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

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16. 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.

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METRIC (SI*) CONVERSION FACTORS

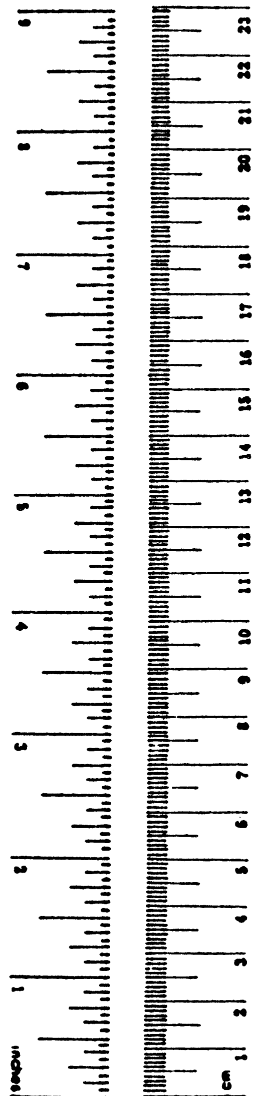
APPROXIMATE CONVERSIONS TO SI UNITS

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^2	square inches	645.2	millimetres squared	mm^2
ft^2	square feet	0.0929	metres squared	m^2
yd^2	square yards	0.836	metres squared	m^2
mi^2	square miles	2.59	kilometres squared	km^2
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^3	cubic feet	0.0328	metres cubed	m^3
yd^3	cubic yards	0.0765	metres cubed	m^3

NOTE: Volumes greater than 1000 L shall be shown in m^3.

	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^2	millimetres squared	0.0016	square inches	in^2
m^2	metres squared	10.764	square feet	ft^2
km^2	kilometres squared	0.39	square miles	mi^2
ha	hectores (10 000 m^2)	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^3	metres cubed	35.315	cubic feet	ft^3
m^3	metres cubed	1.308	cubic yards	yd^3
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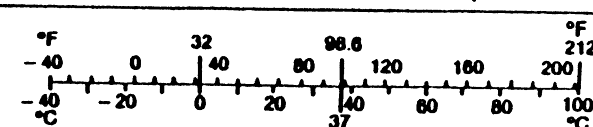

These factors conform to the requirement of FHWA Order 5190.1A.

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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 mg/m^3. PEL's from the personal sample and two area samples were all below 0.5 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 X 10^{-4} cm/sec, whereas the falling head method is applicable for silts or clay with permeability coefficient less than or equal to 1 X 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:

$$q = kiA \tag{2.1}$$

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:

$$v \propto i/\eta \tag{2.2}$$

where: v is the discharge velocity;

i is the hydraulic gradient; and

η 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 η. Viscosity varies with temperature, and increases approximately 30% when there is a temperature change from 20 to 30 degrees centigrade (^{o} C). ASTM D 2434 specifies that all laboratory permeability values be reported as an equivalent permeability at 20 ^{o} C (ASTM, 1987). If a permeability test conducted at T ^{o} C gives a coefficient of permeability of k_t, the corresponding value at 20 ^{o} C can be found from:

$$k_{20} = k_T(\eta_T / \eta_{20}) \qquad (2.3)$$

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):

$$S_y\,(\%) = (V_y / V) \times 100 \tag{4}$$

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^3). The minimum index values ranged from 76.8 to 89.5 pcf (1.20 to 1.40 kN/m^3) and the maximum index values ranged from 90.9 to 109.3 pcf (1.42 to 1.71 kN/m^3). 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 (ø). 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 plan, 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 ø:

1. ø decreases with increasing void ratio.
2. ø increases with increasing angularity.
3. ø 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 ø.

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 annular 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).

Sample	Glass Color	Sorting	Days Extracted	BOD-5 mg/l	COD mg/l	TOC mg/l	pH SU	Specific Conductivity umhos/cm	Antimony	Arsenic	Beryllium	Cadmium	Chromium	Cobalt	Copper	Lead	Mercury	Nickel	Selenium	Silver	Thallium	Zinc
									Total Metals, mg/l													
AZ-1 A	Brown	Dropbox/	1	4.9	29	4.9	10.3	59	ND	ND	ND	ND	ND	ND	0.11	0.002	ND	0.025	ND	ND	ND	0.037
		Barrels -	2	ND	11	1.5	10.1	23	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	0.047
		Unattended	3	ND	ND	1.0	10.1	16	ND	ND	ND	ND	ND	ND	ND	0.002	ND	ND	ND	ND	ND	0.032
			4	ND	ND	1.0	10.0	18	ND	ND	ND	ND	ND	ND	ND	0.002	ND	ND	ND	ND	ND	0.026
			5	ND	ND	1.3	9.9	14	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	0.037
			6	ND	ND	1.0	9.9	17	ND	ND	ND	ND	ND	ND	ND	0.003	ND	ND	ND	ND	ND	0.096
			7	ND	ND	0.9	9.8	18	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	0.026
			8	ND	ND	ND	9.7	19	ND	ND	ND	ND	ND	ND	ND	0.004	ND	0.021	ND	ND	ND	0.048
			9	ND	ND	ND	9.7	21	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	0.022
			10	ND	ND	0.6	9.6	23	ND	ND	ND	ND	ND	ND	ND	0.003	ND	ND	ND	ND	ND	0.032
AZ-1 B	Brown	Dropbox/	1	6.4	32	5.9	10.4	78	ND	ND	ND	ND	0.024	ND	0.14	ND	ND	0.021	0.003	ND	ND	0.031
		Barrels -	2	ND	15	ND	10.0	24	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	0.027
		Unattended	3	ND	ND	0.7	10.0	15	ND	ND	ND	ND	ND	ND	ND	0.004	ND	ND	ND	ND	ND	0.022
			4	ND	ND	1.2	9.9	15	ND	ND	ND	ND	ND	ND	ND	0.003	ND	0.024	ND	ND	ND	0.050
			5	ND	ND	1.2	9.9	17	ND	ND	ND	ND	ND	ND	ND	0.002	ND	ND	ND	ND	ND	0.031
			6	ND	ND	ND	9.8	21	ND	ND	ND	ND	ND	ND	ND	0.004	ND	ND	ND	ND	ND	0.035
			7	ND	ND	1.0	9.7	18	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	0.028
			8	ND	ND	0.9	9.7	15	ND	ND	ND	ND	ND	ND	ND	0.003	ND	0.029	ND	ND	ND	0.025
			9	ND	ND	1.7	9.7	17	ND	ND	ND	ND	ND	ND	ND	0.003	ND	ND	ND	ND	ND	0.035
			10	ND	ND	0.7	9.6	19	ND	ND	ND	ND	ND	ND	ND	0.003	ND	ND	ND	ND	ND	0.025
WA-10	Mixed	Curbside -	1	ND	18	3.2	10.0	42	ND	ND	ND	ND	ND	ND	0.049	0.002	ND	ND	ND	ND	ND	0.023
		Commingled	2	ND	ND	1.0	10.0	23	ND	ND	ND	ND	ND	ND	ND	0.005	ND	ND	ND	ND	ND	0.054
		With Other	3	ND	ND	2.0	10.0	20	ND	ND	ND	ND	ND	ND	ND	0.003	ND	ND	ND	ND	ND	0.025
		(Plastics,	4	ND	ND	1.4	10.0	25	ND	ND	ND	ND	ND	ND	ND	0.007	ND	ND	ND	ND	ND	0.033
		Cans) -	5	ND	ND	1.4	9.9	26	ND	ND	ND	ND	ND	ND	ND	0.004	ND	0.021	ND	ND	ND	0.032
		Negative	6	ND	ND	0.8	10.1	24	ND	ND	ND	ND	ND	ND	ND	0.003	ND	ND	ND	ND	ND	0.033
		Sorting	7	ND	ND	1.6	9.9	26	ND	ND	ND	ND	ND	ND	ND	0.004	ND	ND	ND	ND	ND	0.020
			8	ND	ND	0.7	9.8	28	ND	ND	ND	ND	ND	ND	ND	0.004	ND	0.036	ND	ND	ND	0.028
			9	ND	ND	1.4	9.9	26	ND	ND	ND	ND	ND	ND	ND	0.005	ND	ND	ND	ND	ND	0.026
			10	ND	ND	1.2	9.9	25	ND	ND	ND	ND	ND	ND	ND	0.004	ND	ND	ND	ND	ND	0.024
Range of Surface Water																						

Notes:

ND - Not Detected

Table 2.1 Chemical characterization-sequential batch extraction glass feedstock samples

(from Dames and Moore 1993);

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

Sample	Location	Crystalline Silica [1]	Total Dust [3]
CWC-01	Personnel sample: Daokaun Zhang	< 2.8 % [2]	0.280 mg/m^3
CWC-02	Area sample near mixing trays		0.351 mg/m^3
CWC-03	Area sample: near analytical scale		0.495 mg/m^3
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^3 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^3 for nuisance dust. Nuisance dusts are those which do not contain otherwise regulated particulate such as asbestos or dusts which contain greater then one percent silica (per 29CFR1910.1000).

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

Cullet Sample Number[1]	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^{-2}
CA-14	100	3/4" minus	90 % of ASTM D 698	89.6	26 C 10^{-2}
WA-09	100	1/4" minus	90 % of ASTM D 698	93.6	6.4 C 10^{-2}
WA-09	100	3/4" minus	90 % of ASTM D 698	95.9	18 C 10^{-2}
CA-14	100	1/4" minus	90% of ASTM D 698	94.9	4.4 C10^{-2}
CA-14	100	3/4" minus	90 % of ASTM D 698	89.6	23 C 10^{-2}
WA-09	100	1/4" minus	90 % of ASTM D 698	93.6	4.8 C 10^{-2}
WA-09	100	3/4" minus	90 % of ASTM D 698	95.9	6.5 C 10^{-2}

1. 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^3

Table 2.4 Typical values of ø for aggregate (from Das, 1983)

TYPE OF SOIL		VOID RATIO		
		0.41-0.5	0.51-0.6	0.61-0.7
Coarse Sand	ø°	43	40	38
Medium Sand	ø°	40	38	35
Fine Sand	ø°	38	36	32
Sandy Silt	ø°	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			Void Ratio	
		0.41-0.5	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^3

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_{10}) 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 Southeasts' 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^3) or a 30 pcf (0.47 kN/m^3) 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^3), 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^3) 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^3 (0.0009 m^3) 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^3 (0.0009 m^3)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^3), 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^3). 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^3 (0.0009 m^3). 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:

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

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:

$$y = -11.894 + 26.394x \tag{3.2}$$

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^3). 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^o envelope associated with the lowest confining stress, a 43^o envelope associated with the intermediate confining stress and a 42^o envelope associated with the lowest 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^o 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.

$$\text{CBR @ } 0.1'' = \frac{\sigma_{0.1''}}{1000\,psi} * 100 \tag{3.3}$$

$$\text{CBR @ } 0.2'' = \frac{\sigma_{0.2''}}{1500 \text{ psi}} * 100 \tag{3.4}$$

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.

$$\mathrm{LBR} = \frac{\sigma_{0.1''}}{800\,psi} * 100 \tag{3.5}$$

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 ± 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_2SO_4 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^3 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.)

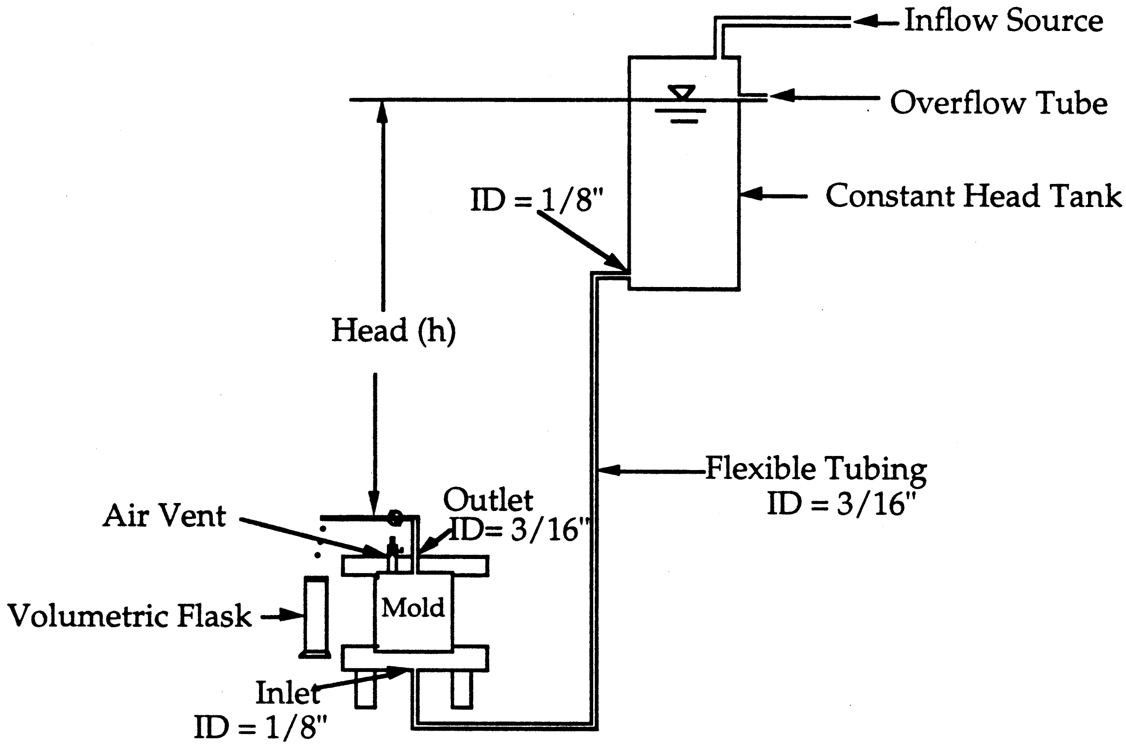


Figure 3.2 Schematic of the standard permeability test apparatus (1 in = 25.4 mm)

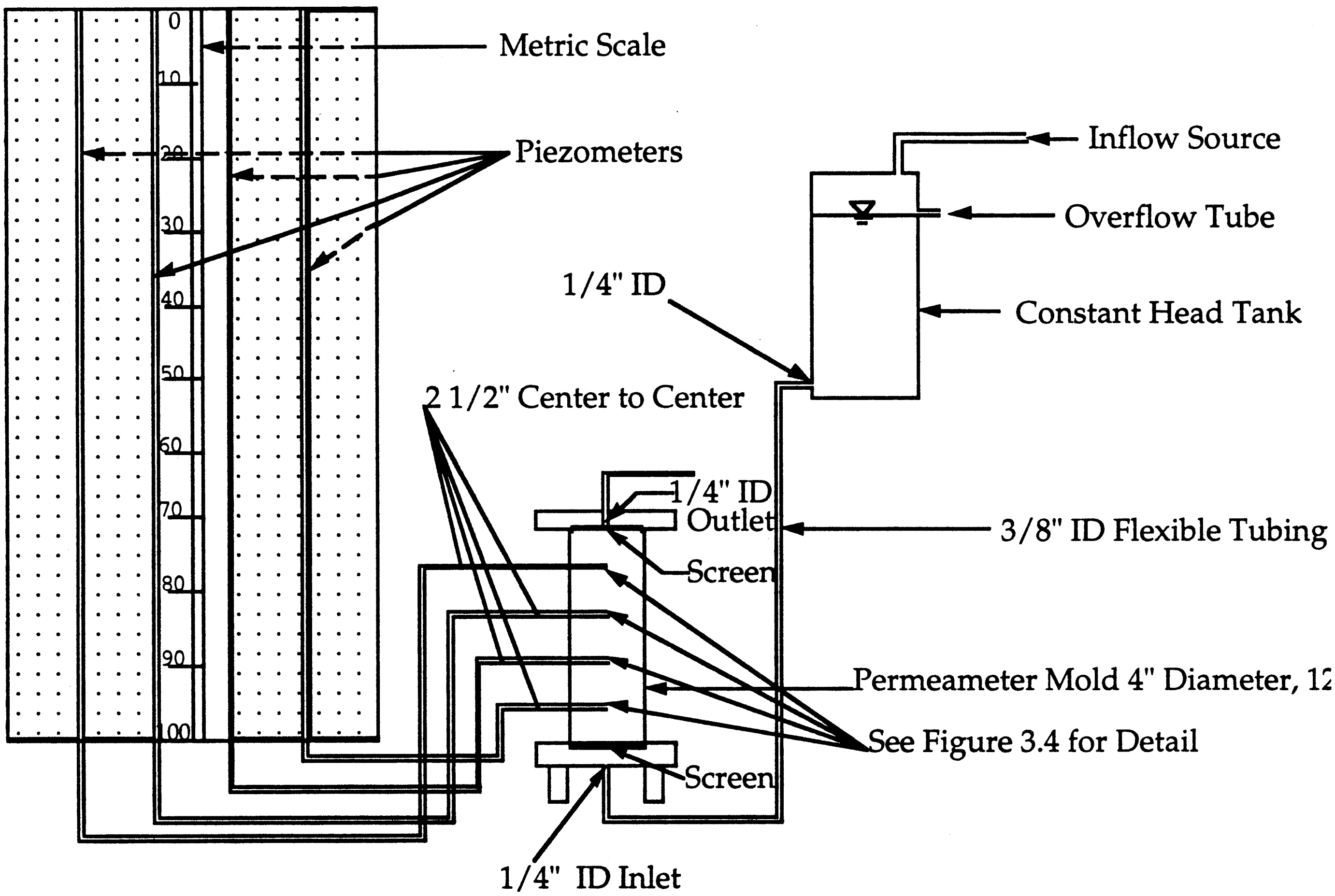


Figure 3.3 Schematic of the modified permeability test apparatus (1 in = 25.4 mm)

