d Variation of secant modulus with density and strain level during CD triaxial testing on mixed cullet meeting ASTM # 10 average gradation (1 psi = 7 kPa) .....	121
Figure 4.17 BSMG leaching column BOD results .....	122
Figure 4.18 BSMG leaching column TP results .....	123
Figure 4.19 BSMG leaching column TKN results .....	124
Figure 4.20 WPBMRF leaching column BOD results .....	125
Figure 4.21 WPBMRF leaching column TKN results .....	126
Figure 4.22 WPBMRF leaching column TP results .....	127

## List of Photographs

Photograph 3.1 Typical BSMG .....	56
Photograph 3.2 Typical WPBMRF waste glass .....	56

## ACKNOWLEDGMENTS

The researchers would like to thank the Florida Department of Transportation for funding this research under contract number C-7754. Additionally, our gratitude goes out to the Southeast Recycling and Palm Beach Counties recycling facilities. The assistance of Dr. Robert K. H. Ho throughout the study was invaluable. The diligent efforts of the following graduate and undergraduate students made timely completion of this project possible: Mario Chavez, Shakeel Syed, Shih-I Wang, Keith Guthrie, Gudny Palsdottir, Massimo Bosso, Rujroj Waiwudhi, Deborah Pandeline, Jennifer Benaman, Daniel Wintermeyer and Dorothy Rhine.

## ABSTRACT

Waste glass (WG), consisting of the non-recyclable portion of disposed glass, from typical sources in Florida was evaluated to determine potential applications for highway fill material. WG has the physical and engineering properties necessary for use in many highway applications. It can be safely handled when sized to meet ASTM D 448 number 8 or finer material, it is suitable for use as a drainage material, it has excellent frictional characteristics and its grains do not break down when subjected to high confined compression. WG has relatively low bearing ratios (i.e., CBR and LBR) and is therefore, not currently recommended for use in base/subbase applications. The geotechnical engineering properties of WG have been evaluated and are summarized.

WG used as a fill material is considered clean debris and requires no special permits or regulatory involvement. However, WG is contaminated with soluble organics that must be washed from the material before use. The shake extraction procedure typically used to determine organic pollutant levels must be modified by using a 1:1 volumetric ratio of glass to water to achieve applicable results.

Based on the study findings developmental specifications have been proposed for using WG in highway applications. They will be incorporated into Standard Specifications for Road and Bridge Construction, Florida Department of Transportation, 1991.

## 1.0 INTRODUCTION

### 1.1 Waste Glass (WG) Quantities

About 200 million tons of municipal solid waste (MSW) is generated yearly in the United States. Florida produced about 20 million tons of MSW in 1993, with about 5.4 million or 27 percent being recycled. Roughly 7 percent (by weight) of the municipal waste stream is glass (U.S. EPA, 1992), while slightly more than 11 million tons of glass containers were sold in 1993 (Glass Packaging Institute, 1993).

According to the Glass Packaging Institute (1993) 35 percent of all glass containers were recycled, with the total glass recycled being 3.8 million tons. A 1992 estimate indicates that Florida generated approximately 582,000 tons of glass out of which 128,000 was recycled, thereby, matching its typical recycling rate of between 20 and 25 percent (Kynes, 1994). The construction industry in Florida uses over 18 million tons of sand and gravel yearly (U.S. Dept. of Interior, 1990) implying that all of the waste glass produced yearly could be utilized by the construction industry.

Waste glass (WG) is generated by most municipalities within Florida due to state mandated recycling quotas. It is defined as the non-recyclable portion of disposed glass. Very few glass recycling facilities exist in Florida, consequently most green and amber glass is recycled out of state. If a shipment is overly contaminated, a recycling facility cannot economically use the WG and will consequently refuse the shipment (Institute of Scrap Recycling Industries, 1994). Transportation costs for recycled glass range from \$1.00 to \$1.50 per mile and the glass is worth \$15 to \$50 per ton upon delivery to a recycling center (Heck et al., 1989).

## 1.2 WG Production

Recycling has become increasingly important as one of today's strategies for solid waste reduction. The practice of recycling glass reduces the quantity of glass disposed of in landfills thereby saving significant landfill space. Approximately 69% of all glass bottles can not be economically recycled due to the contamination resulting from the mixture of amber, flint, and green glass. This nonrecyclable portion is referred to as WG or mixed cullet. The use of WG as an aggregate helps to conserve the naturally occurring aggregates. In aggregate markets, WG would compete with materials ranging from \$5 to \$10 per ton. According to a report released by the Clean Washington Center, processing glass as an aggregate feedstock costs between \$7 and \$12 per ton, while sorting glass for the bottle market can cost between \$20 and \$50 per ton (Dames & Moore, 1993). Highway applications have been proposed where 100% cullet can safely be used. Additional applications that require a mixture of cullet with natural aggregates have also been proposed. According to the Clean Washington Center report, it is less expensive to collect and process glass for recycling than it is to landfill it (Dames & Moore, 1993).

To be recycled curbside, glass must be separated by color into flint, amber, and green. This separation insures color consistency when new containers are manufactured. Current specifications require separated glass to be relatively free of contamination—for example, specifications published by the Institute of Scrap Recycling Industries require furnace ready flint glass to be 95% pure (1994). However, at present no technology exists which can efficiently color sort glass, although research in this area continues (Glass Packaging Institute, 1993).

### 1.3 WG Disposal Techniques

Glass is an interesting component of waste; it will not burn, rust, or decay. The current disposal method for glass is limited to sanitary landfilling. Economic considerations have generally dictated whether or not salvage and reclamation operations are feasible—and in the past, processing and transportation costs have generally ruled out this opportunity (Malisch, *et al.*, 1970). In some areas, sanitary landfills cannot provide a solution because of the lack of available space. As landfills are moved farther from urban centers, transportation costs make them less feasible. The principle of volume also causes problems in glass disposal: bottles take up excessive space unless they are thoroughly crushed.

### 1.4 Secondary WG Uses

A growing number of secondary uses are emerging for WG. Uncontaminated WG is used to a limited degree in the fiber glass insulation industry while clear glass is used in the production of glass beads and reflective paints (Menges, 1990). It can also be used as an aggregate in a form of asphalt known as "glasphalt" although problems such as stripping of the binder from the glass may occur (Malisch, *et al.*, 1970) (Hughes, 1990). It can be used as a replacement for gravel and crushed stone in road base construction, pipe backfill and storm drains (Glass Packaging Institute, 1993). In the U.S., approximately 575,000 tons per year of recycled glass is used in these secondary applications (Glass Packaging Institute, 1993). However, a recent survey of Florida's largest 30 counties, revealed that there were no cullet stockpiles.

## 1.5 Objectives

To meet the overall project goal of developing specifications for the Florida Department of Transportation (FDOT) for utilizing WG as a highway fill material the following individual objectives were set for the WG research.

1. To determine the availability of WG to be used as fill material in Florida.
2. To determine the contamination of WG leachate in terms of biochemical oxygen demand (BOD), total phosphorous (TP), total kjeldahl nitrogen (TKN), and solids content.
3. To develop a method for the determining contamination within WG samples.
3. To determine the highway drainage classifications of WG.
4. To determine the drainage properties of WG thereby establishing relationships between gradations, density, void ratio, permeability, and specific yield.
5. To determine the stress-strain and shearing characteristics necessary for using WG as highway fill material.
6. To develop specifications for using WG as highway fill material.

## 2.0 PREVIOUS WG INVESTIGATIONS

### 2.1 Environmental/chemical investigations

#### 2.1.1 Clean Washington Center

Dames & Moore, (1993) conducted a study for the Clean Washington Center, on the feasibility of using mixed cullet as a construction aggregate. The Clean Washington Center is a division of the Department of Trade and Economic Development. The purpose of this project was to provide the necessary information on cullet properties and processing so that engineers can specify the use of cullet as a construction aggregate with confidence and recycled glass aggregate suppliers can invest in market development with minimal risk. The engineering performance, environmental suitability, cost comparability to natural aggregates, and safety in handling aspects were studied to determine the advantages and disadvantages of using WG.

##### 2.1.1.a Contaminant Leaching

To assess the potential for contaminant leaching over time, sequential batch extractions were conducted in accordance with method ASTM 4793. Two different samples were analyzed: one having a high debris content and the other with a low debris content. Following the sequential batch extractions, the aqueous samples generated were analyzed for biochemical oxygen demand (BOD), chemical oxygen demand (COD), total organic carbon (TOC), pH, specific conductivity, priority pollutant metals, and cobalt. The analytical results are summarized in Table 2.1. These concentrations decreased over time and do not appear to be at concentrations of concern.

### 2.1.1.b Health Evaluation

The purpose of the health evaluation was to determine the potential health effects of working with glass cullet during the experiments conducted at the Dames & Moore laboratories. Testing programs were set-up to evaluate two potential hazards from working with glass cullet: exposure to crystalline silica, and cuts and/or lacerations from sharp edges on the glass. Personnel and bulk samples were analyzed for percent crystalline silica by x-ray diffraction according to NIOSH method 7500 (NIOSH, 1977). Also, two area samples were analyzed for total dust by NIOSH method 500/600. The sample results are presented in Table 2.2.

The bulk sample results indicate that both samples contain less than 1% crystalline silica. As such, the cullet is in the nuisance category like all dust producing aggregates are. The Permissible Exposure Limit (PEL) for nuisance dust is  $10 \text{ mg/m}^3$ . PEL's from the personal sample and two area samples were all below  $0.5 \text{ mg/m}^3$  total dust. Therefore, based on the samples taken during this test program, cullet would not cause a health hazard from crystalline silica or dust. It is recommended however, that safety clothing normally worn while working with natural aggregates, be worn when working with cullet, including heavy gloves, long-sleeve shirts, pants, heavy boots, hard hats, and hearing and eye protection.

## 2.2 Environmental Regulations

Mary Jean Yon, Administrator of the Solid Waste Section of the Florida Department of Environmental Regulation, confirmed in a personal letter to Dr. Howell H. Heck, the regulatory classification of WG. The Florida Department of Environmental Regulation considers glass to be "clean debris" as defined in Rule 17-701.200 (10) Florida Administrative Code (F.A.C.). This classification means it is considered virtually inert,

does not pose a threat to the environment, is not considered a fire hazard, and is likely to retain its physical and chemical structure under expected conditions of disposal or use. Section 17-701.730 (1) of F.A.C. stipulates that clean debris may be used as fill material in any area, including waters of the State, but these waters may require a departmental dredge and fill permit. Clean debris, used as fill material, is not solid waste, and therefore doesn't require a solid waste permit.

## **2.3 Engineering Investigations**

### **2.3.1 WG Properties**

During the Dames & Moore (1993) study, two gradation sizes (1/4 in (6.3 mm) minus and 3/4 in (18.9 mm) minus) were manufactured for the testing program. Also quantified were the physical debris levels typical of different collection and sorting techniques. These debris levels were based on a visual classification of the amount of non-glass materials present. Samples containing debris levels of 10 percent or greater were defined as high debris samples; samples with debris levels between 3 and 10 % were classified as medium debris levels; and samples with less than 3% contamination were classified as low debris levels.

### **2.3.2 Permeability**

Permeability is a measure of the resistance to flow of a liquid through a saturated soil. The permeability of a coarse grained material depends on its gradation and density (Holtz and Kovacs, 1981). Permeability is a function of surface texture which affects: 1) drag or friction between the fluid and the particle surface, 2) density and fluid viscosity, 3) the pressure difference applied to the soil and 4) the mineralogical and

electrochemical properties of fluid and soil (McCarthy, 1988). Therefore, due to surface texture which affects drag or friction between the fluid and the particles, a mixture of aggregate and smooth cullet would likely have a higher permeability than the natural sands and gravels used in drainage application. Well graded aggregate can be compacted to a higher maximum density than a poorly graded aggregate, and it usually has a lower permeability.

The two commonly used laboratory tests for determining the permeability of soils are the constant and falling head methods. The constant head method is applicable for coarse aggregates or granular soils with permeability coefficient values greater than or equal to  $1 \times 10^{-4}$  cm/sec, whereas the falling head method is applicable for silts or clay with permeability coefficient less than or equal to  $1 \times 10^{-4}$  cm/sec (Holtz and Kovacs, 1981).

The relationship presented by Darcy (D'Arcy, 1856), concerning the laminar and streamlined flow of water in sands states that the rate of flow is proportional to hydraulic gradient. The basic form of Darcy's equation is:

$$q = kiA \quad (2.1)$$

where:  $q$  is the rate of flow;

$k$  is the proportionally constant termed the coefficient of permeability with units of length/time;

$i$  is the hydraulic gradient ( $\Delta h/l$ ); and

$A$  is the area of cross-section of the soil.

A more generalized version of Darcy's law states that: the discharge velocity of fluid flow through a porous granular medium under steady flow conditions, is proportional to excess hydrostatic pressures causing the flow, and inversely proportional to the viscosity of the fluid. This relationship can be expressed by the proportionality:

$$v \propto i/\eta \quad (2.2)$$

where:  $v$  is the discharge velocity;  
 $i$  is the hydraulic gradient; and  
 $\eta$  is the dynamic viscosity of the fluid.

Permeability is not constant for a given soil, but is related to the dynamic viscosity of the fluid denoted by  $\eta$ . Viscosity varies with temperature, and increases approximately 30% when there is a temperature change from 20 to 30 degrees centigrade ( $^{\circ}\text{C}$ ). ASTM D 2434 specifies that all laboratory permeability values be reported as an equivalent permeability at 20  $^{\circ}\text{C}$  (ASTM, 1987). If a permeability test conducted at  $T$   $^{\circ}\text{C}$  gives a coefficient of permeability of  $k_t$ , the corresponding value at 20  $^{\circ}\text{C}$  can be found from:

$$k_{20} = k_T \left( \frac{\eta_T}{\eta_{20}} \right) \quad (2.3)$$

The permeability coefficient is an important parameter used for determining the drainage capability of the unbound pavement layers, seepage quantities and pressures through earth dams, as well as the seepage pressures developed on coffer dams and retaining walls. The permeability coefficient is also used in the design of graded filter and the control of leachate through landfill liners.

Table 2.3 shows the permeability coefficients and dry density values for different cullet samples presented in the Dames & Moore (1993) report. The values reported indicate that WG permeabilities would be expected to fall in the  $10^{-2}$  cm/sec range. However, Dames & Moore (1993) also reported that four test samples had permeability coefficients greater than the maximum value the ASTM D 2434 specified permeameter could measure. In order to obtain representative values for the coefficient of permeability for these specimens, the length of the permeameter was increased to 34 in, while the 4 in (100 mm) diameter was maintained.

### 2.3.3 Grain Size Distribution

Gradation has been correlated to the engineering behavior of granular soils, which can be seen when using classification systems such as the AASHTO soil classification system (AASHTO, 1978, Holtz and Kovacs, 1981). The grain size distribution of a soil must be identified by its proper classification (Das, 1990). The grain size distribution curve, or gradation of coarse grained soils, is normally determined by sieve analysis. This curve is generally depicted on a semi-log plot with the particle diameter on a logarithmic scale on the abscissa, and the percent finer particles by weight on an arithmetic scale on the ordinate.

A well-graded soil has a wide range of particle sizes, but a uniform or poorly graded soil is either excessive or deficient in certain particle sizes. A soil with a coefficient of curvature between 1 and 3, is considered to be well-graded as long the coefficient of uniformity is greater than 4 for gravels, and greater than 6 for sands (Holtz and Kovacs, 1981). These coefficients are ratios that define the slope of the curve. Well graded soils can be compacted to a denser state and therefore will have a higher strength but lower permeability than poorly-graded materials (Dames & Moore, 1993). A poorly or uniformly graded soil will have more voids, resulting in higher permeability coefficients.

### 2.3.4 Specific Yield

The specific yield of a soil or rock is the ratio of the volume of water that can be drained by gravity to the total volume (V) of a saturated soil. It has been used in drainage analyses of pavement layers (Carpenter *et al.* 1981). It is expressed as a percentage using the equation (ASTM, 1982):

$$S_y (\%) = (V_y / V) \times 100 \quad (4)$$

where:  $S_y$  is the specific yield expressed in percentage;  
 $V_y$  is the volume of water drained by gravity; and  
 $V$  is the volume of soil specimen.

Specific yield, also referred to as effective porosity, is a direct indication of the degree to which soil will hold water when a saturated sample is allowed to drain under gravity (Carpenter *et al.*, 1981). Currently there is not a ASTM procedure available for determining this value, other than the above definition (ASTM, 1982). Values of specific yield depend on grain size, shape, distribution of pores, compaction of the stratum, and time period of drainage. The specific yield is particularly sensitive to the percentage of fines. It should be noted that fine grained materials drain less water, whereas coarse-grained materials permit substantial amounts. Walton (1970) reported typical specific yield values ranging from 1 to 10 % for clay, 10 to 30 % for sand, 15 to 25 % for sand and gravel, and 15 to 30 % for gravel. Referring to equation 4, the specific yield must be less than or equal to the soil's porosity.

### 2.3.5 Relative Density

Relative density can be defined as the soil's density relative to its possible range of densities. It is used to compare the void ratio of granular soils to the maximum and minimum possible void ratios. If fines are present, the Proctor moisture-density relationship may also be used to categorize density (choosing the proper density classification method can be difficult). ASTM D 2049 (1980) suggests that relative density is most suitable for soils with less than 12% fines. Therefore, the possible range of densities of cohesionless granular soils such as cullet, would be determined by the maximum and minimum density index tests. For loose, firm, dense, and very dense cohesionless soils, the relative density ranges from 0 to 50, 50 to 70, 70 to 90, and 90 to 100 percent, respectively (Sowers, 1979).

The results from Dames & Moore (1993) relative density testing on cullet samples, conforming to ASTM D 4253 and D 4254 yielded an average minimum density index of about 83.5 pcf and an average maximum density index of about 101.5 pcf (1.59 kN/m<sup>3</sup>). The minimum index values ranged from 76.8 to 89.5 pcf (1.20 to 1.40 kN/m<sup>3</sup>) and the maximum index values ranged from 90.9 to 109.3 pcf (1.42 to 1.71 kN/m<sup>3</sup>). Tests were conducted on samples of two cullet sources (WA-09 low debris and CA-14 high debris) and two cullet gradations (1/4 in (6.3 mm) minus and 3/4 in (18.9 mm) minus).

### 2.3.6 Specific Gravity

The specific gravity is the ratio of the particle mass to the mass of an equal volume of water. It is a measure of the density of solids, and is used as a parameter for establishing the density-volume relationship of a soil mass. Typical values of specific gravity of soils are 2.64 to 2.9 (Das, 1990), while typical values of commercial glass are 2.49 to 2.51 (BCIT 1991, HWA 1992). The specific gravity of the cullet ranges from 1.96 to 2.52 (Dames & Moore, 1993). The increased variations of specific gravity is due to the variation in debris content. The specific gravity of the cullet originating from commercial glass, and having a low debris content, is expected to be 2.5 (Dames & Moore, 1993).

### 2.3.7 Strength and Deformation Characteristics of Aggregates

The strength and deformation characteristics of an aggregate can be determined by studying its stress-strain behavior under various loading conditions. Several studies have been conducted on dry granular cohesionless soils subjected to different loading conditions. Lambe and Whitman (1969) provide an excellent discussion of the subject.

Three common types of stress-strain and deformation testing can be conducted: (1) the confined compression test, (2) the direct shear test, and (3) the triaxial shear test. The parameters resulting from these tests are the Modulus of Elasticity: (E), the cohesion (c) and the internal angle of friction ( $\phi$ ). These parameters are used to define the strength and deformation characteristics of granular cohesionless soil.

### 2.3.7.a Friction Angle

Both the direct shear test and the triaxial shear test are used to determine the angle of friction. The direct shear test offers the advantages of a fast, inexpensive and simple method for determining the shear strength of cohesionless soils. Its disadvantages are a pre-determined failure plane, which may not be the weakest plane of failure in the field, and highly non-uniform stress conditions that develop in the specimen during shear. Even with these disadvantages, the test still gives a good indication of the in-situ shear strength although the results have a tendency to be slightly higher than the results of the triaxial shear test (Lambe and Whitman, 1969).

The triaxial shear test was developed to overcome some of the disadvantages of the direct shear test. The triaxial shear test, though more complicated than the direct shear test, has the benefit of being a more versatile test. For example, a confining pressure (i.e., isometric compression) is applied to the soil in the triaxial test but not in the direct shear test.

Table 2.4 shows typical values of the angle of friction obtained for conventional aggregate. A review and summary of the table indicates that the following factors affect  $\phi$ :

1.  $\phi$  decreases with increasing void ratio.
2.  $\phi$  increases with increasing angularity.
3.  $\phi$  increases with increasing grain size distribution.

In addition, Holtz and Kovacs (1981) state that prestress (i.e. past stress history) and particle size (with constant void ratio) have little to no effect on  $\phi$ .

### 2.3.7.b Modulus of Elasticity

The modulus of elasticity (E), defined as the stress per unit strain of an elastic material, can be obtained from both the confined compression test and the triaxial shear test. Lambe and Whitman (1969) determined that the following characteristic stress-strain behavior exists during both confined compression and triaxial compression on granular soils:

1. A highly nonlinear stress-strain relationship occurs.
2. Accumulated compressive strains are developed by cycles of loading and unloading.
3. An increased stiffness, or stress-strain response, is developed by cycles of loading and unloading.

The following characteristics are valid for triaxial compression on granular soils:

1. The stiffness decreases with increasing vertical stress.
2. Dense sands tend to increase in volume as they are compressed while loose sands experience little volume change.
3. Dense sands lose strength when strained beyond peak strength, but loose sands do not.

In order to use the concepts and formulas from the theory of elasticity, the stress-strain curves must be "linearized", (i.e. replaced by straight line segments). Therefore, E is not constant, but rather is a quantity which "approximately describes the behavior of a soil for a particular set of stresses" (Lambe and Whitman, 1969). A secant modulus, (i.e., the slope of a straight line connecting two points on a curve) is used to define E.

For confined compression, the term secant constrained modulus ( $E_S$ ) is used. Typically,  $E_S$  increases as the material is compacted (void ratio decreases) and angular grains are more compressible than rounded grains Rodriguez *et al.*, 1988).

The elastic modulus derived from triaxial testing is often called Young's Modulus and the value usually quoted for soils is the secant modulus: from zero deviator stress to a deviator stress equal to about 1/3 to 1/2 the peak deviator stress (Lambe and Whitman, 1969). Table 2.5 (from Das, 1983) indicates the general effect of void ratio and composition on E for the first loading to one-half the peak deviator stress. Typical elastic moduli ranges for various materials are as follows: stabilized base materials: 50 to 60,000 psi (350 to 420,000 kPa), stiff clay: 7,600 - 17,000 psi (53 to 119,000 kPa), medium clay: 4,700 - 12,300 psi (32,900 to 86,100 kPa), soft clay: 1,800 - 7,700 psi (12,600 to 53,900 kPa), and very soft clay: 1,000 - 5,700 psi (7,000 to 39,900 kPa) (Huang, 1993).

### 2.3.7.c California Bearing Ratio (CBR)

The California Bearing Ratio (CBR) test is a common strength test that applies to highway design. The test is empirical and was developed by the military as a simple and reputable approach for categorizing materials used as highway bases, subbases, or subgrades. The CBR test measures the shearing resistance of a soil or aggregate material in relation to a standard test material. Typically, natural aggregates have CBR values ranging between 30 and 80. A typical minimum CBR specification for a road base is 100 (SCS, 1991). Chesner (1989) reports CBR values between approximately 85 and 155 for bottom ash utilization when the samples were prepared using the compactive energy described in ASTM D-1557. To find the CBR, the stress versus deflection curve for the material of concern is compared to the stress versus deflection curve for a standard crushed limestone. CBR values greater than ten are given very

good subgrade ratings; values from six to ten are considered good; values from three to six are poor (Baker, 1983).

#### 2.3.7.d Limerock Bearing Ratio (LBR)

The LBR test is used for evaluating limerock and other soils for base, stabilized subgrade and subgrade or embankment material encountered in Florida (FDOT, 1993). This value is determined with the same procedure used for CBR testing, however, the calculation of this ratio incorporates the effects of the softer limerock. The LBR can be calculated directly from the data from the CBR test as the ratio of the stress (psi) at 0.1 in (2.54 mm) penetration over 800 psi (5600 kPa) expressed as a percentage.

#### 2.3.8 Using WG in Bound Pavement Layers

Research conducted by the Bureau of Mines in the seventies indicated that a light weight concrete suitable for structural use could be produced utilizing mixed cullet as a primary raw material (Liles and Tyrrell, 1975). Testing conducted by the American Society for Testing Materials indicates that the mixed cullet is susceptible to alkali-aggregate reactions. As a result of this testing, ASTM developed a standard procedure—ASTM C 227—for determining alkali reactivity (ASTM 1990a). The result of the alkali-aggregate reaction is an expansion of the concrete that reduces the concrete's strength. Strengths were affected greatly when high-alkali cement was used. Alkali-silica reactivity severely limits the use of mixed cullet for portland cement concrete mixes.

Considerable research has also been conducted on the use of mixed cullet in the production of hot mix asphalt concrete pavement. The term "glasphalt" has been coined for this type of mix (Malisch *et al.*, 1970). Many states including Florida, New York and

Virginia have developed specifications for the use of mixed cullet as an aggregate in asphalt concrete design. Current specifications typically limit the maximum content of mixed cullet that can be used in the mix design to 5 - 15 % (Hughes, 1990). Generally, the maximum amount of contaminants in the mixed cullet is limited to less than one percent (Hughes, 1990). Although glasphalt does create a potential market for mixed cullet, the extensive processing required to remove the contaminants and to provide the specific gradation requirements for the mix design can become cost prohibitive.

#### **2.4 Existing Highway Applications And Specifications**

Many municipalities and state highway departments currently allow the use of WG in various pavement and drainage applications. A brief summary of some existing specifications is presented below, emphasizing uses in the unbound portions of highways.

- The National Association of Plumbing-Heating-Cooling Contractors, used 3/4 in (18.9 mm) crushed glass as fill around french drains (National Association of Plumbing-Heating-Cooling Contractors, 1992).
  
- The Pennsylvania Department of Transportation (PennDOT) has developed specifications which allow WG to be used as pipe backfill and embankment material. PennDOT has used WG as fill around two culverts beneath a state highway (PennDOT, 1992).

- The practical limits for using recycled materials in highway work is currently being evaluated by the North Carolina Department of Transportation. Crushed glass may be used as drainage aggregate beneath concrete highway slabs (Dames & Moore, 1993).
  
- Specifications drafted by the Connecticut Department of Transportation include the use of recycled glass in roadway embankment fill. These specifications require that no particles be larger than one inch, that glass contents are less than 25 percent by weight of the fill and that no glass mixture is placed within five feet of the embankment slope (Connecticut Specifications, 1992).
  
- The specifications for using WG in aggregate base and subbase, developed by the California Department of Transportation, would eliminate the use of glass for surfacing (CALTRANS Amendments, 1992).
  
- A filtration trench system designed by the Seattle Drainage and Waste Water Utility for the Green Lake Stormwater Project includes the use of recycled glass as stormwater filtration media. A geotextile fabric was wrapped around the glass and was covered with 6 inches of conventional aggregate (City of Seattle, 1992).
  
- The Vermont Department of Transportation amended the specification for subbase materials in 1993 allowing WG to be blended with common aggregates up to in quantities up to 10% by weight (Vermont, 1993).
  
- Specifications developed by the Washington State Department of Transportation for the Standards of Road, Bridge and Municipal construction, allow aggregate to consist of up to 15% cullet. The fraction of aggregate retained on 1/4 in (6.3 mm) or

larger sieves is limited to 10% cullet, 100% must pass the 3/4-in sieve (18.9 mm) and the maximum debris level is 10%. Additionally, the total lead content must not exceed 80 ppm (Washington State Department of Transportation, 1991).

- ☛ Maine's Department of Transportation (MDOT) released specifications in November, 1992 allowing crushed glass to be used in fill sections of projects in place of common borrow. WG layers can not be located within two feet of the ground surface and only one 8-inch layer can be placed in the pavement system (MDOT, 1992).

- ☛ Specifications for the use of processed glass aggregate as a construction material were developed by the New Hampshire Department of Transportation (NHDOT). New Hampshire's Governor's Recycling Program asked NHDOT to specifically study using recycled glass as a building material in state highway construction. Because information was already available on "Glasphalt" the study focused on using WG in other highway application. The specifications limit the top size of the material to 1/2-in (12.5 mm) and they allow up to 5% WG to be mixed into base courses (New Hampshire DOT, 1992).

- ☛ Evaluation of the use of glass cullet in drainage and base course applications was studied under the authorization of the City of Seattle Solid Waste Utility (Dames & Moore, 1993).

- ☛ The use of green cullet as electrical trench backfill was studied under the authorization of British Columbia Hydro (Veith and Guenther, 1991).



Table 2.2 Crystalline silica and dust results (after Dames and Moore, 1993)

Sample	Location	Crystalline Silica <sup>1</sup>	Total Dust <sup>3</sup>
CWC-01	Personnel sample: Daokaun Zhang	< 2.8 % <sup>2</sup>	0.280 mg/m <sup>3</sup>
CWC-02	Area sample near mixing trays		0.351 mg/m <sup>3</sup>
CWC-03	Area sample: near analytical scale		0.495 mg/m <sup>3</sup>
CWC-04	Blank Sample	<0.005 mg	
CWC-05	Blank Sample		0.160 mg
CWC-06	Bulk sample: WA -09 1/4" cullet	0.27 %	
CWC-07	Bulk sample: WA -09 1/4" cullet	0.07 %	

1. The Permissible Exposure Limit is 0.05 mg/m<sup>3</sup> for respirable crystalline silica (per 29CFR1910.1000). However, Federal regulations are not applicable to crystalline silica concentrations less than 1 % by mass.
2. Accuracy of test results limited by restricted sampling time.
3. The Permissible Exposure Limit is 10.0 mg/m<sup>3</sup> for nuisance dust. Nuisance dusts are those which do not contain otherwise regulated particulate such as asbestos or dusts which contain greater than one percent silica (per 29CFR1910.1000).

Table 2.3 WG Permeability and dry density (after Dames and Moore, 1993)

Cullet Sample Number <sup>1</sup>	Cullet Content (%)	Cullet Gradation	Approximate Relative Compaction	Dry Density (pcf)	Permeability (cm/sec)
CA-14	100	1/4" minus	90% of ASTM D 698	94.9	6 C10 <sup>-2</sup>
CA-14	100	3/4" minus	90 % of ASTM D 698	89.6	26 C 10 <sup>-2</sup>
WA-09	100	1/4" minus	90 % of ASTM D 698	93.6	6.4 C 10 <sup>-2</sup>
WA-09	100	3/4" minus	90 % of ASTM D 698	95.9	18 C 10 <sup>-2</sup>
CA-14	100	1/4" minus	90% of ASTM D 698	94.9	4.4 C10 <sup>-2</sup>
CA-14	100	3/4" minus	90 % of ASTM D 698	89.6	23 C 10 <sup>-2</sup>
WA-09	100	1/4" minus	90 % of ASTM D 698	93.6	4.8 C 10 <sup>-2</sup>
WA-09	100	3/4" minus	90 % of ASTM D 698	95.9	6.5 C 10 <sup>-2</sup>

- CA-14 is the high debris (i.e., > 10%) level sample  
WA-09 is the low debris (i.e., < 3%) level sample

Note: 1 pcf = 0.0157 kN/m<sup>3</sup>

Table 2.4 Typical values of  $\phi$  for aggregate (from Das, 1983)

TYPE OF SOIL		VOID RATIO		
		0.41-0.5	0.51-0.6	0.61-0.7
Coarse Sand	$\phi^{\circ}$	43	40	38
Medium Sand	$\phi^{\circ}$	40	38	35
Fine Sand	$\phi^{\circ}$	38	36	32
Sandy Silt	$\phi^{\circ}$	36	34	30

Table 2.5 Young's modulus (E) for initial loading from zero to one-half peak deviator stress (from Das, 1983)

Type of Soil		0.41-0.5	Void Ratio 0.51-0.6	0.61-0.7
Coarse Sand	E (psi)	6,550	5,700	4,700
Medium Sand	E (psi)	6,550	5,700	4,700
Fine Sand	E (psi)	5,300	4,000	3,400
Sandy Silt	E (psi)	2,000	1,700	1,450

Note 1 psi = 6.895 kPa

Table 2.6 CBR values of compacted soils (National Highway Institute, 1990).

Soil Type	Range of Dry Unit Weight (pcf)	Range of CBR Values
Well graded clean gravel	125 - 135	40 - 80
Well graded clean sand	110 - 130	20 - 40
Poorly graded clean sand	100 - 120	10 - 40
Silty sand clay mix	110 - 130	5 - 30
Organic silts - clays	75 - 100	5 or less

Note: 1 pcf = 0.0157 kN/m<sup>3</sup>

### 3.0 METHODOLOGY AND EXPERIMENTAL PROCEDURES

To achieve the project goal of categorizing WG for highway applications, samples meeting ASTM D 448 (ASTM, 1990a) gradations were prepared. These gradations were assumed to bound the possible range of WG sizes for use in field applications. Assuming that primary highway applications for WG would be drainage and backfill, prepared glass samples were tested at various densities and gradations to determine the drainage and shear strength properties: permeability, specific yield, angle of internal friction, elastic moduli and California/Limerock Bearing Ratio (CBR/LBR).

