A Study Of The Strength Of Pervious Pavement Systems

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A STUDY OF THE STRENGTH OF PERVIOUS PAVEMENT SYSTEMS

By

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B.Eng Federal University of Technology, Owerri 2005

A thesis submitted in partial fulfillment of the requirements
for the degree of Master in Science of Civil Engineering
in the Department of Civil, Environmental, & Construction Engineering
in the College of Engineering and Computer Science
at the University of Central Florida
in Orlando, Florida

Summer Term

2010

ABSTRACT

This thesis presents a study on the strength properties of the different pervious pavement systems installed at the Stormwater Management Academy field laboratory at University of Central Florida (UCF), Orlando. The strength tests were performed both in the laboratory and in the field. Laboratory testing was conducted to determine the compressive strength and flexural strength of the various pavement surfaces. Evaluation of field pavement performance was performed by comparing the deflection basins using the Falling Weight Deflectometer test on pervious concrete and porous asphalt with conventional impervious concrete and asphalt pavements of similar layer profile and thickness, respectively. From literature and previous work at the academy, it is evident that pervious pavements should not be used to withstand heavy traffic loading. They are mostly used in low traffic volume areas such as parking lots, driveways, walkways and some sub-divisional roads.

This research studied the compressive strength and flexural moduli. Also it investigated the relationship between the compressive strength and void ratio, unit weight and volume by carrying out laboratory testing of different pervious pavements such as pervious concrete, porous asphalt, recycled rubber tires, recycled glass and porous aggregate. Different sizes of cylinders and beams were cast in place molds for these laboratory tests. Furthermore, the in-situ resilient moduli of the twenty four pavement sections in our research driveway were back calculated with Modulus 6.0 (Liu, et al., 2001) computer program. The calculated deflection basins were compared to the results obtained from a well known computer program called KENPAVE (Huang, 2004).

The design of the requisite pavement layer thickness design was performed by doing hand calculations using American Association of State Highway Transportation Officials (AASHTO) method for flexible and rigid pavements and utilizing a Texas Transportation Institute (TTI) computer software known as FPS 19W (Liu, et al., 2006). The structural number for flexible pavements were calculated and tabulated for two different reliability levels (90% and 95%). Traffic loading was estimated in the absence of actual traffic count measurement devices at the field test site.

Based on the laboratory testing, the maximum compressive strength of the cored pervious concrete was about 1730 psi. Backcalculated pervious concrete and porous asphalt moduli values were within the specified range discussed in literature. The in-situ modulus of elasticity range for pervious concrete is found to be 740 – 1350 ksi, for porous asphalt 300 – 1100 ksi, for permeable pavers 45 – 320 ksi, for recycled rubber tire 20 – 230 ksi, recycled glass pavement 850 ksi and porous aggregate 150 ksi. For low volume traffic loading, the minimum layer thickness was calculated for rigid pavements and it is presented in this study.

In conclusion, this research summarizes the result of laboratory and field testing performed at the University of Central Florida Stormwater Management Academy Research laboratory to determine the strength related properties of pervious pavement systems.

This thesis is dedicated to my grandfather who passed on to eternal glory while I was writing this thesis.

ACKNOWLEDGMENTS

Firstly, I give thanks to God for giving me life and crowning my efforts with success. My profound gratitude goes to Dr Manoj Chopra for his excellent advice and support in ensuring that this research work sees the light of day and for giving me the opportunity to actualize a milestone in my academics. I hereby extend an unwavering appreciation to Dr Marty Wanielista and Dr Lakshmi Reddi for serving on my committee and interest in trying to improve the content of this research. I would also like to thank Ikiensinma Gogo-Abite for his support and encouragement throughout this program.

I am grateful to Florida Department of Transportation (FDOT) for their financial support in this research. Special thanks go to Mr. Charles Holzschuher and Mr. Hyung Suk Lee of the FDOT for their help in providing FWD equipment and backcalculation software.

My indebtedness to my colleagues at the Stormwater Management academy (UCF) especially Erik Stuart, Mike Hardin, Nic and Rafiq Chowdhury for their support, guidance and help in ensuring this research was a success.

Finally, my deepest appreciation is poured out to my parents and siblings whose love, understanding, support and encouragement kept me afloat even in times of struggles and crisis. Not forgetting all my friends especially Jumoke, Susay, Assal, Elie and Theodore for their kind gestures towards me.

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LIST OF ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
ADT	Average Daily Traffic
AADT	Average Annual Daily Traffic
ASTM	American Society of Testing and Materials
BMP	Best Management Practices
C_d	Drainage Coefficient
E_c	Modulus of Elasticity of Concrete
EPA	Environmental Protection Agency
FDOT	Florida Department of Transportation
ft	Feet
FWD	Falling Weight Deflectometer
f'_c	Compressive Strength of Concrete
GY	Total Growth Factor
in.	Inches
J	Load Transfer Coefficient
K	Modulus of Subgrade Reaction in pci

ksi	Kilo pounds per inch
LID	Low impact development
lbs	Pounds
MnDOT	Minnesota Department of Transportation
M_R	Resilient Modulus
NDT	Non-destructive test
P.C	Pervious Concrete (or Porous Concrete)
P.A	Porous Asphalt
PP	Permeable Pavers
P_i	Initial Serviceability Index
P_t	Terminal Serviceability Index
ΔPSI	Change in Serviceability Index
PCA	Portland Cement Association
PCC	Portland Cement Concrete
PCPC	Portland Cement Pervious Concrete
Psi	Pounds per inch
Pci	Pounds per cubic inch