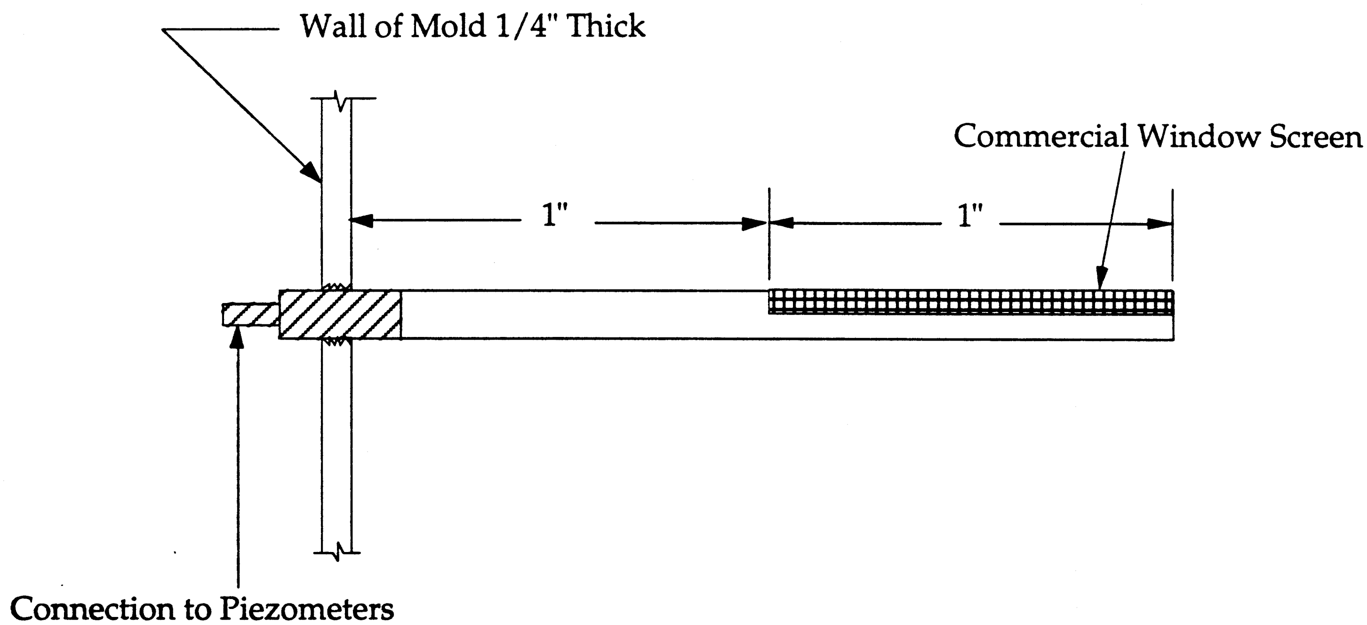


Figure 3.4 Schematic of piezometer inlet within permeameter mold(1 in = 25.4 mm)

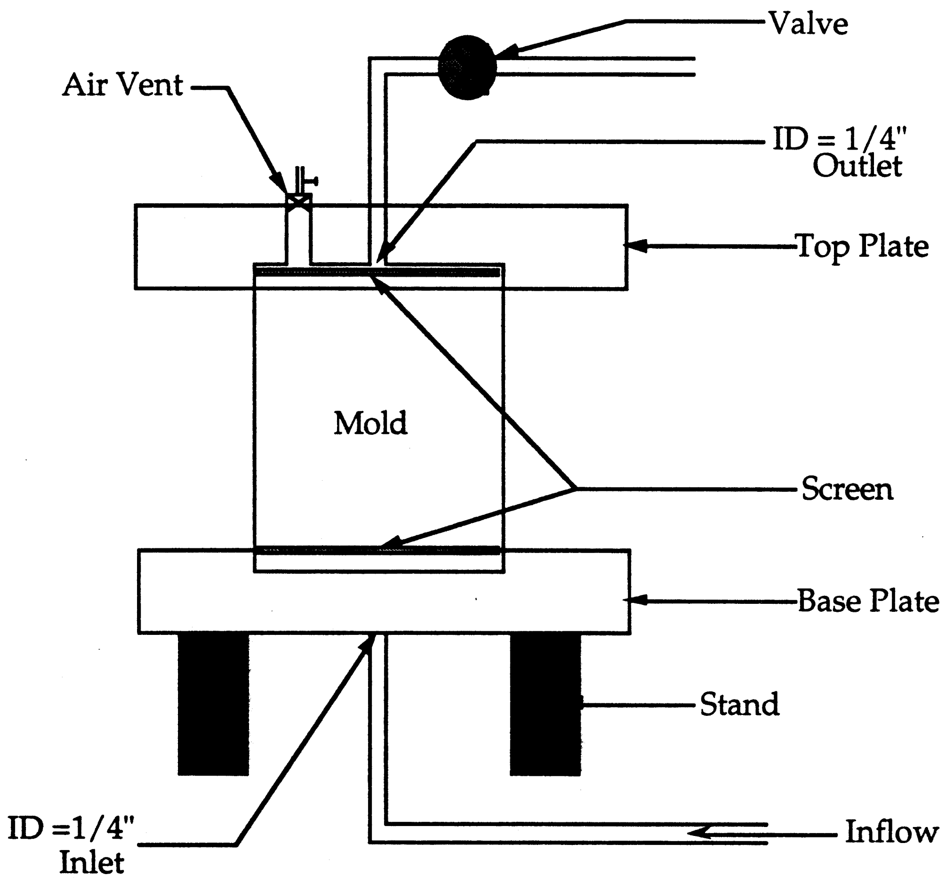


Figure 3.5 Schematic of specific yield test apparatus (1 in = 25.4 mm)

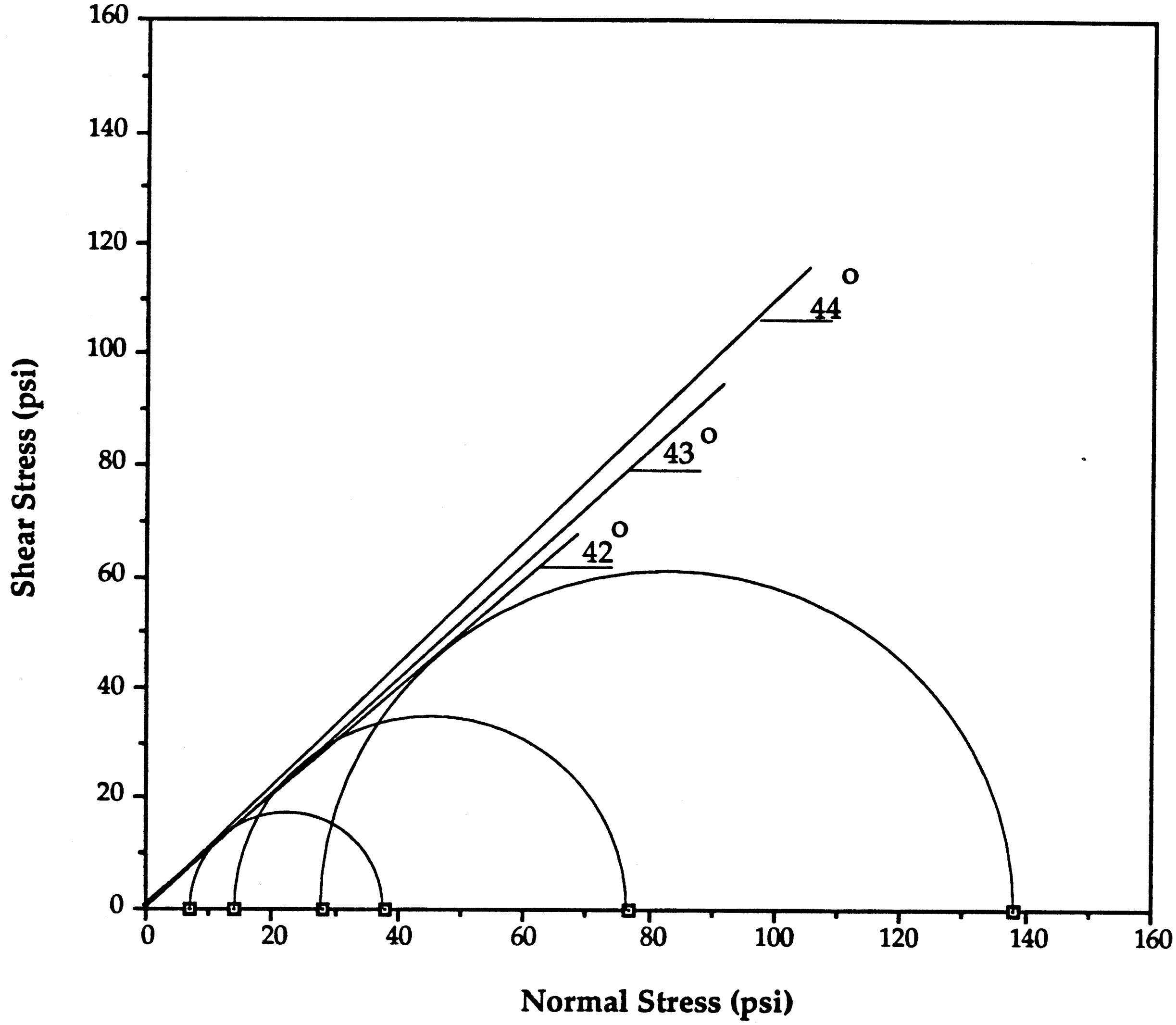


Figure 3.6 Mohr circles for preliminary WPBMRF gradation triaxial testing (1 psi = 6.895 kPa)

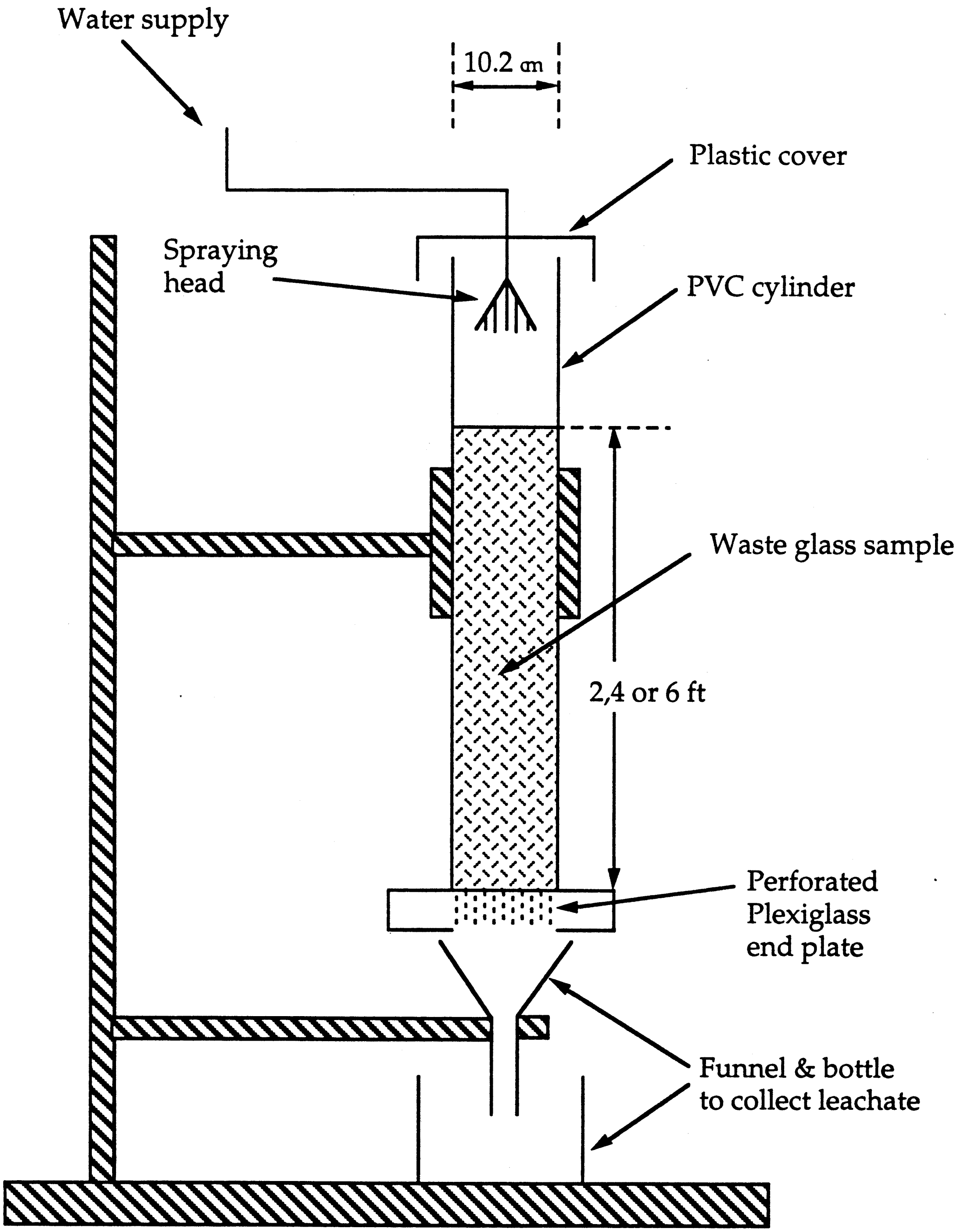
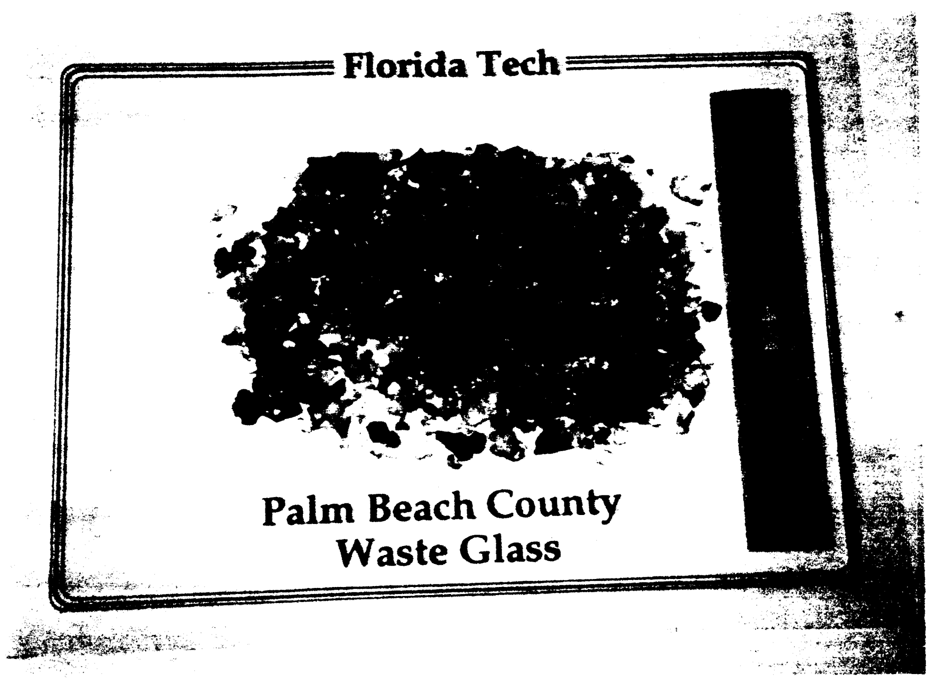


Figure 3.7 Schematic of column leaching apparatus (from Wang, 1993)

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^3), whereas for #10 gradation the maximum density ranges from 81 to 87 pcf (1.27 to 1.37 kN/m^3). These differences are believed to be a results of the smaller D_{10} of ASTM #10 WG. WPBMRF (#89) glass had a higher maximum density (111 pcf, 1.74 kN/m^3) 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^3). These densities are higher than the 33 to 55 pcf (0.52 to 0.87 kN/m^3)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^3), is lower than the minimum density of gravel (100 pcf, 1.57 kN/m^3) 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)

$$k = C\,D_{10}^2 \tag{4.1}$$

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_{10} 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 ≤ 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 ≤ 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^{o}. 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 kN/m^3) has a friction angle of 40^{o}, 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 ^{o} and friction angles at high densities ranged from 24 to 43 ^{o}. 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 ^{o} 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^3. 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 Kg/m^{2}) based on a density of 90 lb/ft^3 (1443 Kg/m^3) to properly conduct sequential batch extraction. As stated above ASTMs' 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_5 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_5 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 BOD5 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:

$$y = L\left(1 - 10^{-kt}\right) \tag{4.2}$$

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:

$$\mathrm{k} = 2.61\frac{\mathrm{B}}{\mathrm{A}}, \tag{4.3a}$$

$$\mathrm{L} = \frac{1}{6\mathrm{A}^2\mathrm{B}}. \tag{4.3b}$$

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_5 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_5 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_5 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_5 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± 0.29 and for TP was 0.76 ± 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
Mean	4.0 ± 1.2	4.2 ± 0.9	4.1 ± 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	A-1-a
			with Sand (GP)	
#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	A-1-a
			with Gravel (SP)	
#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	A-1-a
			with Gravel (SP)	
WPBMRF	9.3	1.4	Well Graded Sand	A-1-a
			with Gravel (SW)	
BSMG	3.1	1.3	Poorly Graded Gravel	A-1-a
			with Sand (GP)	

* 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_{10} (mm)	Coefficient of Permeability (k) (cm/sec)	Specific Yield (N_e) (%)	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

ASTM D 448 Gradation	Unit Weight (pcf)	Angle of Internal Friction (o)
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

ASTM Gradation	Unit Weight (pcf)	CBR (%)	LBR (%)
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?	Production tons/year Clear	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	sort-crush	yes	poss.	no
7	Calhoun	*No response*								
8	Charlotte	*No response*								
9	Citrus	Curbside				total 700	sort-crush	yes	yes	no
10	Clay	*No response*								
11	Collier	Curside	16,732			16,444	-	yes	yes	no
12	Columbia	Bins				total 170	sort	info	no	yes/ 20 tons
13	Dade	MRF	2,466	550	1,334	5,590	sort-crush	yes	yes	no
14	Desoto	Bins				total 35	sort	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	info	no	no
22	Glades	*Do not currently have a recycling program*								
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 used?	Production tons/year Clear	Brown	Green	Mixed	Addn'l Treatment	Would you sell glass at $3/cu.yds.?	Is there storage available?	Stockpile currently
24	Hamilton	*No response*								
25	Hardee	Bins, Curb	60	40	0	0	sort-crush	no	yes	no
26	Hendry	Bins, MRF	35	10	5	na	sort-crush	yes	yes	yes/ 20 tons
27	Hernando	MRF				total 300	-	yes	yes	no
28	Highlands	Bins, MRF	185	63	5	na	sort-crush	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	sort-crush	yes	yes	no
37	Leon	Bins,MRF	955	350	355	na	sort	yes	yes	no
38	Levy	Bins,MRF				total 205	sort	no	no	no
39	Liberty	*No response*								
40	Madison	*No response*								
41	Manatee	Bins,Curb	126	10	11	na	none	no	no	no
42	Marion	Curb				total 303	none	info	poss	no
43	Martin	*No response*								
44	Monroe	Curb	389	365	258	na	sort-crush	yes	yes	no
45	Nassau	*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 used?	Production tons/year Clear	Brown	Green	Mixed	Addn'l Treatment	Would you sell glass at $3/cu.yds.?	Is there storage available?	Stockpile currently
46	Okaloosa	Curb				total 1,400	no	poss	poss	yes/ 40 tons
47	Okeechobee	Bins				total 10	srt	yes	yes	yes/ 0.5 ton
48	Orange	Bins, MRF	2,500	650	775	1,700	sort	yes	no	no
49	Osceola	Bins, MRF				total 178	crush,wash	no	-	-
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	yes	no
52	Pinellas	*No response*								
53	Polk	Bins				total 1,900	none	poss	no	no
54	Putnam	Binx,Curb				total 375	sort	yes	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	no
59	Seminole	MRF,Bins				total 16,300	sort crush	poss	yes	no
60	Sumter	*No response*								
61	Suwanne	Bins	20	30	na	na	sort	yes	yes	no
62	Taylor	Bins	25	20	5	5	sort-cursh	poss	yes	no
63	Union	*No response*								
64	Volusia	Bins,Curb				total 2,965	sort-crush	no	yes	no
65	Wakulla	Bins				total 4	sort	yes	ues	yes/ 5 tons
66	Walton	-	48,000	48,000	48,000	na	sort-crush	no	no	yes/ 40,000 CY
67	Washington	Bins				total 24	sort-crush	poss	yes	yes