To determine the permeability of WG, which is granular, constant head permeability tests were conducted. Laboratory devices were designed for determining both permeability and specific yield. Additional soils' tests were conducted on WG to determine grain size distributions and specific gravities, along with minimum and maximum densities, for computation of void ratio, porosity, and effective diameter (D<sub>10</sub>) of the aggregates.

#### 3.1 WG Sources

Two WG sources were used for the testing program: Southeast Recycling Corporation (Brevard Division, Melbourne, Florida); and West Palm Beach Material Recycling Facility (WPBMRF). These sources are representative of facilities using basic recycling processes.

### 3.1.1 Southeast Recycling WG

Southeast Recycling collects glass through its curbside recycling program and transports the recyclable portions to companies that use recycled glass. Collected glass is color-sorted (according to flint, green, and amber) during transport to Southeast's facility. During transport, the glass is also crushed using the bucket of a front end loader, thus allowing more glass to be placed in the haul trucks. The end product includes a wide range of sizes, beginning at about 4-in (100 mm) in diameter, making it a very difficult material to handle. WG in the form of full and partially broken bottles was transported to the Applied Research Laboratory (ARL) at Florida Tech, where it was stored in separate containers. Due to handling difficulties, these bottles were passed through a commercial yard-waste shredder, and downsized to a 1-in (25.4 mm) minus aggregate. This WG will be termed Brevard Shredded Mixed Glass (BSMG) to distinguish it from WPBMRF WG. Photograph 3.1 shows BSMG after shredding.

### 3.1.2 West Palm Beach WG

The West Palm Beach Material Recycling Facility (MRF) also collects materials through a curbside recycling program. However, glass, cans and plastic bottles are commingled during collection and require sorting at the MRF. Therefore, the sorting process at a MRF requires separation of different colors of glass, various grades of plastics and various types of cans before transporting to recycling companies. During this sorting process, much of the glass is of mixed color and therefore nonrecyclable. Nearly 60% of WPBMRF glass is nonrecyclable, according to information obtained during a visit in Fall 1992. To aid in handling WG, WPBMRF subcontracted with RRT Empire Returns to employ their mixed glass sizing system. This system consists of a Stedman crusher and a three-stage trammel that grinds glass to a specified gradation

(White, 1992). Sizing WG in this manner makes it easier to handle and use in landfill for daily cover, etc. RRT's process enables WPBMRF to produce a 3/8 in (9.5 mm) minus mixed cullet or WG. The sorted glass bottles are passed through a crusher which sizes the glass to an aggregate of maximum particle size ranging from 1/4 in (6.7 mm) to 1/2 in (12.5 mm) depending upon the size of the screens used. The crushed glass, color-sorted according to flint, green or amber, is stockpiled for transport to the nearest glass recycling facility. WG produced during this process has no economic value and is typically landfilled. WPBMRF produces about 40 tons of WG per day. The mixed cullet is similar to a coarse-grained material, and it usually does not have sharp edges. WG from WPBMRF was transported to ARL at the Florida Institute of Technology (Florida Tech) in 55 gallon drums. Photograph 3.2 shows WPBMRF mixed cullet.

### 3.1.3 WG Survey Development

To determine the location, contact person(s), availability, processes and costs of WG, the survey form shown in Figure 3.1 was developed. Copies of this form were sent to the solid waste directors in all of Florida's 67 counties.

## 3.2 Visual Classification

Visual classifications were performed on WG from both sources. Percentages of paper, plastics, ceramic, and miscellaneous materials were determined by weighing hand-sorted, representative samples.

In addition to visual classification, the thickness of each color glass was determined. Thickness measurements in millimeters were taken on ten samples of each color.

### 3.3 Grain Size Distribution Analysis

The grain size distribution of WG was determined in accordance with ASTM C 136-84a, "Standard Method for Sieve Analysis of Fine and Coarse Aggregate" (ASTM, 1987). The U.S. standard sieves used were the 3/8 in (9.5 mm), #4, #8, #16, #30, #50, and #100. Sieve analyses were conducted on ten samples of WG from WPBMRF. Sieve analyses were also conducted on BSMG. Based on the grain sizes the Unified Soils Classification System (USCS) (ASTM D 2487), the American Association of State Highway and Transportation Officials (AASHTO) (ASTM D 3282) soils classification system and ASTM D 448 "Standard Classification for Sizes of Aggregate for Road and Bridge Construction" were used to classify WG (ASTM 1987, 1990a). Also the coefficient of uniformity ( $C_u$ ), coefficient of gradation ( $C_z$ ), and  $D_{10}$  (effective size), were determined from the grain size plots.

### 3.4 Sample Preparation

WG samples were prepared to meet ASTM D 448 specifications ranging from #8, to #10. WG obtained from WPBMRF was sieved through the 3/8 in (9.5 mm), #4, #8, #16, #50 and #100 sieves; and, the fractions retained on each of these sieves were stored in separate labeled containers. These fractions were then proportionately mixed to the required ASTM D 448 gradation using the weight (retained in percent) for the ASTM D 448 #8, #9, and #10 gradations (Table 3.1). After mixing, a representative sample was obtained using a sample splitter following ASTM C 702 (ASTM, 1990a).

Drainage characterization samples meeting each of these ASTM gradations were further classified according to upper, lower, and average limits. The upper limit of gradation was defined as the grain size distribution generated for the upper limit of percent finer for each of the sieves used (ASTM D 448). The lower limit of gradation

was defined as the grain size distribution generated for the lower limit of percent finer for each of the sieves used. The average gradation was defined as the grain size distribution generated for the average range of percent finer for each of the sieves used. For example, if the ASTM D 448 specified percent finer for 3/8 in (9.5 mm) is 100% to 85%, then 100% was used for generating the upper limit of gradation; 85% was used for generating the lower limit of gradation; and 92.5% was used for generating the average gradation. Samples were prepared for the upper, average, and lower gradation limits of ASTM gradations #8, #9, and #10.

For strength and deformation characterization, only the average gradations were used. It was assumed that  $D_{10}$  would significantly affect the drainage characteristics but would not significantly affect the strength-deformation characteristics. This assumption was made because Hazen (1930), conducting research on loose sands, showed that texture, density, confining stress level, gradation (i.e., well, gap or poorly graded) and stress history control strength-deformation characteristics (Lambe and Whitman, 1972), while  $D_{10}$  controls the flow rate.

The maximum size mixed cullet used for this research was 3/8 inch (9.5 mm). Material of this size could be safely handled in a laboratory atmosphere without respiratory or skin irritation problems. The gradations used in this research effectively band the safe handling range of mixed cullet to provide a sufficient range for studying the effects of grain size on the strength and deformation behavior of the mixed cullet.

### **3.5 Engineering Testing**

#### **3.5.1 Density**

Relative density terminology was avoided in the WG investigation because small differences between minimum and maximum WG densities (i.e., 80 to 110 pcf (1.26 to

1.73 kN/m<sup>3</sup>) or a 30 pcf (0.47 kN/m<sup>3</sup>) range) can cause problems. It was believed that problems would arise in interpreting data because small differences in densities would be magnified within this range. For example if the minimum and maximum densities are 80 and 100 pcf and the two densities being analyzed are 81 and 84 pcf (1.26 to 1.73 kN/m<sup>3</sup>), their relative densities would be 5% and 20%. This large difference in relative densities tends to overshadow the fact that 81 and 84 pcf (1.27 to 1.32 kN/m<sup>3</sup>) are very close; however, our current density measuring techniques cannot appropriately distinguish between these values.

#### 3.5.1.a Minimum Density Testing Procedure

Minimum density testing was conducted in accordance with ASTM D 4254-83 on glass meeting ASTM gradations from #8 to #10. Samples, obtained from WPBMRF, were loosely poured through a funnel into the 1/30 ft<sup>3</sup> (0.0009 m<sup>3</sup>) Proctor mold. The weight of the soil divided by the mold volume yielded the minimum density.

#### 3.5.1.b Maximum Density Testing Procedure

A modified maximum density test was developed for WG. Maximum density tests were conducted on prepared WG samples meeting ASTM D 448 gradations from #8 to #10. A Proctor mold 1/30 ft<sup>3</sup> (0.0009 m<sup>3</sup>) in volume was used and WG was placed into the mold in five lifts. Each lift was compacted 25 times using the Marshall compaction hammer specified in ASTM D 1559 (1990a). This method will be referred to as the Modified Marshall-Proctor method. The Marshall compaction hammer has a circular flat tamping surface and a 10 lb (44.48 N) sliding weight with an 18 inch (45 cm) free-fall. A sieve analysis was conducted on the compacted WG to check whether any alteration in the gradation had taken place as a result of this compaction procedure.

The Modified Marshall-Proctor procedure was used because the equipment needed to meet ASTM D 4253-83, "Maximum Index Density of Soils Using a Vibratory Table," was not available. The Modified Marshall-Proctor test was very similar to the Modified Proctor ASTM D 1557 (1987), and it was assumed that most of the FDOT district offices would have both the mold and Marshall hammer to duplicate the results. Maximum densities conducted using the Standard Proctor compaction test ASTM D 698 (1987) resulted in loss of WG particles throughout compaction and is therefore not recommended.

### 3.5.1.c Densities for Drainage Testing

Densities of samples used in the permeability and specific yield tests included minimum, maximum, and two intermediate densities based on ASTM D 4253 and the Modified Marshall-Proctor method. A representative distribution of densities for each gradation was developed by choosing four evenly-spaced densities between the minimum and maximum index values. For example if the minimum and maximum densities for a gradation were 85 and 100 pcf (1.33 to 1.57 kN/m<sup>3</sup>), respectively, the four densities for testing would have been 85, 90, 95 and 100 pcf (1.33, 1.41, 1.49, and 1.57kN/m<sup>3</sup>). The estimated weight of a sample required to fill the molds was calculated according to the desired density. Samples were then placed into the mold with lifts and compacted using a plate vibrator with a 5-lb (22.4 N) surcharge. Following compaction the weight of the sample in the mold was determined; and the final weight per unit volume was then used as the dry density.

### 3.5.1.d Densities for Shear Strength and Deformation Testing

A minimum of five densities between the maximum and minimum densities were used in the shear strength and deformation testing. These minimum and maximum densities were based on ASTM D 4253 and the Modified Marshall-Proctor method, respectively. Due to the relatively small size of the samples required for the direct shear and triaxial tests, the intermediate values tended to vary slightly from test to test. This testing process revealed meaningful relationships between density and the desired parameters to be developed.

### 3.5.2 Permeability

#### 3.5.2.a Standard Constant Head Permeability Testing

A constant head permeability test was conducted on glass in accordance with ASTM D 2434-74 (ASTM 1987). The initial test apparatus consisted of a permeameter connected to a constant head tank by a clear tubing (Figure 3.2). The permeameter consisted of a base plate, a mold, a top plate and filter cloth. The base plate had a 1/8 in (3.1 mm) inside diameter (I.D.) inlet opening and the top plate had a 3/16 in (4.7 mm) (I.D.) outlet opening. The permeameter mold (the same mold used for ASTM D 2434) had a diameter of 4 in (100 mm), a length of 4.5 in (112.5 mm) and a volume of 1/30 ft<sup>3</sup> (0.0009 m<sup>3</sup>). Filter cloth was placed on top of the base plate to prevent washing of fines from the sample. The constant head tank had an overflow tube to maintain the constant head (Figure 3.2).

### 3.5.2.b Preliminary Testing Procedure

Following proper compaction (See Section 3.5.3), the sample was placed on the constant head set-up, and water was made to flow upwards through the sample by providing a head suitable to cause sufficient outflow. During testing, the rate of flow was determined by noting the time (t) taken to collect Q ml of water in a volumetric flask. The permeability (k) was calculated using the constant head equation:

$$k = QL/hAt \quad (3.1)$$

where: h is the total head applied;  
L is the length of the sample;  
Q is the quantity of water collected;  
A is the area of the sample; and  
t is the time in seconds.

### 3.5.2.c Validation of Standard Permeability Testing

The coefficient of permeability, calculated from eighteen tests on WG (WPBMRF) using the ASTM apparatus, yielded a value of approximately 0.02 cm/sec irrespective of the density at which it was compacted. These results were questionable. Therefore, the permeability was determined using #57 limestone (ASTM D 448) and a "play" sand which also yielded a value of 0.02 cm/sec. Because these results were again questionable, the limiting permeability of the permeameter was determined. Tests conducted using an empty permeameter again gave similar results, suggesting that the permeameter could only measure permeabilities below 0.02 cm/sec. It was also noted that water coming out of the outlet opening had sufficient velocity head to indicate that the pressure head at the outlet was not zero. If water velocity at the exit is too great, the assumption of zero head for the sample would be inaccurate.

### 3.5.2.d Modification of the Permeameter

The original permeameter was modified by increasing the diameter of the inlet opening from 1/8 in to 1/4 in (3.1 to 6.2 mm), and the outlet opening from 3/16 in to 1/4 in (4.7 to 6.3 mm). The results from several permeability tests yielded a permeability of 0.02 cm/sec, indicating that the porous stone in the original mold was restricting the flow. The next modification was to increase the height of the permeameter mold from 4 inches to 12 inches (100 to 300 mm). Four openings each 1/8 in (3.1 mm) in diameter, were also added to the perimeter of the cylindrical permeameter mold where four pinchcocks were attached (Figure 3.3). Commercial window screen was placed at the top and bottom of the mold to retain the sample and ensure a free-draining condition. The large-scale permeameter provided a more realistic sample size for coarse-grained testing. This permeameter worked well with granular materials because it could measure permeabilities up to 10 cm/sec, depending upon the gradations used. The piezometers, consisting of clear nylon tubing, were mounted on a panel and connected to the permeameter by 1/8" (3.1 mm) bore nylon tubing. Each piezometer passed through a gland in the wall of the permeameter and extended to the centerline of the cell (Figure 3.4). As shown in Figure 3.4, the last 1 in (25.4 mm) of tubing was sliced in half and wrapped with commercial screen to allow water to enter the piezometer.

### 3.5.2.e Final Testing Procedure

The permeameter shown schematically in Figure 3.3 was used to determine the permeability of the various WG gradations. Vibratory compaction was used as described in Section 3.5.3. Once the sample was prepared, a suitable constant head was maintained so that water flowed through the sample and exited through the outlet.

During a test, the time ( $t$ ) taken to collect  $Q$  ml of water was recorded. The height to which water rose in each of the four piezometers was the corresponding head at those elevations. The difference in water level between any two piezometers was the head lost over the corresponding length. Using the permeability values calculated for the head loss between piezometers 1 and 2; 2 and 3; and 3 and 4, each test yielded three  $k$ -values. The average of these values was used to determine the coefficient of permeability for each test.

A total of forty WG samples were tested to determine permeability characteristics. Four samples were prepared at each of the following ten gradations: #8 upper limit; #8 average; #8 lower limit; #9 upper limit; #9 average; #9 lower limit; #10 upper limit; #10 average; #10 lower limit; and WPBMRF. Each sample was prepared at a different density so that the minimum, maximum and two intermediate densities were tested. Variation of permeability within a gradation was thus studied in this testing scheme

### 3.5.3 Specific Yield

#### 3.5.3.a Description of Specific Yield Apparatus

The apparatus for conducting specific yield consists of a Proctor mold having a diameter of 4 in (100 mm), and length of 4.5 in (112.5 mm), a base plate and a top plate (Figure 3.5). The base plate and the top plate have inlet and outlet of 1/4 in (6.3 mm) diameter (Figure 3.5). The inlet opening is connected to clear flexible tubing to saturate the sample. The top plate has both an outlet opening connected to a valve which regulates the outflow of water and an air vent for removing air during saturation. Commercial window screen was placed at the top and bottom of the mold to prevent the washing of fines and ensure a free-draining condition.

### 3.5.3.b Testing Procedure for Specific Yield

WG samples were placed in the Proctor mold and compacted to the desired density using the plate vibrator. By providing a suitable head, water flowed through the inlet opening, passed through the sample and exited through the outlet. To ensure that the sample was saturated, the outlet valve was opened so that there was no restriction to the flow of water through the outlet. After saturation was complete, the outlet valve was closed to prevent the flow of water through the outlet; and the pipe, connecting the inlet opening to the overhead tank, was disconnected. The air vent was opened, and the apparatus was placed in an upright position so water from the sample drained through the opened inlet opening and collected in the measuring jar. Sufficient time was allowed for water to drain out. Typically a test required approximately 10 minutes for water to drain by gravity flow. The volume of water collected was recorded, and the specific yield was calculated as the ratio of volume of water drained by gravity to total volume of sample. A total of forty samples were tested for specific yield at four different densities using ASTM D 448 gradations #8, #9, and #10 upper, average, and lower gradation limits.

### 3.5.4 Specific Gravity

The specific gravity of WG was determined in accordance with ASTM D 854-83 (1990) "Standard Test Method for Specific Gravity of Soils". A total of eight tests were conducted on WG passing the #4 sieve. A 500 ml volumetric flask was used as the pycnometer and entrapped air was removed by connecting the top of the pycnometer to a vacuum pump.

### 3.5.5 Confined Compression Testing

Confined compression tests were performed to determine the stress-strain behavior of the mixed cullet under one-dimensional axial loading. Tests were conducted using the Brainard-Kilman model S 610 CBR/UCC loading frame. The results were used to determine the variation of the secant constrained modulus ( $E_S$ ) with stress level.

The tests were conducted using a Brainard-Kilman Terraload loading device equipped with a pressure gauge having a 100 psi (700 kPa) capacity. The samples' applied load was determined from the pressure dial gauge reading using the following calibration equation:

$$y = -11.894 + 26.394x \quad (3.2)$$

where  $y$  = load on sample (lbs)

$x$  = dial gauge reading (psi)

Specimens were prepared and tested in a 4 in (100 mm) diameter Proctor Mold having a capacity of 1/30 cubic foot (950 cm<sup>3</sup>). An initial seating load of 1 psi gauge (7 kPa) was applied to all specimens. For each load increment, deflection readings were taken for one hour. At selected load increments, the samples were unloaded to 10 psi (70 kPa) and reloaded. Typically three unload-reload loops were conducted during a test so that the unload pressure was maintained at 10 psi (70 kPa). After each test, the samples were removed from the mold and a gradation analysis was conducted according to ASTM C-136 "Standard Method for Sieve Analysis of Fine and Coarse Aggregate". The results of the gradation analyses were overlaid on to the original grain size plot to determine if any degradation of the sample occurred during loading.

The confined compression testing program consisted of four confined compression tests on each gradation of mixed cullet. The density, and hence void ratio, was controlled during testing. Within each gradation, confined compression tests were

conducted at various relative densities. Unload/reload cycles were conducted on the mixed cullet to determine the stress-strain response to cycle loading. The confined compression tests were conducted using seven load increments from 0 to 100 psi (700 kPa) (dial reading). The unload/reload cycles were conducted at three stress levels, with a 10 psi (70 kPa) unloading dial gage pressure maintained during unloading. Each loading increment was maintained for one hour.

The primary confined compression testing utilized pressure intervals of 5, 20, 30, 50, 70, 90, 100 psi (35, 140, 210, 350, 490, 630 and 700 kPa)(dial gage reading). These dial gage pressures allowed WG samples in the Proctor mold to be tested to about 210 psi (1470 kPa) vertically. Deflection readings were taken for each pressure at 0.0, 0.25, 0.50, 1, 2, 4, 6, 9, 16, 36, and 60 minutes. At the end of the 20, 50, and 90 psi (140, 350 and 630 kPa) pressure increments the sample was incrementally unloaded to 10 psi (70 kPa) and reloaded to the previously induced stress. The change in pressure increments during the unload/reload cycle was determined by the amount of deflection occurring during a one minute time interval. Specifically, when no deflection was recorded (i.e., less than 0.0001 in (0.00025 mm)) for a one minute interval, the pressure was changed. After the cycles were completed, the sample was removed from the mold and a gradation analysis was conducted according to ASTM C136 "Standard Method for Sieve Analysis of Fine and Coarse Aggregate."

### 3.5.6 Direct Shear Testing

Direct shear tests were conducted on WG samples using a Geotest model S2213 Digital Direct Residual Shear device. This machine has a square shear box with 3-inch (7.5 mm) sides. The gap between the shear box halves was kept between 0.01 and 0.02 (0.25 to 0.50 mm) inches and the rate of strain was set to 0.03 inches (0.75 mm) per

minute. The direct shear test was conducted in general accordance with ASTM D 3080 "Method for Direct Shear Tests of Soil Under Consolidated Conditions".

The testing program consisted of conducting four direct shear tests on each WG gradation where, the average gradation for ASTM D 448 #8, #9 and # 10 along with WPBMRF were used. Each direct shear test included testing three samples, at the same density, using normal stresses of 1,000 psf (49 kPa), 2,000 psf (98 kPa), and 4,000 psf (195 kPa). The ultimate shear strength of each sample was determined by plotting the shear stress versus horizontal deflection. A shear strength envelope was developed for each density and the range of values for the angle of friction was determined. Data from these tests was used to calculate the variation of angle of internal friction versus gradation and density for WG.

### 3.5.7 Triaxial Shear Testing

Consolidated drained triaxial tests were conducted to determine the stress-strain behavior of the mixed cullet. Brainard-Kilmans' triaxial shear testing equipment was used. The consolidated drained test is particularly suitable for free-draining cohesionless soil, such as mixed cullet, in which pore pressures will not build up during loading. Data from these tests was used to determine elastic moduli, failure stresses, typical stress-strain behavior and the angle of internal friction. Samples meeting the average gradation for ASTM D 448 #8, #9 and # 10, along with WPBMRF, were tested.

A split mold forming jacket was utilized while preparing each sample in a 0.025 inch (0.625 mm) thick rubber membrane. The mixed cullet was placed and compacted in the mold in a minimum of ten equal lifts. Following compaction, the CD tests were conducted in general accordance with procedures outlined by Bishop (1961) and the Army Corps of Engineering Testing Manual (1970).

Because WG is a cohesionless material, it is likely that the Mohr-Coulomb failure envelope can be determined from a single triaxial test. Therefore, preliminary triaxial testing was conducted to determine if the friction angle of WG would be relatively constant for three confining stresses. Tests were thus conducted at confining pressures of 7 psi (49 kPa), 14 psi (98 kPa), and 28 psi (196 kPa). Preliminary test results were used to plot the Mohr-Coulomb failure envelope and determine the variability of the internal friction angle (Figure 3.6). Figure 3.6 illustrates three failure envelopes: a  $44^{\circ}$  envelope associated with the lowest confining stress, a  $43^{\circ}$  envelope associated with the intermediate confining stress and a  $42^{\circ}$  envelope associated with the highest confining stress. This variability was assumed to be reasonable for a test as complex as the triaxial test and an average friction angle of  $43^{\circ}$  was recommended for these results. It was concluded that WG triaxial testing could be accurately studied if only one confining stress was used. Any of the three confining stresses could have been used however, research has shown that the true Mohr-Coulomb envelope is curved with the friction angle decreasing with increasing confining stress (Lambe and Whitman, 1969). Realizing that the 14 psi (98 kPa) confining pressure is near the upper limit of confining stresses expected for WG highway applications it was chosen as the confining stress for the triaxial testing program.

### 3.5.8 California Bearing Ratio (CBR) Testing

CBR testing was conducted in accordance with ASTM D 1883 "Standard Test Method for Bearing Ratio of Laboratory-Compacted Soils". Tests were conducted using the Brainard-Kilman model S 610 CBR/UCC loading frame. Samples meeting the average gradation for ASTM D 448 #8, #9 and # 10 were tested. Four samples were prepared for each gradation, and the relative density was varied within each gradation

to provide a range of CBR values. Data from these tests was used to calculate both CBR and LBR values for WG.

Samples were not saturated prior to testing for two reasons: First, it was believed that WG will not be saturated during field use; and, second, water is not held in the pore spaces of glass. Samples were compacted using a vibratory plate. Two 5-lb (22.4 N) weights were added during testing to simulate a pavement surcharge.

The ASTM specification defines the CBR number as, "the ratio of the unit stress required to effect a certain depth of penetration of the standard into a compacted specimen of soil at some water content and density to the standard unit stress required to obtain the same depth of penetration on a standard sample of crushed stone." The CBR number is usually based on the ratio of these stresses for a penetration of 0.1 in (2.54 mm). However, if the CBR number at a penetration of 0.2 in (5.08 mm) is greater than that at 0.1 in (2.54 mm), the test must be repeated. If the second test yields the same results, the CBR number at 0.2 in (5.08 mm) penetration is used. These relationships are shown in the following equations.

$$\text{CBR @ 0.1"} = \frac{\sigma_{0.1}}{1000 \text{ psi}} * 100 \quad (3.3)$$

$$\text{CBR @ 0.2"} = \frac{\sigma_{0.2}}{1500 \text{ psi}} * 100 \quad (3.4)$$

where: CBR = California Bearing Ratio in percent and  
 $\sigma_{0.1}, \sigma_{0.2}$  = corrected stress at piston penetrations of 0.1 or  
0.2 in (2.54 or 5.08 mm)

### 3.5.9 Limerock Bearing Ratio (LBR) Testing

Limerock Bearing Ratio (LBR) values were calculated for WG using the data collected from the CBR tests. The previously collected data was used because the procedure for the CBR tests (ASTM D-1883-92) is in accordance with the procedure for the LBR tests (FDOT FM-5-515). The LBR was determined to evaluate WG in various highway applications encountered in Florida.

The LBR number equals the stress at 0.1 in (2.54 mm) penetration divided by 800 psi (5600 kPa) as shown in the following equation.

$$\text{LBR} = \frac{\sigma_{0.1''}}{800 \text{ psi}} * 100 \quad (3.5)$$

where: LBR = Limerock Bearing Ratio in percent  
 $\sigma_{0.1''}$  = Corrected stress @ 0.1 in (2.54 mm)

## 3.6 Environmental Testing

### 3.6.1 Column Leaching

The column leaching test was modified from ASTM D 4874 - 89 "Standard Test Method for Leaching Solid Waste in a Column Apparatus." The leaching was performed using 2 ft (61 cm); 4 ft (122 cm); and 6 ft (183 cm) PVC cylinders 4 in. (100 mm) in diameter. The PVC cylinder was mounted on a perforated Plexiglas® plate and attached to a solid support to prevent it from tipping over. Its vertical alignment was checked with a level. The leaching fluid, distilled water with a pH of 4.5, was applied to the top of the column and collected from the bottom. The base of the column apparatus had 0.4 in. (1.0 cm) diameter holes for the leachate to exit. A glass fiber was placed above the holes to hold the glass in place. The maximum sample particle size never

exceeded 0.4 in (1.0 cm). A schematic of the column leaching apparatus is shown in Figure 3.7.

The leaching fluid was applied at a rate of 7 ml/min through the dripping spray head tubes (Figure 3.7). Seven ml/min is equivalent to a 2 in./hr rainfall event. The leaching fluid was evenly spread over the column diameter using 17 small tubes. The tubes have a 0.0228 in (0.58 mm) inside diameter and distribute the flow from a 1/8 in (3.18 mm) diameter tube coming from the pump. Transition from the small to large tube was sealed with polyurethane glue (3M Marine Adhesive/Sealant, Part No. 05203). Leachate was applied and collected until the effluent appeared clear. The collection intervals varied depending on the column length and the time required to get a clear leachate.

### 3.6.2 Biochemical Oxygen Demand (BOD)

A 5-day BOD test, method 5210B five-day BOD test (Standard Methods, 1989) was performed. Neither nitrification inhibitors nor seeding were used. Samples whose 5-day BOD did not exceed 7 mg/l were not diluted, samples whose BOD was greater than 7 mg/l were analyzed using the dilution method. For each sample requiring dilution, two different dilutions were used. All samples were analyzed using duplicates. Several trial runs were necessary to determine appropriate dilution ratios.

### 3.6.3 Dissolved Oxygen (DO)

The oxygen levels were measured using method 4500-OC azide modification (Standard Methods, 1989) of the winkler method, employing the full bottle technique (EPA, 1974).

### 3.6.4 Total Phosphorous (TP)

The total phosphorous levels were measured with the single reagent method and persulfate digestion (EPA, 1974). The digestion method converts the various forms of phosphorous into soluble orthophosphate. The orthophosphate concentration was measured on a Shimadzu® UV-160A spectrophotometer.

A calibration curve was prepared and checked against two standards for each series of specimens. The standards should agree within  $\pm 2\%$  of the true value. Results are reported as mg/l P. If a value has above 2.0 mg/l P, the sample was diluted so that the concentration fell into the 0-2.0 mg/l P range.

### 3.6.5. Solids

#### 3.6.5.a Total Dissolved Solids (TDS)

A well mixed 50-200 ml sample was filtered through a Whatman 934AH 4.7 cm glass fiber filter. The filtrate was evaporated to near dryness in a weighed dish on a hot plate at 180°C. It was then placed in a drying oven at 105°C until dry. The sample was cooled to room temperature in a desiccator, then weighed. Cycles of drying, cooling, desiccating and weighing was repeated until a constant weight was obtained.

#### 3.6.5.b Total Suspended Solids (TSS)

Residue retained on the fiber filter used for TDS measurement was dried for at least one hour to a constant weight at 105°C. The sample was cooled in a desiccator at room temperature, then weighed. The increased filter weight represents TSS. Cycles of

heating, cooling, desiccating, and weighing were repeated until a constant weight was obtained.

#### 3.6.5.c Fixed and Volatile Suspended Solids

The residue and filter from TSS measurements were ignited for 30 minutes to a constant weight at 550°C in a muffle furnace. The sample was cooled in a desiccator, to room temperature, then weighed. Cycles of heating, cooling, desiccating, and weighing were repeated until a constant weight was obtained. The weight after ignition allows determination of the fixed suspended solids. The volatile suspended solids are determined by the difference in the residue and filter weight before and after ignition.

#### 3.6.6 Total Kjeldahl Nitrogen (TKN)

TKN analysis was analyzed according to method 351.3 (Standard Methods, 1989) by Brevard Teaching and Research Labs, Inc. in Palm Bay, Florida. The 500 ml samples were preserved by adding H<sub>2</sub>SO<sub>4</sub> to the sample, until the pH was below 2.0, and kept refrigerated until analysis.

Table 3.1 Standard sizes of selected road aggregates (ASTM D 448)

ASTM	% Finer								
No.	1 in	3/4 in	1/2 in	3/8 in	No.4	No.8	No.16	No.50	No.100
8			100	85-100	10 to 30	0-10	0-5		
9				100	85-100	10 to 40	0-10	0-5	
10				100	85-100				10 to 30

Note: 1 inch = 25.4 mm

**Survey of Statewide Glass Recyclers for Florida Department  
of Transportation Research Study**

1. Name and phone number of the contractor (or government employees) who are in charge of selling or disposing of waste glass

---

---

---

---

2. Do you use curbside sorting, collection bins, or MRF's?

3. Annual production of:

- a. Clear glass \_\_\_\_\_ tons/yr.
- b. Brown glass \_\_\_\_\_ tons/yr.
- c. Green glass \_\_\_\_\_ tons/yr.
- d. Mixed color glass \_\_\_\_\_ tons/yr.

4. Are any of your waste glass streams sold at a price below the transportation costs required to deliver the glass to the reprocessing facility?

---

---

5. Please note if any additional treatment is performed on the waste glass before delivery to the processor, such as crushing, washing, metals removing, hand sorting, etc. \_\_\_\_\_

---

---

6. Would you be interested in selling any part of your glass waste stream for use as fill for approximately \$3.00/yd<sup>3</sup> on FDOT construction projects?

---

---

7. Do you have a location where you could accumulate large quantities of waste glass for use on FDOT projects? \_\_\_\_\_

---

---

8. Are you currently stockpiling any waste glass? \_\_\_\_\_  
If yes, approximately how much do you have stockpiled?

---

---

Figure 3.1 WG survey form

9. Are you disposing (i.e. not recycling) any of your waste glass stream after collection which has been sorted or partially sorted?

---

---

10. If available, please provide printed materials (i.e. brochures) about you facility or glass recycling programs. Mail the material along with the completed survey to:

Paul J. Cosentino, Ph.D., P.E.  
Assistant Professor  
Civil Engineering Department  
Florida Institute of Technology  
Melbourne, Fl 32901-6988  
(407) 768-8000 ext. 7555

Figure 3.1 WG survey form (cont.)