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

MRF= Material Recovery Facility
Curb=Curbside 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)	BOD5 (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)	BOD5 (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)	BOD5 (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 BOD5 and TP

Column length (ft)	WASTE GLASS SOURCE			
	Southeast Recycling		WPBMRF	
	BOD_5	TP	BOD_5	TP
2	$y = 6.00\ E6\ X^{-1.62}$ $R^2 = 0.9407$	$y = 2.44\ E2\ X^{-0.67}$ $R^2 = 0.8933$	$y = 6.00\ E7\ X^{-1.93}$ $R^2 = 0.9982$	$y = 2.99\ E2\ X^{-0.82}$ $R^2 = 0.9876$
4	$y = 3.00\ E7\ X^{-1.62}$ $R^2 = 0.9945$	$y = 9.67\ E2\ X^{-0.85}$ $R^2 = 0.9013$	$y = 1.63\ E5\ X^{-1.08}$ $R^2 = 0.9166$	$y = 1.84\ E2\ X^{-0.65}$ $R^2 = 0.9346$
6	$y = 2.00\ E7\ X^{-1.38}$ $R^2 = 0.9899$	$y = 6.53\ E2\ X^{-0.87}$ $R^2 = 0.9457$	$y = 4.00\ E6\ X^{-1.34}$ $R^2 = 0.9735$	$y = 1.90\ E2\ X^{-0.67}$ $R^2 = 0.8787$

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)
WPBMRF Mixed Cullet				
	2	5880	664.5	3.33
	4	14280	358.5	12.47
	6	22260	1852.5	12.06
BSMG				
	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_5 (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		
		Upper	Average	Lower	Upper	Average	Lower	Upper	Average	Lower
AASHTO Classification	A-1-a (0)	A-1-a (0)	A-1-a (0)	A-1-a (0)	A-1-a (0)	A-1-a (0)	A-1-a (0)	A-1-a (0)	A-1-a (0)	
USCS Classification	Table 4.2 (SW)	Table 4.2 (GP)	Table 4.2 (GP)	Table 4.2 (SP)	Table 4.2 (SP)	Table 4.2 (SP)	Table 4.2 (SP)	Table 4.2 (SP)	Table 4.2 (SP)	Table 4.2 (SW)
Maximum Unit Weight	111 pcf 1.74 kN/m3	99 pcf 1.55 kN/m3	102 psf 1.60 kN/m3	102 psf 1.60 kN/m3	102 psf 1.60 kN/m3	105 psf 1.64 kN/m3	102 psf 1.60 kN/m3	81 pcf 1.27 kN/m3	87 pcf 1.37 kN/m3	87 pcf 1.37 kN/m3
Minimum Unit Weight	83 pcf 1.30 kN/m3	85 pcf 1.33 kN/m3	85 pcf 1.33 kN/m3	83 pcf 1.30 kN/m3	83 pcf 1.30 kN/m3	85 pcf 1.33 kN/m3	85 pcf 1.33 kN/m3	53 pcf 0.83 kN/m3	60 pcf 0.94 kN/m3	75 pcf 1.18 kN/m3
Effective Grain Size (D 10)	0.5 mm	2.38 mm	3.00 mm	4.75 mm	2.38 mm	1.50 mm	1.30 mm	1.19 mm	0.075 mm	0.075 mm
Permeability	0.3 to 5 cm/s	4 to 8 cm/s	6 to 8 cm/s	5 to 7 cm/s	1 to 3 cm/s	4 to 10 cm/s	4 to 8 cm/s	0.003 to 0.02 cm/s	0.003 to 0.01 cm/s	0.7 to 2 cm/s
Specific Yield	18 to 33 %	38 to 47 %	38 to 43 %	38 to 47 %	28 to 38 %	34 to 41%	39 to 45 %	4 to 13 %	9 to 18 %	33 to 42 %
Triaxial Friction Angles	Figure 4.15		Figure 4.15			Figure 4.15			Figure 4.15	
Direct Shear Friction Angles	45 to 45		45 to 51			37 to 45			34 to 46	
CBR	0.4 to 3.3 %		0.9 to 2.7 %			0.8 to 2.8 %			0.5 to 1.7 %	
LBR	0.5 to 4.0 %		1.1 to 3.4 %			1.0 to 3.5 %			0.6 to 2.1 %	

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.

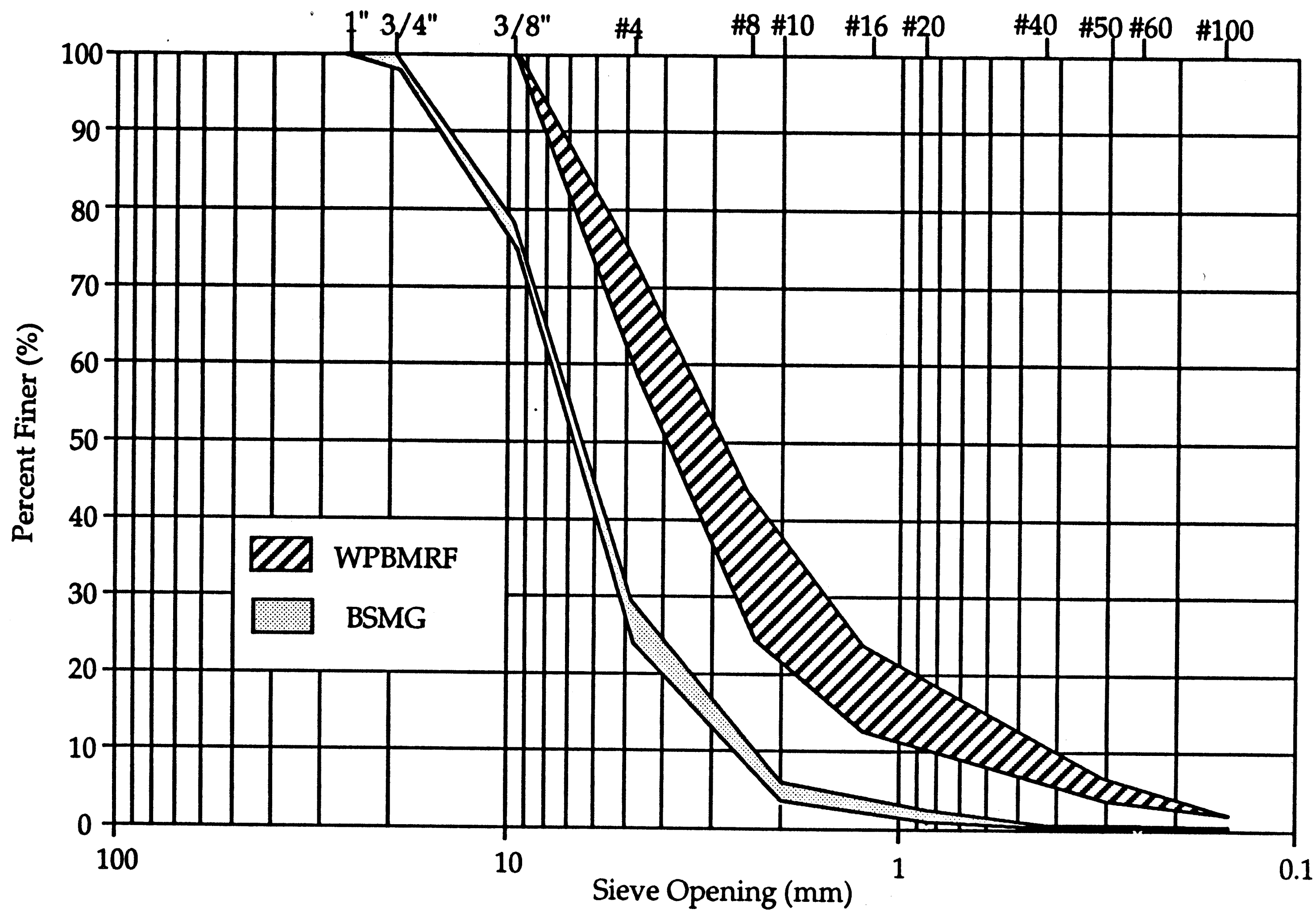


Figure 4.1 WG grain size distributions for WPBMRF and BSMG

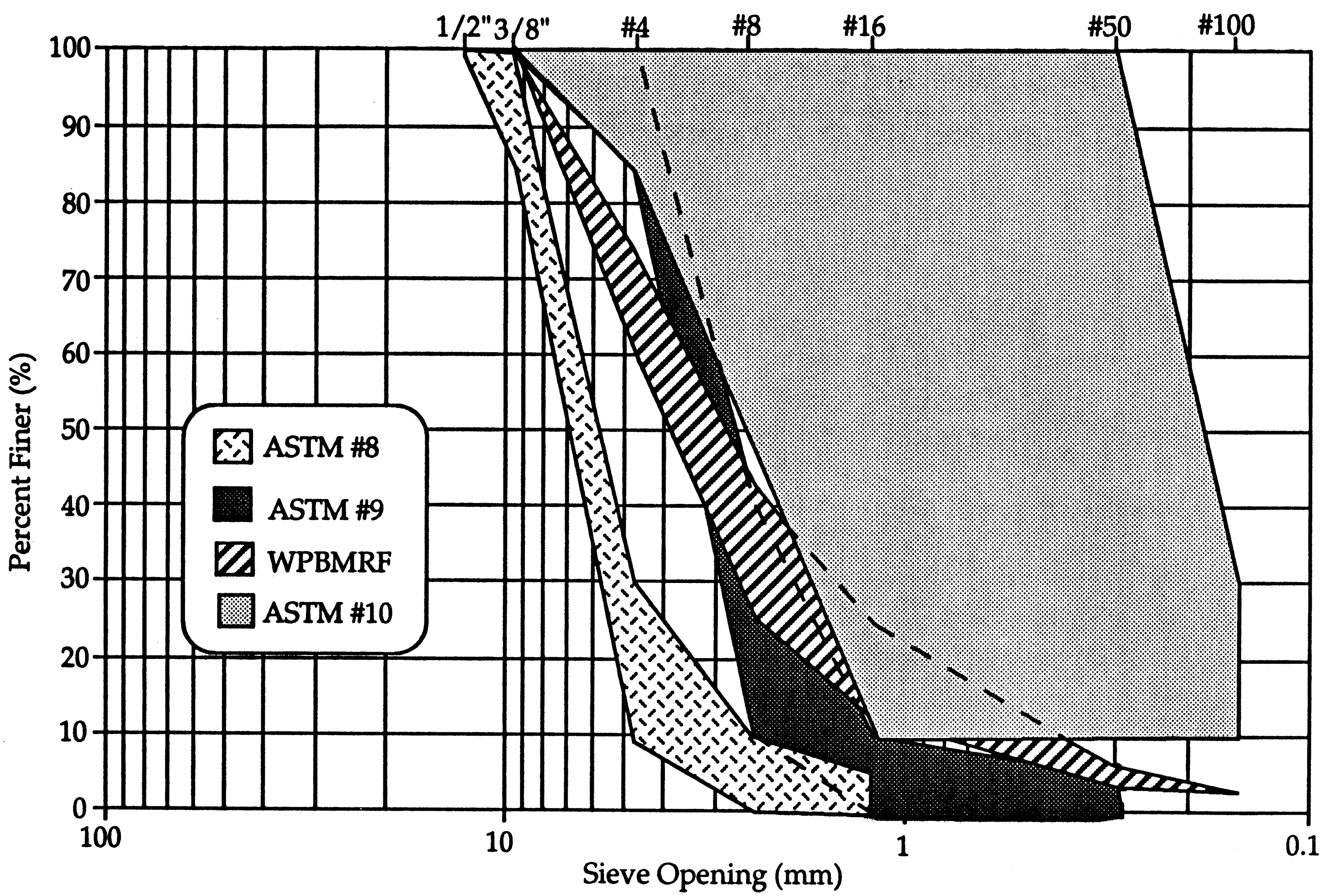


Figure 4.2 WG grain size distributions for ASTM D 448 coarse aggregate designations #8, #9, #10, and WPBMRF

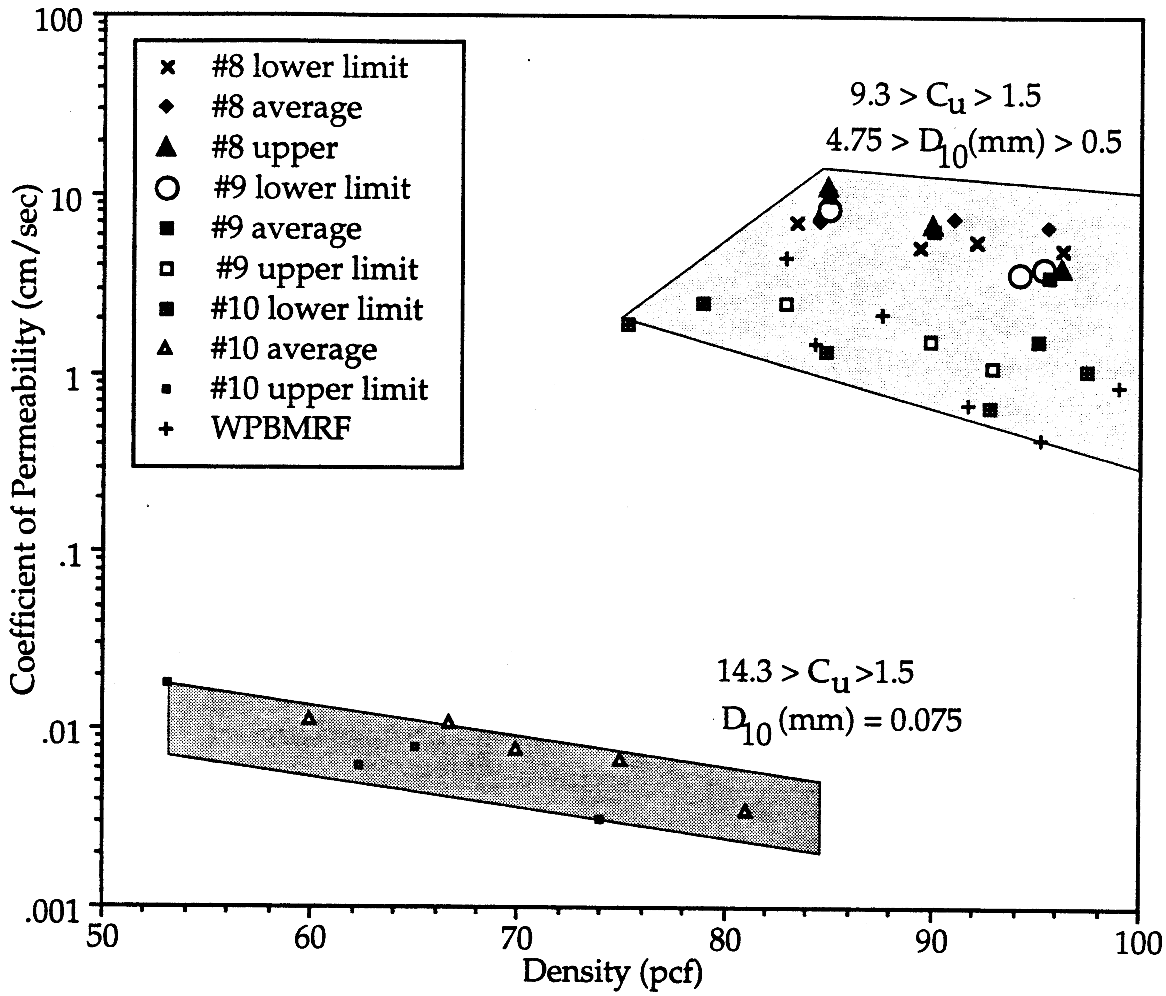


Figure 4.3 Relationship between density and permeability for WG meeting ASTM D 448 #8, #9, #10, and WPBMRF gradations

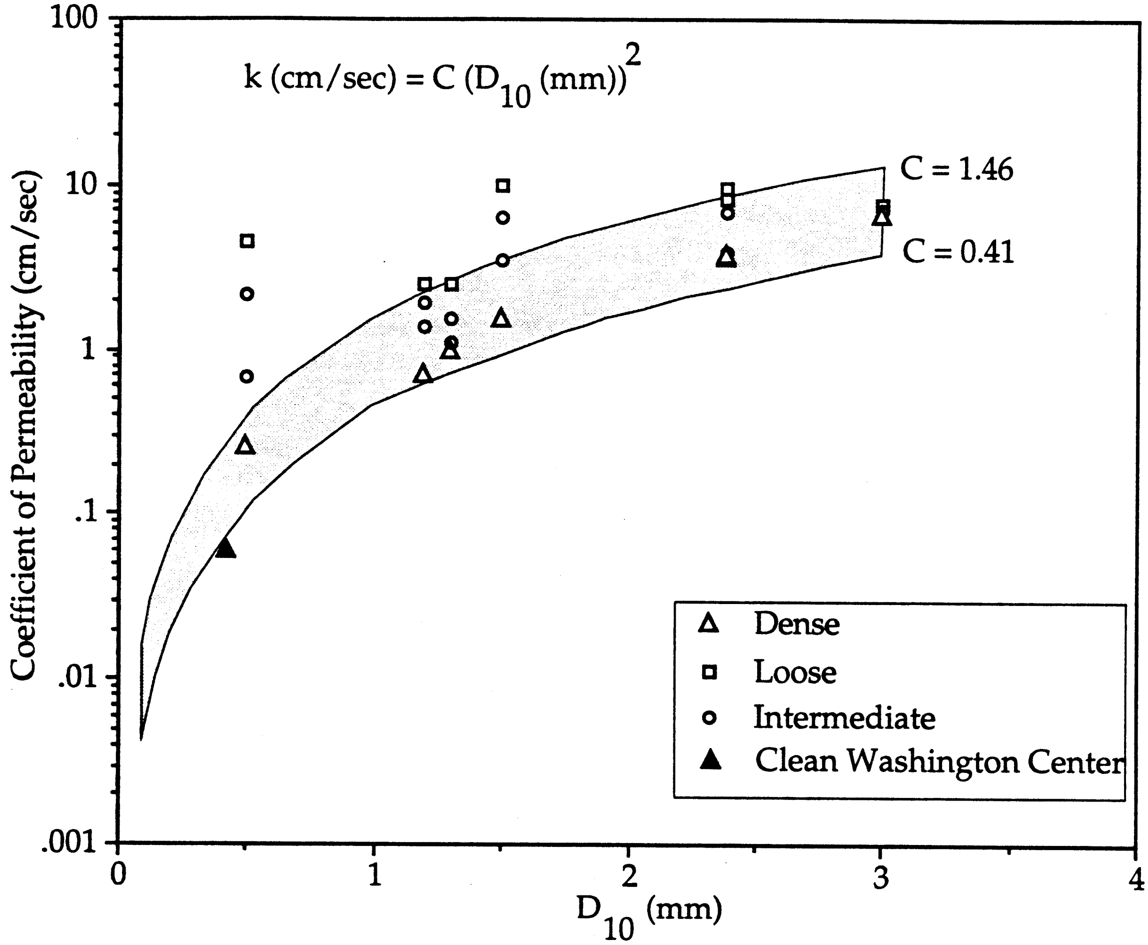


Figure 4.4 WG permeability and D_{10} (mm) relationship compared to Hazens' empirical formula with $C_u < 5$ and $0.1 < D_{10}$ (mm) < 3

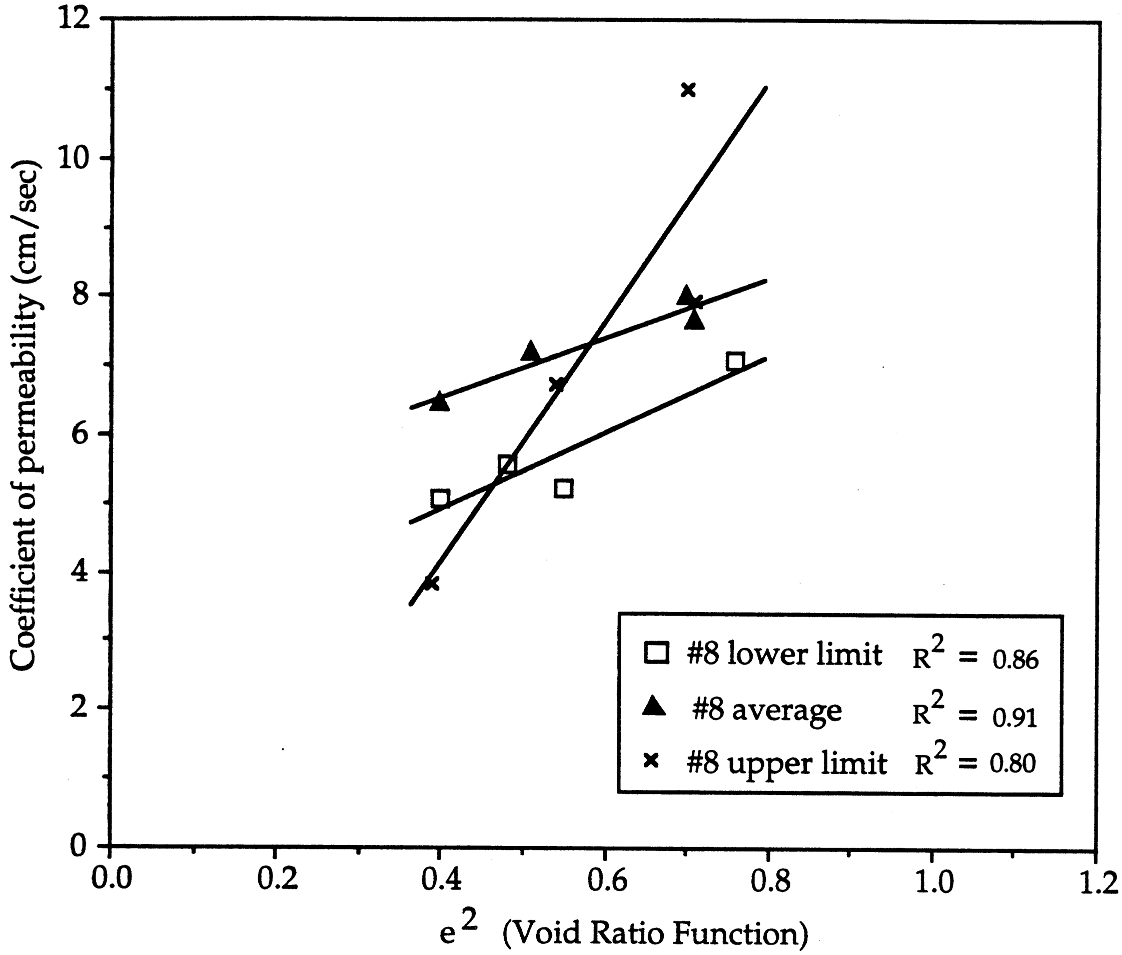


Figure 4.5 Relationship between e^2 and coefficient of permeability for WG meeting ASTM #8 gradation

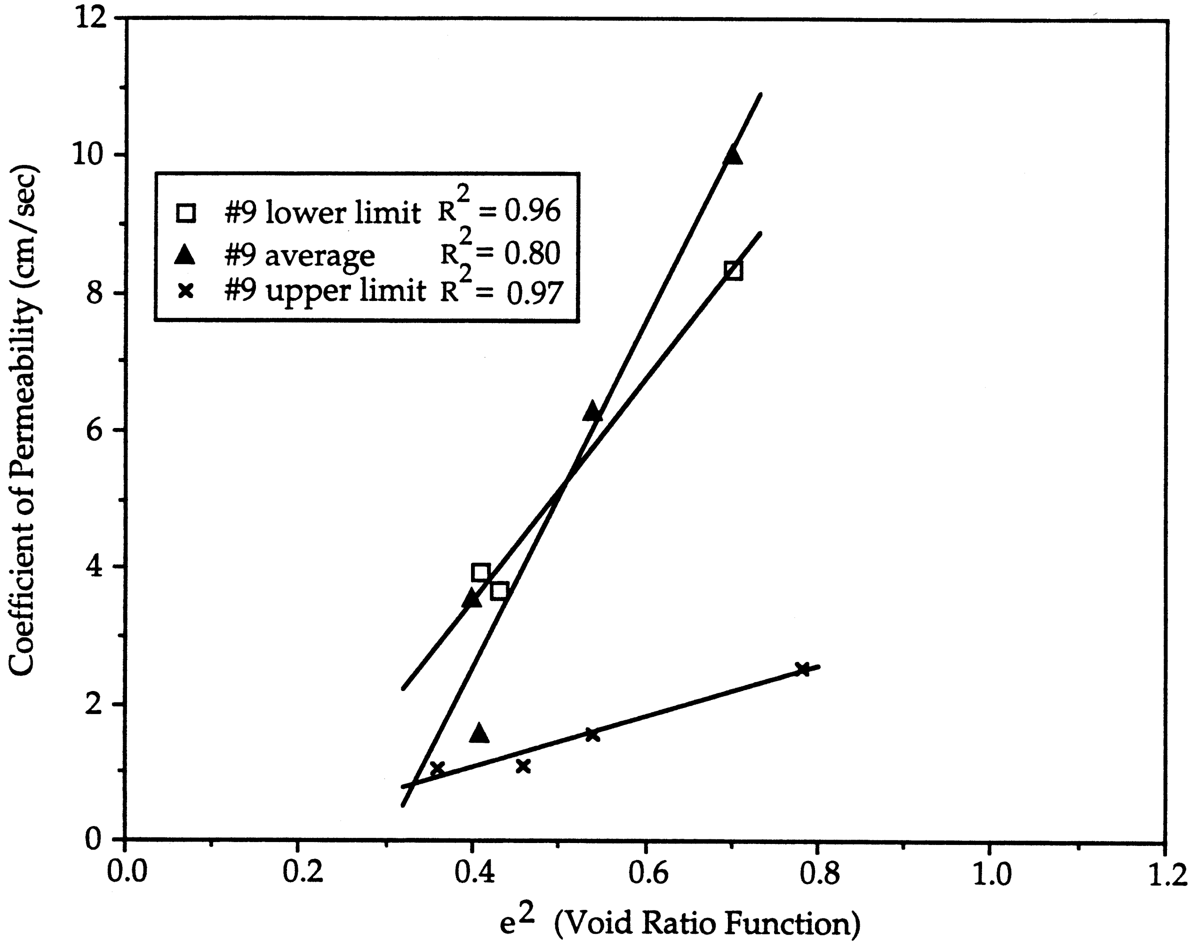


Figure 4.6 Relationship between e^2 and coefficient of permeability for WG meeting ASTM D 448 #9 gradation

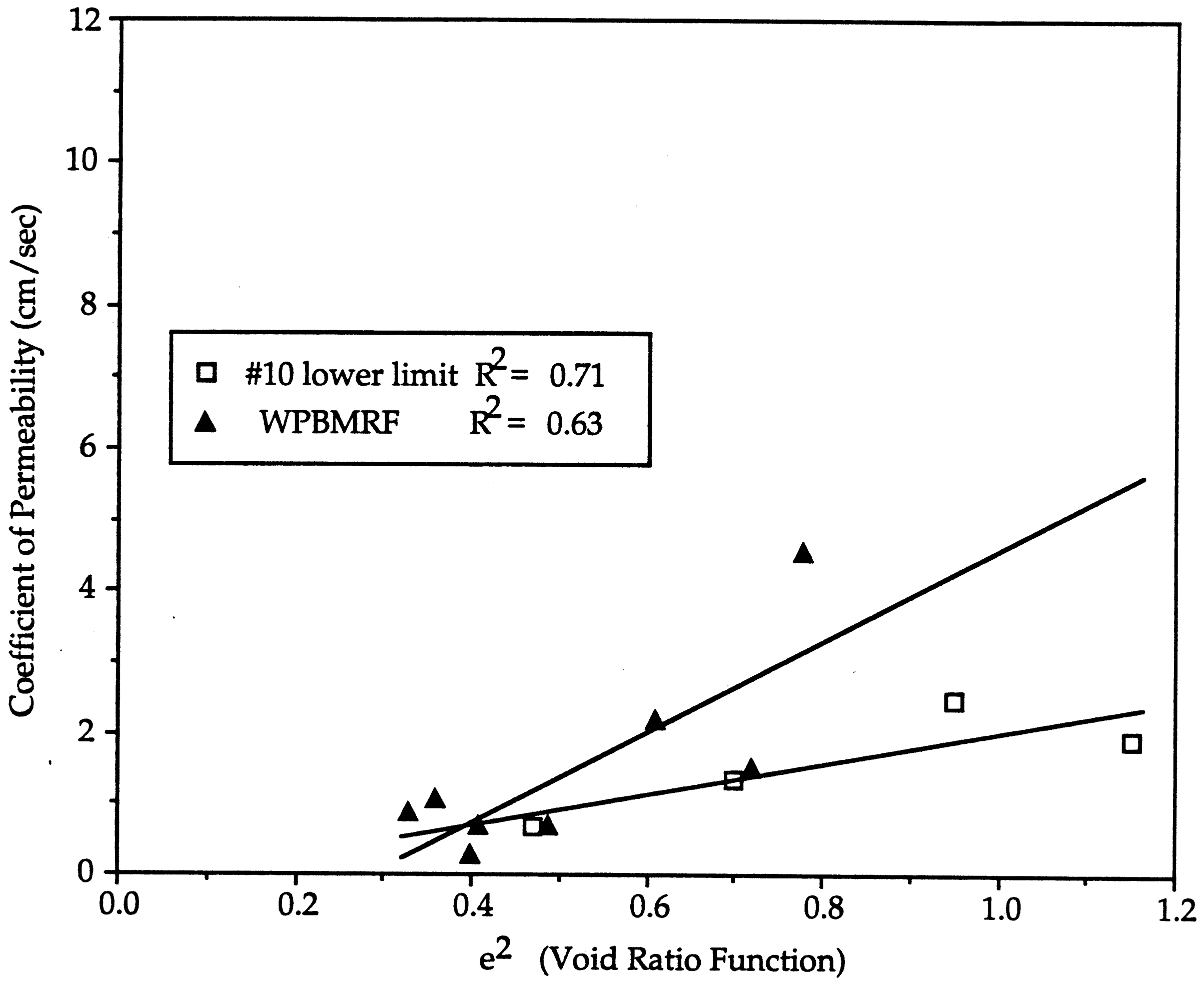


Figure 4.7 Relationship between e^2 and coefficient of permeability for WG meeting ASTM D 448 #10 lower and WPBMRF gradations

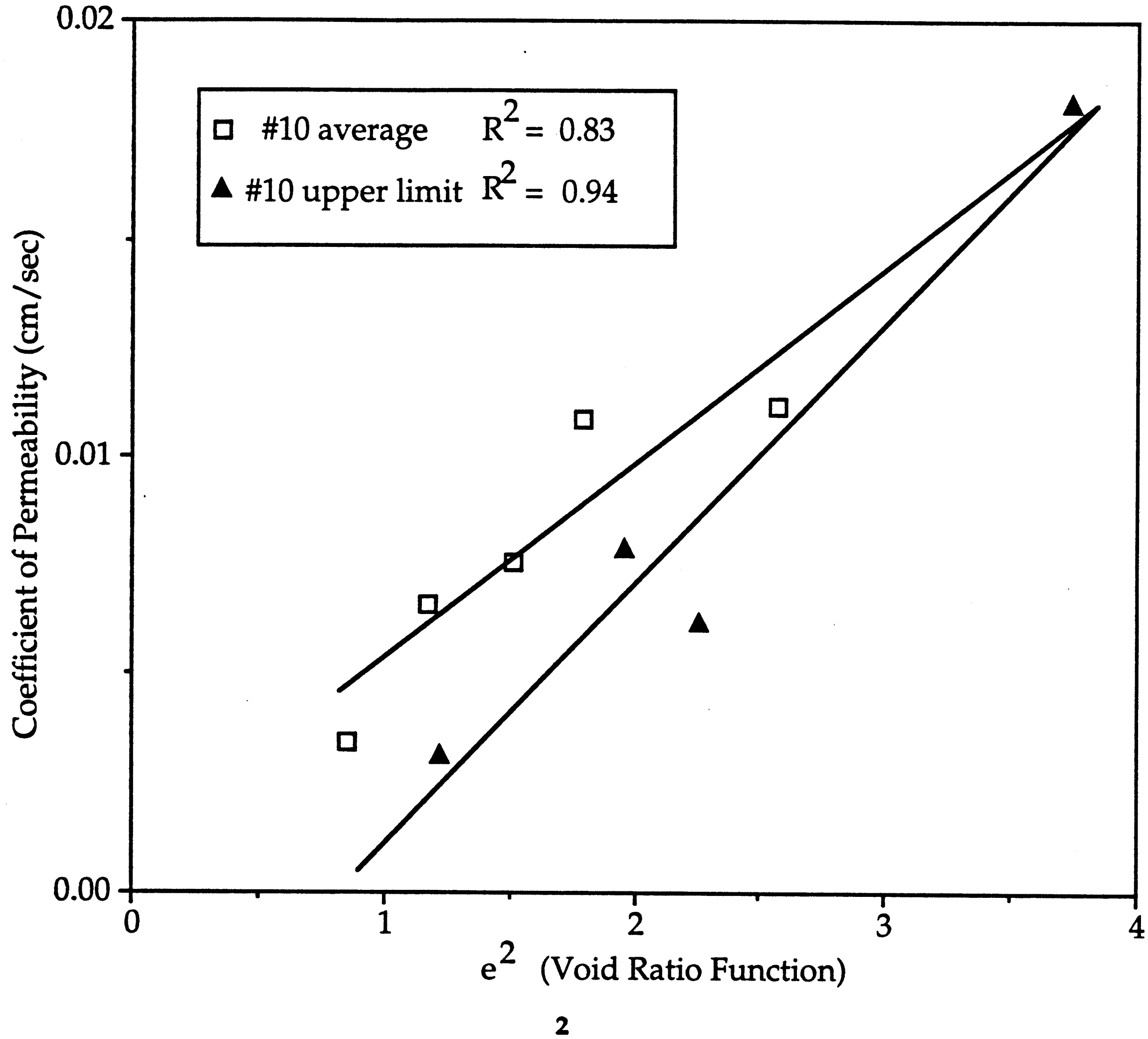


Figure 4.8 Relationship between e^2 and coefficient of permeability for WG meeting ASTM D 448 #10 average and upper limit gradations