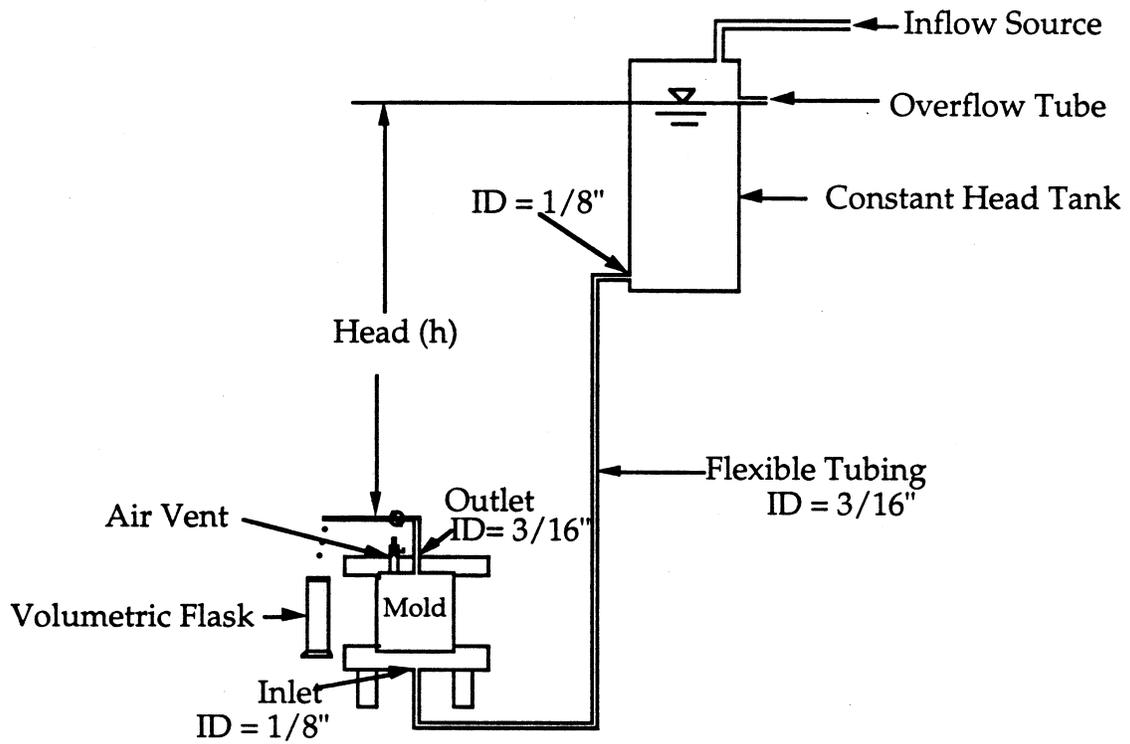


Figure 3.2 Schematic of the standard permeability test apparatus (1 in = 25.4 mm)

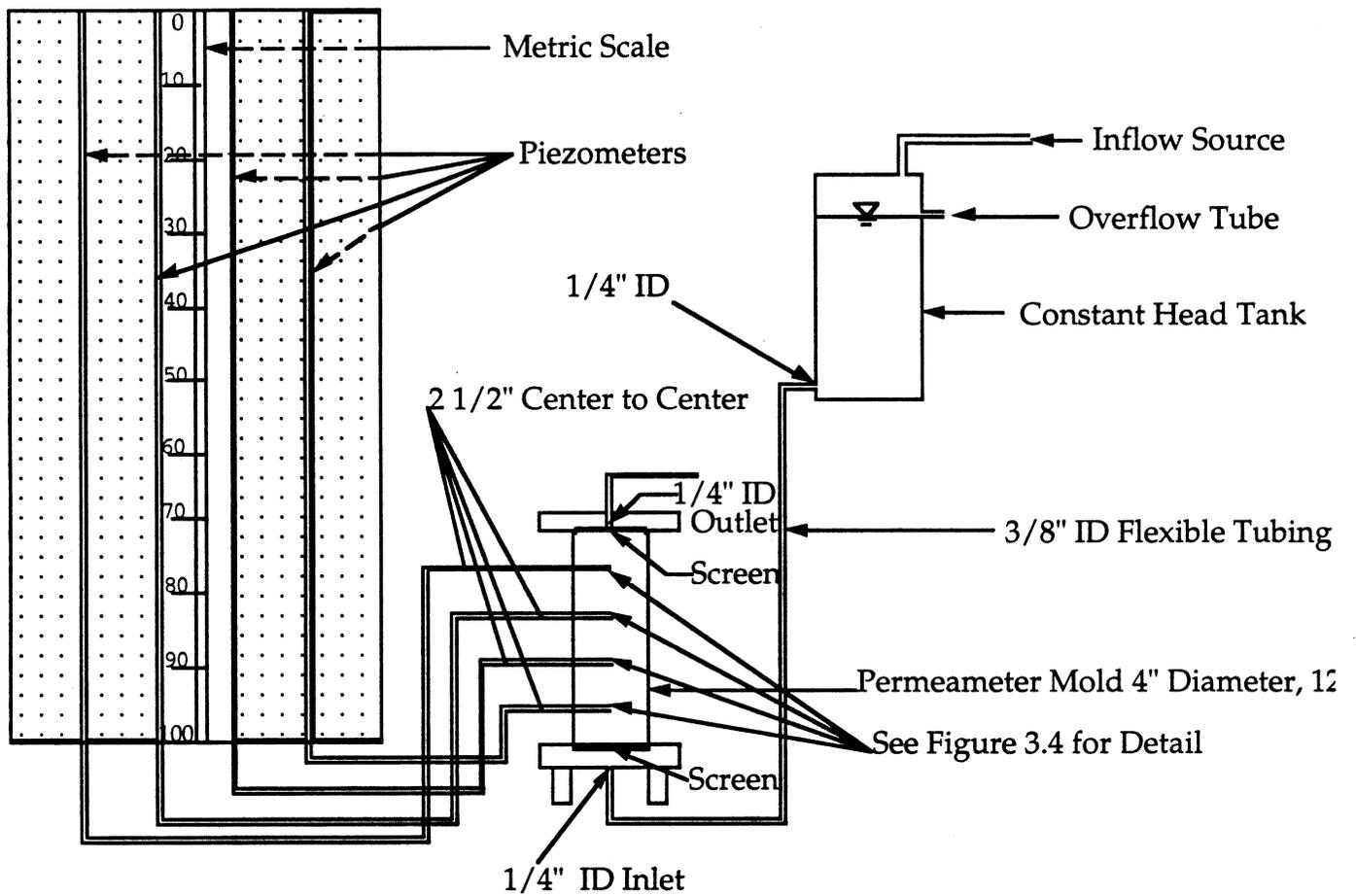


Figure 3.3 Schematic of the modified permeability test apparatus (1 in = 25.4 mm)

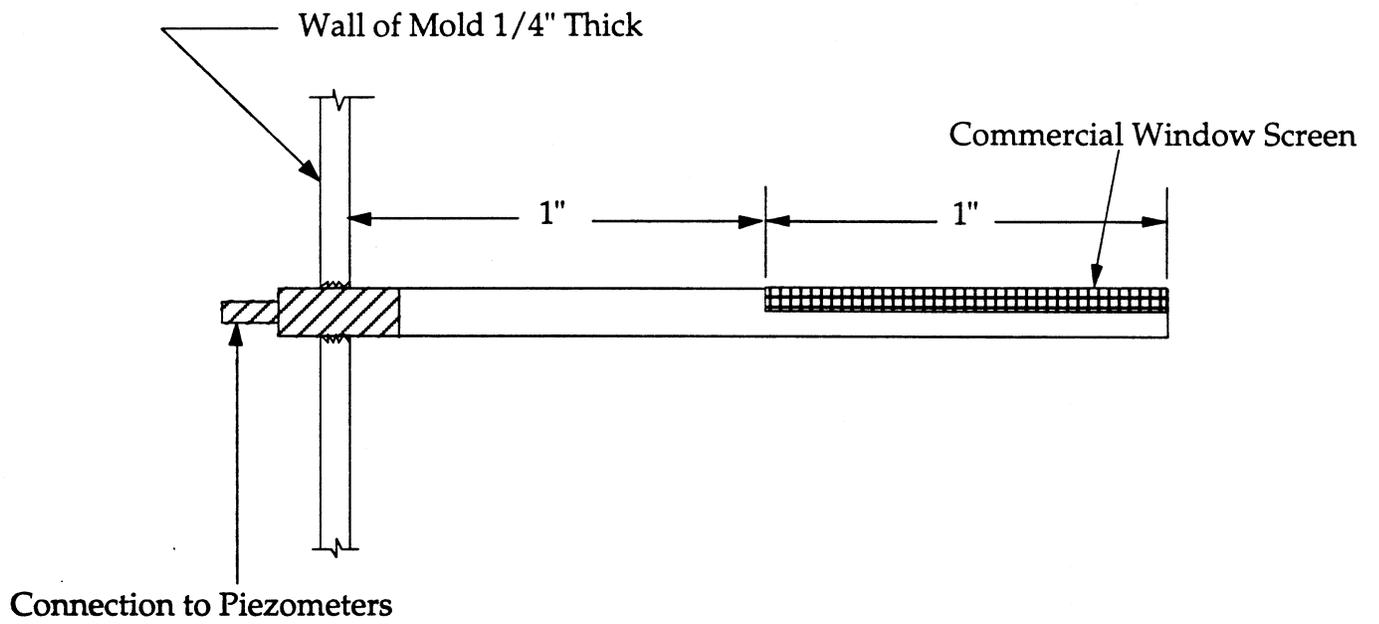


Figure 3.4 Schematic of piezometer inlet within permeameter mold(1 in = 25.4 mm)

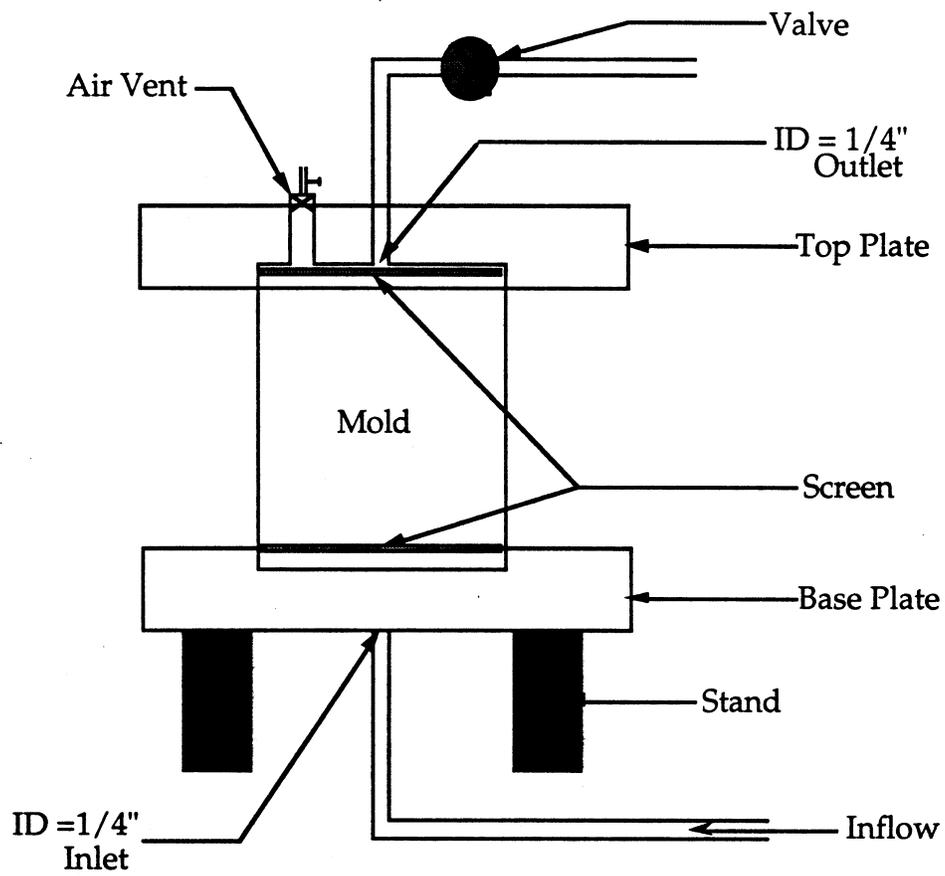


Figure 3.5 Schematic of specific yield test apparatus (1 in = 25.4 mm)

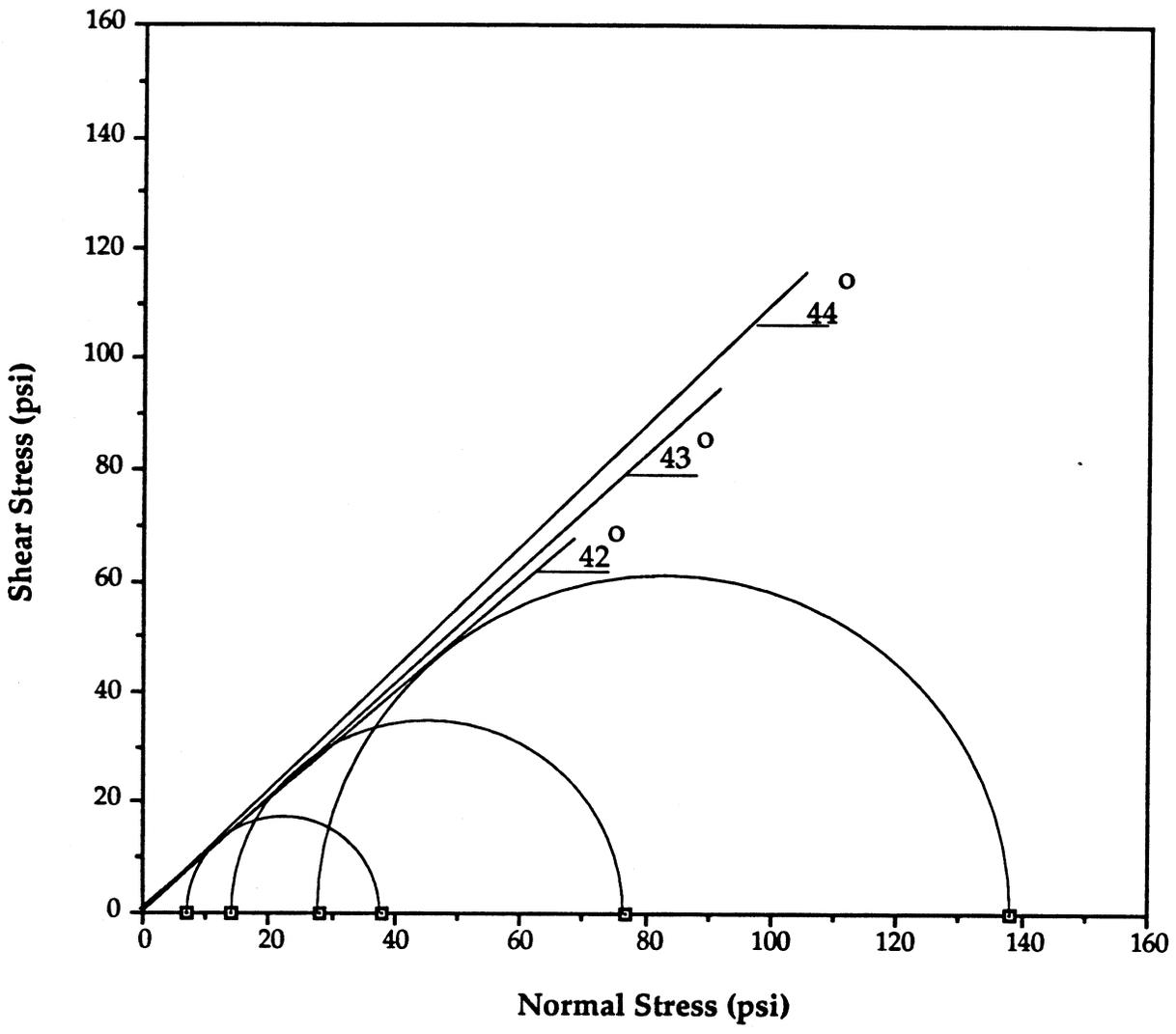


Figure 3.6 Mohr circles for preliminary WPBMRF gradation triaxial testing (1 psi = 6.895 kPa)

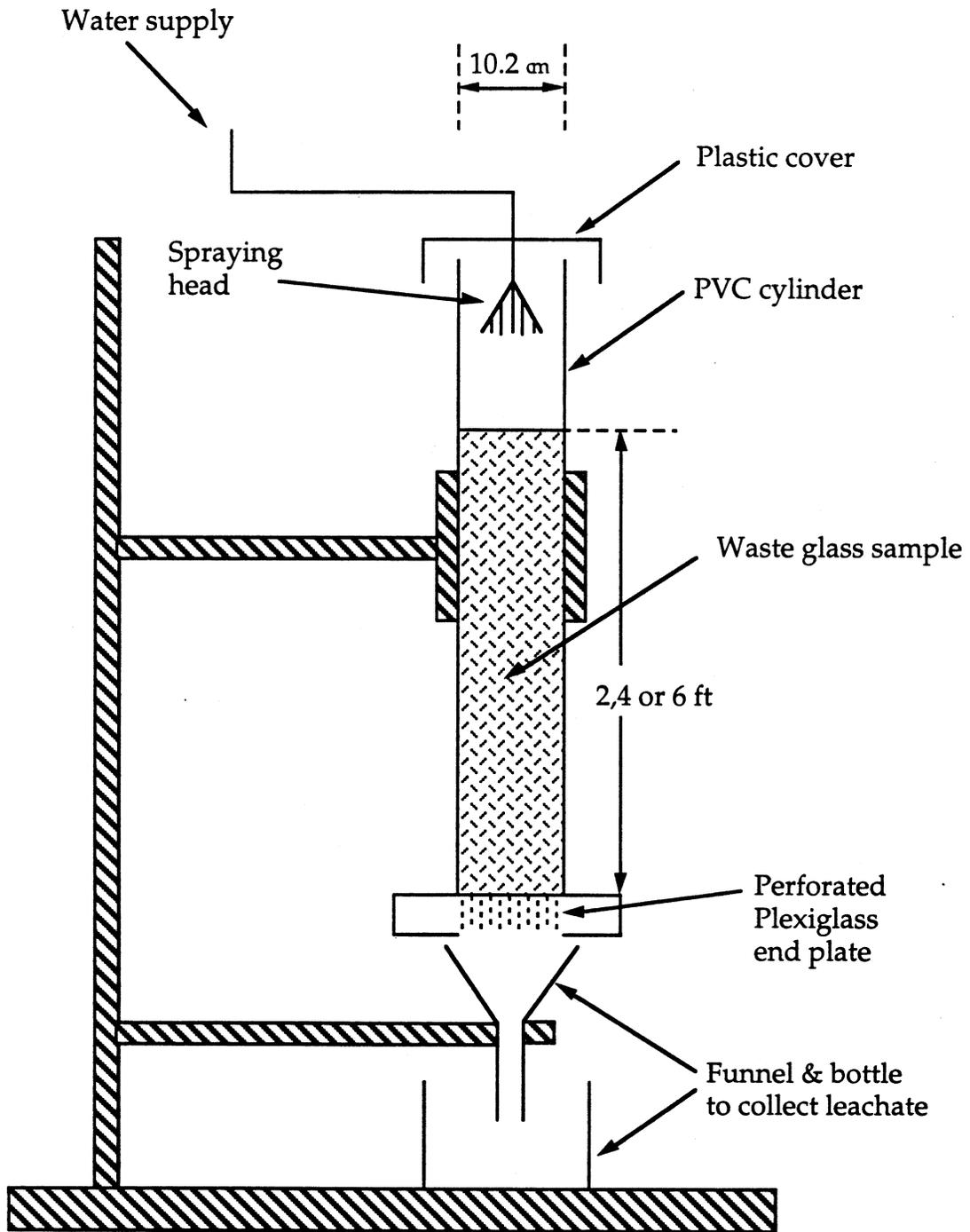
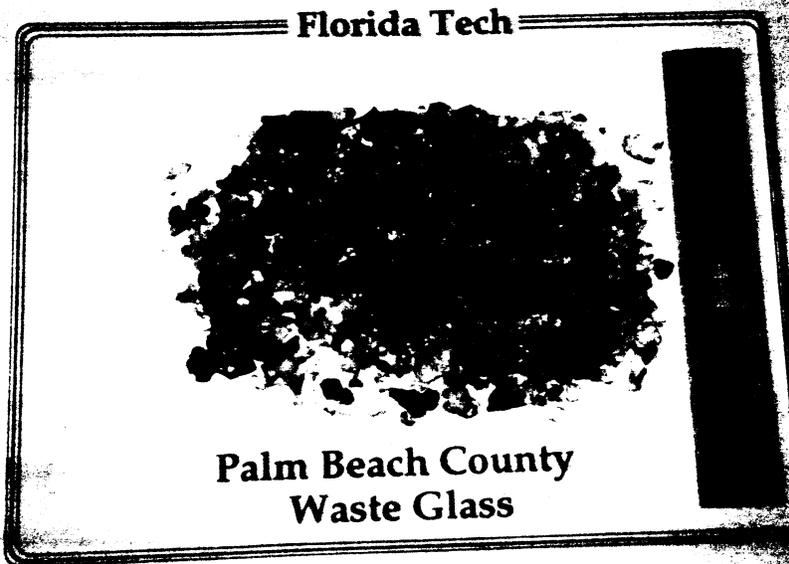


Figure 3.7 Schematic of column leaching apparatus (from Wang, 1993)



Photograph 3.1 Typical BSMG



Photograph 3.2 Typical WPBMRF waste glass

## 4.0 PHYSICAL, GEOTECHNICAL AND ENVIRONMENTAL PROPERTIES

### 4.1 Visual Classification

Visual classifications were performed only on WPBMRF cullet. The cullet from Southeast Recycling was difficult to sample and visual classifications of this material would have been biased from the shredding process that was used at Florida Tech to determine the final gradation. Though no visual classifications of the WG from Southeast Recycling were performed, Photograph 3.1 shows that more paper was present in this WG than in the WPBMRF WG. Because the debris in the BSMG was loose paper and plastics, it was assumed they could be removed using an air separation process. By analyzing a 500 g sample it was found that BSMG contains about 0.7 percent plastic and about 1.0 percent paper.

WPBMRF mixed cullet is similar to a coarse grained material and usually does not have sharp edges if a top size of 3/8-inch (9.4 mm) is maintained. When stored in a 55-gallon barrel, it develops an odor similar to stale beer and a white mold forms on the glass at the top of the barrel. By analyzing 1000 g samples, it was found that WG contains 0.48 percent crushed paper, 0.31 percent plastics and 0.01 percent ceramic/others by weight (Syed, 1994).

Thickness measurements of ten samples of flint, green and amber glass revealed an average thickness of 4.0 mm (0.16 inches). The thickest portion of the bottles is the base and, as can be seen from Table 4.1, green glass is the thickest of the three, most likely due to the fact that wine bottles are included in this group. The base of the bottles measured on average about 5 mm (0.20 inches) or about one-half of the 3/8 inch (9.4mm) top size produced by the RRT facility at WPBMRF.

## 4.2 Grain Size Distribution

The average results from ten sieve analyses conducted on WPBMRF (Figure 4.1) indicate that WPBMRF could be categorized as ASTM #89 material (ASTM D 448). The 89 categorization implies that the top size would correspond to the top size of a #8 and the bottom size would correspond to the bottom size of a #9 material. The average grain size distribution of WPBMRF mixed cullet is compared to the grain size distribution of BSMG in Figure 4.1. WPBMRF mixed cullet has a maximum particle size of 3/8 in (9.4 mm) which is smaller than the 1 in (25 mm) maximum particle size of BSMG. The effective size ( $D_{10}$ ) of WPBMRF ranges from 0.3 to 0.7 mm which is much smaller than the 1.3 to 1.6 range of  $D_{10}$  for BSMG. Less than one percent passes the #200 sieve for both WPBMRF and BSMG indicating that minimal fines exist in the sample. There was less variation in the grain size distribution of WPBMRF glass than in BSMG glass. WG samples were prepared to meet ASTM gradations #8, #9, and #10 to ensure safety in handling.

The grain size distribution of WPBMRF is compared to ASTM D 448 gradations #8, #9, and #10 in Figure 4.2. In Table 4.2, WPBMRF and ASTM D 448 gradations #8, #9, and #10 are further classified as highway materials according to AASHTO and as a soil according to USCS. The revised ASTM standard for USCS is used for the description of the materials (ASTM D 2487). According to AASHTO, WPBMRF, BSMG and the glass meeting the lower limits of ASTM #8, #9 and #10 can be classified as granular material with a group classification of A-1-a, and a group index of zero. ASTM D 448 was an ideal method for classifying WG because it was developed for the classification of construction aggregates and drainage materials.

No scientific analysis was conducted on the safety aspects of handling glass. The WPBMRF WG maximum particle size of 3/8 in (9.4 mm) minus, however, could easily be hand picked and handled without causing injury. Common sense indicated that

BSMG was unsafe to handle due to the 1 inch (25 mm) maximum particle size. The drainage properties of BSMG were therefore not investigated. Because the amount of fines present in WPBMRF, BSMG, and ASTM D 448 #8, #9, and #10 is less than one percent, health concerns inhalation of air-borne glass particles are minimized.

### **4.3 Specific Gravity**

The results of the eight specific gravity tests on WG passing the #4 sieve are discussed below. The average specific gravity was between 2.45 - 2.50. This value compares to the value of 2.5 obtained by Dames & Moore (1993). The specific gravity of WG is approximately 10 percent lower than that of conventional granular fill materials which are in the range of 2.65 - 2.70.

### **4.4 Density**

Several different methods were used to investigate WG's maximum density. First, a vibratory maximum density was determined by placing WG samples in a Proctor mold, attaching the mold to a plate vibrator, placing a 5 lb (22.2 N) surcharge on the sample and vibrating until a maximum was obtained. This test produced lower densities than the maximum values reported by Dames & Moore (1993), and therefore, was not considered accurate enough to give reliable results.

A second vibratory maximum density was determined by placing WG in a calibrated five gallon bucket and using a concrete vibrator to compact glass. Besides being time consuming and nonrepeatable, there were problems associated with accurately measuring the WG volume in the bucket. In the end, this procedure was not considered accurate enough to give reliable results.

When a maximum density test was conducted using Proctor compaction energy in accordance with ASTM D 698-83, glass particles spilled from the mold as the compaction hammer contacted the WG surface. It was assumed that this phenomenon could be attributed to the low surface tension and rigidity of the glass particles. This problem indicated that the conventional Proctor moisture-density relationship did not exist.

Maximum densities obtained using the Modified Marshall-Proctor method produced results close to that of the Clean Washington Center as reported by Dames & Moore (1993). The grain size distribution of the glass determined from a sample after compaction indicated no change in grain size distribution and therefore no significant degradation of the particles. The Modified Marshall-Proctor method for compaction was found to be satisfactory to determine the maximum densities of glass meeting ASTM #8, #9, #10, and WPBMRF.

The maximum and minimum density values for WG are listed in Table 4.3. The maximum densities obtained for WG meeting ASTM D 448 #8, and #9 range from 99 to 105 pcf (1.55 to 1.65 kN/m<sup>3</sup>), whereas for #10 gradation the maximum density ranges from 81 to 87 pcf (1.27 to 1.37 kN/m<sup>3</sup>). These differences are believed to be a results of the smaller D<sub>10</sub> of ASTM #10 WG. WPBMRF (#89) glass had a higher maximum density (111 pcf, 1.74 kN/m<sup>3</sup>) than the #8, #9, and #10 materials because it is classified as a well-graded sand with gravel as per USCS. Therefore when it was compacted, it yielded a higher maximum density than the poorly graded #8, #9, and #10 materials. The maximum densities of the WG is approximately 20 percent lower than that of the conventional fill material.

The minimum density of glass meeting #8, #9, and WPBMRF gradations ranges from 83 to 85 pcf while for the #10 gradations it ranges from 53 to 75 pcf (0.83 to 1.17 kN/m<sup>3</sup>). These densities are higher than the 33 to 55 pcf (0.52 to 0.87 kN/m<sup>3</sup>) specified by FDOT for lightweight aggregate (Florida Department of Transportation, 1991). The

minimum density of the WG, which is about 83 pcf (1.30 kN/m<sup>3</sup>), is lower than the minimum density of gravel (100 pcf, 1.57 kN/m<sup>3</sup>) by approximately 20 percent (Bowles, 1979). This lower density could be advantageous in highway applications such as when it is used behind a retaining wall where it would exert less pressure against the wall than conventional gravels.

## 4.5 Permeability

The coefficient of permeability measured was found to range from 0.003 to 8 cm/sec, depending upon the gradation, type of glass and density. Table 4.4 summarizes the range of permeability for each gradation tested. Permeability values reported by Dames & Moore (1993) for WG were less than 1 cm/sec.

### 4.5.1 Relationship between Permeability and Density

The range of permeabilities for WG meeting ASTM #8, #9, and #10 classification at upper, average and lower limit of gradations, and WPBMRF are listed in Table 4.4. The variation of permeability for ASTM #8, #9, #10, and WPBMRF gradations with respect to density was studied (Syed, 1993). A linear relationship does exist between density and permeability. The relationship between permeability and density showed less than one order of magnitude (cm/sec) difference between the permeabilities at the minimum and maximum density. Fine-grained soils inherently have much larger variations (Holtz and Kovacs, 1981).

A summary plot (Figure 4.3), presents the relationship between permeability and density for WG meeting ASTM D 448 #8, #9, #10 and WPBMRF gradations. These gradations represent coarse to fine gradations. This plot shows that the permeability coefficient decreased linearly by less than one order of magnitude with an increase in

density due to the reduction in void ratio at higher densities. The results indicated that density had a relatively small influence on the permeability.

Figure 4.3 shows results indicated that the permeability coefficient for #10 average, and #10 upper was lower than #8, #9, #10 lower, and WPBMRF gradations by two orders of magnitude. Therefore, finer WG gradations have a lower permeability coefficient than the coarser gradations (Figure 4.3). The percent finer corresponding to #16 and #50 sieves for ASTM #8, #9, and #10 lower limit material is in the range of 0 to 5 percent (Figure 4.2) which indicated that these materials are coarser and resulted in higher permeabilities in the range of 0.7 to 10 cm/sec. The percent finer corresponding to #100 sieve for #10 average and #10 upper limit of gradation is in the range of 10 to 30 percent (Figure 4.2) indicating that these materials are finer, and resulted in a lower permeability in the range of 0.003 to 0.01 cm/sec. The plot shows that effective grain size controls the permeability because  $D_{10}$  of #10 average and upper is 0.075 which is much lower than  $D_{10}$  of #8, #9, #10 lower limit, and WPBMRF gradations which lie in the range of 0.5 and 4.75 mm. The results also indicate that the coefficient of uniformity did not influence permeability as much as variations in  $D_{10}$ .

#### 4.5.2 Relationship between Permeability and Gradation

The results from the permeability tests show that; 1) at the lower limit of gradation the permeability coefficient is higher because it represents a coarser gradation and, 2) at the upper limit of gradation the permeability coefficient is lower because it represents a finer gradation. Figure 4.3 shows that  $D_{10}$  has a significant effect on permeability. This conclusion would be supported by Hazens' work which focused on predicting permeabilities of loose clean filter sands with  $D_{10}$  between 0.1 and 3.0 mm (Holtz and Kovacs, 1981).

#### 4.5.3 Relationship between Permeability Coefficient and Effective Grain Size ( $D_{10}$ )

The permeabilities determined using the constant head test apparatus were compared with the permeability coefficients which would be predicted using Hazens' relation (Figure 4.4). Hazens' empirical relation to determine permeability coefficient is expressed as (Lamb and Whitman, 1979)

$$k = C D_{10}^2 \quad (4.1)$$

where:  $k$  is the coefficient of permeability in cm/sec,  
 $C$  is a constant ranging from 0.41 to 1.46 with 1.00 commonly chosen and  
 $D_{10}$  is the effective size in millimeters.

Hazens' relation is applicable for clean sands with  $D_{10}$  in the range of 0.1 to 3.0 mm, and coefficient of uniformity less than 5. A band was plotted for arbitrary  $D_{10}$ 's in the range of 0.1 to 3.0 mm using  $C = 0.41$  and 1.46, where 0.41 and 1.46 represent dense and loose states, respectively (Figure 4.4). The laboratory permeability values were then plotted on the same graph. The plot shows that the permeabilities calculated from laboratory tests lie within or are greater than the band (Figure 4.4). The permeability coefficient of WG as reported by Dames & Moore (1993) also lies close to the band. Based on this data Hazens' equation with  $C = 0.41$  would yield a conservative permeability.

#### 4.5.4 Relationship between Permeability and Void Ratio ( $e$ )

Figures 4.5 through 4.8 are plots showing the variation of permeability with respect to  $e^2$  for ASTM D 448 gradations #8, #9, #10 and WPBMRF, respectively. The relationship between  $e^2$  and permeability coefficient is relatively linear. Figures 4.5 through 4.8 show that the correlation coefficients ( $R^2$ ) for #8, #9, and #10 average and upper limit, and #10 lower limit material lie in the range of 0.71 to 0.97. As  $R^2$

approaches unity, it is assumed that a linear relationship exists between the two variables. Although no standard minimum  $R^2$  is used to determine whether or not there is a linear relationship, the consistency in the  $R^2$  values suggests a linear relationship for WG does exist between  $e^2$  and permeability. The regression coefficient for WPBMRF is 0.63 and is lower because  $D_{10}$  of WPBMRF varied between 0.3 and 0.7 mm. This relationship has been reported for various fine-grained soils (Lamb & Whitman, 1979). These graphs also indicates that for fine-grained materials like #10 average and upper, data points are oriented towards the origin whereas, for the coarser material, like #8 upper, it is directed above the origin.