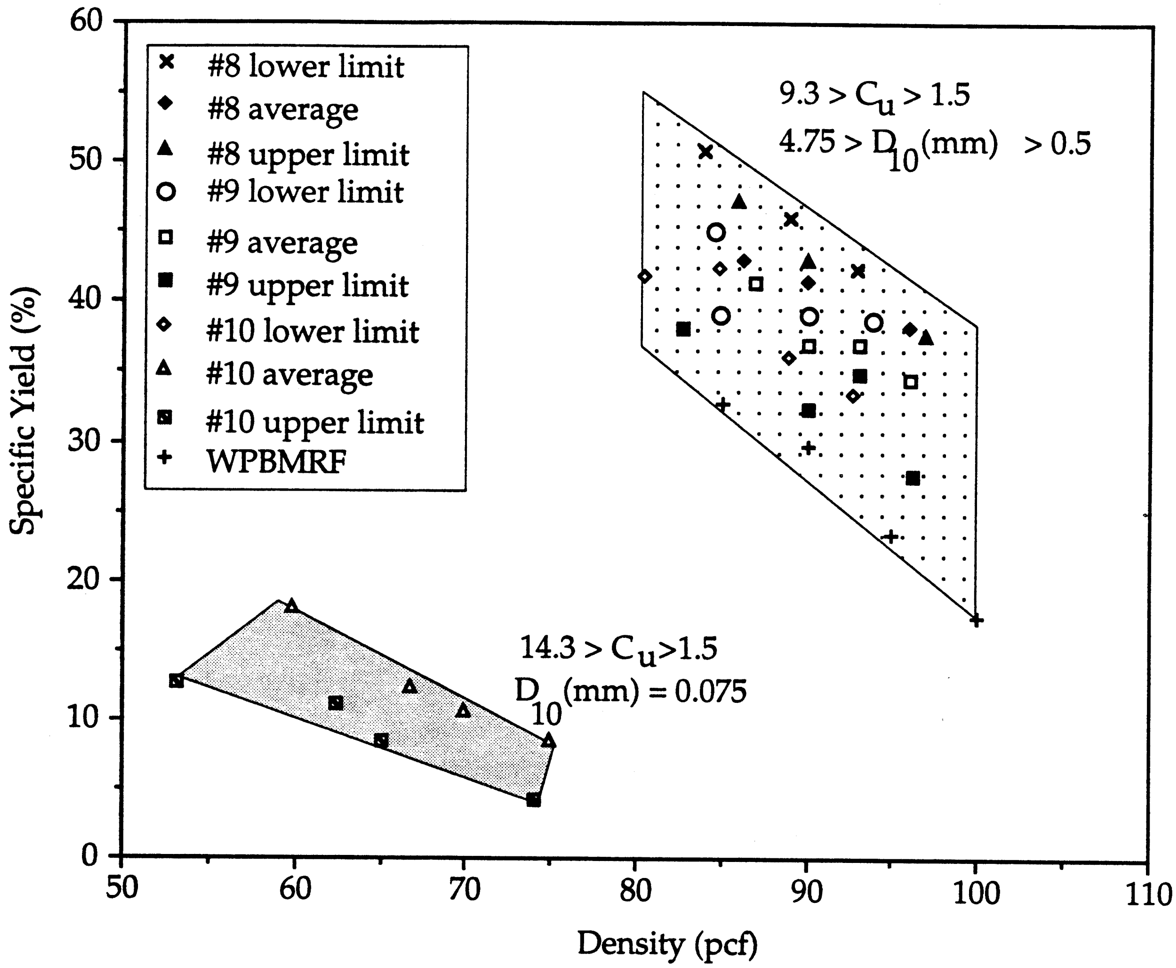


Figure 4.9 Relationship between density and specific yield for WG meeting ASTM D 448 #8, #9, #10, and WPBMRF gradations

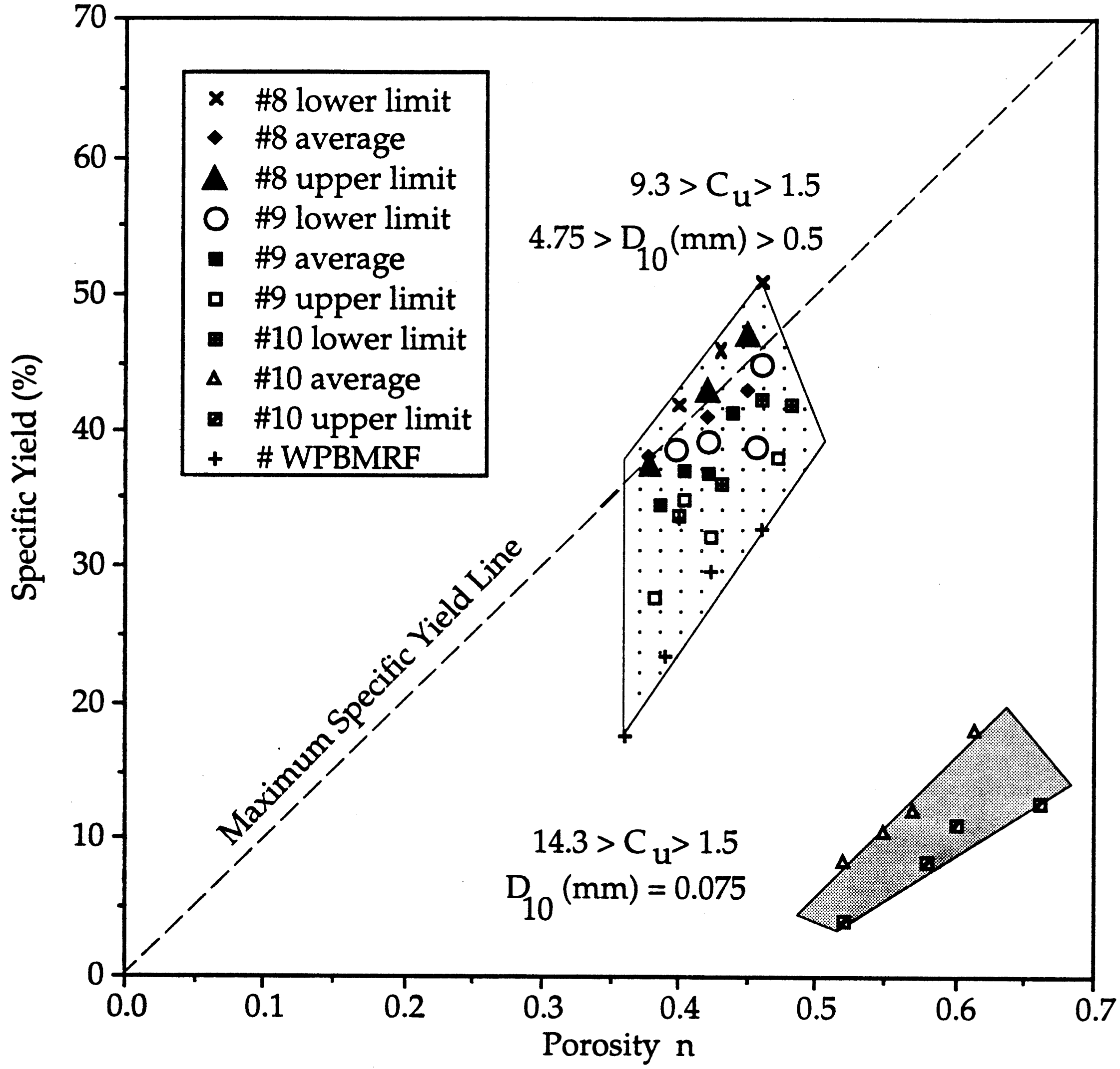


Figure 4.10 Relationship between porosity and specific yield for WG meeting ASTM D 448 #8, #9, #10, and WPBMRF gradations

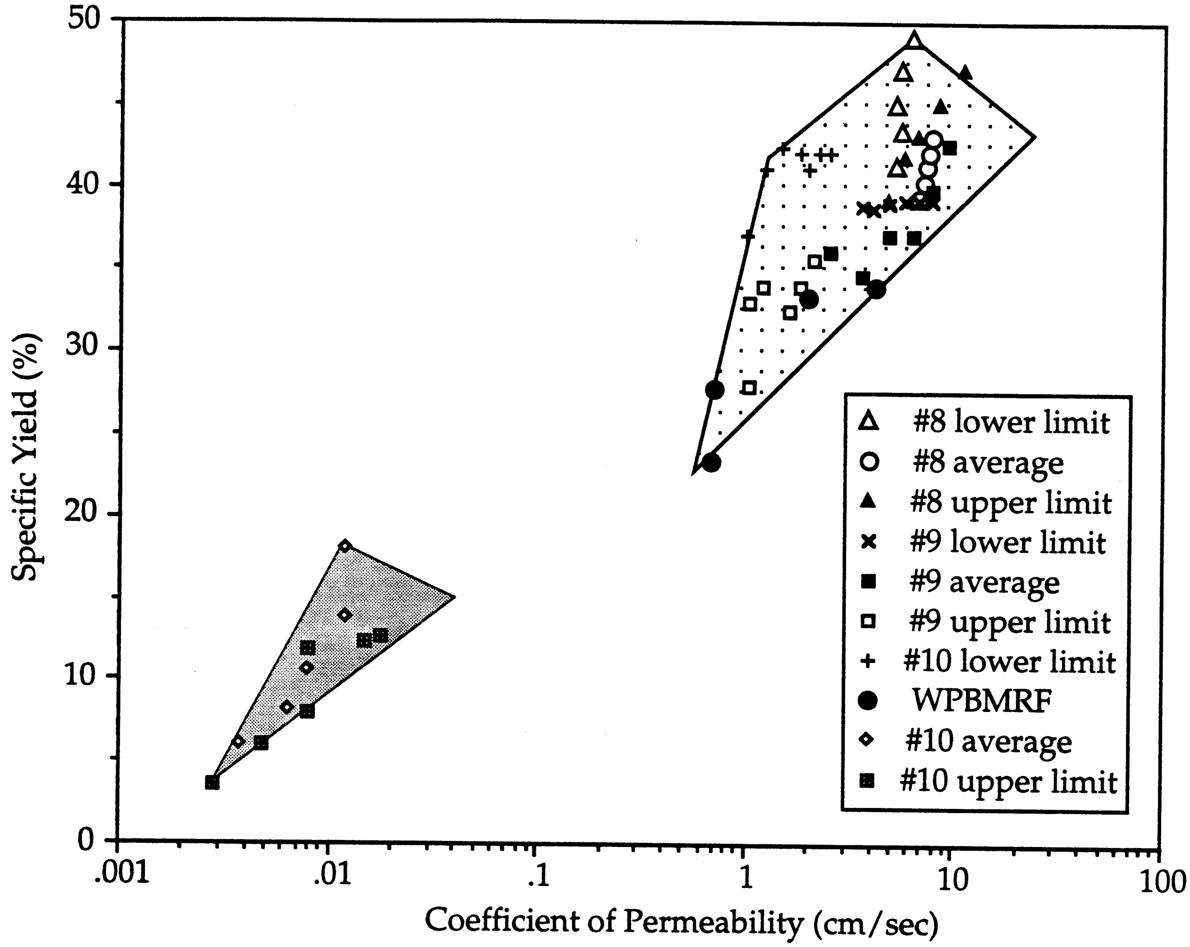


Figure 4.11 Relationship between permeability and specific yield for WG meeting ASTM D 448 #8, #9, #10, and WPBMRF gradations

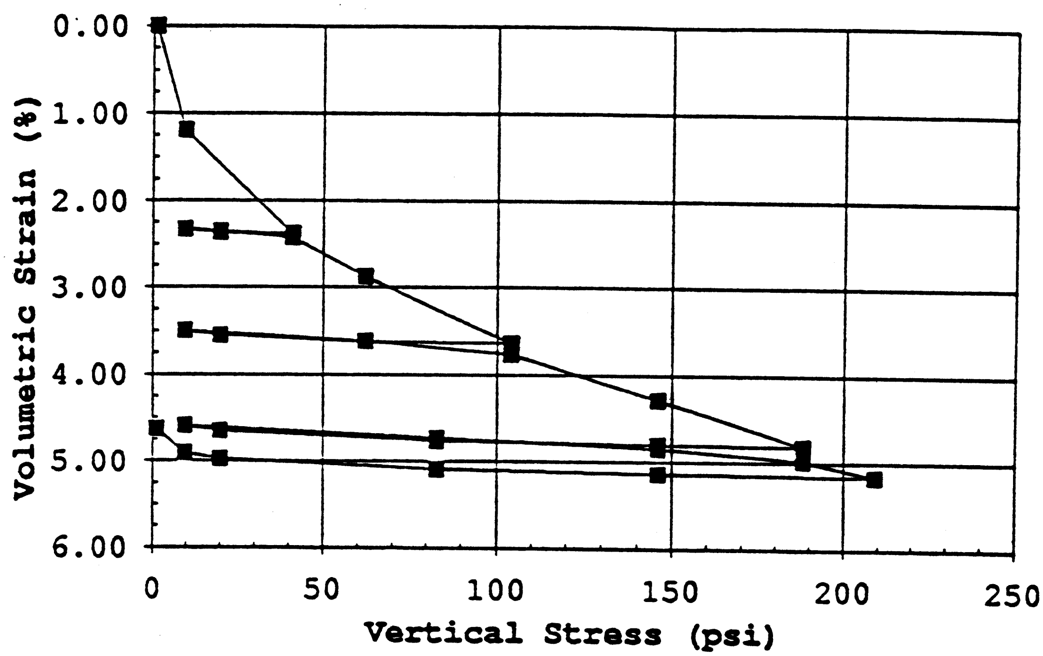


Figure 4.12 Typical confined compression results for mixed cullet meeting WPBMRF gradation (1 psi = 7 kPa)

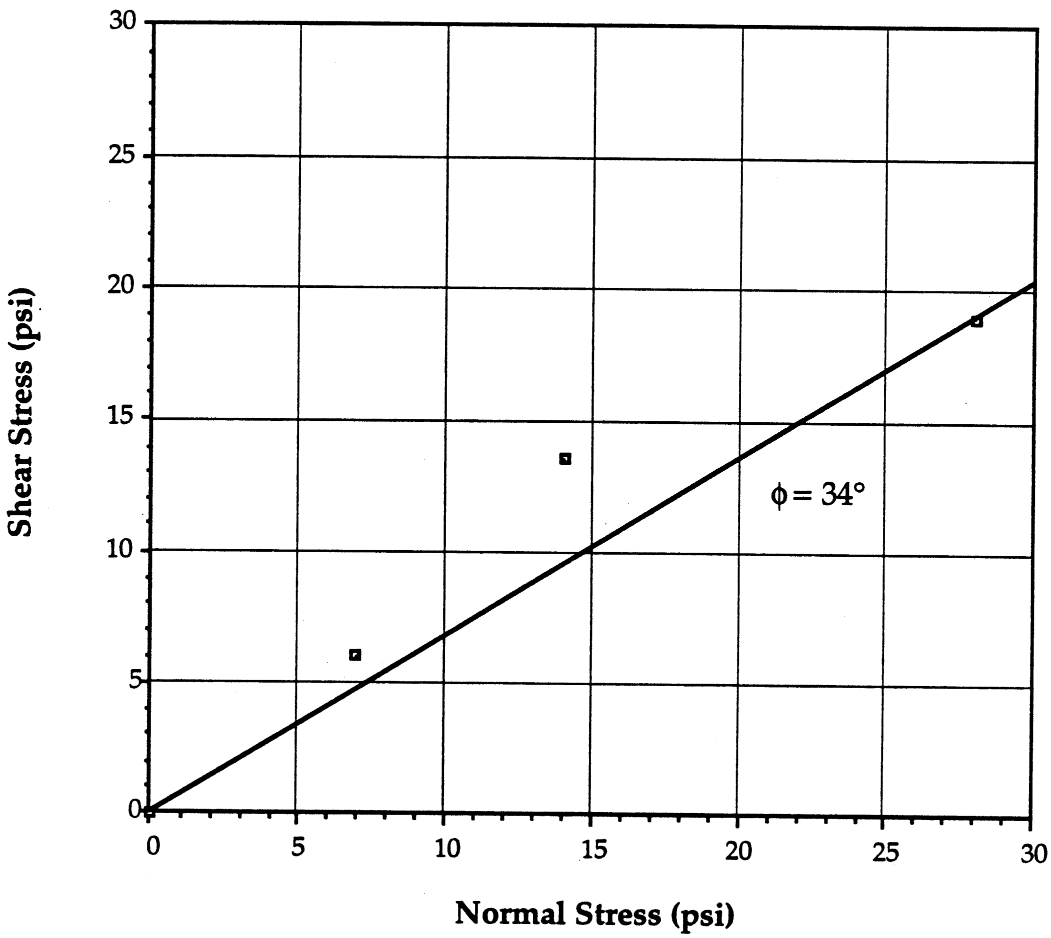


Figure 4.13 Nonlinear Mohr-Coulomb failure typical of WG direct shear tests

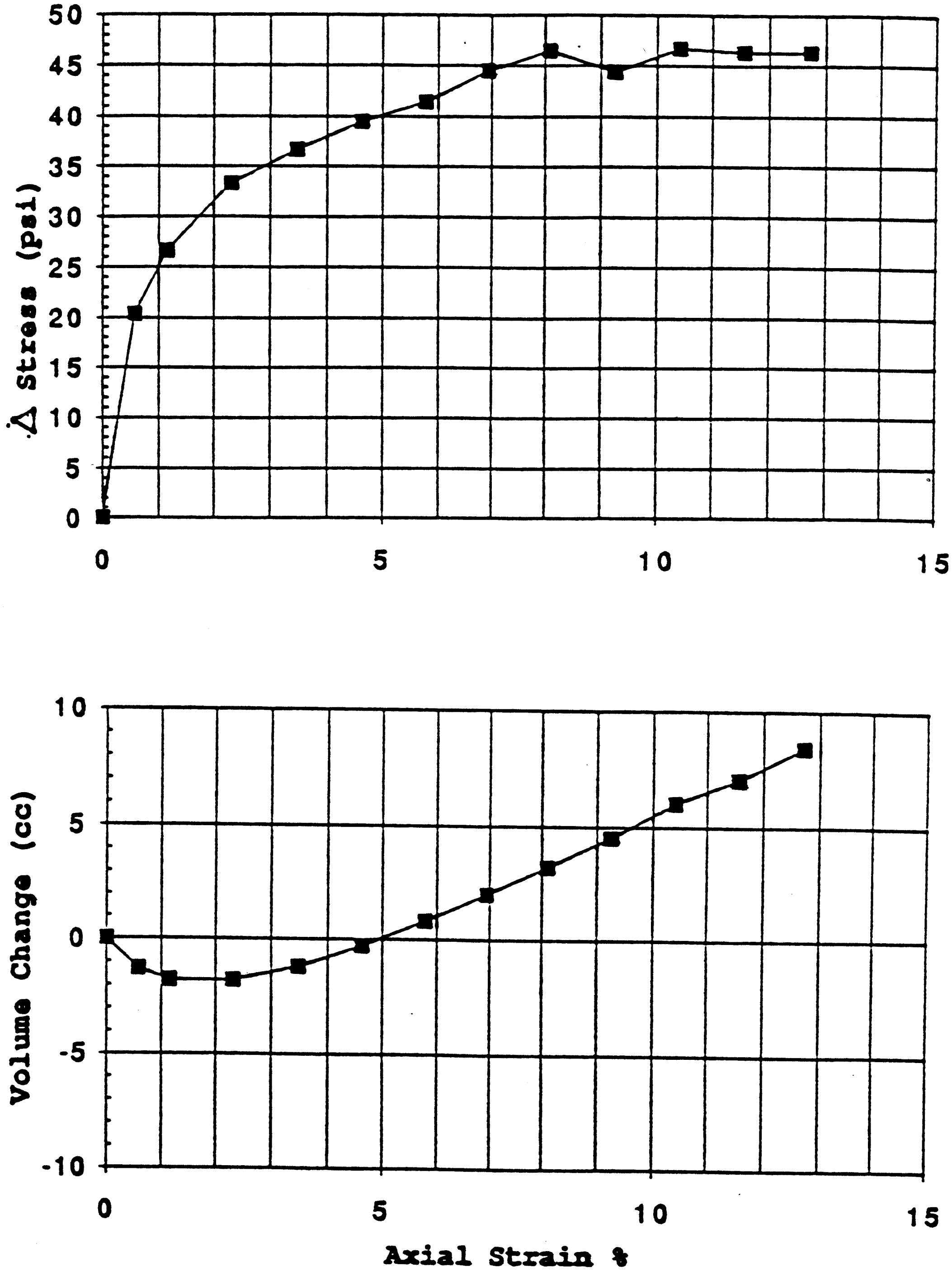


Figure 4.14 Typical vertical stress versus vertical strain and volumetric strain versus vertical strain plots for WG triaxial shear testing.

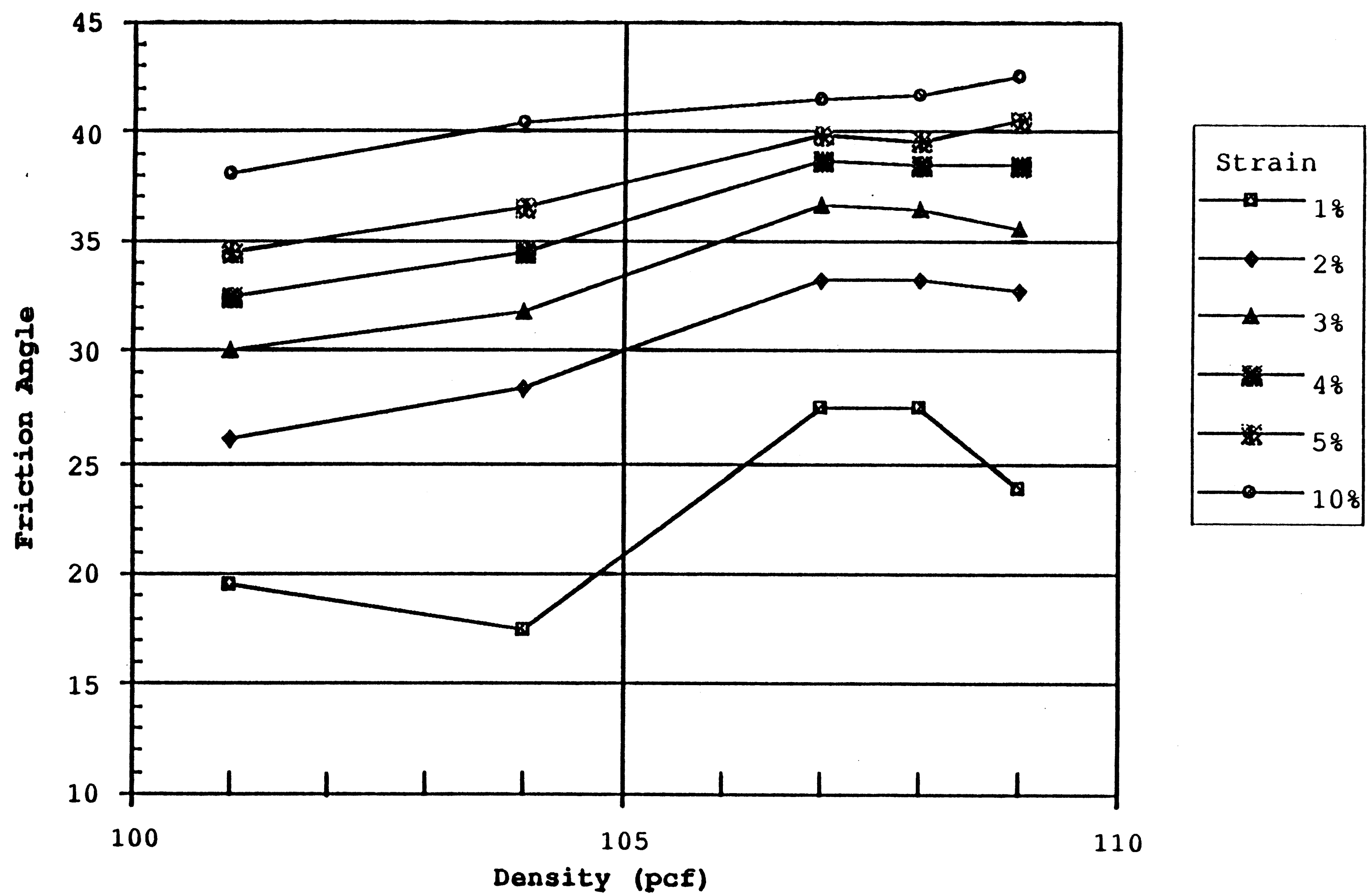


Figure 4.15a Variation of friction angle with density and strain level during CD triaxial testing on mixed cullet meeting WPBMRF gradation (1 psi = 7 kPa)

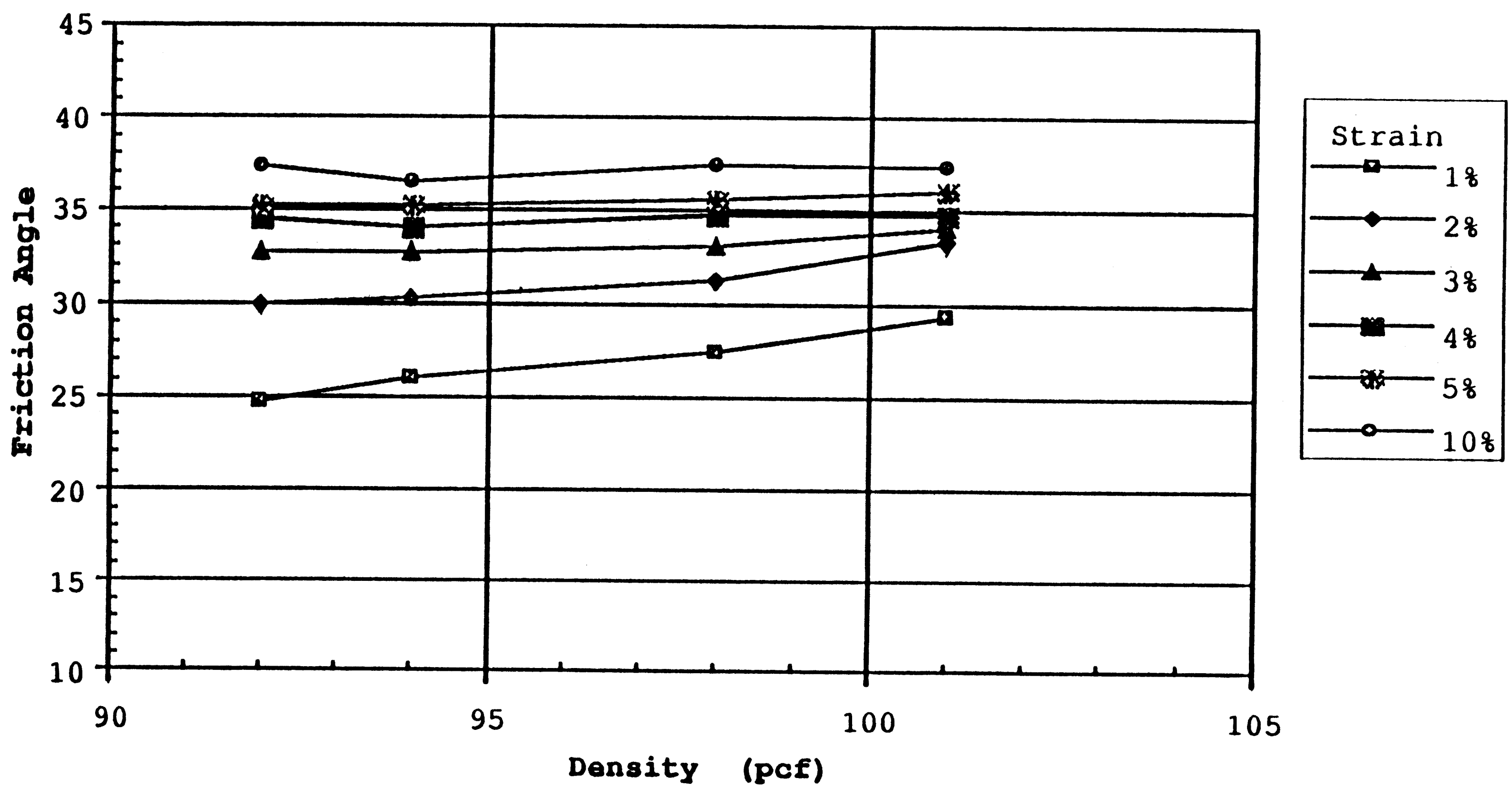


Figure 4.15b Variation of friction angle with density and strain level during CD triaxial testing on mixed cullet meeting ASTM # 8 average gradation (1 psi = 7 kPa)

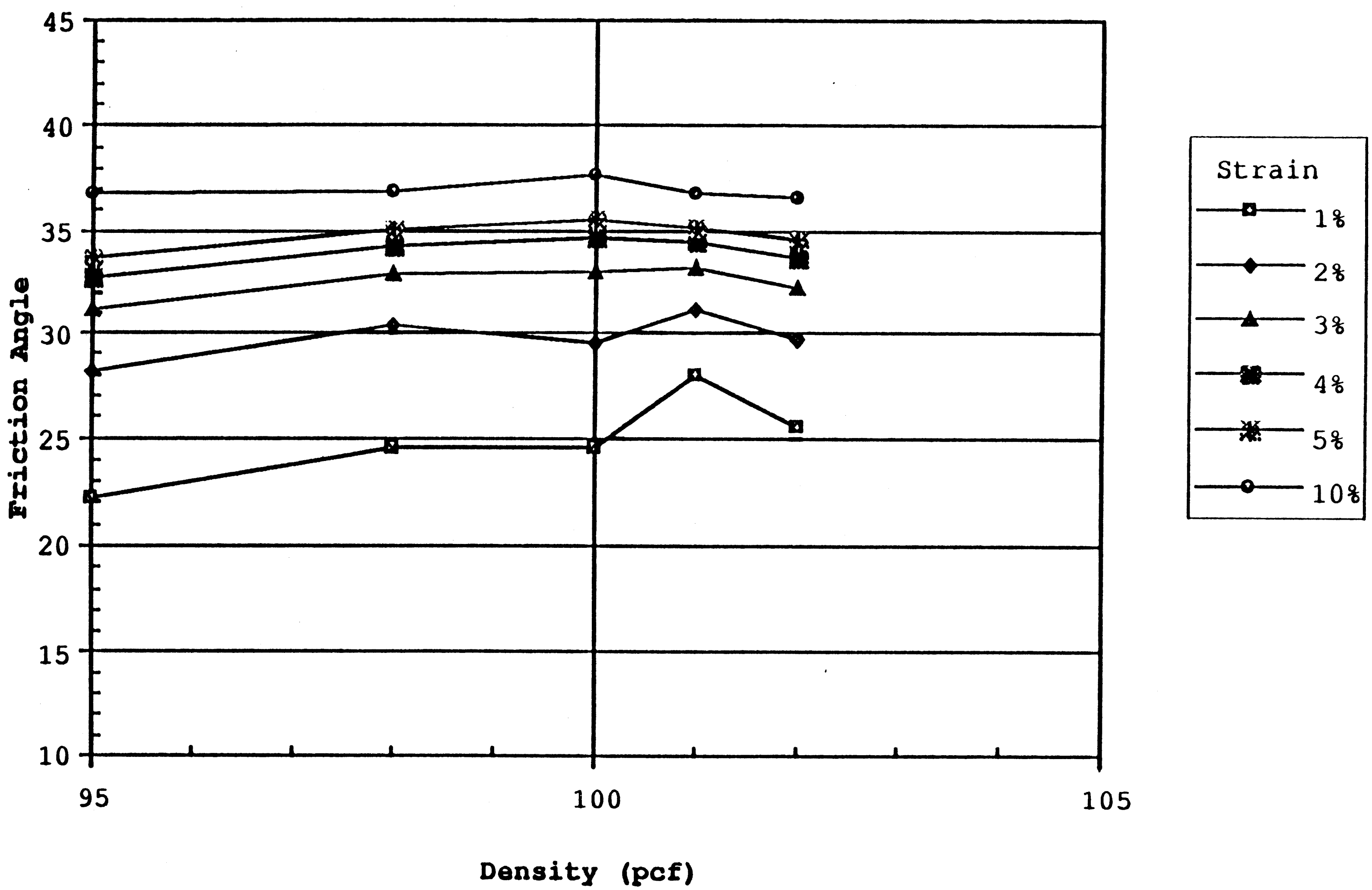


Figure 4.15c Variation of friction angle with density and strain level during CD triaxial testing on mixed cullet meeting ASTM # 9 average gradation (1 psi = 7 kPa)

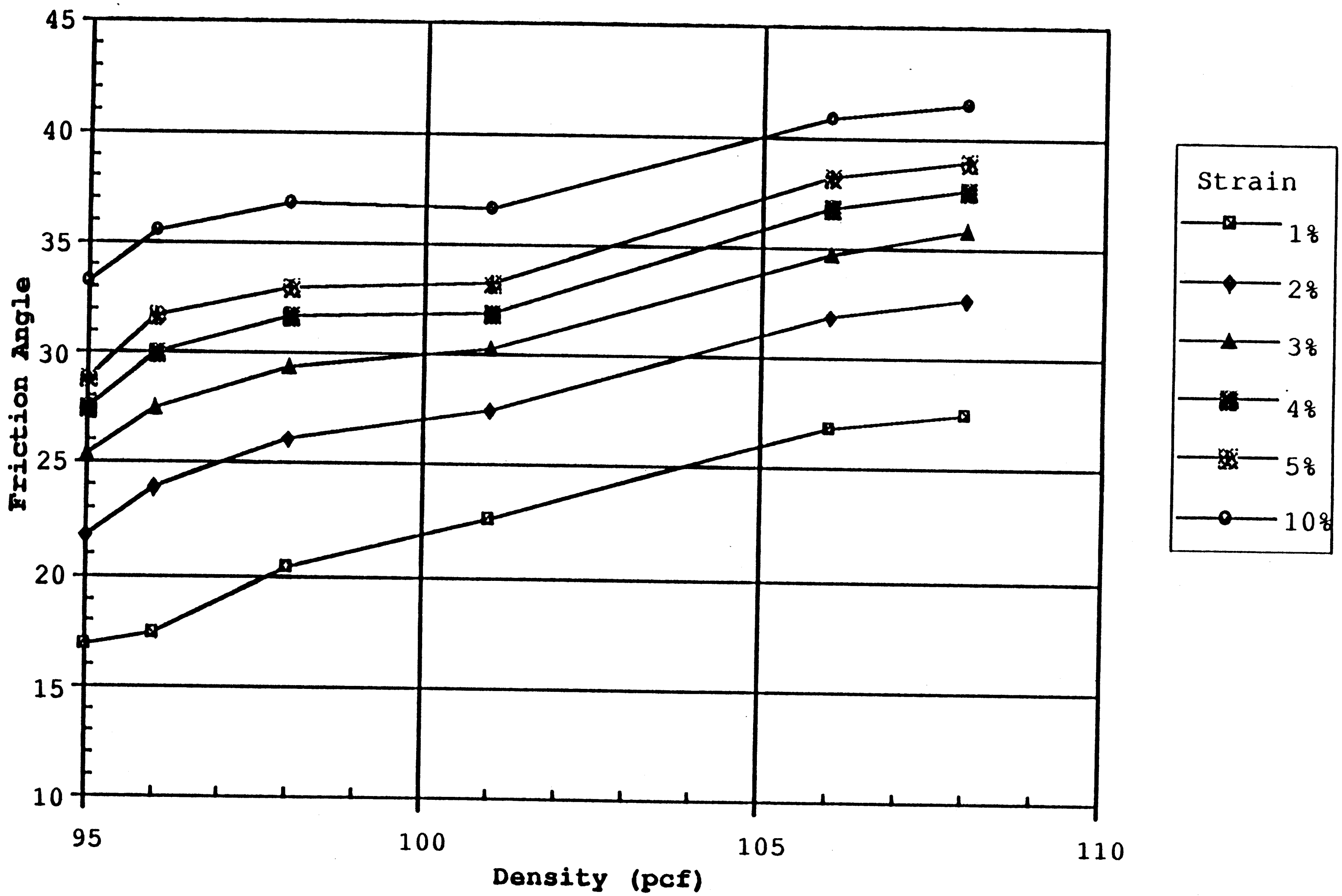


Figure 4.15d Variation of friction angle with density and strain level during CD triaxial testing on mixed cullet meeting ASTM # 10 average gradation (1 psi = 7 kPa)