#### 4.6 Specific Yield

The range of specific yield values of WG are listed in Table 4.5. Individual plots showing the variation of specific yield with respect to density for a particular gradation were developed. Conclusions were not able to be developed from the individual plots, therefore, a summary plot was developed. This summary plot (Figure 4.9) shows the relationship between density and the coefficient of permeability for ASTM D 448 gradations #8, #9, #10, and WPBMRF. The specific yield changes by 10 percent, approximately with a 12 percent increase in density for the ASTM #8, and #9 gradations. For the ASTM #10 gradation, the specific yield changes were 10 percent with a 35 percent increase in density. The specific yield for WPBMRF decreases by a magnitude of 16% with an increase of density by 15 pcf (or 20 percent). These results indicated that the specific yield increases with a decrease in density which is due to a decrease in the void ratio. These results also indicated that for all gradations, specific yield is at a minimum when the sample is tested at its maximum density. By comparing the range of specific yield for #10 lower and #10 upper limit gradations, it is clear that the specific yield for #10 upper is lower than that at #10 lower by one order of

magnitude. This decrease implies that for finer gradations like #10 average and upper, the effect of D<sub>10</sub> on the specific yield is greater than density. This behavior of specific yield with respect to density is similar to permeability. These results indicated that glass aggregate meeting ASTM #8, #9 and #10 lower limit of gradation had a higher specific yield than most of the conventional fill materials (Todd, 1980).

#### 4.6.1 Relationship between Specific Yield and Gradations

Specific yield values were higher at the lower than at the average and upper limits of gradation which was expected because the lower limit represents a coarser gradation.

#### 4.6.2 Relationship between Specific Yield and Porosity

The relationship between porosity and specific yield for ASTM #8, #9 and #10 designation, and WPBMRF is shown in Figure 4.10. The relationship between porosity and specific yield is relatively linear with specific yield increasing as porosity increases. Specific yield is maximum when the volume of water, drained by gravity from a saturated specimen, is equal to the volume of voids. The diagonal line, extending from the origin to the right corner, represents the maximum specific yield condition. The graph (Figure 4.10) shows that the data points for #8, #9 and #10 lower and WPBMRF lie below the maximum specific yield line with the exception of the data points corresponding to #8 upper or #8 lower limit of gradation. These points lie on or near the maximum specific yield line, with one data point of #8 lower limit lying slightly above the maximum specific yield. An error of 8% was determined for this data point, which corresponds to the loosest density for #8 lower limit. When the loose sample was saturated in testing, a settlement of less than 1/4 inch (6.3 mm) occurred.

An increase in effective size increases the specific yield; and, the specific yield for #10 average and upper limit is much lower than #8, #9 and #10 lower limit. The relatively high specific yield of WG meeting ASTM #8, #9, WPBMRF, and #10 lower could be due to lower surface tension effect between glass and water. The uniformity coefficient ( $C_u$ ) does not play a significant role in controlling permeability.

#### 4.6.3 Relationship between Specific Yield and Permeability

The summary plot (Figure 4.11) shows the relationship between coefficient of permeability and specific yield for ASTM D 448 #8, #9, #10, and WPBMRF gradations. As permeability decreases, the specific yield decreases. The relationship between coefficient of permeability and specific yield is relatively linear. A large gap can be seen between data points corresponding to #10 upper and average when compared to #8, #9, #10 lower, and WPBMRF gradations. This gap occurs due to a large difference in permeability and specific yield range for #10 average and upper limit as compared to #8, #9, and #10 lower limit. This variation is due to the large difference in the  $D_{10}$ . The behavior of specific yield is similar to that of permeability because both are functions of void ratio,  $D_{10}$ , and gradation. The individual plots showing the relationship between permeability and specific yield for ASTM gradation #8, #9, and #10 were excluded from this report because the conclusions from the summary plot (Figure 4.11) show the effect of  $D_{10}$ .

#### 4.7 Drainage Analysis

One of the major causes of pavement deterioration is moisture due to poorly designed drainage layers. Clogging of filters in the pavement results in a poor drainage system, thus leading to premature pavement distresses. Carpenter *et al.*, (1981)

developed a procedure for determining how efficiently a pavement layer could remove free water. A drainage analysis using this procedure was performed on WG meeting ASTM D 448 gradations #8, #9 and #10. Input parameters in this analysis were found based on grain size analyses, moisture-density relationships and permeability testing. They are used in conjunction with the pavement cross-section geometry to determine the time required for the specified drainage layer to achieve a saturation level below 85%, assuming the layer is initially saturated. All input parameters needed to conduct this analysis were determined during WG drainage research.

The shorter drainage time indicates that water from the pavement will drain very fast, thereby preventing significant deterioration. A well-designed drainage layer in the pavement system will remove free water quickly keeping the saturation level is  $\leq 85\%$  (Carpenter et al., 1981). Drainage time is a direct indication of a pavements' drainage quality. For an excellent base, the typical drainage time, corresponding to 85% saturation, must be less than 2 hours. Drainage quality of the layer is considered good if the 85% saturation time is 2 to 5 hours fair if it takes 5 to 10 hours poor if it takes more than 10 hours and very poor if it is greater than 10 hours.

Results of drainage analysis on WG meeting ASTM #8, #9, #10 and WPBMRF are presented in Table 4.6. Results indicate that drainage time of material is controlled significantly by  $D_{10}$ . The trend shows that lower values of  $D_{10}$  yielded significantly longer drainage times. The drainage time for ASTM D 448 #8 and #9 gradations to reach 85 percent saturation is  $\leq 0.01$  hours (less than 2 hours) and is therefore, an excellent drainage material. The WPBMRF and ASTM #10 lower limit of gradation behaves as an excellent draining material because the drainage time is  $< 0.1$  hours (less than 2 hours). Drainage times of #10 average and upper are 61 and 70 hours much greater than the 10 hours thus indicating poor drainage quality. This pavement geometry is believed typical for a single-lane roadway in Florida. Although other

geometry's may change these results (extremely good i.e.  $\leq 0.01$  hours or extremely poor i.e. 70 hours) geometric variations they will not change these conclusions.

#### 4.8 Confined Compression

Four confined compression tests were conducted on each of the four WG gradations. Tests included unload-reload cycles at three stress levels to determine variation in response. Volumetric strain versus vertical stress curves were developed to analyze this one-dimensional response. WG samples were subjected to stresses up to 210 psi (1470 kPa). Additionally, gradation analyses conducted before and after each confined compression test showed negligible change in the grain size distribution from testing pressures up to 210 psi (1470 kPa). The largest variation in size occurred at the No. 4 sieve with about a 10% decrease passing for 2 of the 16 samples analyzed. However, the remaining grain size checks showed less than a 2% change passing throughout the curves. This analysis indicates that very little degradation will occur in confined compression for stresses up to 210 psi (1470 kPa). These results translated to the field, mean that very little degradation would occur for WG subjected to high static stresses in a confined zone. However, this conclusion does not mean that field compaction equipment will not crush the grains because WG will not be confined near the surface.

Figure 4.12 shows a typical strain-stress plot for a confined compression test on WG. During this type of loading, WG experiences a strain-hardening. There is also a significant difference in strain-stress responses for unload-reload cycles and virgin loading conditions. Typically, there was an order-of-magnitude increase in the slopes from the "virgin" loading curve to the unload-reload curve. Consequently, elastic moduli calculated from point-to-point values increased from about 5000 psi (42,000 kPa) to 50,000 psi (420,000 kPa).

## 4.9 Direct Shear

Four direct shear tests were conducted on each gradation of mixed cullet. Each consisted of three samples at the same relative density with normal stresses of 1000 psf (49 kPa), 2000 psf (98 kPa), and 4000 psf (196 kPa). This resulted in 52 direct shear tests. Shear strength envelopes for each density and gradation were developed by plotting peak shear strength against corresponding normal stress. These envelopes are somewhat nonlinear as shown in the typical plot given in Figure 4.13. This nonlinearity implies that a constant friction angle should not be used for WG unless it is conservatively chosen. Based on data from Figure 4.13, a conservative friction angle would be approximately  $34^{\circ}$ . Table 4.7 presents the friction angle for each gradation from the direct shear tests. Reported values are based on an average friction angle from data points shown in Figure 4.13. These friction angles are considered relatively high for granular materials. Das (1994) reports friction angles of 30 to 45 degrees for loose to dense angular sands respectively, and 27 to 38 degrees for loose to dense rounded sands respectively. For this study, it was assumed that #8, #9 and WPBMRF materials are angular and #10 material is somewhat rounded. Table 4.7 has a range of densities for each gradation, with the lowest density corresponding to the lowest friction angle. These results imply that WPBMRF at 91 pcf ( $14.3 \text{ kN/m}^3$ ) has a friction angle of  $40^{\circ}$ , significantly above the typical 30 to 35 degrees reported by Das (1994). Problems inherent in the direct shear test included maintaining the proper gap between shear boxes and development of stress concentrations. These problems have significantly affected results and indicate that direct shear testing will significantly overestimate WG friction angles.

#### 4.10 Triaxial Shear

The triaxial shear testing program consisted of conducting CD triaxial tests at 14 psi (98 kPa) confining pressure with different densities for each gradation. WG has no cohesion. Each test was therefore used to determine the angle of internal friction for the gradation and density tested. A total of 23 tests were conducted: 5 tests on WPBMRF gradation, and 6 on the ASTM gradations. In addition to vertical stress-strain plots, volumetric strain versus vertical strain plots were developed as shown in Figure 4.14. This figure shows a typical stress-strain response for all WG triaxial samples.

Figures 4.15a through 4.15d illustrate the variation in friction angle versus density for strain levels from 1 to 10% for the WG gradations. Generally, the friction angle increases slightly as density increases. The most significant increase is depicted for ASTM #10 (Figure 4.15d). Friction angles at low densities ranged from 20 to 38 ° and friction angles at high densities ranged from 24 to 43 °. These values are more consistent with friction angles for angular materials than those determined from direct shear testing. Das (1994) reports typical friction angles from 40 to 45 ° for dense angular sands.

Secant moduli from the origin of the stress-strain plots to strain levels from 1 to 5% for these same gradations are illustrated in Figure 4.16. The highest secant moduli correspond to 1% strain, and the lowest correspond to 5% strain as expected. Secant moduli increased slightly as density increased and decreased as strain level increased. Similar to friction angles, the most significant increase in secant moduli was associated with the # 10 gradation.

## **4.11 CBR/LBR**

Four sets of CBR/LBR values were determined from the CBR testing on each of the four WG gradations. Table 4.8 summarizes these results. Both sets of bearing ratios are considered low. Baker, (1983) suggests that subgrade CBR values between 3 and 6 are poor. The CBR values in Table 4.8 are nearly all below this range. Bearing ratio testing on WG was difficult. The top of the samples were easily disturbed which allowed the piston to penetrate into WG.

## **4.12 Environmental WG Analysis**

### **4.12.1 Availability of WG**

To determine the availability of WG for use as fill material, a survey was conducted among all the counties in Florida. A copy of the survey is presented in Figure 3.1 and the results are presented in Table 4.9. Forty-four of the 67 counties responded. Of those that responded, most were interested in selling their WG for \$3/yd<sup>3</sup>. Of those that were interested, 9 had WG stockpiled and 22 had storage space available for stockpiling.

### **4.12.2 Method of Leachate Generation**

The Clean Washington Center (1994) in Seattle performed a similar WG investigation. The method they used to generate leachate was the sequential batch extraction in accordance with ASTM D 4793. This method calls for a 100g sample to be agitated continuously for 18 hours at 30 rpm in a one gallon container. The solids concentration should be at least 5%, and the container should be 90% full. They

reported BOD concentrations ranging from non-detectable to 6.4 mg/l for the first extraction's and no detectable levels for subsequent extraction's, as was shown in Table 2.1.

The sequential batch extraction method to predict leachate concentrations from an actual fill appears inappropriate. If a 6 ft (1.83m) fill were constructed, it would require 540 lb (2400 N) of glass per square foot ( $2640 \text{ Kg/m}^2$ ) based on a density of  $90 \text{ lb/ft}^3$  ( $1443 \text{ Kg/m}^3$ ) to properly conduct sequential batch extraction. As stated above ASTM's required ratio is 100 g of sample per one gallon container and the container should be 90 percent full. Assuming a porosity of 0.4, the glass to water ratio would be 1:34 when calculated on a weight basis. The use of the column extraction method simulates actual field conditions. The waste was placed in the column in an unsaturated condition, and water was applied at the required 2 in/hr rate (5.1cm/hr) to simulate a typical Florida rainfall event. Leachate was collected from the bottom of the column, similar to water draining from a fill area behind a retaining wall.

Prior to using the column apparatus to generate a leachate, the shake extraction method was used in accordance with ASTM D 3987. This method calls for a 140g sample in a one gallon container. The water and sample should occupy 80% to 90% of the container. The weight ratio of glass to water was approximately 1:24. The sample was agitated continuously for 18 hours at 30 rpm. The leachates' BOD<sub>5</sub> was 1 mg/l, which was comparable to the values that were presented in the Clean Washington Report (Dames and Moore, 1994). The low level of BOD<sub>5</sub> was due to the high dilution ratio and does not simulate any field conditions where glass would be used as a fill material.

The column leaching test was conducted with column heights of 2, 4 and 6 ft (60, 120 and 180 cm), to simulate field conditions where the glass would be used as fill material. Several trial runs were performed to determine the appropriate dilution ratio

for BOD<sub>5</sub> testing. Once found, the samples exhibited a large range of concentrations that decreased exponentially.

The column leaching analysis of BSMG mixed cullet are presented in Tables 4.10, 4.11, and 4.12 and WPBMRF results are presented in Tables 4.13, 4.14 and 4.15. The results are discussed in the following sections.

#### 4.12.3 Biochemical Oxygen Demand (BOD)

BOD testing is essentially a bioassay procedure involving the measurement of oxygen consumed by living organisms (mainly bacteria) while utilizing the organic matter present in a waste, under conditions similar to those that occur in nature. The total or ultimate BOD is generally symbolized as  $L$ ; and BOD exerted at any time  $t$  is symbolized by  $y$ . The usual form of the equation to determine BOD is:

$$y = L(1 - 10^{-kt}) \quad (4.2)$$

A variety of methods can be used to evaluate  $k$  and  $L$  from BOD versus time data. The Thomas graphical method (Sawyer, 1978) was chosen for its simplicity. The procedure for determining BOD constants by this method is as follows:

1. From the experimental results of  $y$  and  $t$ , calculate the value of  $\left(\frac{t}{y}\right)^{-1/3}$  for each day.
2. Plot  $\left(\frac{t}{y}\right)^{-1/3}$  versus  $t$  on arithmetic paper and draw a best fit straight line.
3. Determine the intercept  $A$  and slope  $B$  from the plot or regression.
4. Calculate  $k$  and  $L$  from:

$$k = 2.61 \frac{B}{A}, \quad (4.3a)$$

$$L = \frac{1}{6A^2B}. \quad (4.3b)$$

As part of determining the dilution ratios used to analyze the column leachate, a study of the oxygen uptake was performed by measuring the oxygen concentrations

every day for 5 days, and calculating the ultimate BOD,  $L$ , and the rate of constant  $k$ . Only WPBMRF mixed glass was used for this study. Three leachate samples were collected from the 2 ft. (60 cm) column at 36 minute time intervals generating three - 250 ml samples. The average rate constant was 0.08. The ultimate BOD of the three samples was approximately 600 mg/l, 400 mg/l, and 375 mg/l. The average BOD<sub>5</sub> was 60% of the ultimate BOD. This percentage can be used to convert any of the five day BOD's reported in this study to ultimate BOD's.

Column leaching tests BOD<sub>5</sub> values for BSMG had initial concentrations of 435, 1470 and 2880 mg/l at leaching times of 14, 22, and 33 hours, and final BOD<sub>5</sub> concentrations of 6.6, 10.8 and 49.5 mg/l, at these times, respectively, for the 2, 4 and 6 foot (60, 120 and 180 cm) columns.

WPBMRF WG had 479, 235 and 855 mg/l initial BOD<sub>5</sub> concentrations at leaching times of 14, 34, and 53 hours, and final concentrations of 12, 12, and 6 mg/l at these times for the for the 2, 4 and 6 foot (60, 120 and 180 cm) columns. This WG leachate is considerably less contaminated, although it still exhibits pollutant levels greater than raw domestic waste water.

#### 4.12.4 Total Kjeldahl Nitrogen (TKN)

The TKN data was presented in the same tables (4.10 to 4.15) as the BOD data. TKN analyses were performed on the first and last leachate samples. The initial TKN values are very high and could be contributing to the large BOD through nitrification processes. The BSMG WG, TKN concentrations for the 2, 4, and 6 ft (60, 120 and 180 cm) columns were 32, 114 and 345 mg/l initially and 2, 1 and 7 mg/l in the final samples, respectively. WPBMRF WG, TKN concentrations were 37, 55, and 67mg/l initially and 1, <1, and 2 mg/l in the final sample for the 60, 120 and 180 cm) columns. The TKN typical value for raw domestic waste water is 40 mg/l and for treated

domestic waste water 5 mg/l. Therefore, TKN would be high initially and acceptable after some time when washed.

#### 4.12.5 Total Phosphorous (TP)

Total phosphorus (TP) concentrations ranged from 0.4 to 2.0 mg/l for BSMG, and 1.4 to 2.8 mg/l for WPBMRF WG, for the first samples, respectively, and final samples had concentrations of 0.03 to 0.14 mg/l and 0.17 to 0.31 mg/l, respectively. Typical TP concentrations in raw domestic waste water are 8 mg/l and treated waste water are 1 mg/l. Both WG sources have leachate concentrations similar to treated wastewater. Once WG is processed to remove the other contaminants, the phosphorus is not expected to be a problem.

#### 4.12.6 Solids

Since a filter was used to hold the glass in place, low suspended solid concentrations were expected. No or negligible suspended solids concentrations were measured. The dissolved solids concentrations were so low that they hardly enough to measure. A low level of solids were observed on the filter at the bottom of the column. The solids levels are not expected to create any environmental concerns.

#### 4.12.7 Rate Constants

Various models were investigated to determine the trend of both BOD and TP concentrations in the leachate. Inspection of Figures 4.17 to 4.22, revealed that neither linear nor logarithmic models could be used to model the data. Power series models were therefore selected to develop equations for these relationships. The equations and

corresponding  $R^2$ -values are summarized in Table 4.16. The average power series exponent for the removal of BOD was  $1.5 \pm 0.29$  and for TP was  $0.76 \pm 0.10$ . The calculated exponent did not correlate with column length, but it was significantly different for each contaminate. The power series equation had high  $R^2$  values ranging from 0.87 to 0.99, indicating a good correlation exists between the variables. The use of these equations are limited, however, because the leaching studies were not performed in duplicate.

#### 4.12.8 Mass of Pollutants Released

Determining the total mass of pollutants released, requires determining the area under the curve in Figures 4.17 to 4.22. Table 4.17 shows the total mass of BOD and TP released according to this technique. These values are useful when comparing leachate concentrations to the volume of liquid used to extract the pollutants.

To provide usable quantities, all facilities would have to accumulate WG for at least six months. During stockpiling, biological degradation and rainfall occurrences could be sufficient to "clean-up" the glass so that the leachate would exhibit pollutant concentrations similar to normal storm water. The total quantity of rainfall required to clean WG would be a function of the height of the glass piles. Storing glass in 2 ft (60 cm) layers or less would significantly reduce the required rainfall for cleaning. If accumulation periods of years are necessary, then the storage height of the glass piles could exceed 6 ft (180 cm). It is likely that degradation processes would greatly reduce the required storage periods. It is also possible that recirculating leachate over the WG piles could provide treatment in a few weeks.

#### 4.12.9 Comparison to Shake Extraction

Column leaching is not a very convenient method for determining WG pollution potential. It takes up to three days to run the column extraction and during testing samples must be taken continuously. The shake extraction method is much more convenient, even though it takes 18 hours to complete, because after testing begins the lab technician only records data after 18 hours. This advantage lead to a proposed volumetric glass to water dilution ratio for the shake extraction method of 1:1. Using this ratio the expected concentrations for BSMG and WPBMRF WG would be 9 to 38 mg/l and 3 to 12 mg/l, respectively. These ranges represent about 10% of the average concentrations (Tables 4.10 to 4.15) and 1.3% of the peak concentrations. An analysis of BOD, TKN and TP was done on five materials including three materials typically used in highway construction and the two WG sources. The WG samples were WPBMRF WG, BSMG, and the highway fill materials tested were marl, coquina, and sand. Fill materials were obtained from the Blackhawk Quarry in Melbourne, Florida. All samples were agitated on a stirring table for 18 hours at 30 rpm in 1 liter containers with airtight closures. The results are in Table 4.18. The WG concentrations are within expected ranges, using the percentages above would yield average and peak concentrations of 81 and 623 mg/l for WPBMRF WG and 107 and 823 mg/l for BSMG, respectively.

#### 4.13 WG Data Base

The following tables have been assembled as a data base for suppliers, costs, quantities, engineering and environmental properties of mixed cullet. As more information is compiled it can be added to the existing tables.

A three page summary of the results from the WG survey distributed to all 67 counties in Florida was presented in Table 4.9. It shows that 44 of the counties responded, 9 have WG stockpiles, most were interested in selling WG for \$3.00/cubic yard and 22 had storage space available for stockpiling.

Table 4.19 is a summary of the WG physical and engineering properties. The reader is either given the properties directly or referred to the appropriate table or figure in the text. A series of remarks have been included to give the engineer a basic understanding of the particular property of interest.

#### 4.14 WG Materials Cost

Cost estimates for WG have been developed from various phone conversations with industry experts in Florida. These costs can be compared to costs for conventional base course materials and general fills. According to Dan Christy of the Blackhawk Quarry in Palm Bay, Florida (1995) cemented coquina base materials can be purchased and loaded into trucks for \$5.25/ton, while general fill can be purchased and loaded for \$1.00 to \$1.50 per ton. Tarmac Incorporated of Cocoa, Florida sells limerock base for \$6.10/ton loaded (Russo, 1995). WG currently is stockpiled at West Palm Beaches' MRF and could be purchased for less than \$1.00/ton. This price would include the costs for environmental testing plus loading the material into haul trucks. According to the results presented in the survey (Table 4.9), WG throughout Florida would be sold by many counties for approximately \$3.00 per cubic yard, or approximately \$3.00 per ton. This cost may be misleading for two reasons; first a minimum cost was not specified in the survey and second, the survey did not specify whether or not this price was to include loading and delivery. WG could very likely be sold for less than the \$3.00 per cubic yard if the county MRF owned the WG, because they could make income from stockpiled WG that currently is used for daily landfill cover.

Table 4.1 Thicknesses of recycled glass bottle fragments

Sample No.	Brown glass Thickness (mm)	Clear glass Thickness(mm)	Green glass Thickness(mm)
1	4.5	2.5	5.5
2	5.0	2.5	4.5
3	4.0	4.0	5.0
4	1.8	4.5	4.0
5	2.5	4.0	4.0
7	2.0	4.5	5.0
8	2.0	2.0	5.0
9	4.0	4.5	4.5
10	2.0	4.0	2.0
<b>Mean</b>	$4.0 \pm 1.2$	$4.2 \pm 0.9$	$4.1 \pm 1.2$

Table 4.2 WG highway classifications

ASTM D 448 Gradation	Coefficient of Uniformity	Coefficient of Gradation	USCS Classification	AASHTO * Classification
# 8 Upper Limit	2.6	1.4	Poorly Graded Gravel with Sand (GP)	A-1-a
#8 Average	2.3	1.3	d.o.	A-1-a
#8 Lower Limit	1.6	0.9	Poorly Graded Gravel (GP)	A-1-a
#9 Upper Limit	2.3	0.9	Poorly Graded Sand (SP)	A-1-a
#9 Average	2.3	1.2	d.o.	A-1-a
#9 Lower Limit	1.5	0.8	Poorly Graded Sand with Gravel (SP)	A-1-a
#10 Upper Limit	1.6	1.1	Poorly Graded Sand (SP)	A-3
#10 Average	14.3	0.8	d.o.	A-1-b
#10 Lower Limit	2.4	0.9	Poorly Graded Sand with Gravel (SP)	A-1-a
WPBMRF	9.3	1.4	Well Graded Sand with Gravel (SW)	A-1-a
BSMG	3.1	1.3	Poorly Graded Gravel with Sand (GP)	A-1-a

\* The Group Index (GI) = 0

Table 4.3 Maximum and minimum densities for WG meeting ASTM #8, #9, #10 and  
WPBMRF gradations

ASTM D 448	Maximum Density (pcf)	Minimum Density (pcf)
# 8 Lower Limit	102	83
# 8 average	102	85
# 8 Upper Limit	99	85
# 9 Lower Limit	102	85
# 9 average	105	85
# 9 Upper Limit	102	83
# 10 Lower Limit	87	75
# 10 average	87	60
# 10 Upper Limit	81	53
WPBMRF	111	83

Table 4.4 Range of permeabilities for WG meeting ASTM D 448 #8, #9, #10 and  
WPBMRF gradations

ASTM D 448 Gradations	Permeability Range (cm/sec)
#8 Lower Limit	5 - 7
#8 Average	6 - 8
#8 Upper Limit	4 - 8
#9 Lower Limit	4 - 8
#9 Average	4 - 10
#9 Upper Limit	1 - 3
#10 Lower Limit	0.7 - 2
#10 Average	0.003 - 0.01
#10 Upper Limit	0.003 - 0.02
WPBMRF	0.3 - 5

Table 4.5 Range of specific yield for WG meeting ASTM D 448 gradations

ASTM D 448	Specific Yield Range (%)
#8 lower limit	38 to 47
#8 average	38 to 43
#8 upper limit	38 to 47
#9 lower limit	39 to 45
#9 average	34 to 41
#9 upper limit	28 to 38
#10 lower limit	33 to 42
#10 average	9 to 18
#10 upper limit	4 to 13
WPBMRF	18 to 33

Table 4.6 Results of WG drainage analysis

**Pavement Layer Geometry**  
 Width of granular layer = 13'  
 Thickness of granular layer = 0.75'  
 Longitudinal slope = 2 %  
 Transverse slope = 1 %

ASTM D 448 Gradation	Maximum Dry Density (pcf)	D <sub>10</sub> (mm)	Coefficient of Permeability (k) (cm/sec)	Specific Yield (N <sub>e</sub> ) (%)	Time to 85% Saturation (hour)
#8 lower limit	92.92	4.75	5.4	42.38	< 0.01
#8 average	95.61	3	6.46	38.14	< 0.01
#8 upper limit	96.22	2.4	3.85	37.61	< 0.01
#9 lower limit	95	2.4	3.92	39	< 0.01
#9 average	96	1.5	3.54	34	0.01
#9 upper limit	97	1.3	1.05	28	< 0.01
#10 lower limit	93	0.15	0.66	33	< 0.1
#10 average	81	0.075	0.003	8	61
#10 upper limit	81	0.075	0.002	4	70
WPBMRF	111	0.5	0.87	20	< 0.1

Table 4.7 Direct shear friction angles for various WG gradations and densities

<b>ASTM D 448 Gradation</b>	<b>Unit Weight (pcf)</b>	<b>Angle of Internal Friction (o)</b>
WPBMRF	91 - 103	40 - 45
# 8 Average	88 - 93	45 - 51
# 9 Average	91 - 101	37 - 45
# 10 Average	95 - 108	34 - 46

Table 4.8 Summary of CBR/LBR testing

<b>ASTM Gradation</b>	<b>Unit Weight (pcf)</b>	<b>CBR (%)</b>	<b>LBR (%)</b>
WPBMRF	88 -107	0.4 - 3.3	0.5 - 4.0
# 8 Average	89 - 95	0.9 - 2.7	1.1 - 3.4
# 9 Average	83 - 97	0.8 - 2.8	1.0 - 3.5
# 10 Average	84 - 98	0.5 - 1.7	0.6 - 2.1

**Table 4.9 Summary of WG Surveys**

No.	County	Collection system used?	Clear	Production tons/year	Brown	Green	Mixed	Addn'l Treatment	Would you sell glass at \$3/cu.yds.?	Is there storage available?	Stockpile currently	
1	Alachua	No response										
2	Bay	No response										
3	Bradford,											
4	Baker & Union	Bins	50	16	10	na	na	green-poss.		no	no	
5	Brevard	Curbside					total 6,614	yes		no	no	
6	Broward	MRF					total 17,500	yes		poss.	no	
7	Calhoun	No response										
8	Charlotte	No response										
9	Citrus	Curbside					total 700	yes		yes	no	
10	Clay	No response										
11	Collier	Curside	16,732				16,444	yes		yes	no	
12	Columbia	Bins					total 170	info		no	yes/ 20 tons	
13	Dade	MRF	2,466	550	1,334		5,590	yes		yes	no	
14	Desoto	Bins					total 35	yes		no	no	
15	Dixie	No response										
16	Duval	No response										
17	Escambia	MRF					156	sort-crush	no	no	no	
18	Flagler	No response										
19	Franklin	Bins	12	8.5	6	3		sort-crush	yes	yes	yes/ 20 tons	
20	Gadsden	No response										
21	Gilchrist	Curb, MRF	14	5	8	na	na	na	info	no	no	
22	Glades	<i>Do not currently have a recycling program</i>										
23	Gulf	No response										

MRF= Material Recovery Facility  
 Curb=Curbside collection  
 Bins=Collection bins or drop-off  
 info=Please send more information  
 poss=Possibly

Table 4.9 Summary of WG Surveys - continued

No.	County	Collection System	Production tons/year			Mixed	Addn'l Treatment	Would you sell glass at \$3/cu.yds.?	Is there storage available?	Stockpile currently
			Clear	Brown	Green					
24	Hamilton	No response								
25	Hardee	Bins, Curb	60	40	0	0	no	yes	no	
26	Hendry	Bins, MRF	35	10	5	na	yes	yes	yes/ 20 tons	
27	Hernando	MRF				total 300	yes	yes	no	
28	Highlands	Bins, MRF	185	63	5	na	info	no	no	
29	Hillsborough	Bins,Curb				total 3,220	-	-	-	
30	Holmes	MRF					yes	yes	no	
31	Indian River	No response								
32	Jackson	No response								
33	Jefferson	No response								
34	Lafayette	The landfill director does not have time to fill out the survey								
35	Lake	No response								
36	Lee	MRF	1,444	740	726	<2,900	yes	yes	no	
37	Leon	Bins,MRF	955	350	355	na	yes	yes	no	
38	Levy	Bins,MRF				total 205	no	no	no	
39	Liberty	No response								
40	Madison	No response								
41	Manatee	Bins,Curb	126	10	11	na	no	no	no	
42	Marion	Curb				total 303	info	poss	no	
43	Martin	No response								
44	Monroe	Curb	389	365	258	na	yes	yes	no	
45	Nassau	No response								

MRF= Material Recovery Facility  
 Curb=Curb-side collection  
 Bins=Collection bins or drop-off  
 info=Please send more information  
 poss=Possibly

Table 4.9 Summary of WG Surveys - continued

No.	County	Collection system used?	Production tons/year			Mixed	Addn'l Treatment	Would you sell glass at \$3/cu.yds.?	Is there storage available?	Stockpile currently
			Clear	Brown	Green					
46	Okaloosa	Curb				total 1,400	no	poss	yes/ 40 tons	
47	Okeechobee	Bins				total 10	srt	yes	yes/ 0.5 ton	
48	Orange	Bins, MRF	2,500	650	775	1,700	sort	no	no	
49	Osceola	Bins, MRF				total 178	crush, wash	-	-	
50	Palm Beach	MRF	1,300	450	1,000	10,000	yes-mixed	poss	yes	
51	Pasco	MRF	825	175	290	1,300	sort	yes	no	
52	Pinellas	No response								
53	Polk	Bins				total 1,900	none	no	no	
54	Putnam	Binx,Curb				total 375	sort	poss	no	
55	St. Johns	-				-	-	no	no	
56	St. Lucie	No response								
57	Santa Rosa	No response								
58	Sarasota***	MRF	1,800	70	1,200	0	sort	no	no	
59	Seminole	MRF, Bins				total 16,300	sort crush	yes	no	
60	Sumter	No response								
61	Suwannee	Bins	20	30	na	na	sort	yes	no	
62	Taylor	Bins	25	20	5	5	sort-cursh	yes	no	
63	Union	No response								
64	Volusia	Bins,Curb				total 2,965	sort-crush	yes	no	
65	Wakulla	Bins				total 4	sort	ues	yes/ 5 tons	
66	Walton	-	48,000	48,000	48,000	na	sort-crush	no	yes/ 40,000 CY	
67	Washington	Bins				total 24	sort-crush	yes	yes	

\*\*\* Sarasota County is interested, but survey was filled out by recyclers

MRF= Material Recovery Facility

Curb=Curb-side collection

Bins=Collection bins or drop-off

info=Please send more information

poss=Possibly

Table 4.10 2 Ft. Leaching column results for BSMG

Leaching Time (hrs)	Throughput Volume (mL)	BOD <sub>5</sub> (mg/L)	TDS (mg/L)	SS Volatile (mg/L)	SS Total (mg/L)	TKN (mg/L)	Total Phosphorous (mg/L)
0-2	840	435	0.32	0.003	0.019	31.6	0.42305
2-3	1260						
3-5	2100	27.75	0.112	-0.001	0.002		0.1362
5-6	2520						
6-8	3360	9	0	0.001	0.002		0.1553
8-9	3780						
9-11	4620	14.1	-0.002	0.001	0.001		0.0967
11-12	5040						
12-14	5460	6.6				1.6	0.05995