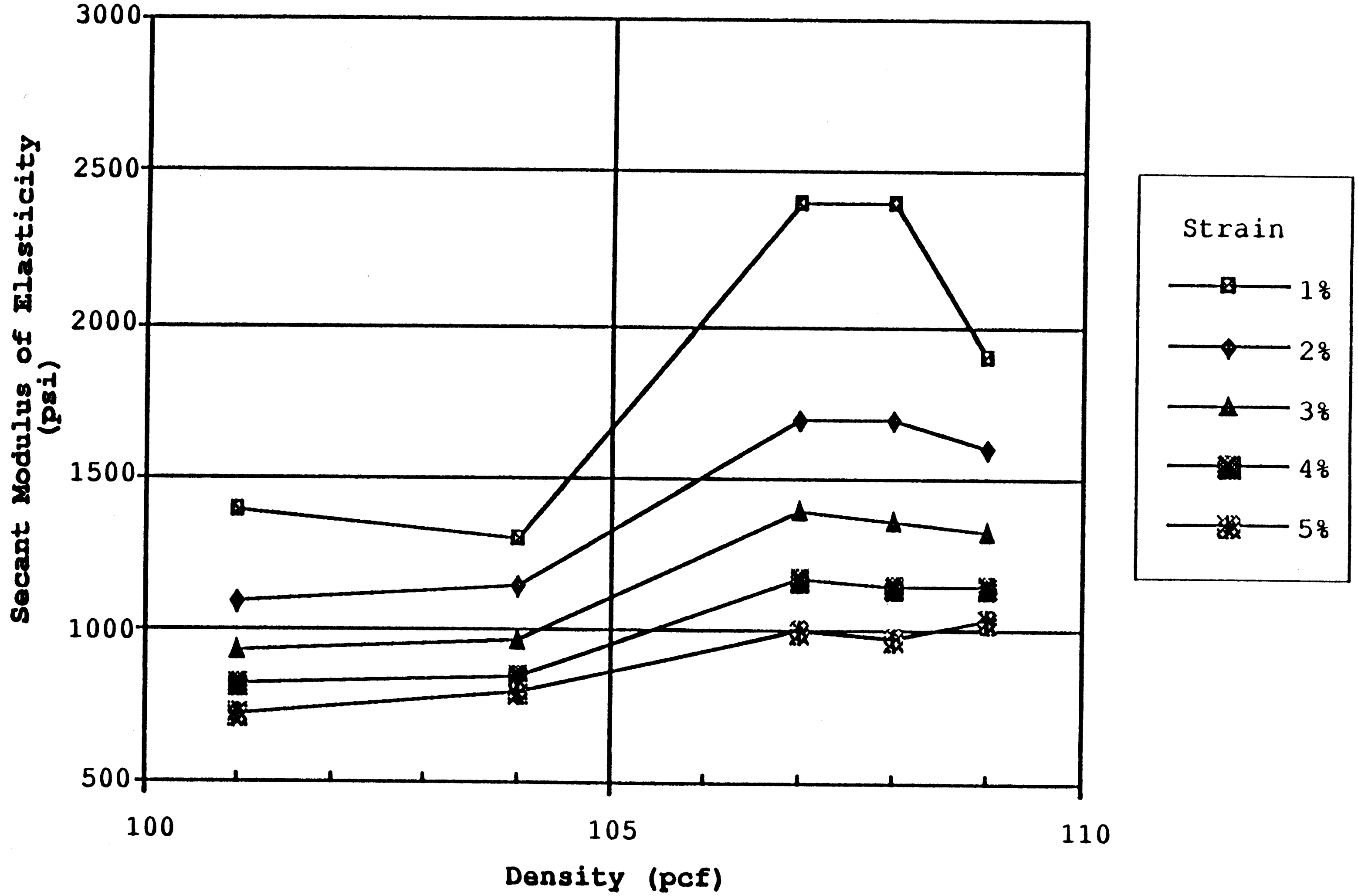


Figure 4.16a Variation of secant modulus with density and strain level during CD triaxial testing on mixed cullet meeting WPBMRF gradation (1 psi = 7 kPa)

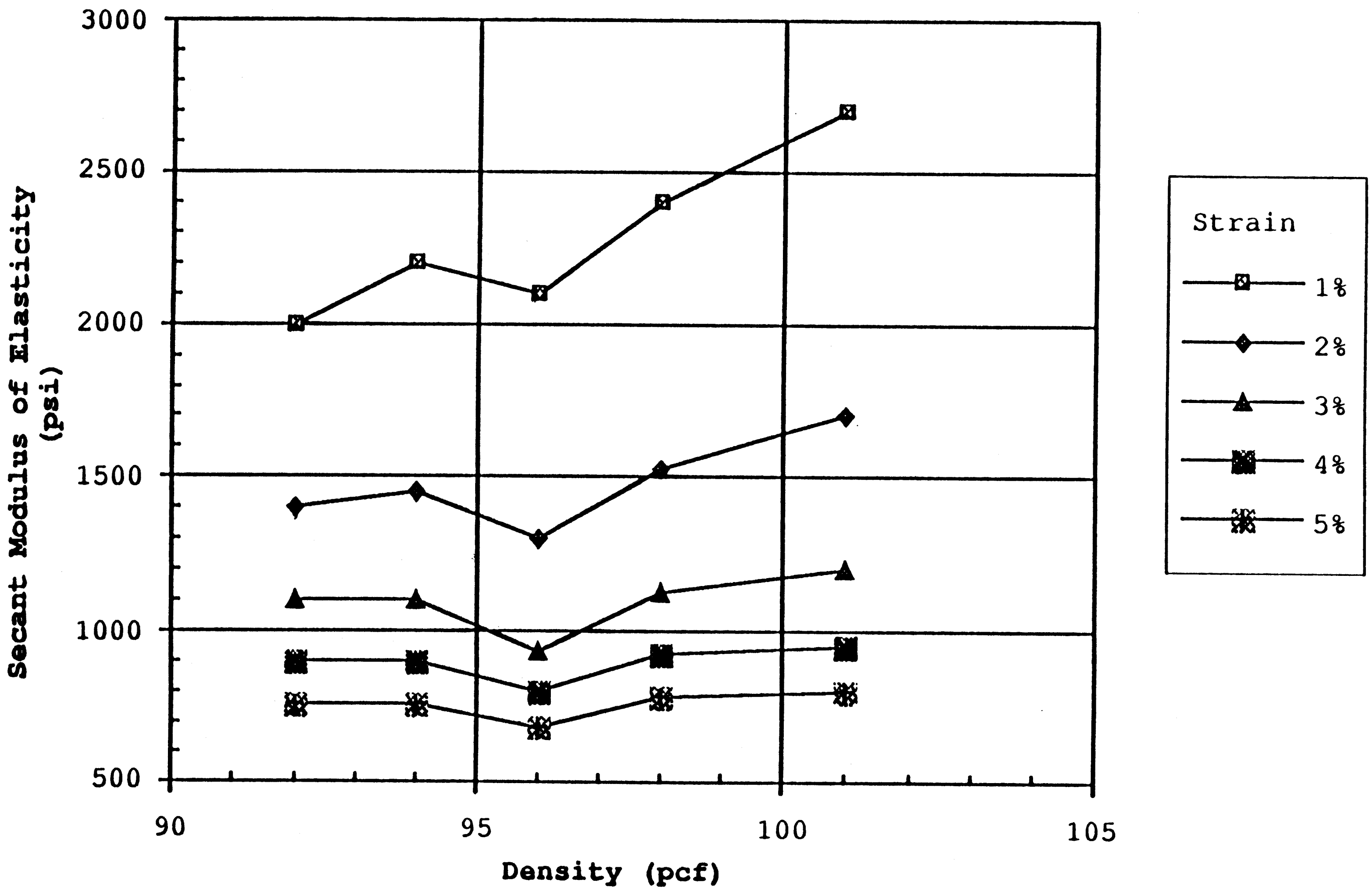


Figure 4.16b Variation of secant modulus with density and strain level during CD triaxial testing on mixed cullet meeting ASTM # 8 average gradation (1 psi = 7 kPa)

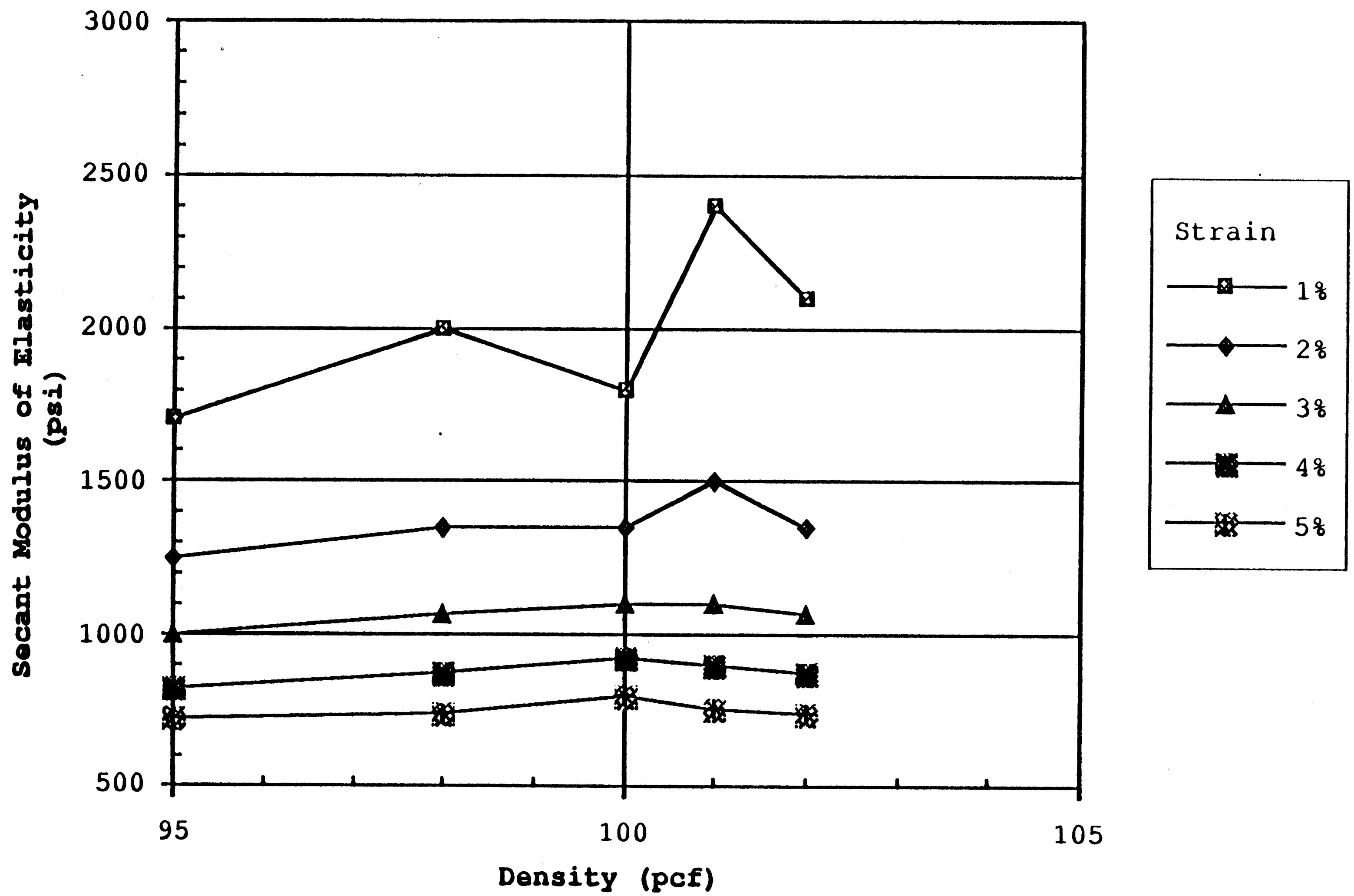


Figure 4.16c Variation of secant modulus with density and strain level during CD triaxial testing on mixed cullet meeting ASTM # 9 average gradation (1 psi = 7 kPa)

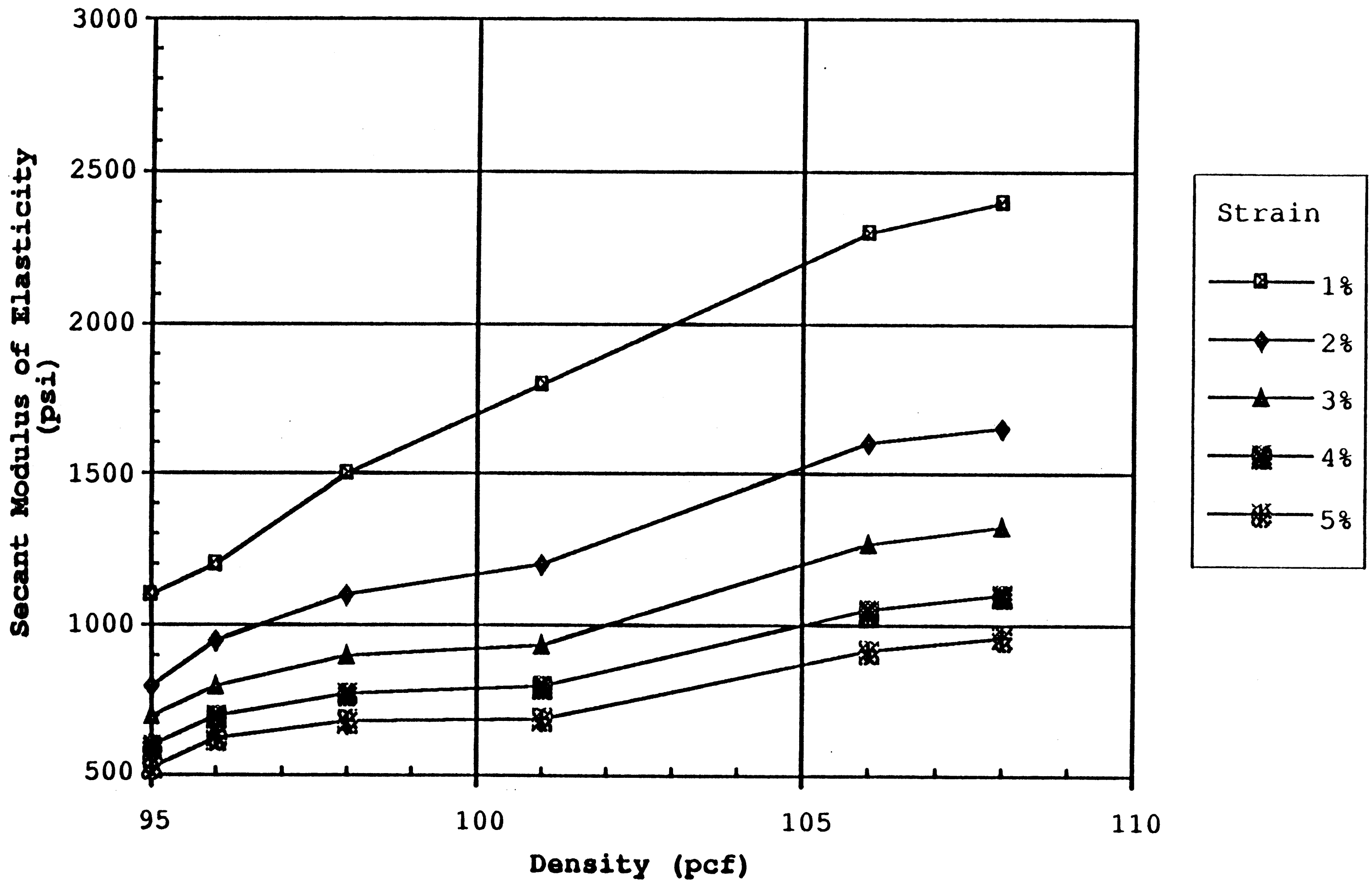


Figure 4.16d Variation of secant modulus with density and strain level during CD triaxial testing on mixed cullet meeting ASTM # 10 average gradation (1 psi = 7 kPa)

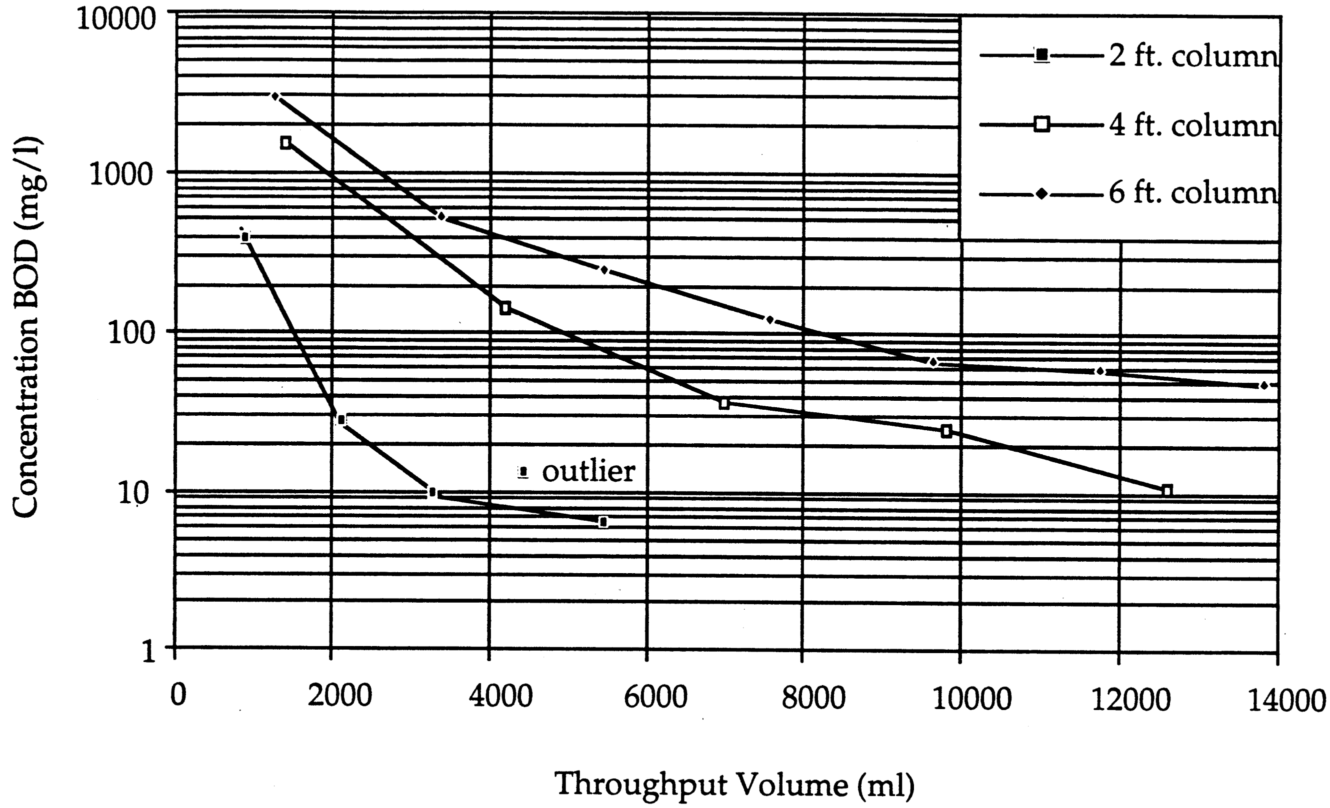


Figure 4.17 BSMG leaching column BOD results

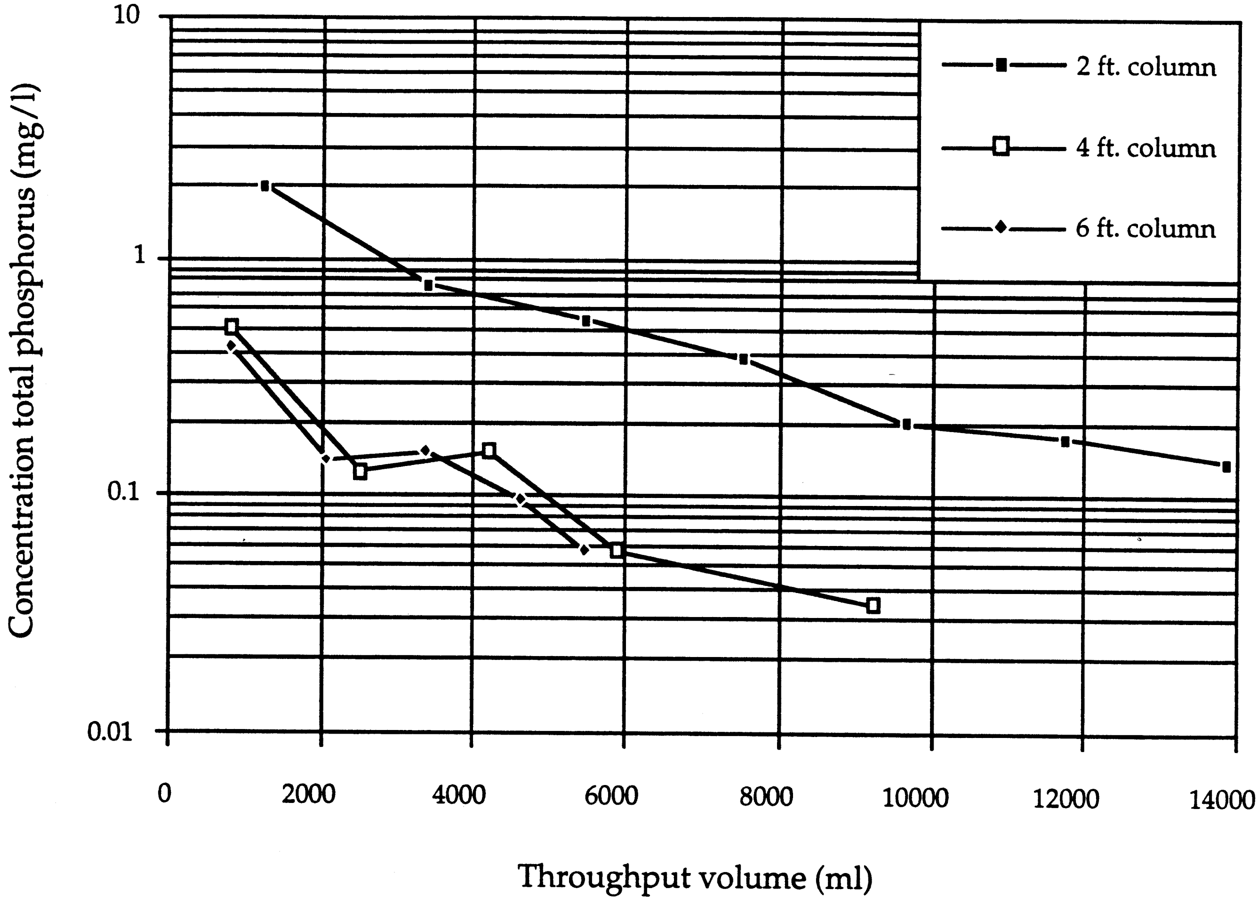


Figure 4.18 BSMG leaching column TP results

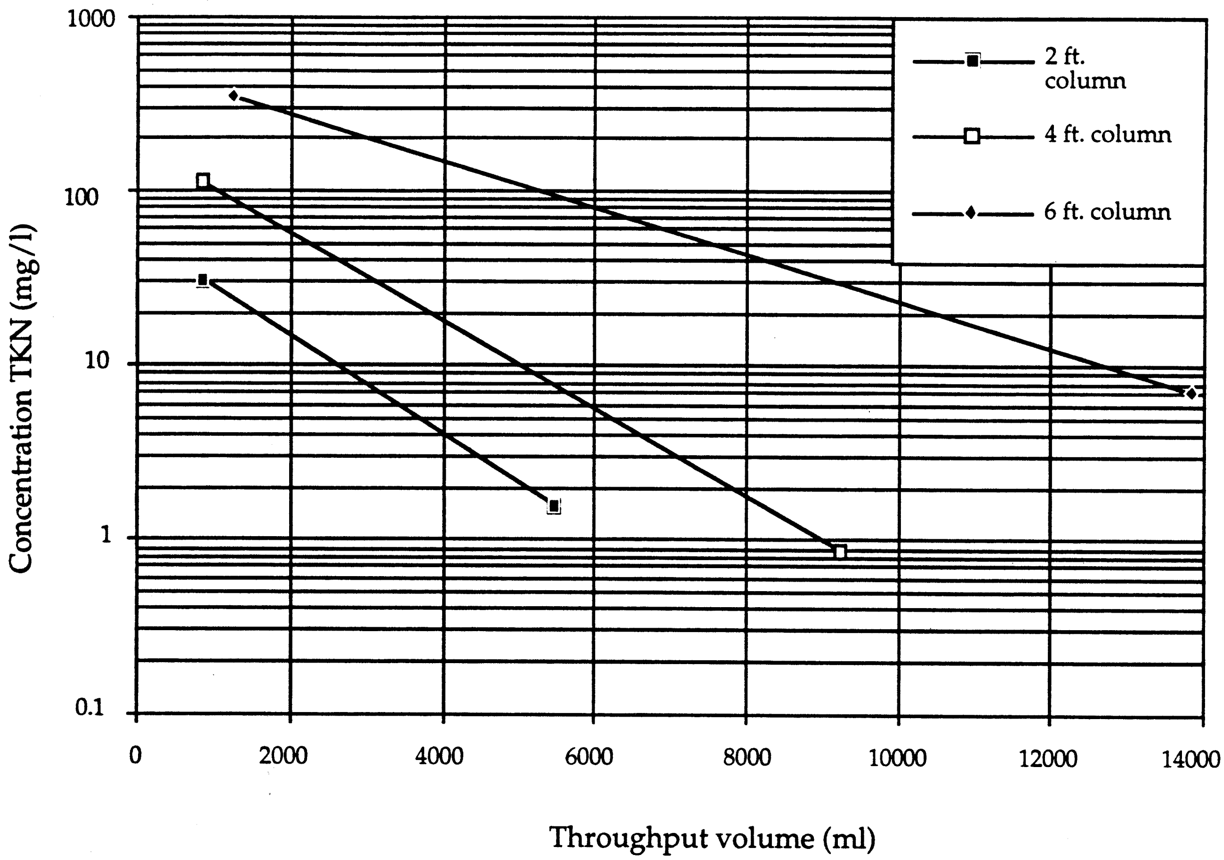


Figure 4.19 BSMG leaching column TKN results

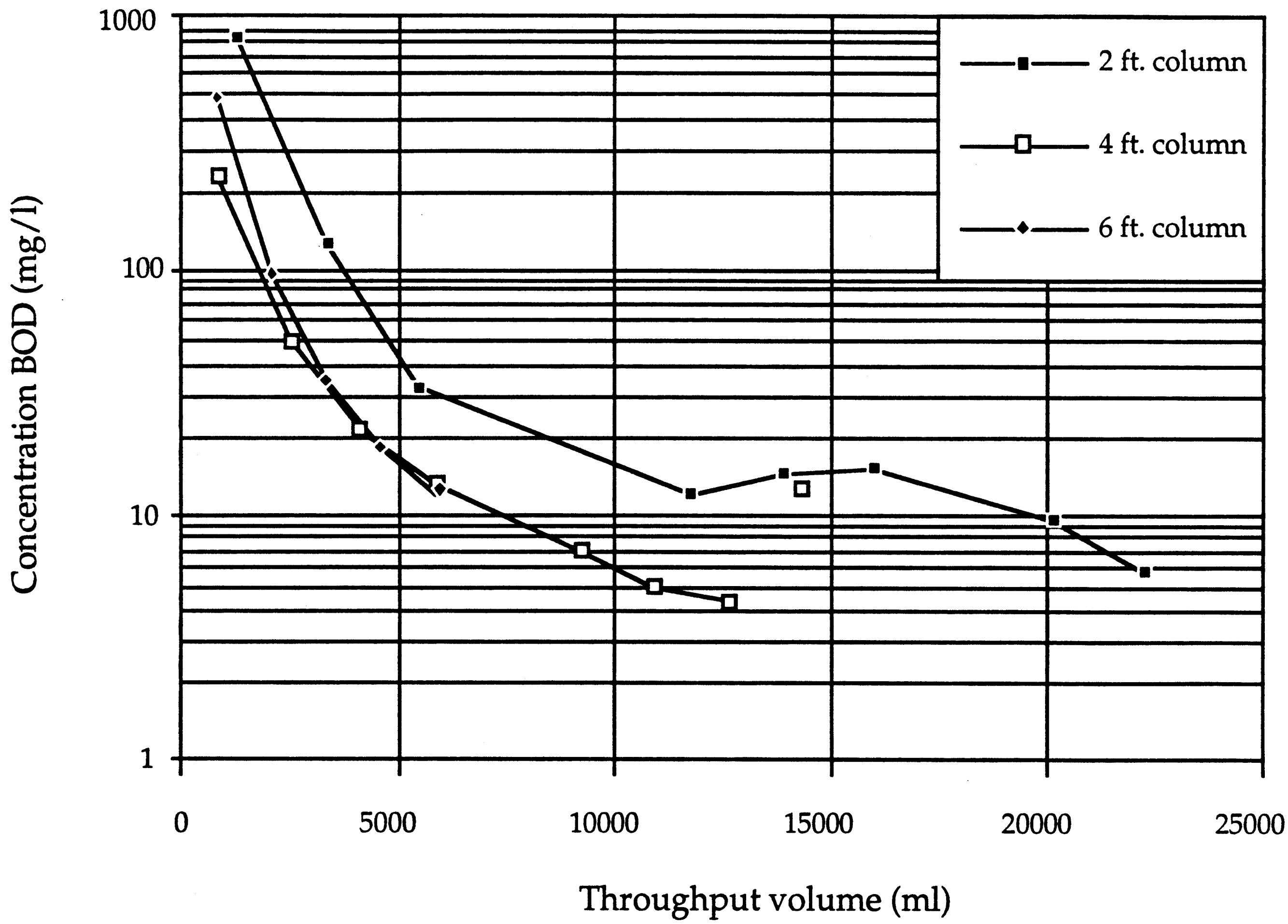


Figure 4.20 WPBMRF leaching column BOD results

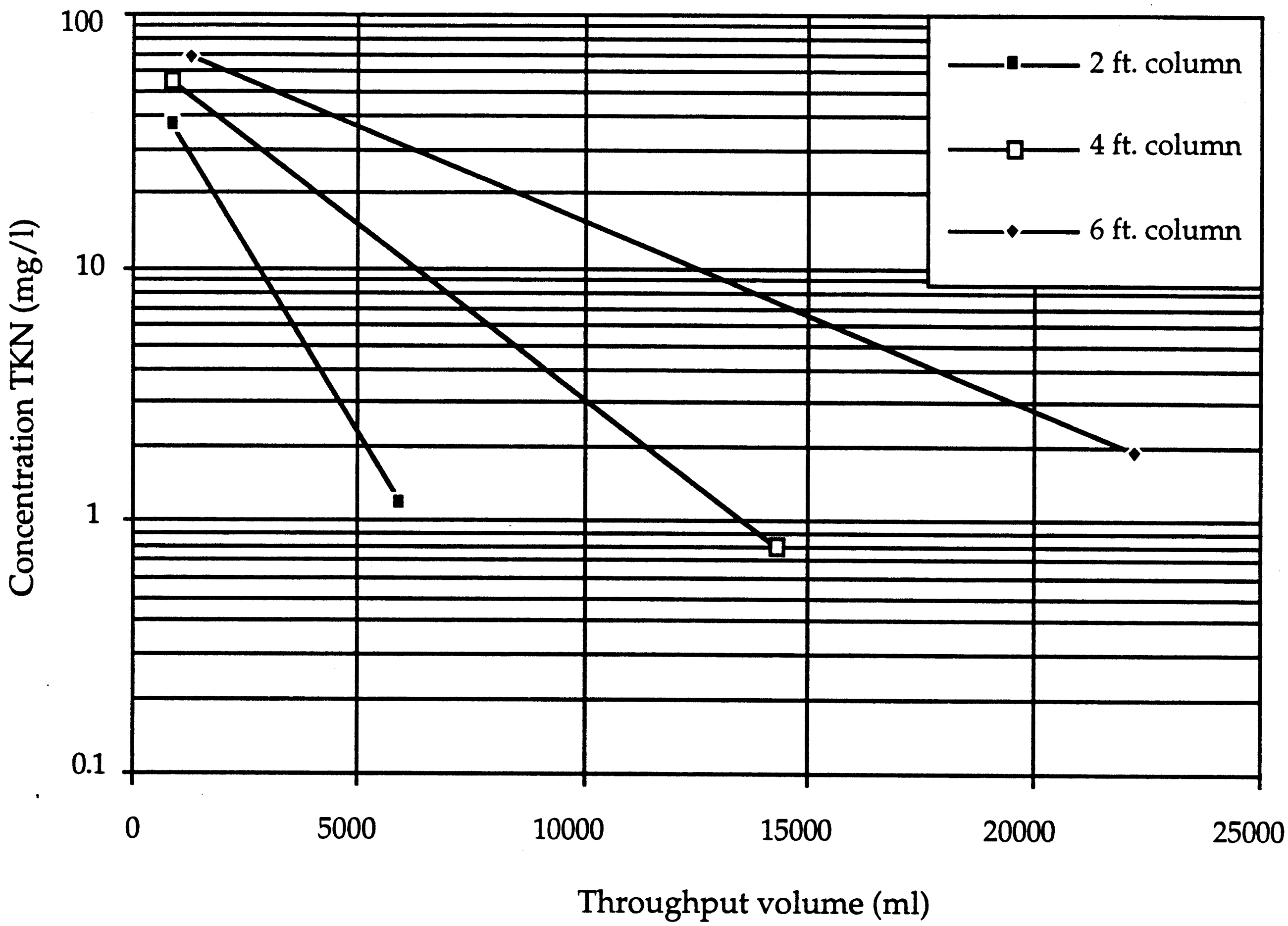


Figure 4.21 WPBMRF leaching column TKN results

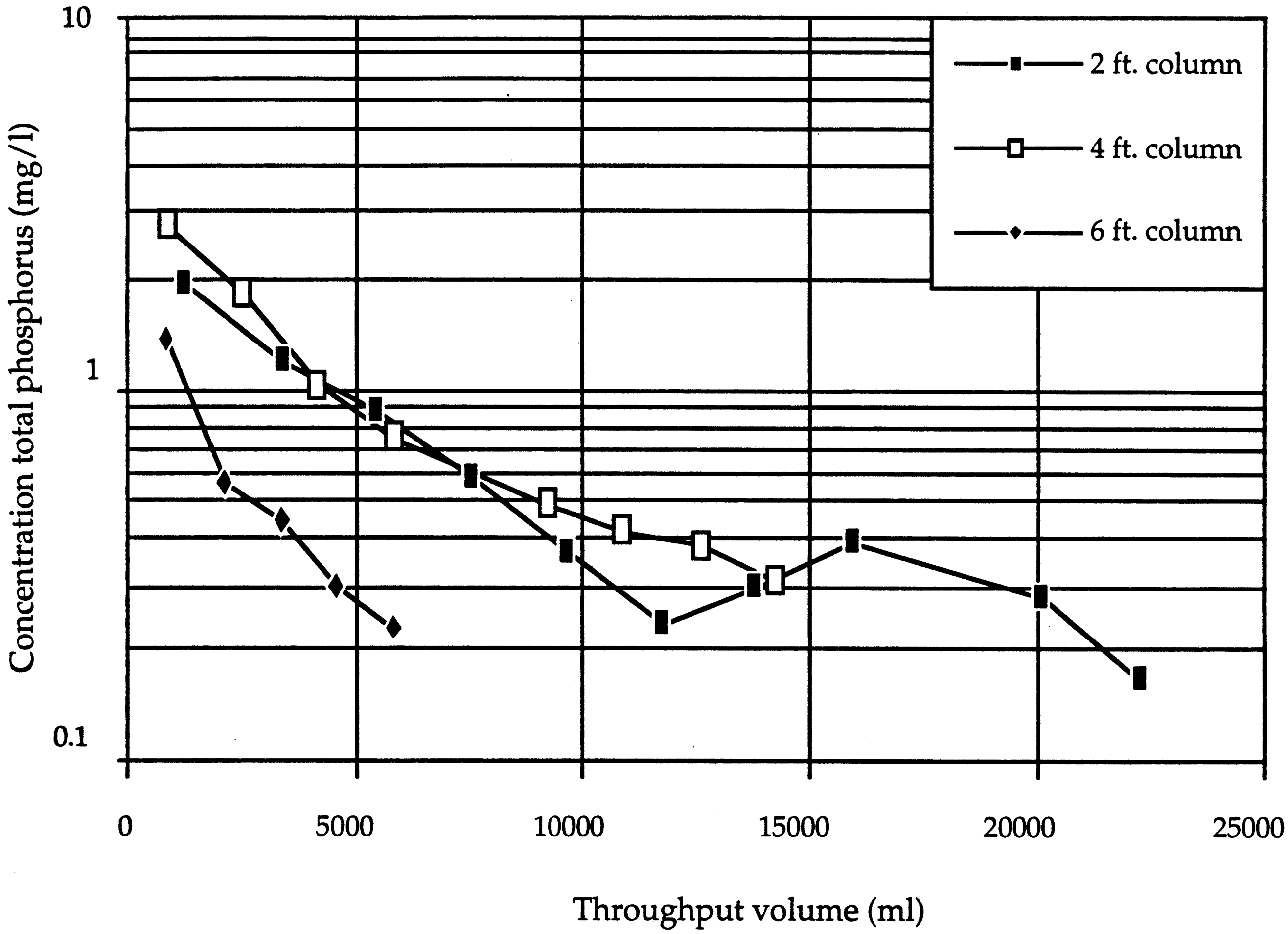


Figure 4.22 WPBMRF leaching column TP results

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 ^{o} and friction angles at high densities ranged from 24 to 43 ^{o}. 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.

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