Table 4.11 4 Ft. Leaching column results for BSMG

Leaching Time (hrs)	Throughput Volume (mL)	BOD <sub>5</sub> (mg/L)	TDS (mg/L)	SS Volatile (mg/L)	SS Total (mg/L)	TKN (mg/L)	Total Phosphorous (mg/L)
0-2	840	1470	1.36	0.009	0.01	114.5	0.50215
2-4	1680						
4-6	2520	143.25	0.234	0	0.003		0.12655
6-8	3360						
8-10	4200	37.875	0.176	0	0		0.14945
10-12	5040						
12-14	5880	25.5	0.152	0	0		0.0598
14-16	6720						
16-18	7560						
18-20	8400						
20-22	9240	10.8	0.12	0	0	0.9	0.0339

Table 4.12 6 Ft. Leaching column results for BSMG

Leaching Time (hrs)	Throughput Volume (mL)	BOD <sub>5</sub> (mg/L)	TDS (mg/L)	SS Volatile (mg/L)	SS Total (mg/L)	TKN (mg/L)	Total Phosphorous (mg/L)
0-3	1260	2880	4.328	0.013	0.153	345	2.00615
3-5	2100						
5-8	3360	510	0.604	0.003	0.048		0.76485
8-10	4200						
10-13	5460	240	0.318	0	0.037		0.54755
13-15	6300						
15-18	7560	123	0.286	0	0.032		0.3864
18-20	8400						
20-23	9660	67.5	0.244	0.01	0.034		0.20255
23-25	10500						
25-28	11760	58.5	0.252	0.008	0.031		0.17695
28-30	12600						
30-33	13860	49.5	0.064	0.01	0.032	7	0.14155

Table 4.13 2 Ft. Leaching column results for WPBMRF WG

Leaching Time (hrs)	Throughput Volume (mL)	BOD5 (mg/L)	TDS (mg/L)	SS Volatile (mg/L)	SS Total (mg/L)	TKN (mg/L)	Total Phosphorous (mg/L)
0-2	840	479	2.28	0.02	0.03	37	1.3816699
2-3	1260						
3-5	2100	95	0.5	0	0.02		0.5602571
5-6	2520						
6-8	3360	34	0.28	0	0		0.44642
8-9	3780						
9-11	4620	18	0.18	0	0		0.29575325
11-12	5040						
12-14	5880	12				1.2	0.2299063

Table 4.14 4 Ft. Leaching column results for WPBMRF WG

Leaching Time (hrs)	Throughput Volume (mL)	BOD5 (mg/L)	TDS (mg/L)	SS Volatile (mg/L)	SS Total (mg/L)	TKN (mg/L)	Total Phosphorous (mg/L)
0-2	840	235	3.42	0.076	0.186	55	2.7741
2-4	1680						
4-6	2520	51	0.384	0.027	0.074		1.83215
6-8	3360						
8-10	4200	21.35	0.12	0.012	0.036		1.0369
10-12	5040						
12-14	5880	12.95	0.012	0.003	0.011		0.73925
14-16	6720						
16-18	7560						
18-20	8400						
20-22	9240	7.15	-0.002	0	0		0.494
22-24	10080						
24-26	10920	5	0.01	0	0		0.415
26-28	11760						
28-30	12600	4.5	0	0	0		0.3751
30-32	13440						
32-34	14280	12.3	0.014	0.004	0.004	<1.0	0.30585

Table 4.15 6 Ft. Leaching column results for WPBMRF WG

Leaching Time (hrs)	Throughput Volume (mL)	BOD5 (mg/L)	TDS (mg/L)	SS Volatile (mg/L)	SS Total (mg/L)	TKN (mg/L)	Total Phosphorous (mg/L)
0-3	1260	855	5.12	0.029	0.089	67	1.9242375
3-5	2100						
5-8	3360	122.25	5.52	0.029	0.069		1.202509
8-10	4200						
10-13	5460	33	1.86	0.008	0.028		0.8801295
13-15	6300						
15-18	7560		0.5	0	0.026		0.573367
18-20	8400						
20-23	9660		0.172	0.015	0.024		0.363653
23-25	10500						
25-28	11760	12	0.12	0.008	0.005		0.2376015
28-30	12600						
30-33	13860	14.25	0.12	0.009	0.023		0.294492
33-35	14700						
35-38	15960	15.375	0.1	0.006	0.017		0.3915405
38-40	16800						
40-43	18060						
43-45	18900						
45-48	20160	9.45	0.04	0.001	0.008		0.2755285
48-50	21000						
50-53	22260	5.925	0	-0.001	0.004	1.9	0.1662095

Table 4.16 Regression analysis results for BOD<sub>5</sub> and TP

Column length (ft)	WASTE GLASS SOURCE			
	Southeast Recycling		WPBMRF	
	BOD <sub>5</sub>	TP	BOD <sub>5</sub>	TP
2	$y = 6.00 \text{ E}6 \text{ X}^{-1.62}$ $R^2 = 0.9407$	$y = 2.44 \text{ E}2 \text{ X}^{-0.67}$ $R^2 = 0.8933$	$y = 6.00 \text{ E}7 \text{ X}^{-1.93}$ $R^2 = 0.9982$	$y = 2.99 \text{ E}2 \text{ X}^{-0.82}$ $R^2 = 0.9876$
4	$y = 3.00 \text{ E}7 \text{ X}^{-1.62}$ $R^2 = 0.9945$	$y = 9.67 \text{ E}2 \text{ X}^{-0.85}$ $R^2 = 0.9013$	$y = 1.63 \text{ E}5 \text{ X}^{-1.08}$ $R^2 = 0.9166$	$y = 1.84 \text{ E}2 \text{ X}^{-0.65}$ $R^2 = 0.9346$
6	$y = 2.00 \text{ E}7 \text{ X}^{-1.38}$ $R^2 = 0.9899$	$y = 6.53 \text{ E}2 \text{ X}^{-0.87}$ $R^2 = 0.9457$	$y = 4.00 \text{ E}6 \text{ X}^{-1.34}$ $R^2 = 0.9735$	$y = 1.90 \text{ E}2 \text{ X}^{-0.67}$ $R^2 = 0.8787$

Table 4.17 Total mass of BOD and TP released during column leaching

Glass Source	Column (ft)	Total Leachate (ml)	BOD Total (mg)	TP Total (mg)
<b>WPBMRF Mixed Cullet</b>	2	5880	664.5	3.33
	4	14280	358.5	12.47
	6	22260	1852.5	12.06
<b>BSMG</b>	2	5460	494.5	0.99
	4	9240	1925.3	1.23
	6	13860	6547.1	8.00

Table 4.18 Shake extraction results

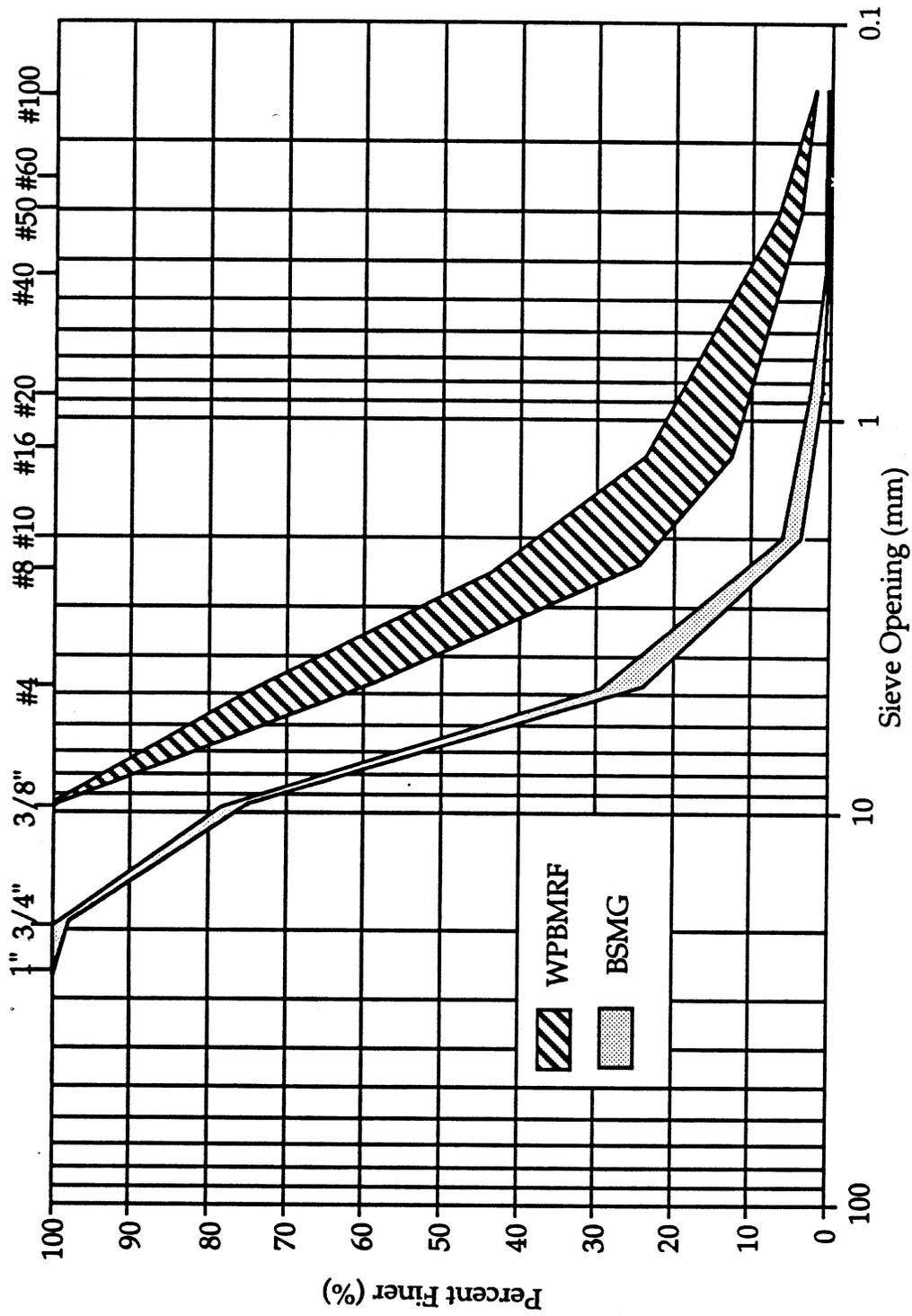
Sample	Total Phosphorous (mg/L)	TKN (mg/L)	BOD <sub>5</sub> (mg/L)
Coquina	0.12	3.9	5.3
Fill	0.16	4.3	3.0
Marl	1.06	3.6	3.5
WPBMRF	1.88	7.2	8.1
BSMG	0.39	1.1	10.7

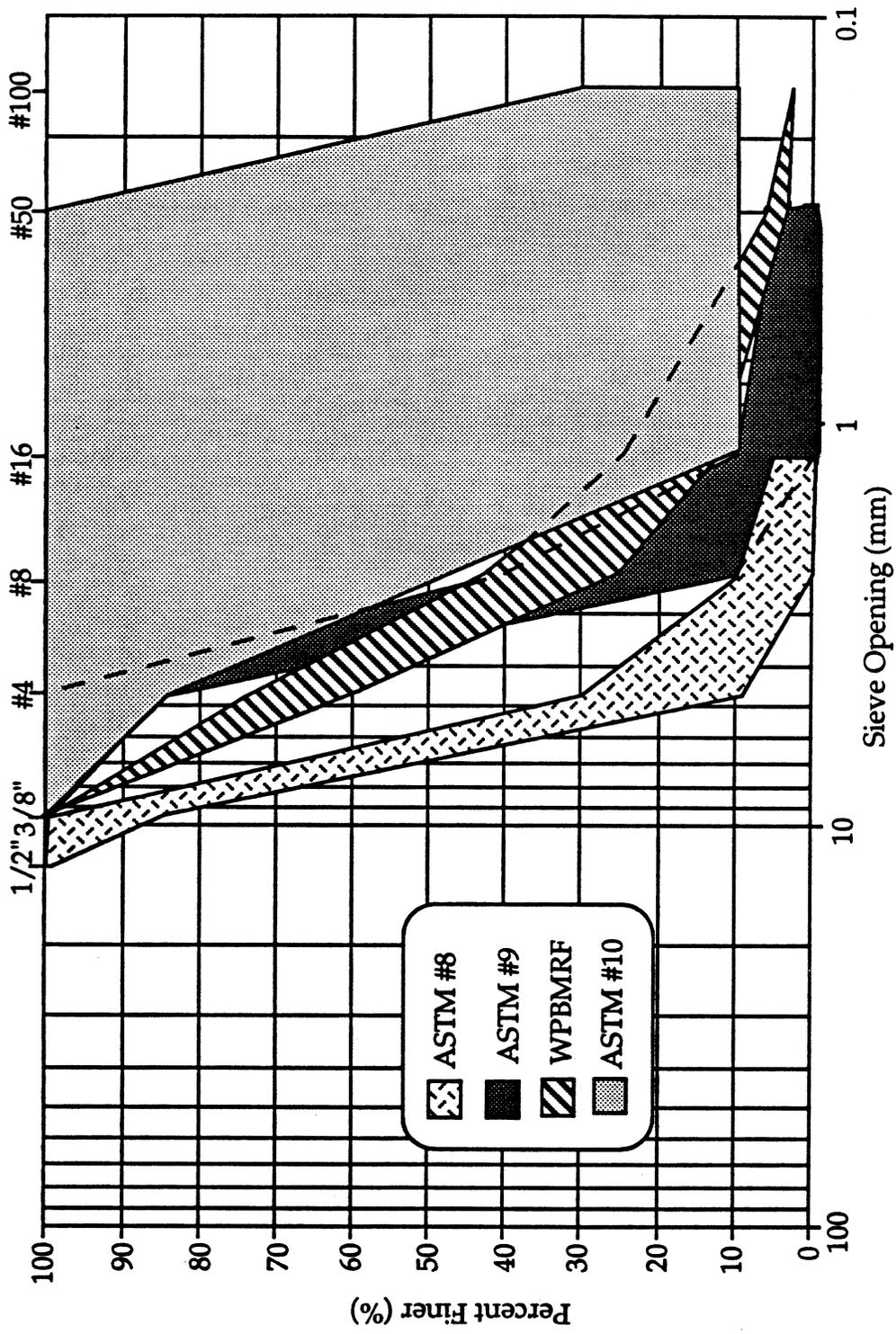
**Table 4.19 Data base of physical and engineering WG properties**

Parameter	Gradation													
	WPBMRF		ASTM # 8			ASTM # 9			ASTM # 10			Lower		
	Upper	Average	Lower	Upper	Average	Lower	Upper	Average	Lower	Upper	Average			
AASHTO Classification	A-1-a (0)	Table 4.2 (SW)	A-1-a (0)	Table 4.2 (SP)	A-1-a (0)	Table 4.2 (SP)	A-1-a (0)	Table 4.2 (SP)	A-1-a (0)	Table 4.2 (SP)	A-1-a (0)	Table 4.2 (SP)	A-1-a (0)	Table 4.2 (SW)
USCS Classification	Table 4.2 (SW)	111 pcf 1.74 kN/m <sup>3</sup>	Table 4.2 (GP)	99 pcf 1.55 kN/m <sup>3</sup>	Table 4.2 (SP)	102 psf 1.60 kN/m <sup>3</sup>	Table 4.2 (SP)	102 psf 1.60 kN/m <sup>3</sup>	Table 4.2 (SP)	105 psf 1.64 kN/m <sup>3</sup>	Table 4.2 (SP)	102 psf 1.60 kN/m <sup>3</sup>	Table 4.2 (SP)	81 pcf 1.27 kN/m <sup>3</sup>
Maximum Unit Weight	Table 4.2 (SW)	83 pcf 1.30 kN/m <sup>3</sup>	Table 4.2 (GP)	85 pcf 1.33 kN/m <sup>3</sup>	Table 4.2 (SP)	83 pcf 1.30 kN/m <sup>3</sup>	Table 4.2 (SP)	83 pcf 1.30 kN/m <sup>3</sup>	Table 4.2 (SP)	85 pcf 1.33 kN/m <sup>3</sup>	Table 4.2 (SP)	85 pcf 1.33 kN/m <sup>3</sup>	Table 4.2 (SP)	53 pcf 0.83 kN/m <sup>3</sup>
Minimum Unit Weight	Table 4.2 (SW)	0.5 mm	Table 4.2 (GP)	2.38 mm	Table 4.2 (SP)	4.75 mm	Table 4.2 (SP)	4.75 mm	Table 4.2 (SP)	1.50 mm	Table 4.2 (SP)	1.30 mm	Table 4.2 (SP)	1.19 mm
Effective Grain Size (D 10)	Table 4.2 (SW)	0.3 to 5 cm/s	Table 4.2 (GP)	4 to 8 cm/s	Table 4.2 (SP)	5 to 7 cm/s	Table 4.2 (SP)	5 to 7 cm/s	Table 4.2 (SP)	4 to 10 cm/s	Table 4.2 (SP)	4 to 8 cm/s	Table 4.2 (SP)	0.003 to 0.02 cm/s
Permeability	Table 4.2 (SW)	18 to 33 %	Table 4.2 (GP)	38 to 47 %	Table 4.2 (SP)	38 to 47 %	Table 4.2 (SP)	38 to 47 %	Table 4.2 (SP)	34 to 41 %	Table 4.2 (SP)	39 to 45 %	Table 4.2 (SP)	4 to 13 %
Specific Yield	Table 4.2 (SW)	Figure 4.15	Table 4.2 (GP)	Figure 4.15	Table 4.2 (SP)	Figure 4.15	Table 4.2 (SP)	Figure 4.15	Table 4.2 (SP)	Figure 4.15	Table 4.2 (SP)	Figure 4.15	Table 4.2 (SP)	Figure 4.15
Triaxial Friction Angles	Table 4.2 (SW)	45 to 45	Table 4.2 (GP)	45 to 51	Table 4.2 (SP)	45 to 51	Table 4.2 (SP)	45 to 51	Table 4.2 (SP)	37 to 45	Table 4.2 (SP)	37 to 45	Table 4.2 (SP)	34 to 46
Direct Shear Friction Angles	Table 4.2 (SW)	0.4 to 3.3 %	Table 4.2 (GP)	0.9 to 2.7 %	Table 4.2 (SP)	0.9 to 2.7 %	Table 4.2 (SP)	0.9 to 2.7 %	Table 4.2 (SP)	0.8 to 2.8 %	Table 4.2 (SP)	0.8 to 2.8 %	Table 4.2 (SP)	0.5 to 1.7 %
CBR	Table 4.2 (SW)	0.5 to 4.0 %	Table 4.2 (GP)	1.1 to 3.4 %	Table 4.2 (SP)	1.1 to 3.4 %	Table 4.2 (SP)	1.1 to 3.4 %	Table 4.2 (SP)	1.0 to 3.5 %	Table 4.2 (SP)	1.0 to 3.5 %	Table 4.2 (SP)	0.6 to 2.1 %
LBR	Table 4.2 (SW)		Table 4.2 (GP)		Table 4.2 (SP)		Table 4.2 (SP)		Table 4.2 (SP)		Table 4.2 (SP)		Table 4.2 (SP)	

**Remarks**

- Gradation: All usable gradations are similar to poorly graded sands or gravels (avoid use of ASTM # 10).
- Density: All gradations have densities about 20% lighter than conventional aggregate.
- Drainage: Use only WPBMRF (i.e. # 89) or # 8 and # 9 for drains.
- Shear Strength: Excellent shear strength for embankment and retaining wall uses.
- Pavement Layers: Very poor bearing ratios if not mixed with conventional base materials.





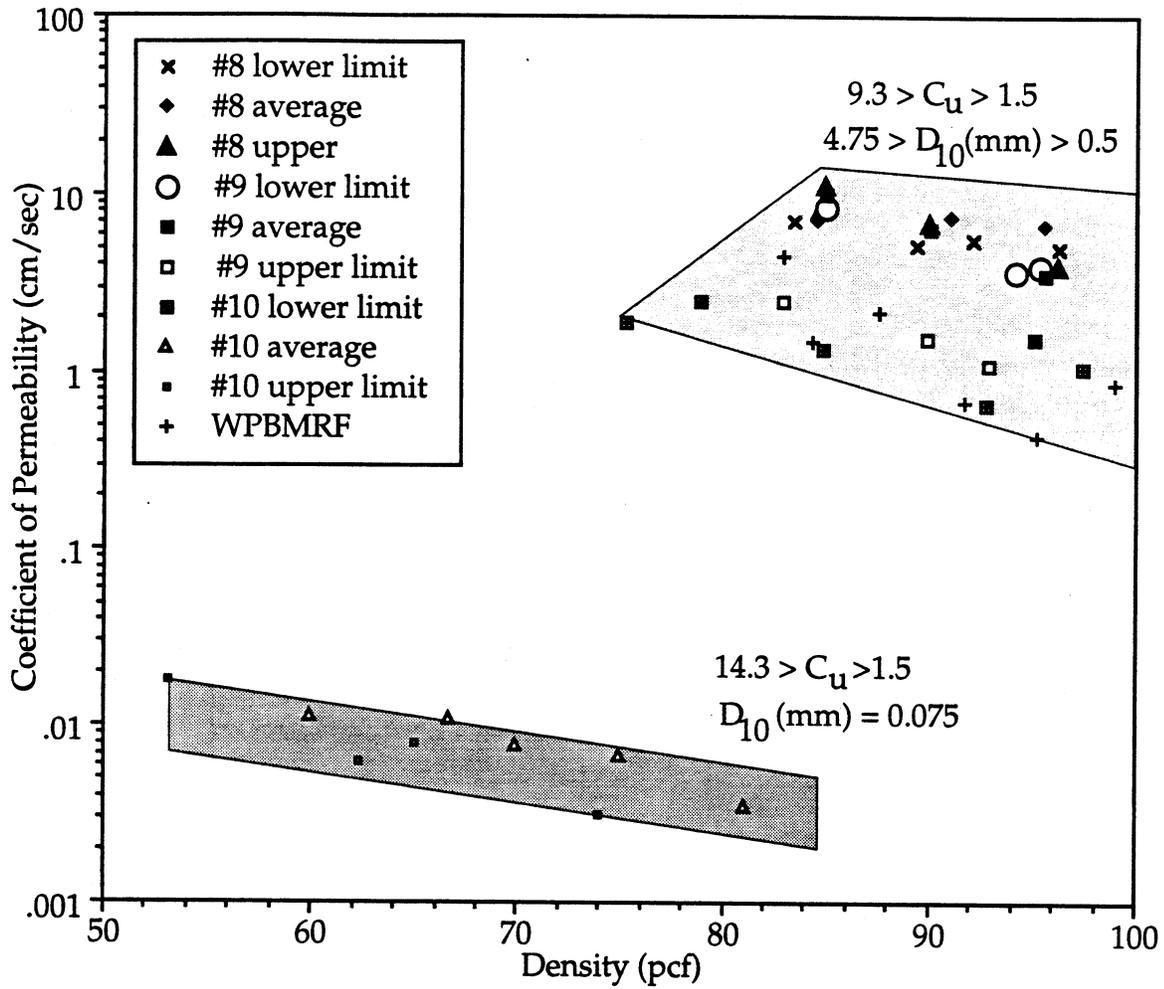


Figure 4.3 Relationship between density and permeability for WG meeting ASTM D 448 #8, #9, #10, and WPBMRF gradations

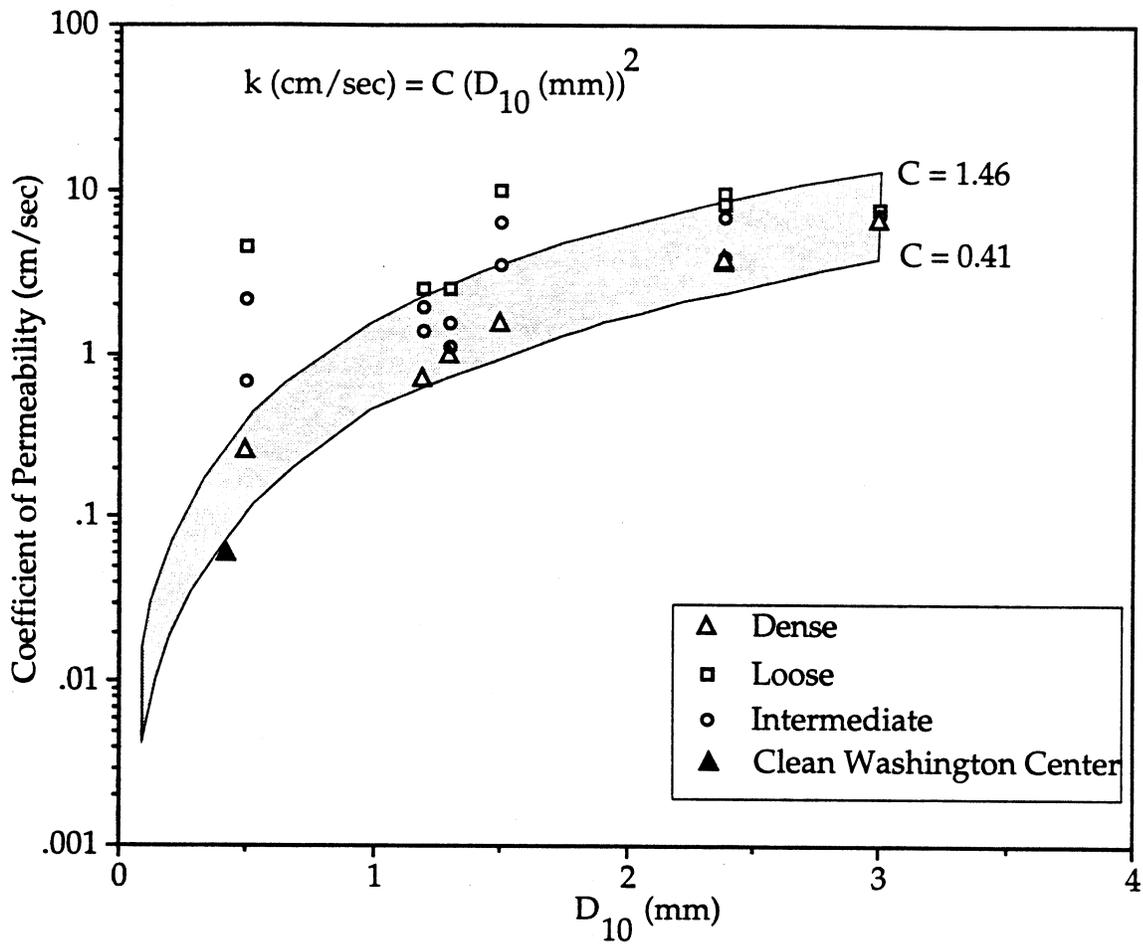


Figure 4.4 WG permeability and  $D_{10}$  (mm) relationship compared to Hazens' empirical formula with  $C_u < 5$  and  $0.1 < D_{10}$  (mm) < 3

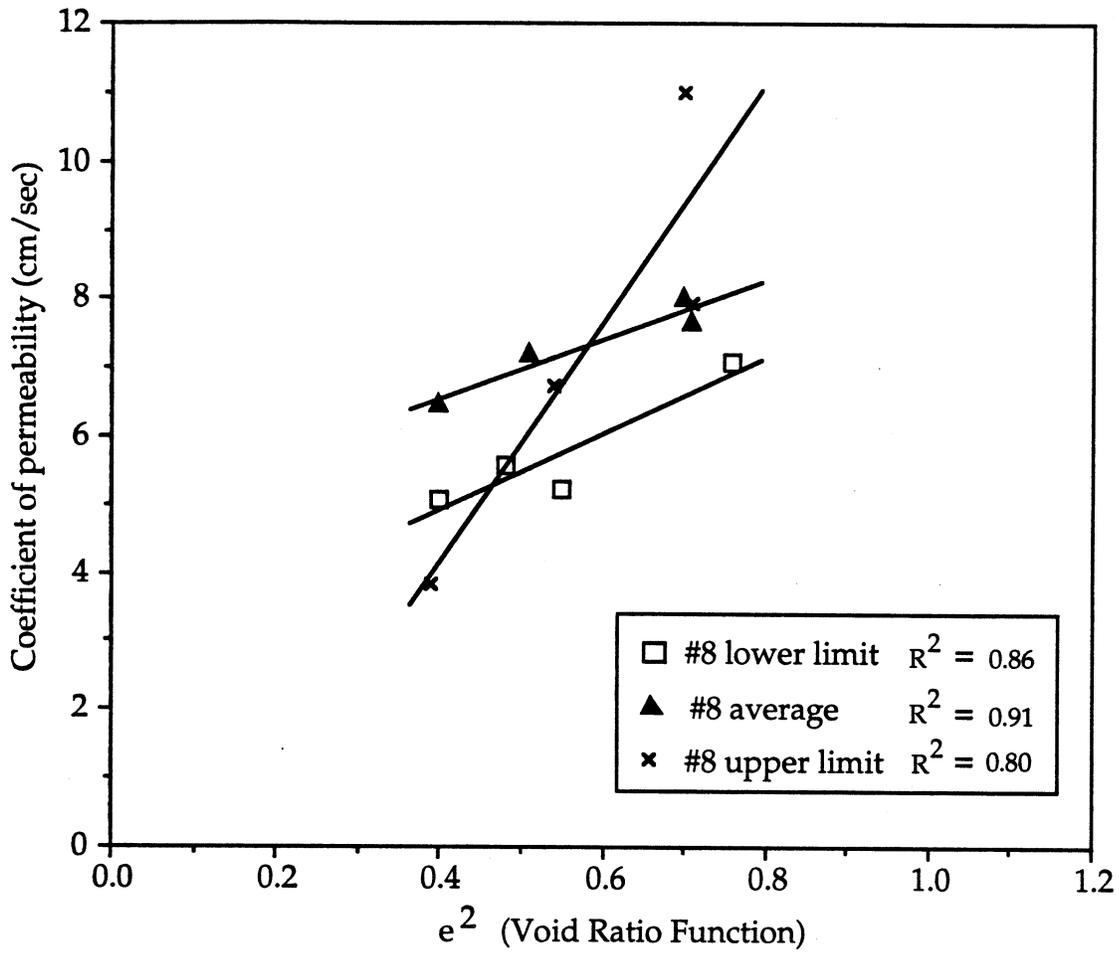


Figure 4.5 Relationship between  $e^2$  and coefficient of permeability for WG meeting ASTM #8 gradation

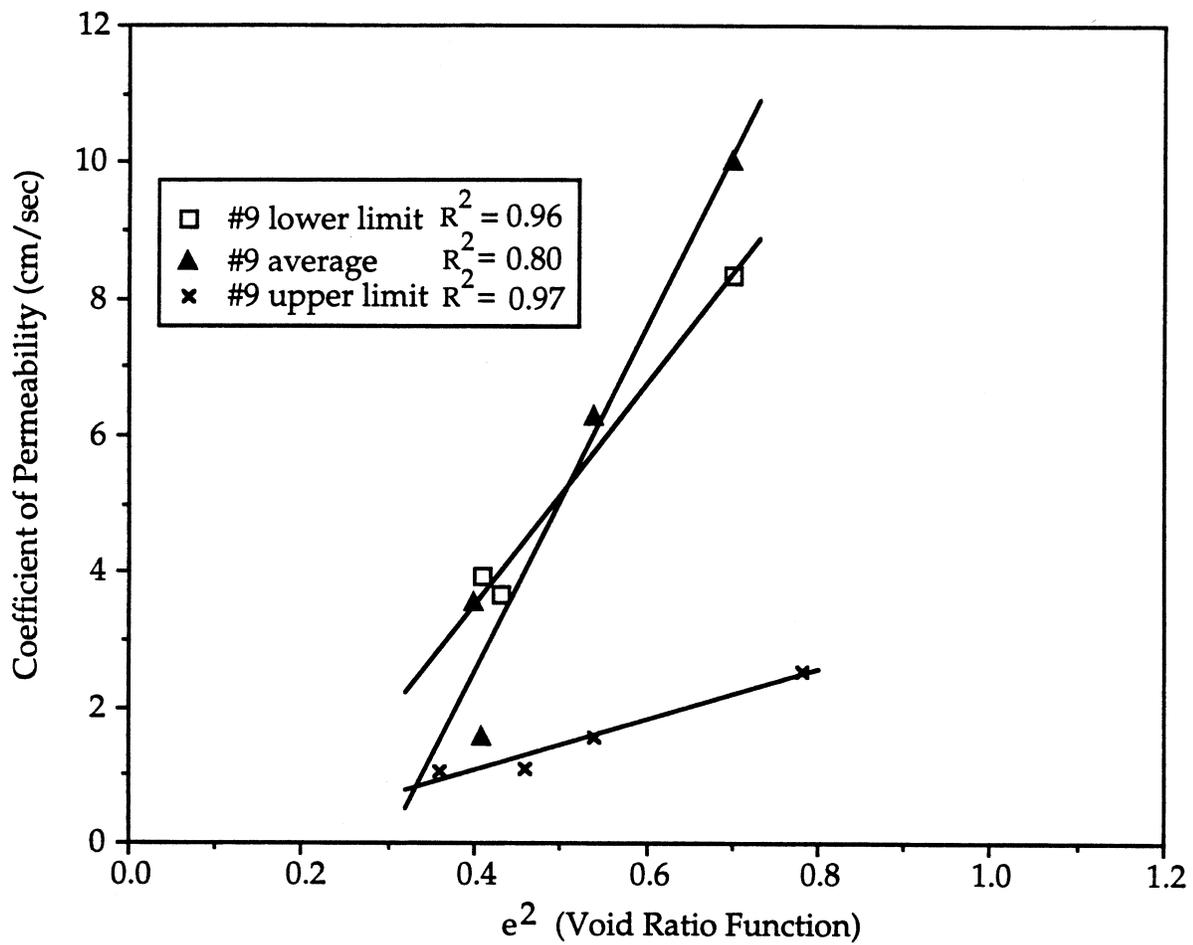


Figure 4.6 Relationship between  $e^2$  and coefficient of permeability for WG meeting ASTM D 448 #9 gradation

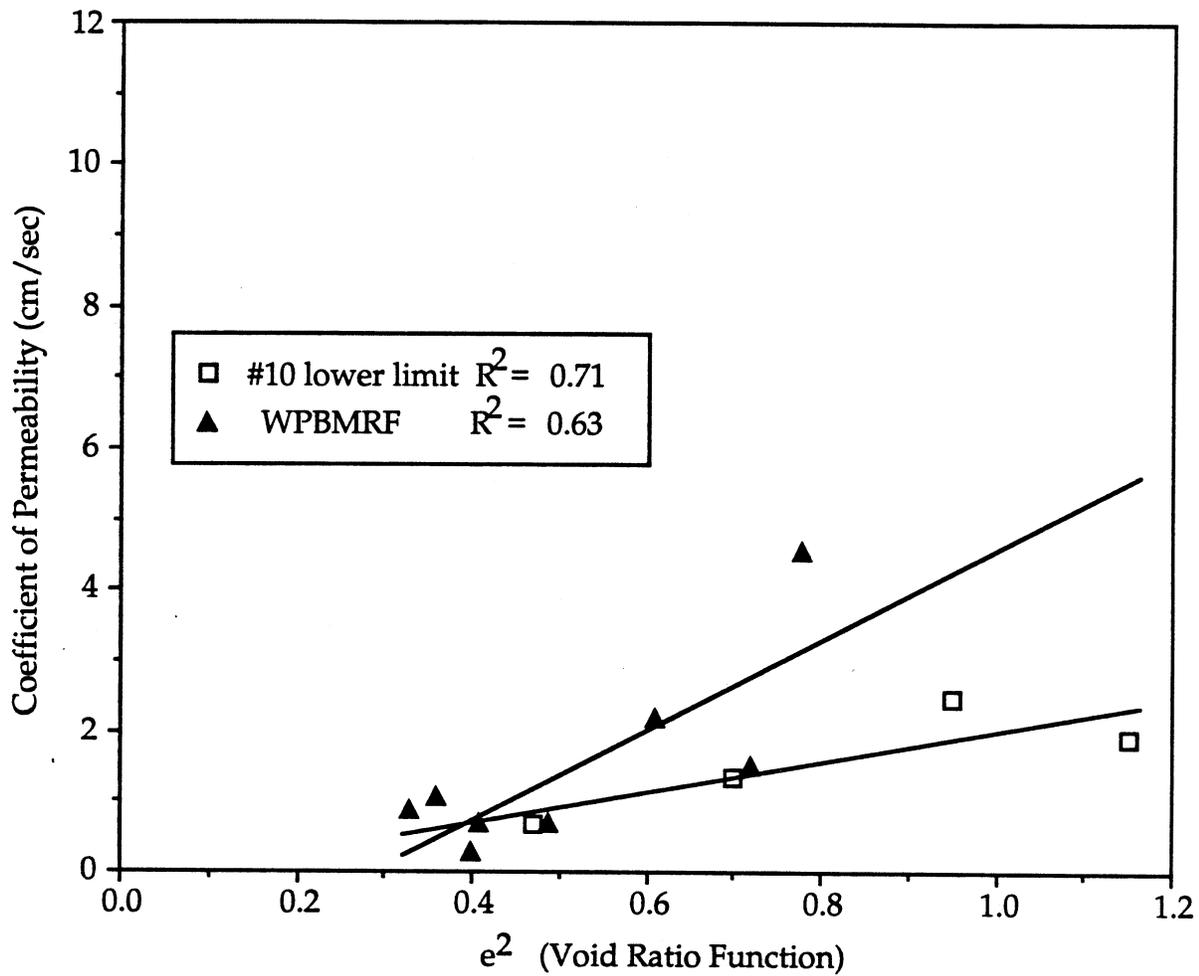


Figure 4.7 Relationship between  $e^2$  and coefficient of permeability for WG meeting ASTM D 448 #10 lower and WPBMRF gradations

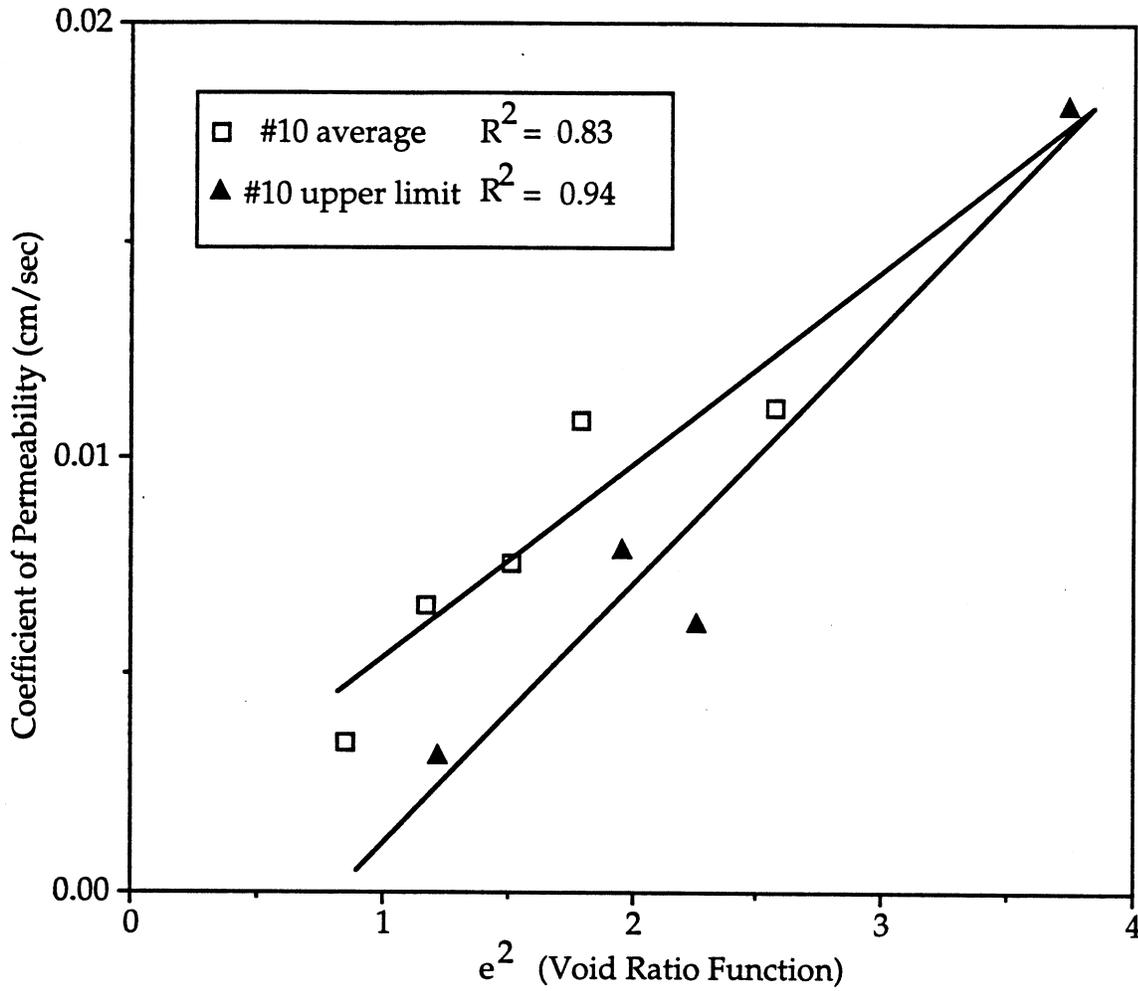


Figure 4.8 Relationship between  $e^2$  and coefficient of permeability for WG meeting ASTM D 448 #10 average and upper limit gradations

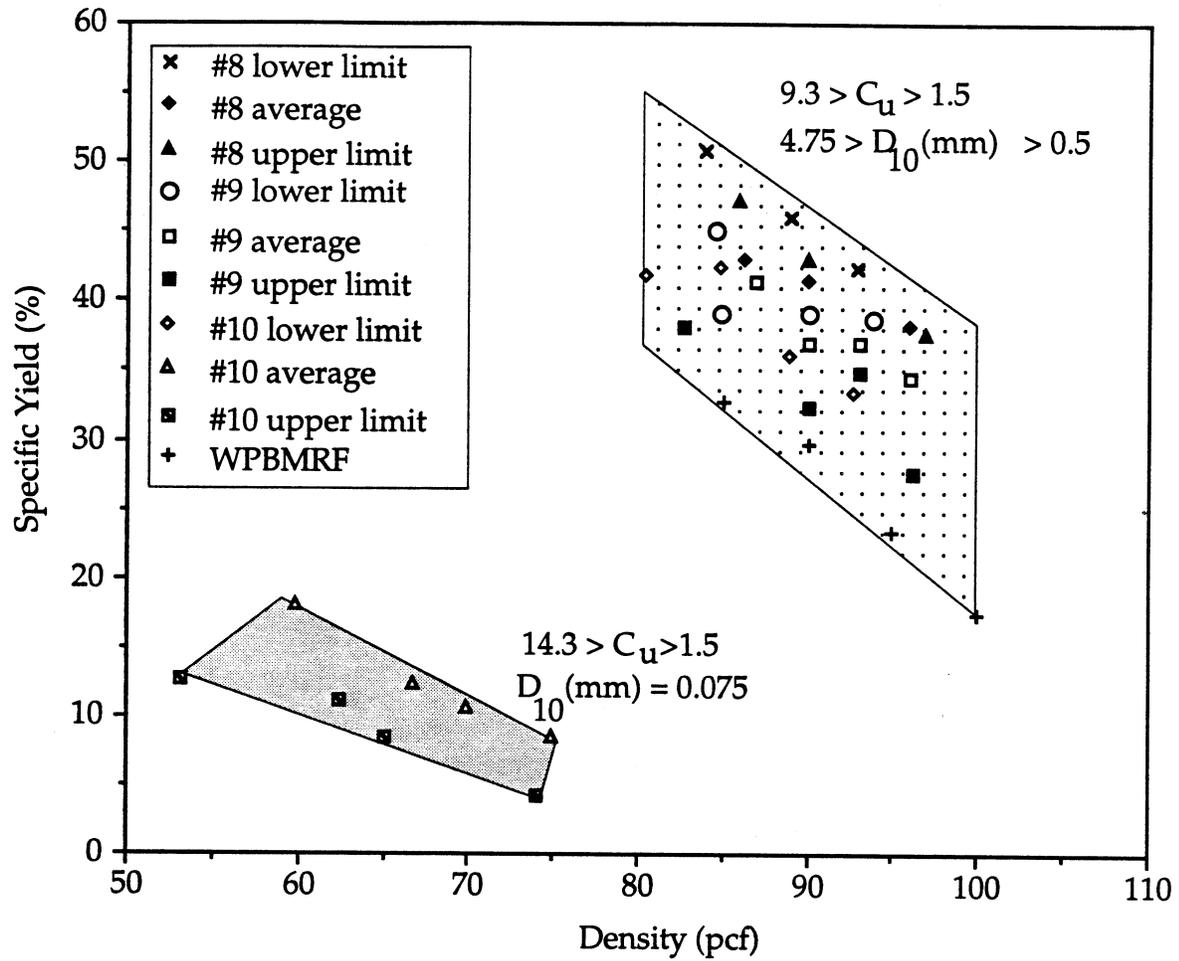


Figure 4.9 Relationship between density and specific yield for WG meeting ASTM D 448 #8, #9, #10, and WPBMRF gradations

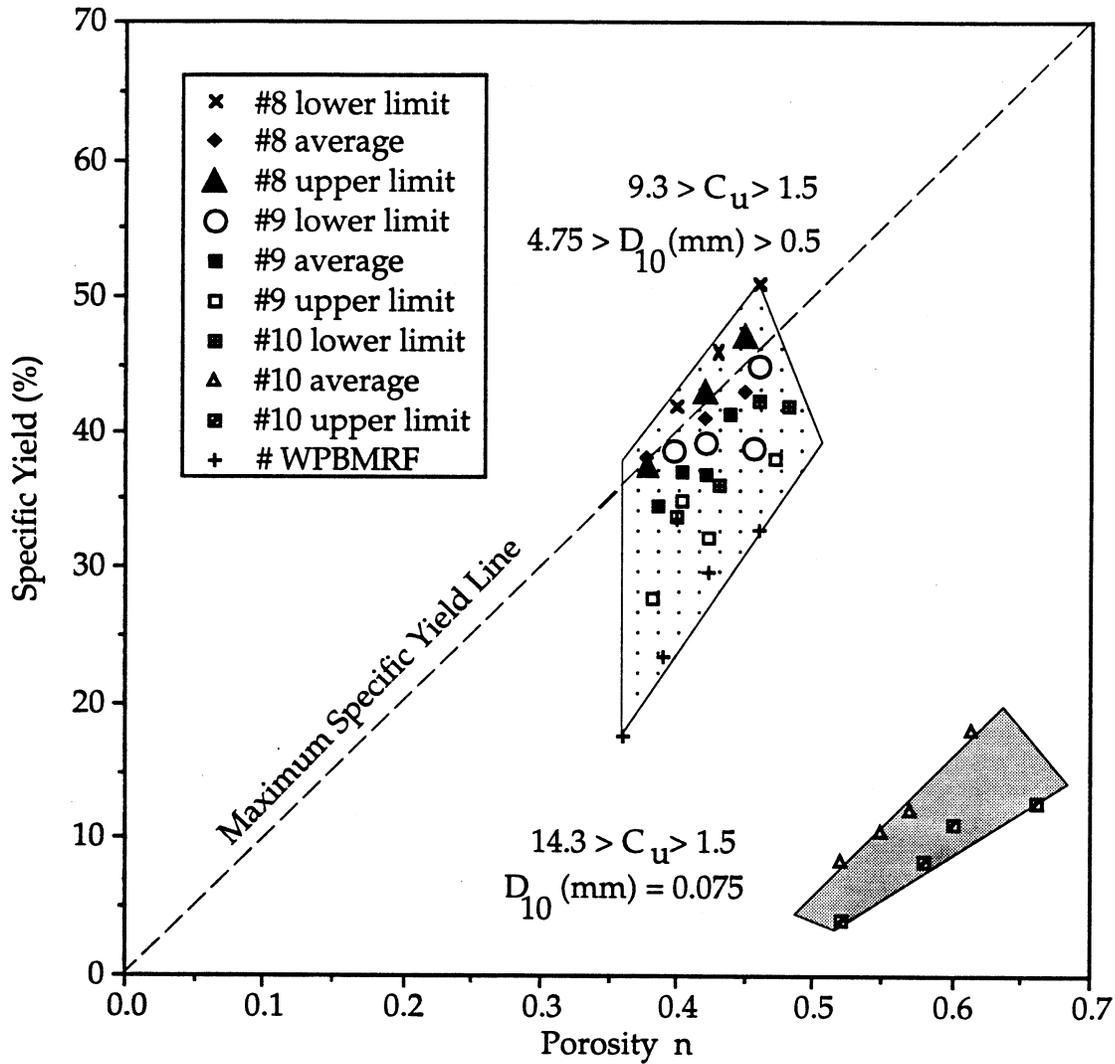


Figure 4.10 Relationship between porosity and specific yield for WG meeting ASTM D 448 #8, #9, #10, and WPBMRF gradations

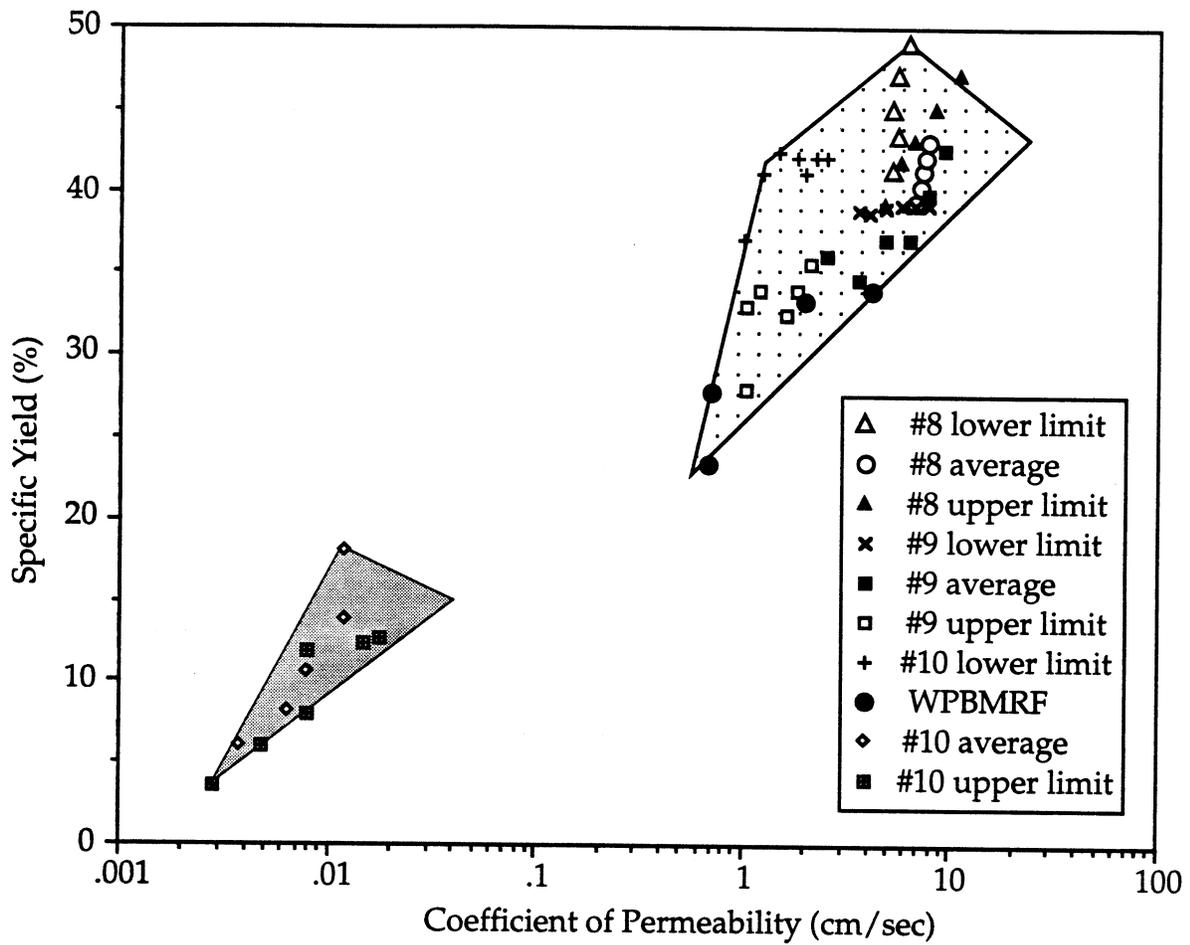


Figure 4.11 Relationship between permeability and specific yield for WG meeting ASTM D 448 #8, #9, #10, and WPBMRF gradations

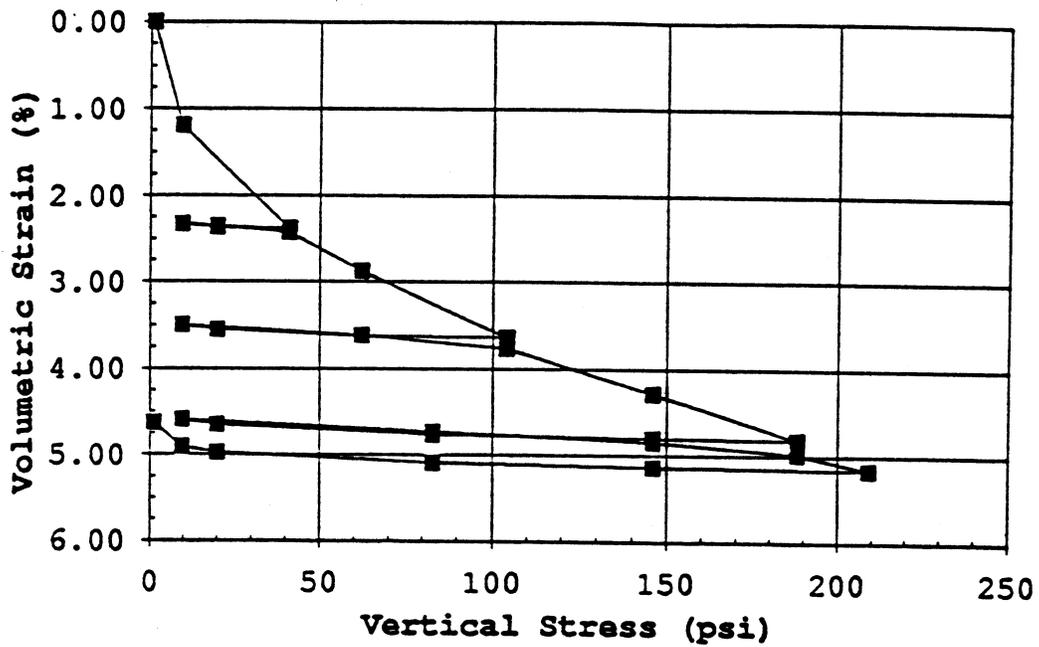


Figure 4.12 Typical confined compression results for mixed cullet meeting WPBMRF gradation (1 psi = 7 kPa)

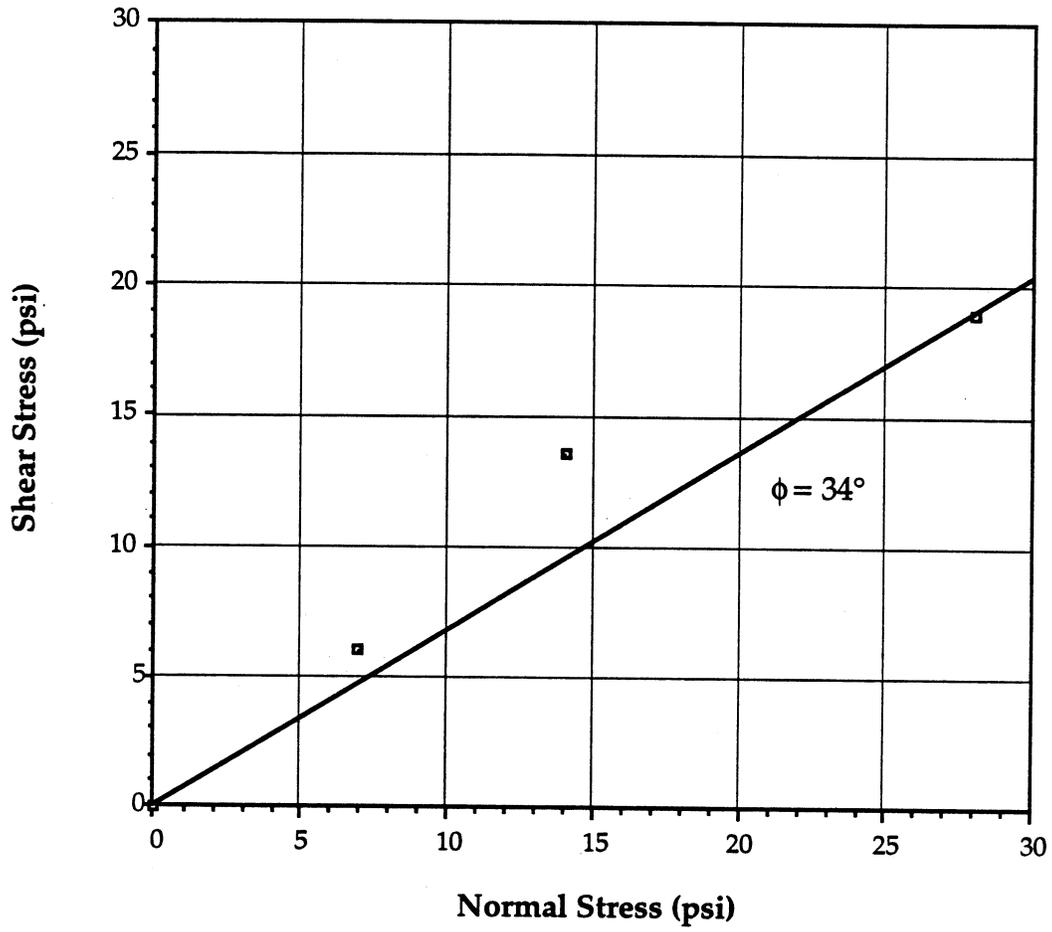


Figure 4.13 Nonlinear Mohr-Coulomb failure typical of WG direct shear tests

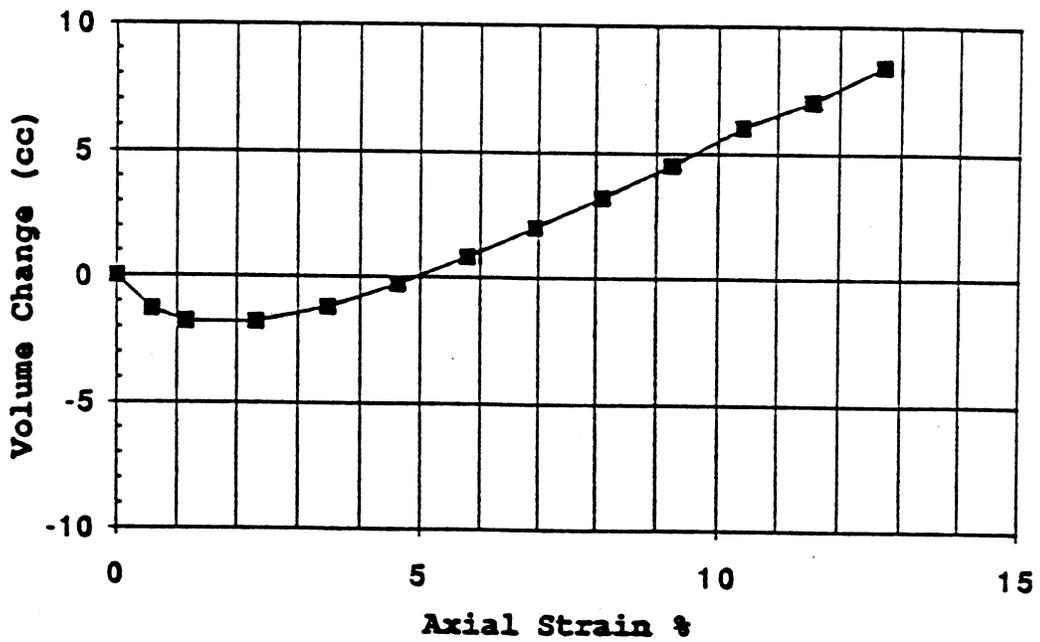
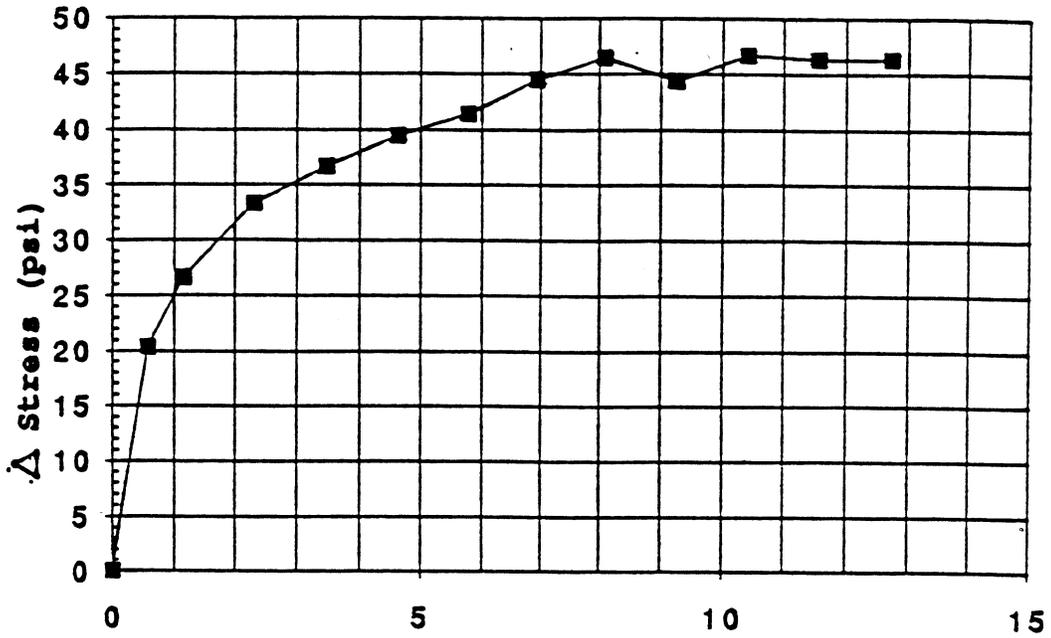


Figure 4.14 Typical vertical stress versus vertical strain and volumetric strain versus vertical strain plots for WG triaxial shear testing.

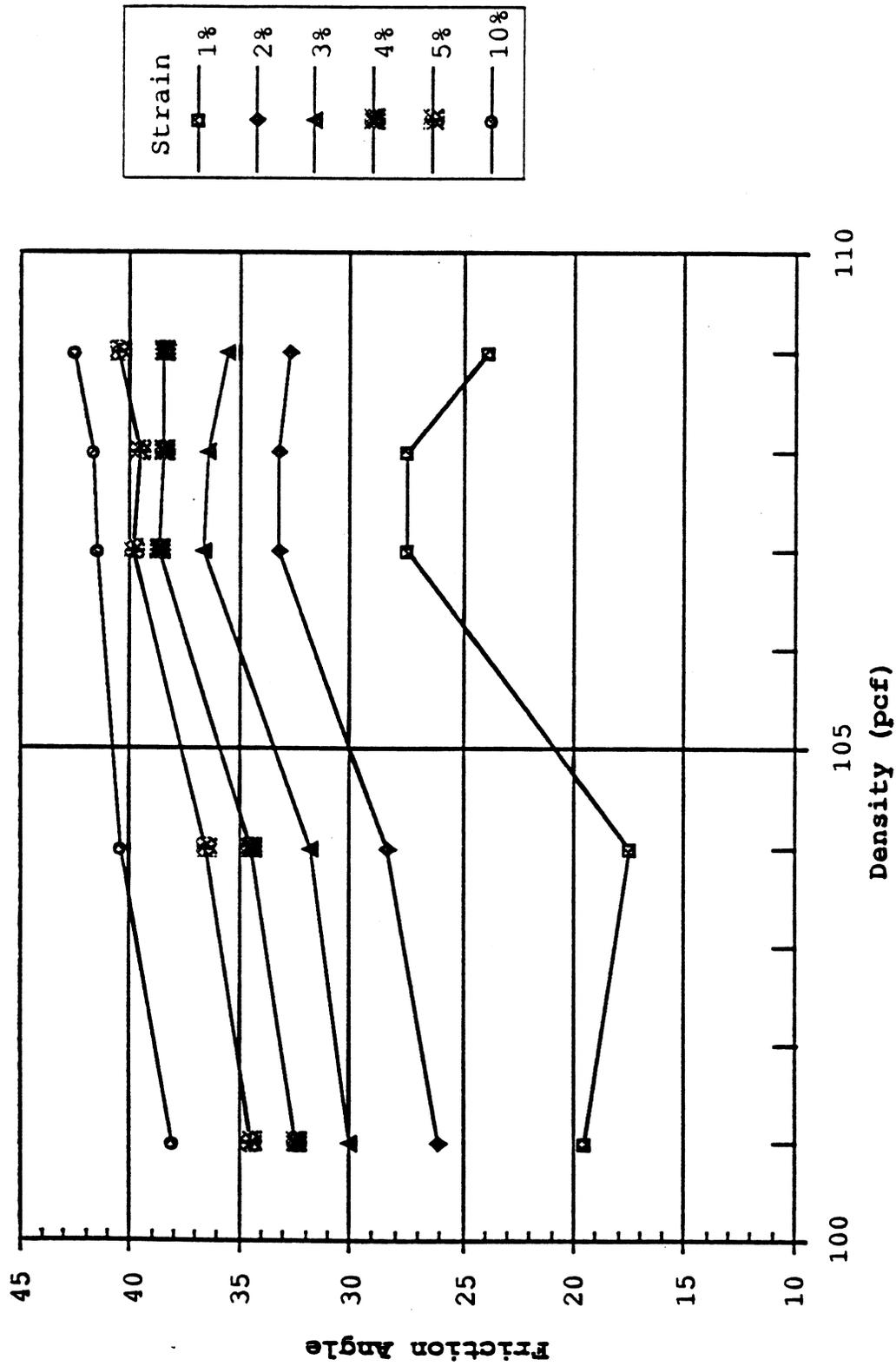


Figure 4.15a Variation of friction angle with density and strain level during CD triaxial testing on mixed cullet meeting WPBMRF gradation (1 psi = 7 kPa)

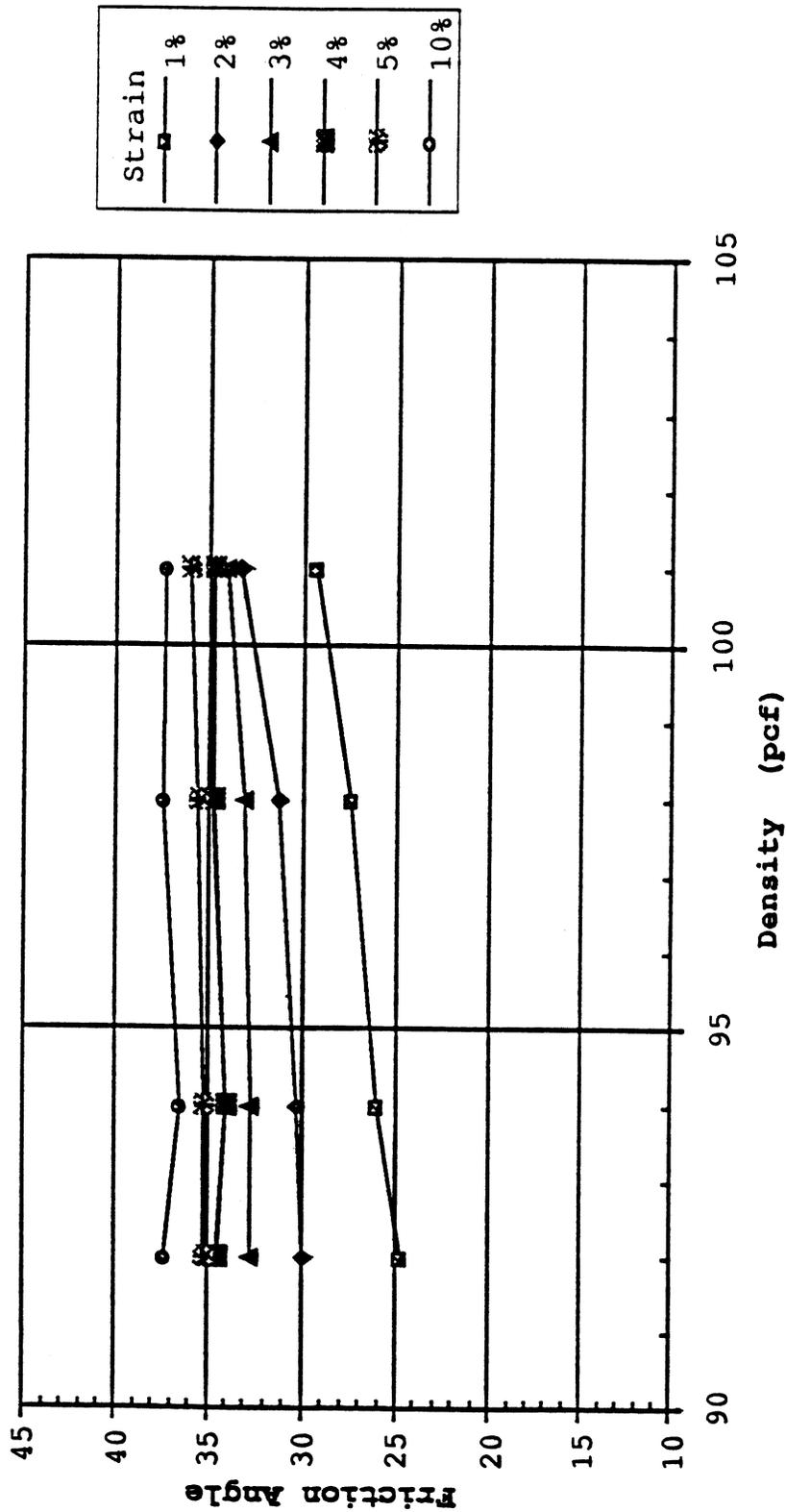


Figure 4.15b Variation of friction angle with density and strain level during CD triaxial testing on mixed cullet meeting ASTM # 8 average gradation (1 psi = 7 kPa)

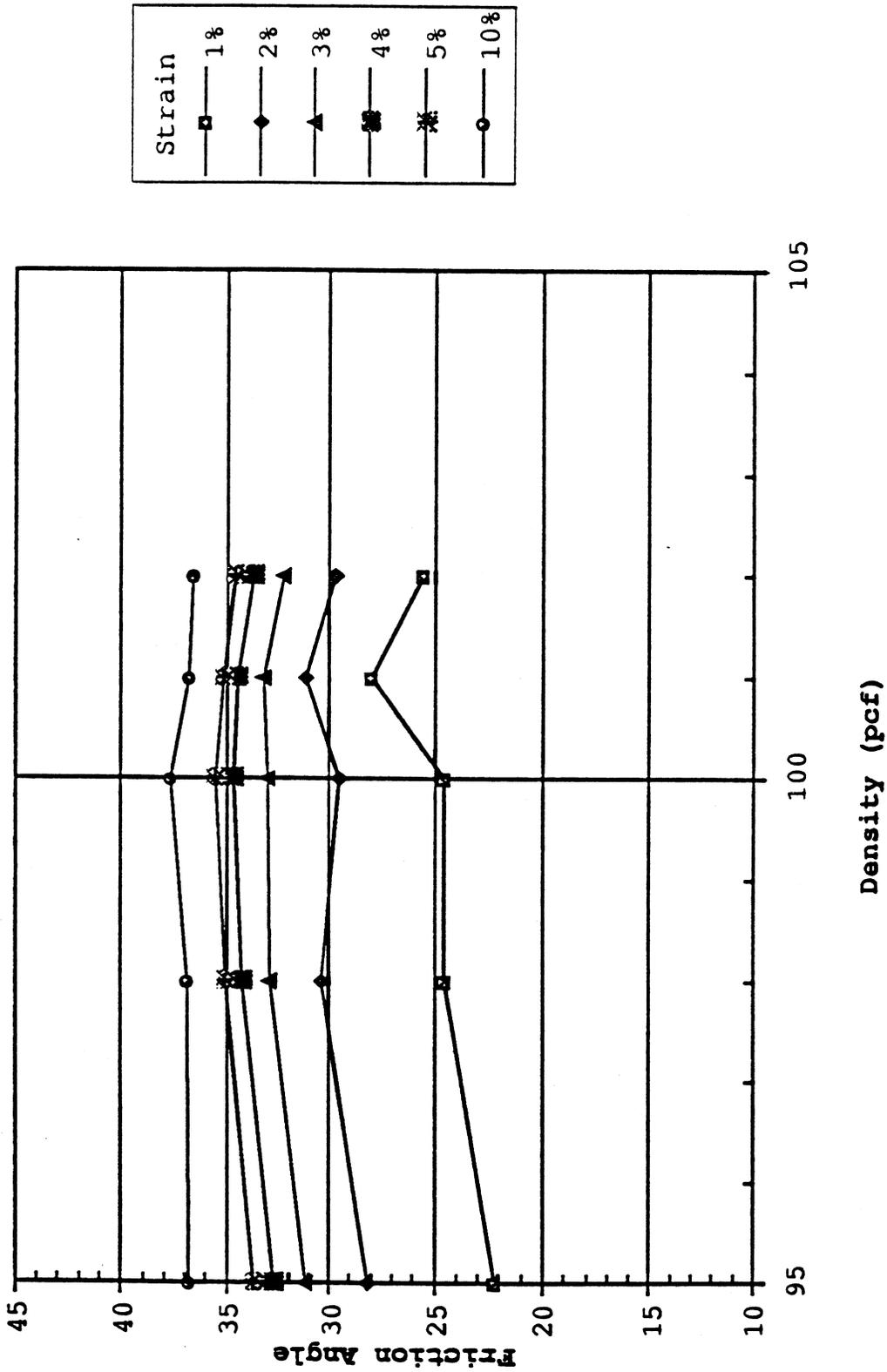


Figure 4.15c Variation of friction angle with density and strain level during CD triaxial testing on mixed cullet meeting ASTM # 9 average gradation (1 psi = 7 kPa)

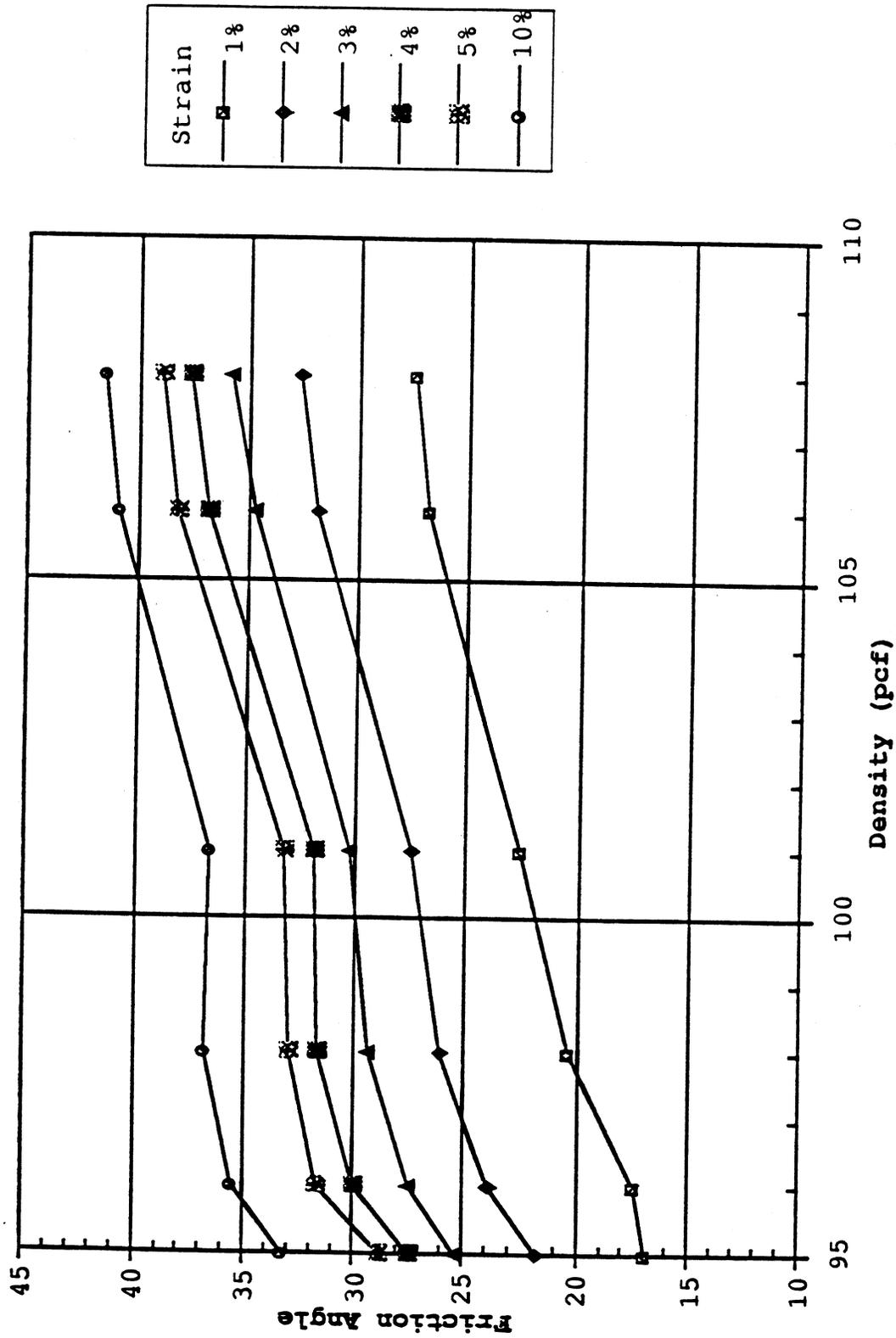


Figure 4.15d Variation of friction angle with density and strain level during CD triaxial testing on mixed cullet meeting ASTM # 10 average gradation (1 psi = 7 kPa)

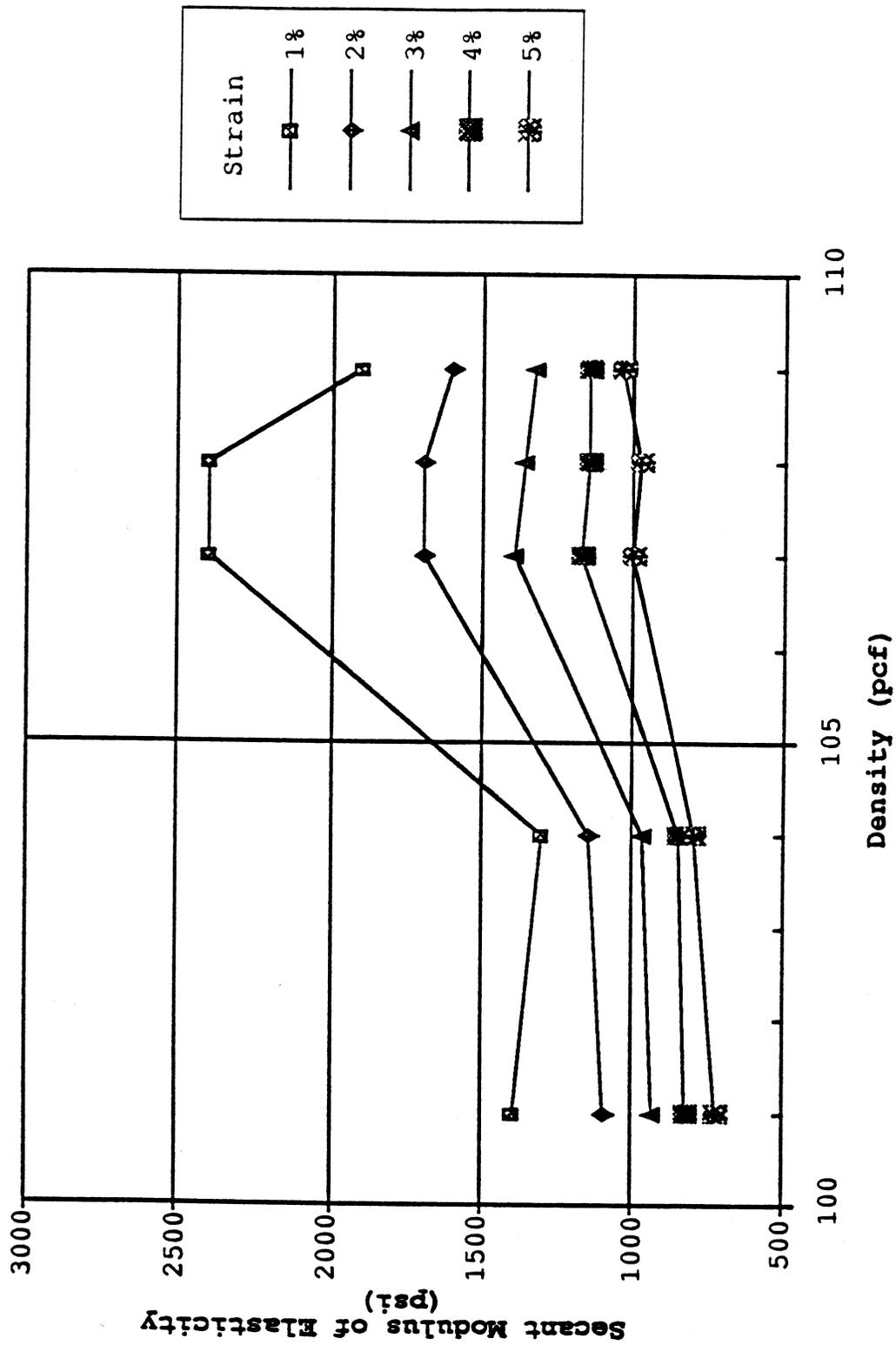


Figure 4.16a Variation of secant modulus with density and strain level during CD triaxial testing on mixed cullet meeting WPBMRF gradation (1 psi = 7 kPa)

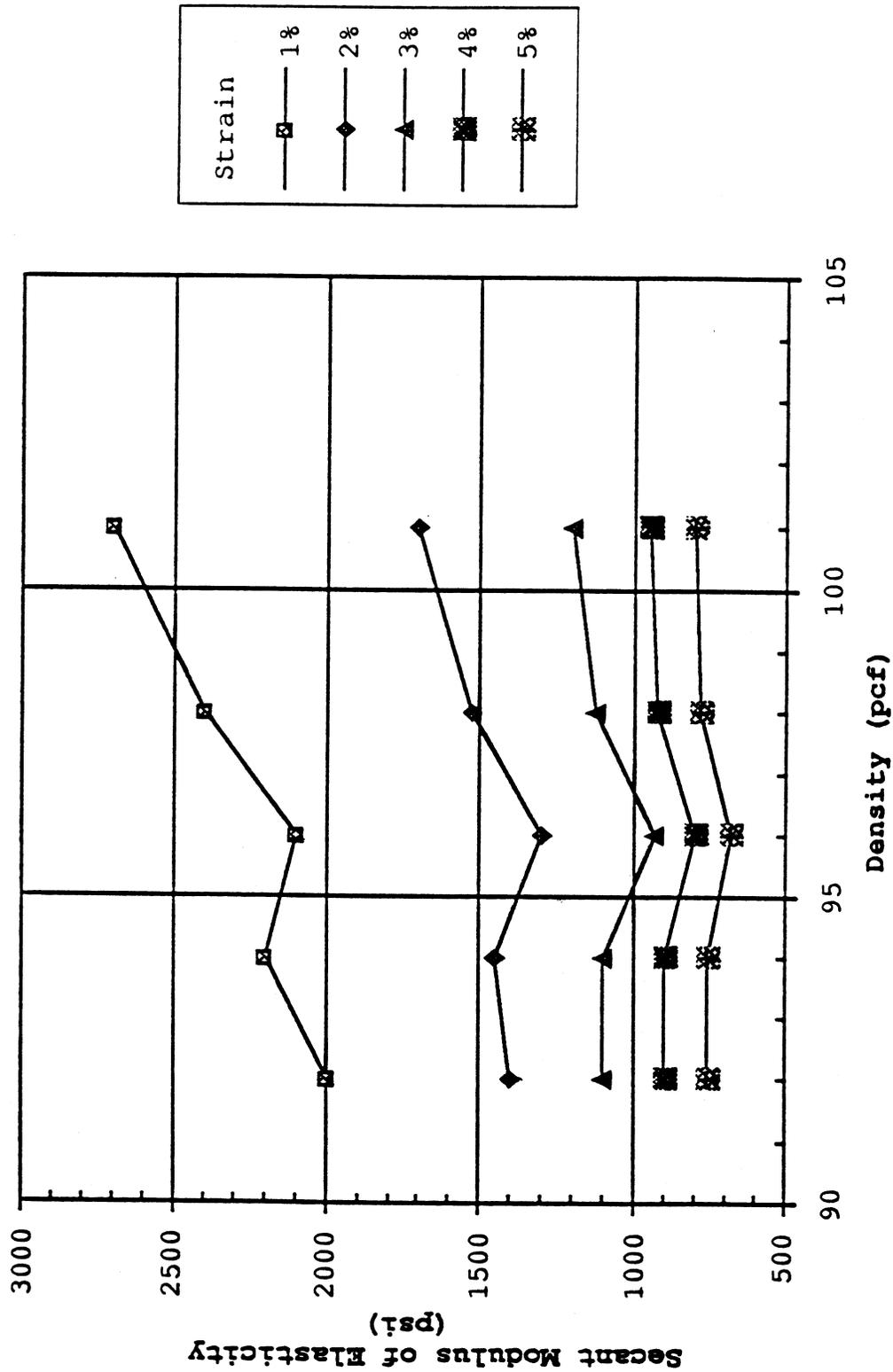


Figure 4.16b Variation of secant modulus with density and strain level during CD triaxial testing on mixed cullet meeting ASTM # 8 average gradation (1 psi = 7 kPa)

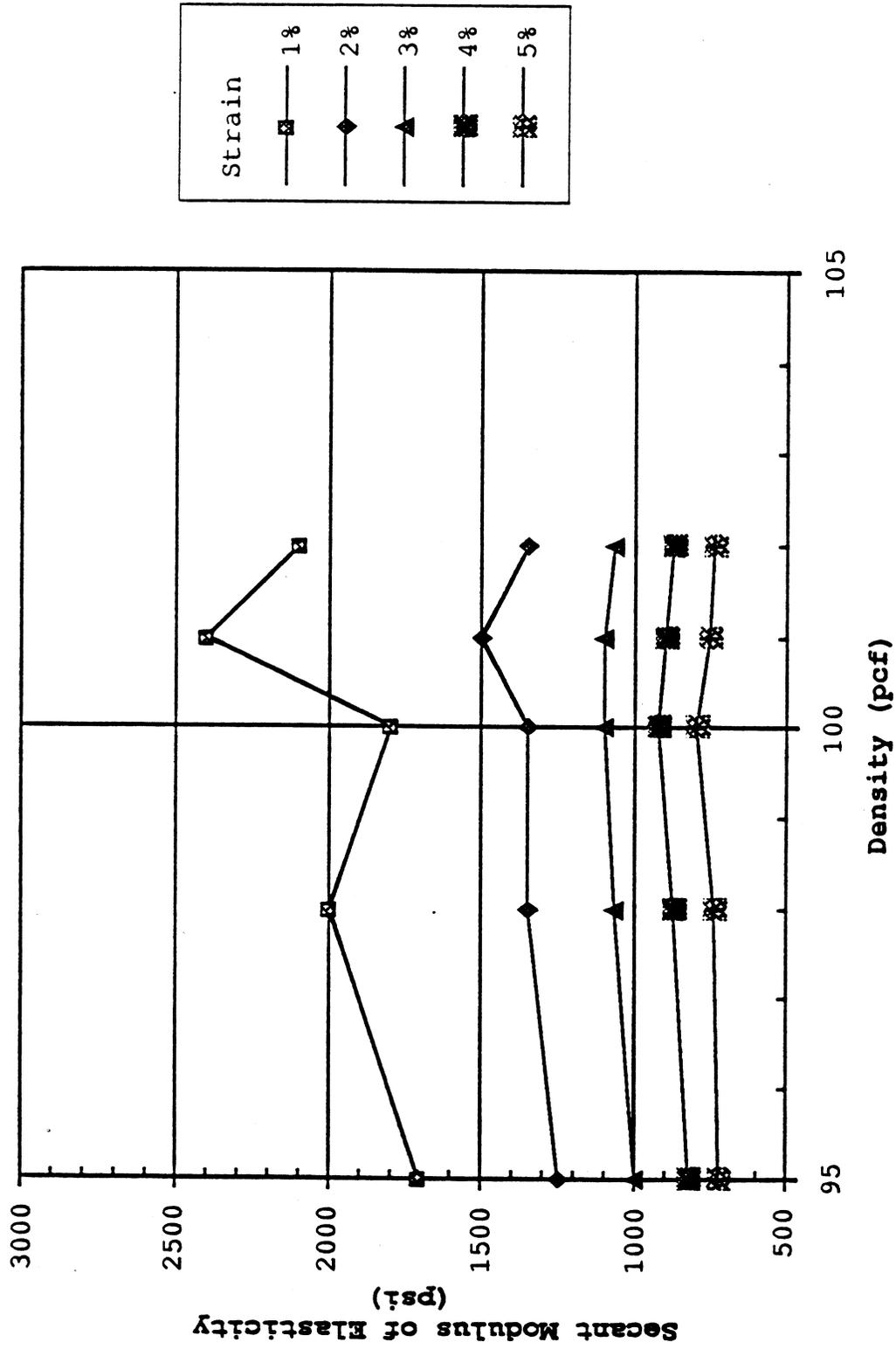


Figure 4.16c Variation of secant modulus with density and strain level during CD triaxial testing on mixed cullet meeting ASTM # 9 average gradation (1 psi = 7 kPa)

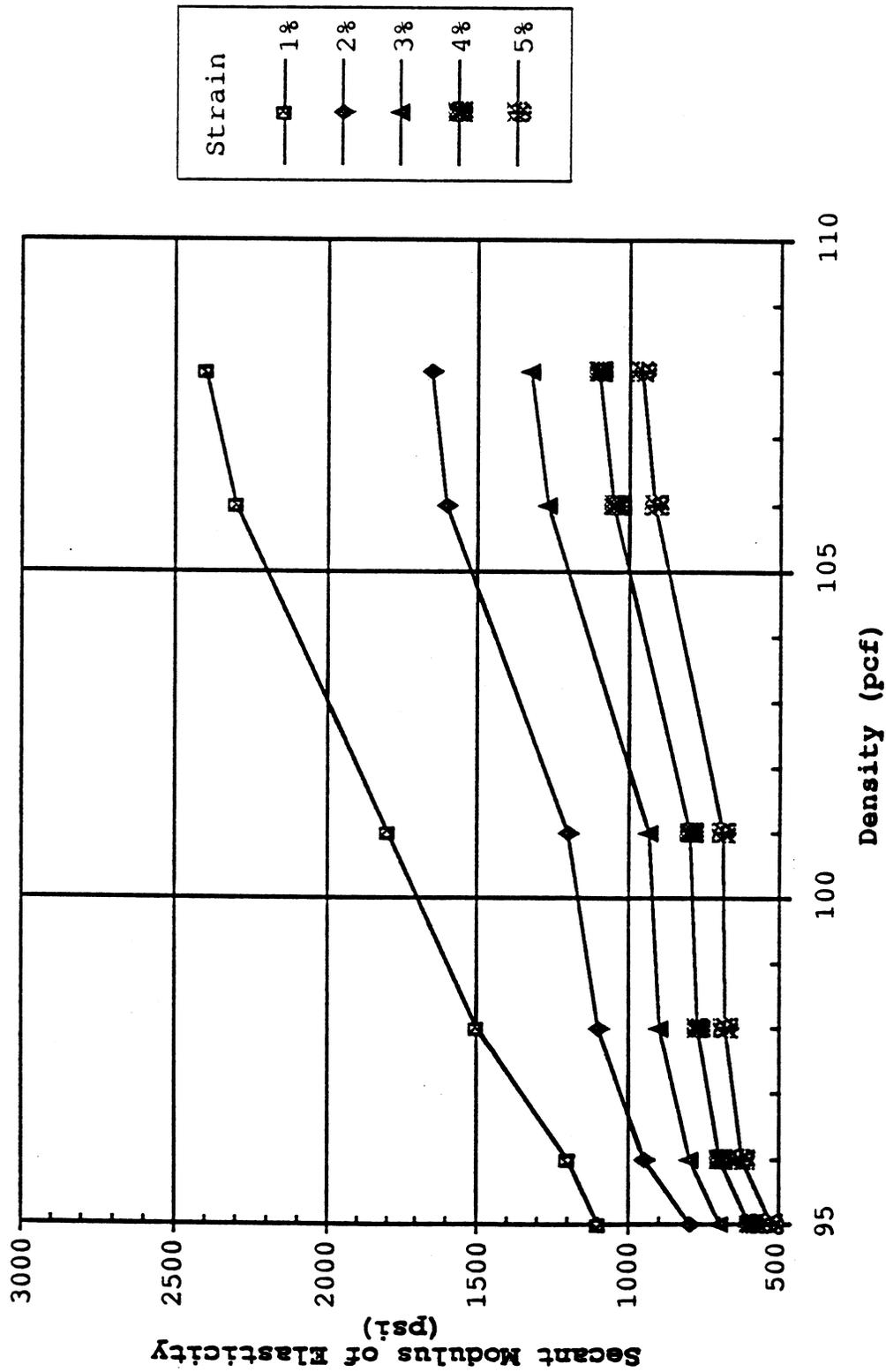


Figure 4.16d Variation of secant modulus with density and strain level during CD triaxial testing on mixed cullet meeting ASTM # 10 average gradation (1 psi = 7 kPa)

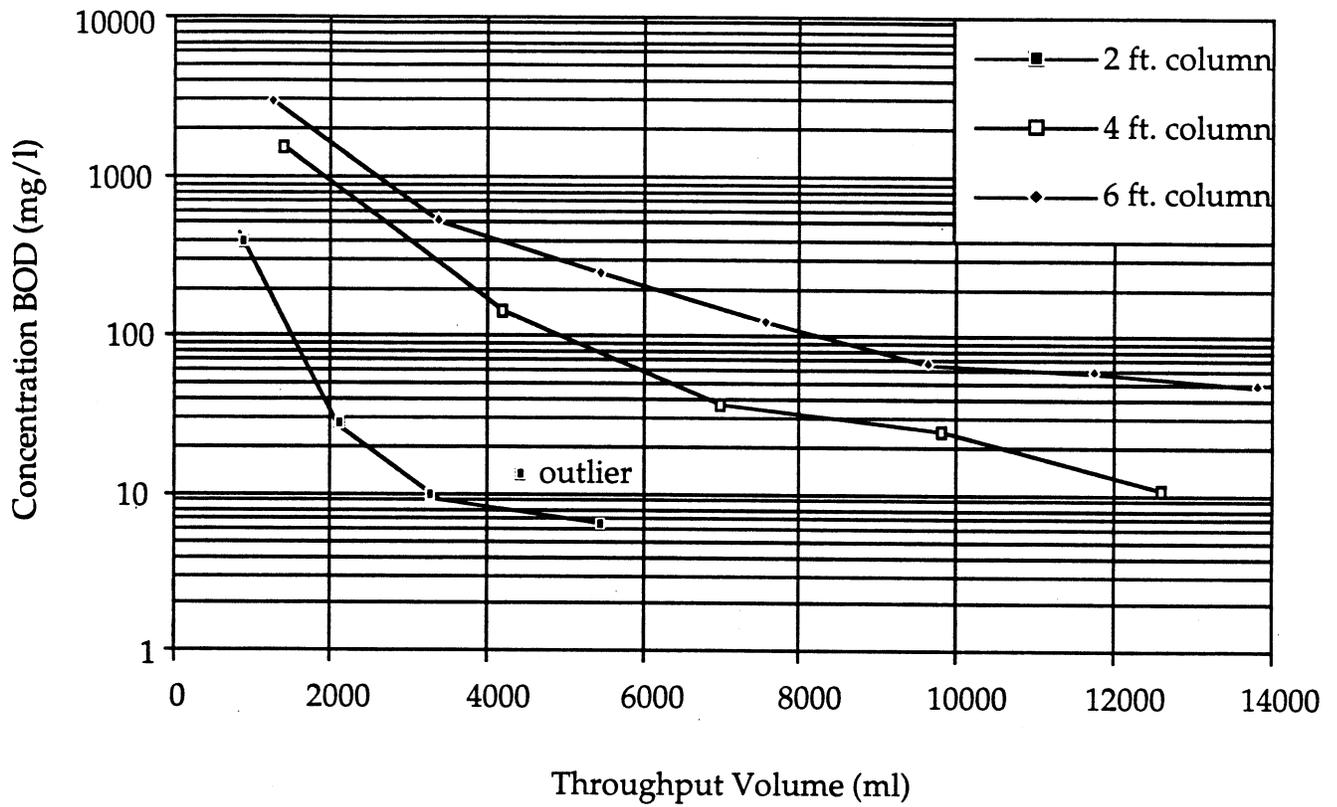


Figure 4.17 BSMG leaching column BOD results

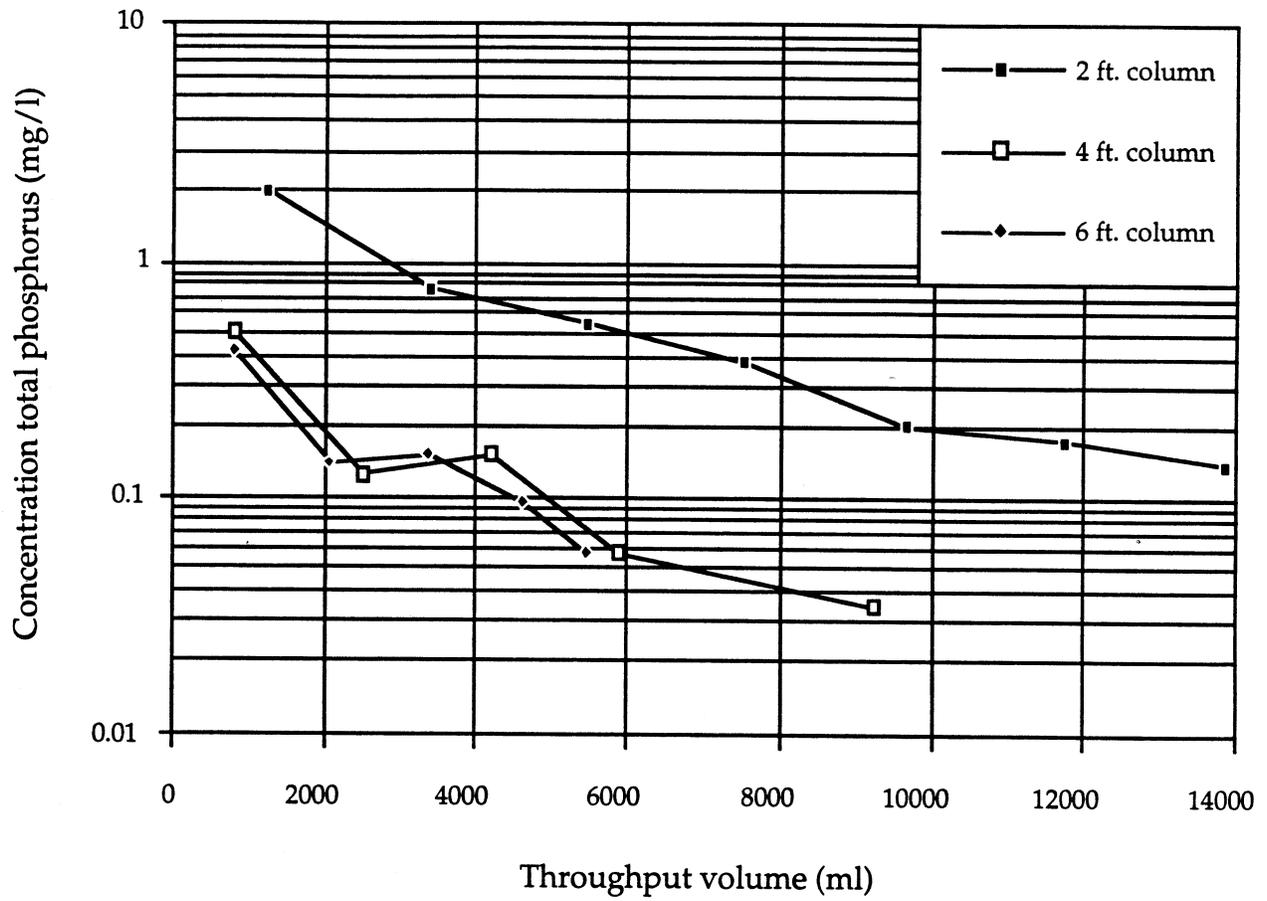


Figure 4.18 BSMG leaching column TP results

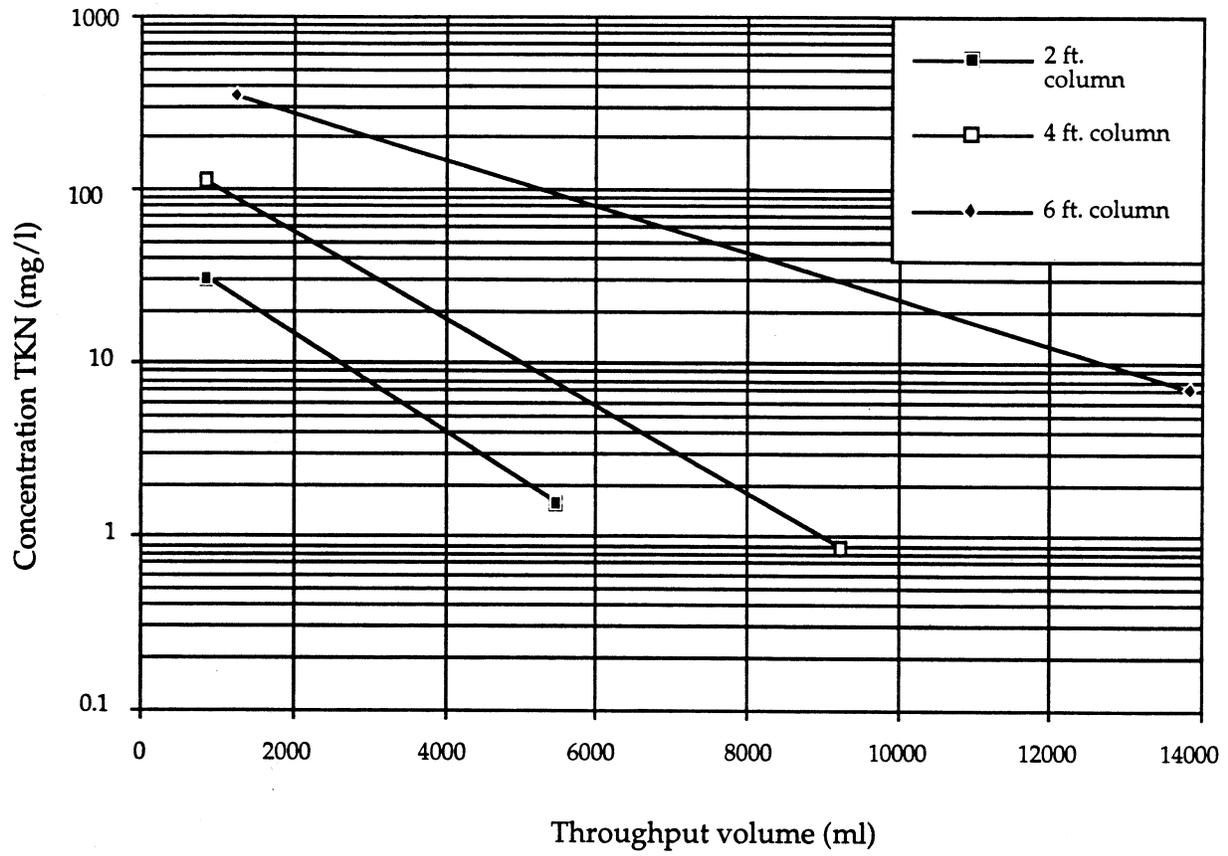


Figure 4.19 BSMG leaching column TKN results

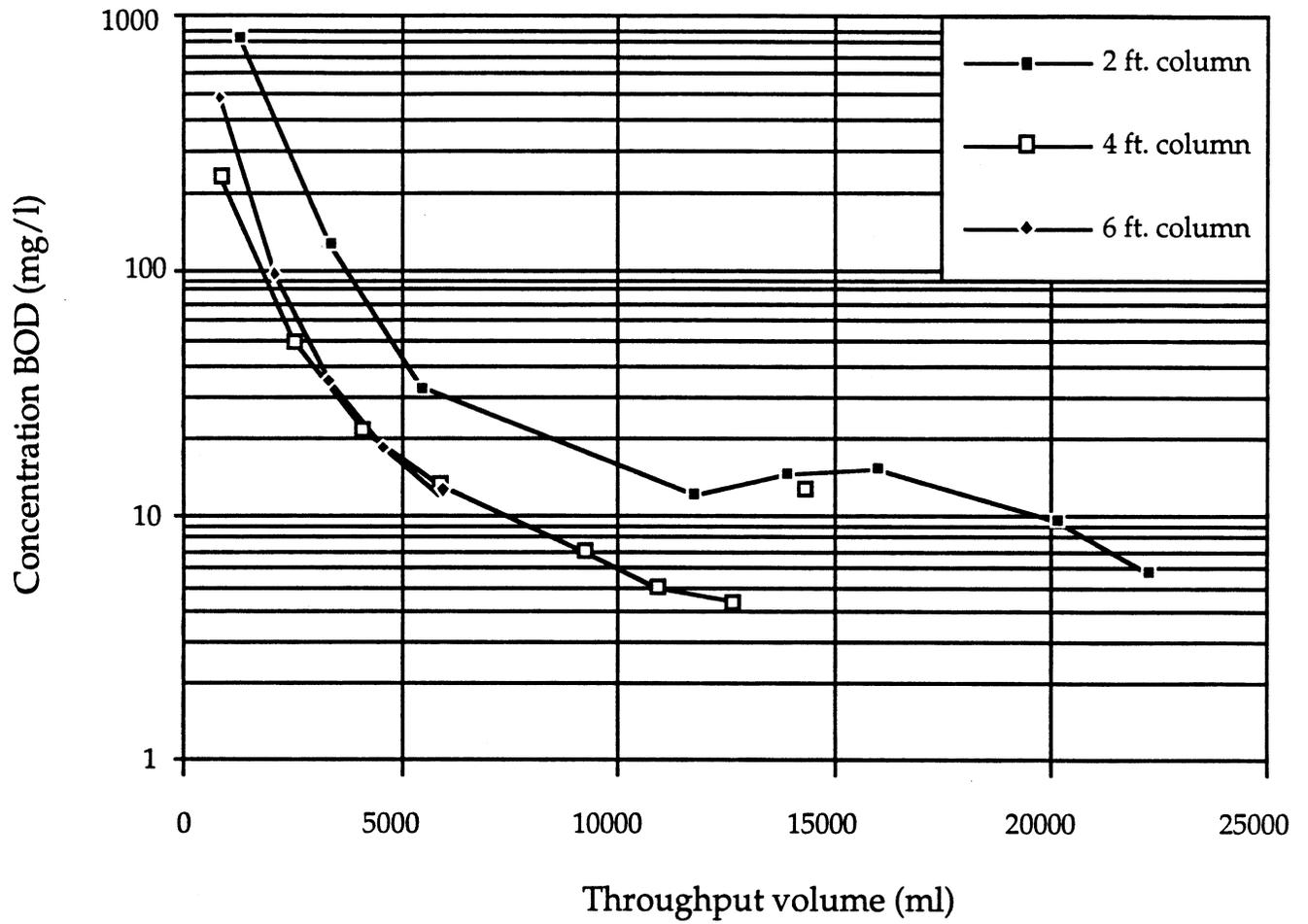


Figure 4.20 WPBMRF leaching column BOD results

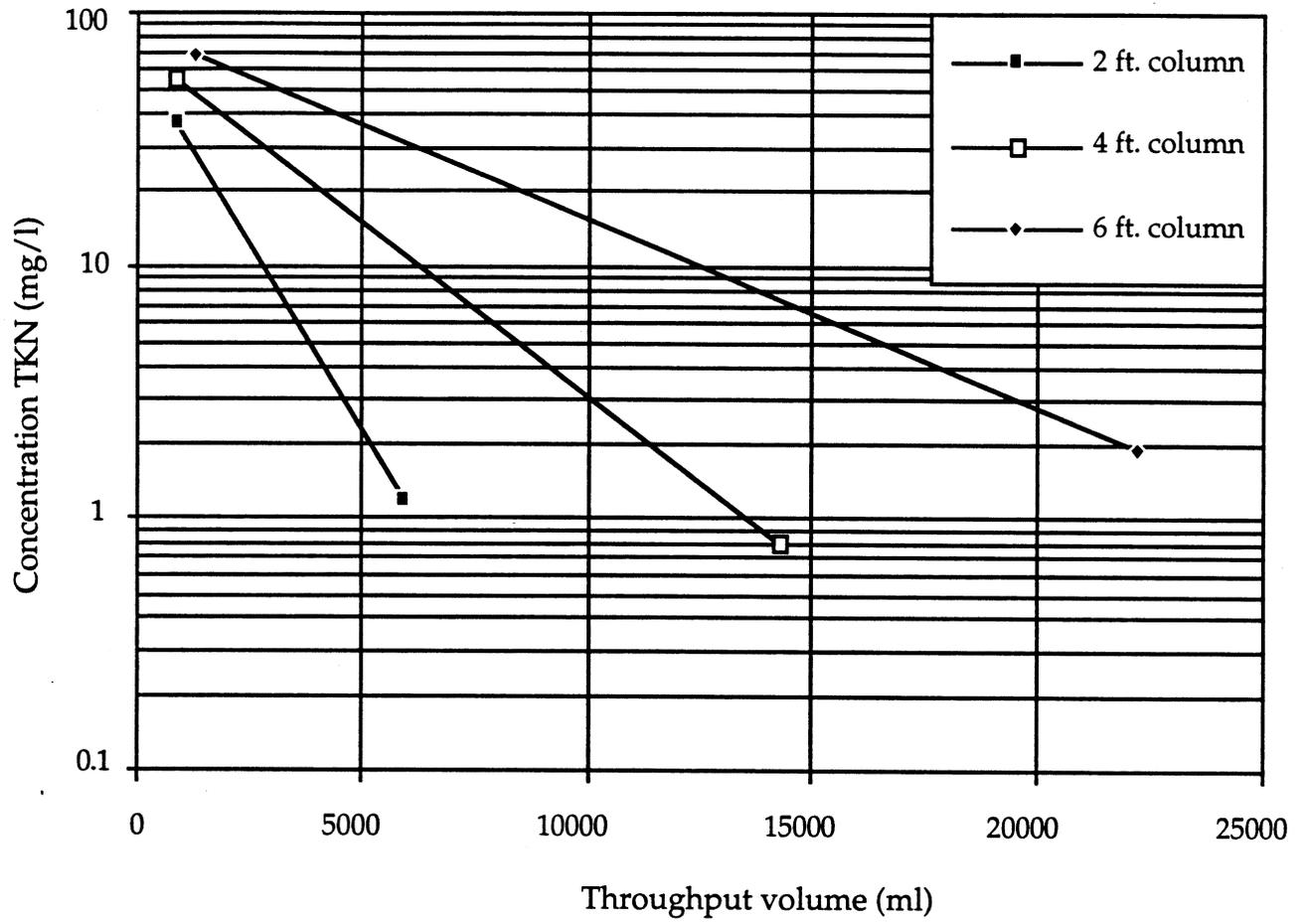


Figure 4.21 WPBMRF leaching column TKN results

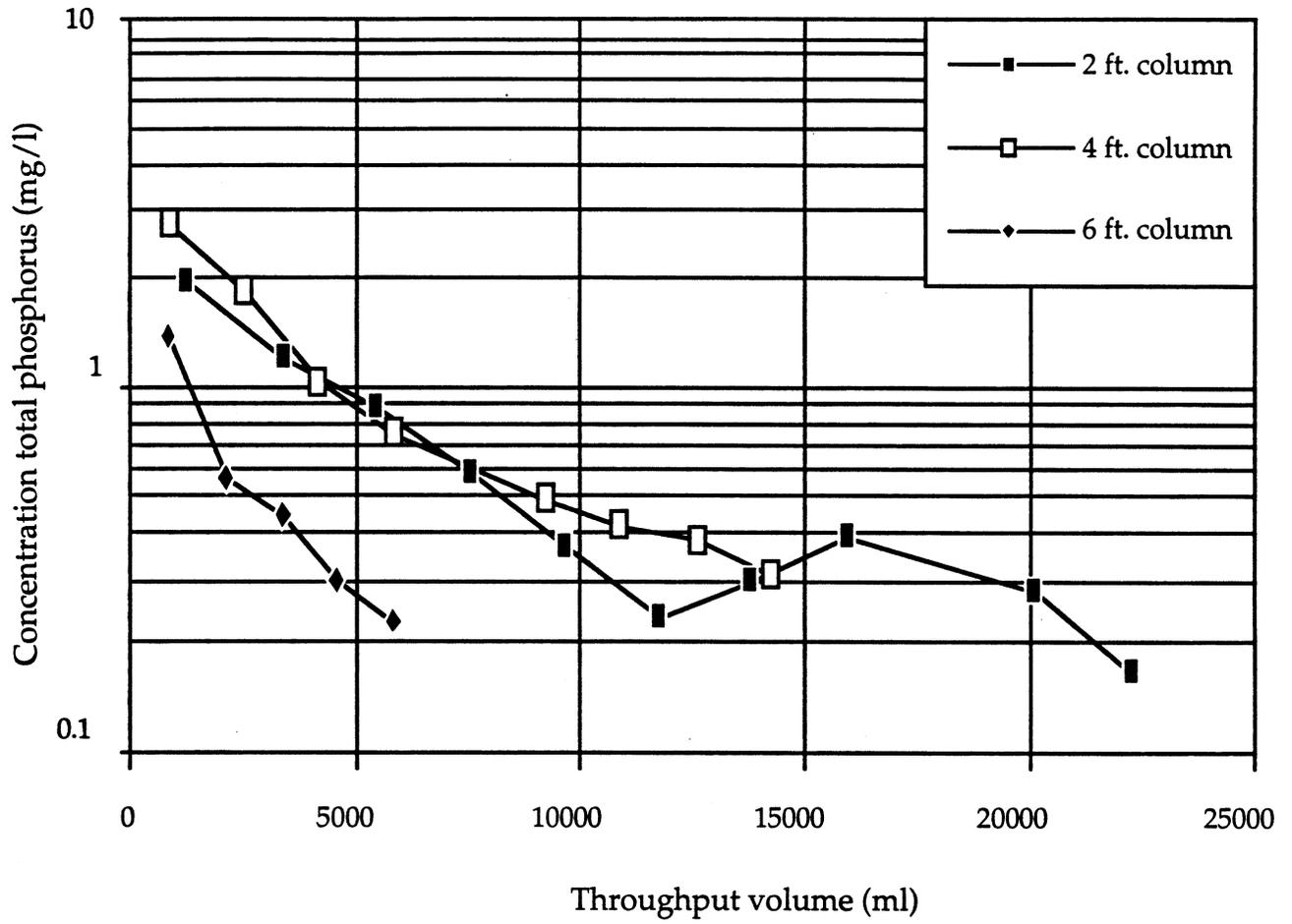


Figure 4.22 WPBMRF leaching column TP results

## 5.0 CONCLUSIONS FOR WG HIGHWAY APPLICATIONS

The results of this study show that WG has the physical and geotechnical properties necessary for application as highway fill material and meets existing environmental acceptability regulations. Its costs should be competitive to costs for conventional fill materials.

*The following specific findings support this conclusion:*

- ☛ WG or mixed cullet can be processed to meet ASTM D 448 gradations #8, #9, #10 for construction purposes.
- ☛ The glass processed and obtained from WPBMRF meets the ASTM D 448 #89 gradation.
- ☛ WG meeting ASTM D448 specifications as #8, #9 or #89 gradations behaves as an excellent drainage material and is recommended for use in drainage applications.
- ☛ WG is approximately 20 percent less dense than conventional natural aggregates. This property is useful when WG is used as backfill material behind retaining walls.
- ☛ Hazen's equation yielded a good prediction of WG permeabilities.
- ☛ The permeability coefficients did not decrease significantly with increase in density and the permeability at minimum density was about half an order of magnitude higher than the permeability at maximum density.

- Both permeability and specific yield increased linearly with a decrease in density, for all of the ASTM D 448 gradations studied (#8, #9, #10, and WPBMRF).
- A comparison of test results indicated that there was a relationship between permeability and specific yield such that specific yield increased with increasing permeability.
- The drainage analysis clearly shows that  $D_{10}$  controls the drainage time as well as the permeability and specific yield.
- Confined compression testing up to pressures of 210 psi (1470 kPa) proved that very little degradation would occur for WG subjected to high static stresses in a confined zone. However, field compaction equipment may crush the grains near the surface where low confining pressures exist. This problem needs to be addressed during field testing.
- During confined compression loading WG experiences a strain-hardening. There is a significant difference in the strain-stress responses for the unload-reload cycles and the virgin loading conditions. Typically there was an order of magnitude increase in the slopes from the "virgin" loading curve to the unload-reload curve.
- The inherent problems associated with the direct shear test, including maintaining the proper gap between the shear boxes and the development of stress concentrations, have affected the results and lead to the conclusion that direct shear testing will significantly over estimate WGs' friction angle.

- For all triaxial tests, failure occurred at about 5 % strain, at which point the stress-strain curve was relatively horizontal. In general, there is a slight increase in friction angle with increasing density. Friction angles at low densities ranged from 20 to 38 ° and friction angles at high densities ranged from 24 to 43 °. These values are consistent with friction angles reported for angular materials.
- The CBR/LBR values are considered very poor, typically falling below 3. Bearing ratio testing on WG was difficult because the top of the samples were easily disturbed allowing the piston to easily penetrate into WG. These results led to the conclusion that WG should not be used for highway bases or subbases.
- WG used as a fill material is classified as clean debris by DER. Clean debris requires no special permits or regulatory involvement when used as fill.
- Even though WG is classified as clean, it is contaminated with soluble organics and capable of producing a leachate with high BOD and TKN concentrations.
- WG may become clean due to rainfall and biodegradation during its accumulation and storage at the solid waste handling facility.
- Testing using the shake extraction method ASTM D 3987 modified by using a 1:1 volume ratio of glass to water, is an appropriate way to produce a leachate for contaminant measurement.
- WG should not be used as a fill material without testing its pollution potential.

## **6.0 DEVELOPMENTAL SPECIFICATIONS FOR WG HIGHWAY APPLICATIONS**

*Based on the results from this study the following developmental specifications are proposed. These specifications have been formatted to fit into the general section on Earthwork and Related Operations in "Standard Specifications for Road and Bridge Construction" (1991) from the Florida Department of Transportation. Section number 180 was developed such that any new specifications for use of waste materials could be added at the end of the section as they were approved. For completeness of this report the description (Section 180-1) associated with this new section has been presented in both volumes of this final report. Section 180-2 Municipal Waste Combustor Bottom Ash is presented in Volume 1 of 2.*

### **DEVELOPMENTAL SPECIFICATION SECTION 180 REUSE OF DISCARDED MATERIALS AND BYPRODUCTS**

#### **180-1 Description**

Discarded materials and byproducts shall consist, in general of municipal waste combustor bottom ash and waste glass generated from state mandated recycling quotas. The specification requirements for various discarded materials as contained in this Section are to govern their use only when these materials are used as a source of borrow material.

Sources of supply shall be approved by the Department.

### 180-3 Waste Glass

**180-3.1 Composition:** Waste Glass shall consist of collected glass, available from recycling facilities.

**180-3.2 Gradation:** Waste glass for borrow or drainage applications shall meet the following gradation requirements:

Passing the 1/2-inch sieve	Minimum 97% (max. dimension , 1-inch)
Passing the No. 200 sieve	Maximum 2 % (by weight)

**180-3.3 Characteristics:** Waste glass shall contain no more than 1 percent by weight of paper, plastics or other deleterious materials.

**180-3.4 Furnishing of Material:** Except as might be specifically shown otherwise, all waste glass material and sources thereof shall be furnished by the contractor.

**180-3.5 Storage of Material:** Waste glass shall be stockpiled for a sufficient time period to allow reduction of leachable materials to acceptable environmental levels.

**180-3.6 Chemical Properties:** Prior to usage, leachate from waste glass stockpiles must meet treated domestic waste water standards, for land application. In addition, the contractor must comply with regulatory issues of other environmental regulatory agencies.

**180-3.7 Construction Methods:** The contractor must comply with construction methods specified in DOT Standard Specifications for constructing embankments Section 120-8. Waste glass shall not be placed in contact with synthetic liners, geogrids

or geotextiles. Waste glass shall not be left exposed to the air for extended periods of time to be determined by the Engineer.

180-3.7.1 *Support of Vegetation:* Areas to be covered with grass shall be covered with a minimum of twelve inches of topsoil over the waste glass. Prior to planting trees and shrubs, the depth of the topsoil shall be adjusted to accommodate the root system.

180-3.7.2 *Compaction Requirements:* Waste glass shall be compacted to a minimum density of 100 pcf unless otherwise approved by the Engineer.

180-3.8 **Safety and Health:** The contractor must comply with the requirements of Section 7-1.4 of the Florida DOT Standard Specifications.

## 7.0 RECOMMENDATIONS FOR WG HIGHWAY APPLICATIONS

There is a need to continue and expand the scope of this project due to the promising results from the first phase. The proposed continuation study of WG should include; 1) stockpiling WG and testing its environmental properties to ensure they are acceptable; 2) studying the variation in engineering and physical properties when WG is combined with FDOT conventional bases courses; 3) expansion of the current data base; and 4) field demonstration projects with emphasis on: a) how *in situ* densities will be achieved and measured and, b) how the *in situ* drainage-strength properties perform in a rigid pavement system.

*Based on the results from this study the following recommendations have been formulated.*

- Research on the effects of the WG combined with conventional coarse aggregates needs to be conducted to determine: 1) if bearing ratios acceptable for

base/subbase applications can be achieved and 2) the drainage properties of the conventional base courses are improved.

- The type of compaction equipment needed to compact glass aggregates in the field needs to be investigated. WGs' lack of surface tension make it insensitive to water and may cause problems during compaction. Glass' inherent insensitivity to moisture enables it to be placed and compacted in wet weather, keeping construction downtime to a minimum.
- WGs' excellent frictional characteristics will allow it to be used as embankment material or drainage material, therefore, it is recommended that a field test site be constructed to verify these applications.
- Environmental concerns exist if WG is not washed, however, if properly washed WG can be used in highway applications.
- Soluble contamination level testing in WG should be based on shake extraction with a 1:1 ratio volumetric ratio of glass to water or should be performed using a leaching column analysis.
- To provide usable quantities, facilities would have to accumulate WG for at least six months. During stockpiling, biological degradation and rainfall occurrences may be sufficient to "clean" the glass so that the leachate would exhibit pollutant concentrations similar to normal storm water. The total quantity of rainfall required to clean WG would be a function of the height of the glass piles. Storing glass in 2 ft (60 cm) layers or less would significantly reduce the required rainfall for cleaning. If long accumulation periods are possible, then the storage height of the glass piles could exceed 6 ft (180 cm). It is likely that degradation processes

would greatly reduce the required storage periods. It may also be possible that recirculating leachate over the WG piles could provide treatment in shorter periods. The variability of the clean-up times should be the subject of further investigations.

## 8.0 REFERENCES

- American Association for State Highway and Transportation Officials. (1978) *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*. 12th Edit. Wash. D.C. Part I, Specifications, pp. 828; Part II, Tests, pp. 998.
- American Association for State Highway and Transportation Officials. (1990) *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*. 15th Edit. Wash. D.C. Part I, Specifications, pp. 828; Part II, Tests, pp. 998.
- American Public Health Association (1989) *Standard Methods for Examination of Water and Waste Water*. 17th Edit.
- American Society for Testing and Materials. (1990a). *Annual Book of ASTM Standards, Concrete and Aggregates*. Volume 04.02. Phil. Pa.
- American Society for Testing and Materials. (1990b) *Annual Book of ASTM Standards, Road and Paving Materials; Traveled surface Characteristics*. Volume 04.03. Phil. Pa.
- American Society for Testing and Materials. (1987) *Soil and Rock; Building Stones; Geotextiles*. Annual Book of ASTM Standards. Volume 04.08. Phil. Pa.
- American Society for Testing and Materials. (1982) *Compilation of ASTM Standard Definitions*. Annual Book of ASTM Standards. Fifth Ed. Phil. Pa.
- American Society for Testing and Materials. (1980) *Natural Building Stones; Soil and Rock*. Annual Book of ASTM Standards, Part 12, Phil., Pa. pp. 634.
- Baker, R., (1983) *Handbook of Highway Engineering*. Malabar Fl.: Robert E. Kreiger
- Barksdale, R. E., (1991) *The Aggregate Handbook*. National Stone Association,.
- Bishop, A.W., and Henkel, D.J., (1964) *The Measurement of Soil Properties in the Triaxial Test*. London: Edward Arnold.
- Bishop, A.W., (1961) *The Strength of Soils as Engineering Materials*. Sixth Rankine Lecture.

- Bowles, J. E., (1979) *Physical and Geotechnical Properties of Soils*. New York: Mc Graw-Hill.
- California Department of Transportation., (1992) Aggregate subbases. *Standard Specifications*. Section 25, 26 (July).
- City of Seattle, Engineering Department, (1992) *Green Lake Stormwater Project*. Grant Number WFG 91029.
- Carpenter, S. H., Darter, M. I. and Dempsey B.J., (1981) *A Pavement Moisture-Accelerated Distress (MAD) Identification System*. Vol. I FHWA/RD-81/079, Vol. II, FHWA/RD-81/080 (September).
- Christy, Dan, (1995) Personal Communications May 5.
- Connecticut Department of Transportation, Office of Research and Materials. (1992) Crushed glass option. *Specifications*. Section 4.06.
- D'Appolonia, D.J. and Whitman. R.V., (1969) "Sand Compaction with Vibratory Rollers." *Journal of Soil Mechanics and Foundations Division*. ASCE. Vol 95, No. SM1 January, pp. 263-284.
- D'Arcy, H., (1856) *Les Fontaines Publiques de la Ville de Dijon*. Dalmont, Paris.
- Dames & Moore., (1993) *Glass Feedstock Evaluation Project*. Clean Washington Center.
- Das, B. M., (1983) *Advanced Soil Mechanics*. McGraw-Hill.
- Das, B. M., (1990) *Principles of Foundation Engineering*. Boston: PWS-KENT.
- Das, B. M., (1994) *Principles of Geotechnical Engineering*. 3rd Edit. Boston: PWS-KENT p. 320.
- Fisher, D., Interview with author, Melbourne, Florida, July (1994)
- Florida Department of Transportation., (1991) *Standard Specifications for Road and Bridge Construction*.
- Florida Department of Transportation., (1993) *Florida Method of Test for Limerock Bearing Ratio*. Designation: FM-5-515.
- Glass Packaging Institute, (1994) Glass recycling rate. *News*. (April).

- Hazen, A., (1911) *Discussion of Dams on Sand Foundations* by A.C. Koenig *Transactions, ASCE* Vol 73 pp. (199-203).
- Head, K.H., (1982) *Manual of Soil Laboratory Testing Vol. 2: Permeability Shear Strengths and Compressibility Tests*. New York: Halsted Press.
- Head, K.H., (1986) *Manual of Soil Laboratory Testing Vol. 3: Effective Stress Tests*. New York: John Wiley and Sons.
- Heck, H. H. III, Korzun, E. A. Stephens, N. T., (1989) "The Impact Of Increased Recycle Rates On Markets For Recycled Paper, Plastic, Metals, Glass And Rubber In Florida." *The Journal of the Air and Waste Management Association*. Vol. 39, No.12 (December).
- Huang, Y. H., *Pavement Analysis and Design*. (1993) Prentice Hall.
- Hughes, C.S., (1990) *Feasibility of Using Recycled Glass in Asphalt*. Virginia Transportation Research Council, Charlottesville, Va.
- Holtz, D. R. and Kovacs, D. W., (1981) *An Introduction to Geotechnical Engineering*. New Jersey: Prentice-Hall.
- HWA (Hong West and Associates, Inc.), (1992) *Design Technical Memorandum*.
- Institute of Scrap Recycling Industries., (1994) Guidelines for nonferrous scrap, ferrous scrap, glass cullet, paper stock, plastic scrap. *Scrap Specifications Circular*.
- Jumikis A., (1984) *Foundation Engineering*. Malabar Fl., Kreiger Publishing.
- Jumikis A., (1984) *Soil Mechanics*. Malabar Fl., Kreiger Publishing.
- Kynes, J., (1994) Update on RMAC activities. *State of Florida Department of Commerce*.
- Lambe, T.W., and Whitman, R. V., (1979) *Soil Mechanics, SI Version*. New York: John Wiley and Sons.
- Lambe, T.W., (1969) *Soil Mechanics*. New York: John Wiley and Sons.
- Lee, K. and Singh, A., (1971) "Compaction of Granular Soils." *Proceeding of the 9th Annual Engineering Geology and Soils Engineering Symposium*. April. pp. 161-174.

- Lee, K. and Seed. H.B., (1978) "Drained Strength Characteristics of Sands." *Journal of Soil Mechanics and Foundations Division*. ASCE. Vol 93, No. SM6 November, pp. 117-141.
- Maine Department of Transportation, (1992) *Special Provisions Section 203 Excavation and Embankment (Crushed Glass)*. November, 16.
- Malisch, W.R., Day, D. E., and Wixson, B.G., (1970) Use of domestic waste glass as aggregate in bituminous concrete. *Highway Research Record* No. 307. Highway Research Board.
- Mattox, R., and Lyndon M. H., (1990) *National Highway Institute Highway Materials Engineering Module II: Soils and Foundations*. Report No. FHWA - HI - 90-005.
- McCarthy, D.F., (1988) *Essentials of Soil Mechanics and Foundations: Basic Geotechniques*. Prentice Hall, Third Edition.
- Menges, D. A., (1990) Potential markets for mixed cullet. *RRT Empire Returns*, Aug., 3-5.
- National Association of Plumbing-Heating-Cooling-Contractors., (1992) Storm drains. *National Standard Plumbing Code*. Amend Section 13.1.5.
- New Hampshire DOT., (1992) Amendments to section 304-aggregate base courses. *Special Provision*. October.
- NIOSH Manual of Analytical Methods., (1977) 2nd Edit V. 1 P&CAM 106 U.S. Department of Health Education and Welfare, Publ. NIOSH 77-157-A
- PennDOT., (1992) Waste glass as pipe backfill. *Pennsylvania Draft Amendments*. Section 703.2.
- Russo, David, (1995) Personal communication May 31.
- Syed, S.A., (1994) *Classification and Drainage Properties of Waste Glass for Highway Applications*. Master of Science Thesis, Florida Institute of Technology.
- Sowers, G. F., (1979) *Introductory Soil Mechanics and Foundations*. 4<sup>th</sup> edition. New York: Macmillan Publishing Company.

- Rodriguez A., del Castillo, R. H. and Sowers, G.F., (1988) *Soils for Highway Engineering*, Trans Tech Publications
- Sawyer, C.N. and McCarty P.L., (1978) *Chemistry for Environmental Engineers*. McGraw-Hill Inc., New York.
- Todd, D. K., (1980) *Groundwater Hydrology*. New York: John Wiley & Sons.
- U.S. Department of the Interior, (1990) *Minerals Yearbook*. U.S. Government Printing Office, Washington D.C.,
- U.S. EPA, (1992) *Characterization of Municipal Solid Waste in the United States: (1992 Update)*. EPA 530-R-92-019, Washington D.C.
- U.S. EPA, (1974) *Methods for Chemical Analysis of Water and Wastes*. EPA 625-6-74-003, Office of Technology Transfer, Washington D.C.
- Veith, D and Guenther I., (1991) The use of green cullet in electrical trenches, feasibility study. *British Columbia Institute of Technology*.
- Vermont Department of Transportation. (1992) *Standard Specifications for Roads*. Section 301-Subbases.
- Walton, W.C., (1963) *Groundwater Resource Evaluation*. McGraw-Hill Book Co., N.Y., N.Y. p. 664.
- Wang, S. I., (1993) *Environmental Acceptability of Using Municipal Waste Combustor Ash as Highway Fill Material*. Master of Science Environmental Science, Florida Tech.
- Washington State Department of Transportation. (1991) *Standard Specification for Road, Bridge and Municipal Construction*. Section 9-03.
- White, K. M., (1992) *MRF of the Month Palm Beach County*. Waste Age.