Pcf	Pounds per foot
R	Reliability
SCI	Surface Curvature Index
S_c	Modulus of rupture in psi
S_o	Standard Deviation

INTRODUCTION

Background

A pavement is a surface treatment or covering laid over soils or rocks intended to carry vehicular or foot traffic. It is the ever present man-made structure (Ferguson, 2005). Asphalt pavements have being used since the early twentieth century. All over the world, engineers strive to strike a balance between cost of a project and its environmental impact on humans and species. As the world becomes environmentally more conscious, solutions are constantly being sought on ways to make stormwater more effective. All these concerns triggered the development and subsequent evolution of pervious pavements.

Impervious pavements, which are the majority of pavements laid worldwide, are responsible for two-thirds of the excess runoff and also hydrocarbon pollutants in urban settlements (Ferguson, 2005). Most of the stormwater runoff issues arise due to loss of the water retaining function of the soil in the urban settlements (Booth & Leavitt, 1999). The major issues with stormwater are the volume of the runoff water and the pollutants carried by this water.

Pervious pavements, which are also known as porous pavements, are pavement systems with inter-connected network of void spaces (Ferguson, 2005). Pervious pavements are an important step towards improving the environment. Hydrocarbon pollutants may pass through this pavement but they will eventually disintegrate in the soil. This new pavement technology potentially reduces the reliance on retention ponds and other traditional stormwater management devices such as curbs, gutters and underground piping (Huang et al., 1999). Primarily, these type of pavements were developed to decrease the amount of stormwater runoff in urban areas (Scholz, et al., 2007). Porosity and permeability of pervious pavements are significant enough to influence the hydrology, environmental effects, and mechanical properties of the entire pavement-soil system (Ferguson, 2005).

Research into pervious pavements at the Stormwater Management Academy Research and Testing Laboratory (SMARTL) at the University of Central Florida (UCF) started in the year 2005 with financial support from Florida Department of Transportation (FDOT) and the Florida Department of Environmental protection (FDEP). The main objective was to tackle stormwater runoff problems and look for a potential best management practice (BMP). FDOT interest vested on the increased application of pervious pavements to withstand higher volume of traffic loadings, linear projects such as shoulders and rest area parking lots. While FDEP interest was mainly on all low traffic applications.

Several different pervious pavement sections were constructed at the SMA Field Laboratory (SMARTL) to investigate its mechanical, hydraulic and environmental properties. This thesis presents the portion of the overall effort that deals with the structural properties of pervious pavements research all with the ultimate goal of improving the durability and design life of these pavements. It looks at the design requirements (strength, deflection and thickness) of such systems.

Brief Historical Perspective

Pervious concrete is undoubtedly the most investigated pervious pavement type. The use of conventional concrete (impervious) as a pavement surface dates back to the 19th century. According to Croney & Croney (1998), the first experimental construction of dense concrete pavements was done in Scotland in 1865. At the early stage, use of conventional concrete as pavements was not supported in cities because it was believed to affect the access to underground utilities (Ferguson, 2005). Pervious concrete were first used as load bearing walls and precast slabs in buildings due to lack of construction materials. The economic benefit of pervious concrete was the main reason for its use and acceptance. Pervious concrete requires less amount of cement when compared to conventional concrete.

After the World War II, some countries in Europe and United States began the use of pervious concrete as a type of pavement. It was used in California as drainage layers under conventional concrete surfaces in highways (Ferguson, 2005), and some countries in Europe used it as a surface friction course. Subsequently, pervious concrete was used as overlays on conventional concrete roads to increase drainage. It was in the 1970's that pervious concrete made a significant mark in the United States. Florida was the first State to use pervious concrete because of its hydrological properties. Its porosity and hydraulic storage capacity made it a great solution for the Florida roadways system which was plagued with increased runoff volumes. According to Ferguson (2005), the use of this type of pavements has gradually spread to States such as Washington, California, North Carolina, Minnesota and Iowa.

Asphaltic concrete, otherwise known as asphalt, is composed of asphalt cement which bounds the aggregates. Early asphalt pavements were installed in Europe in the 1850's (Croney and Croney 1998). In 1870's, the first installation in the United States of America was completed in Washington D.C. Dense asphalt surfaces were supported by a layer of aggregate base. Porous asphalt was first developed by Edmund Thelen and his colleagues in the 1970’s at the Franklin Institute in Philadelphia (Ferguson, 2005). This type of pavement consists of very low amounts of particles greater than 600 microns or the U.S. standard sieve No. 30 or no fine aggregates to fill the void spaces between the aggregates, which allows for the free flow of water through it.

The use of block pavers dates back to many centuries ago (Ferguson, 2005). Most paving blocks were made of stone and bricks. However, after World War II, European companies began production of concrete block pavers for the first time to reconstruct the cities affected by the War (Ferguson, 2005; Borgwardt, 1998; Fischmann, 1999; Rollings and Rollings, 1992; Shackel, 1990; Smith, 1999). These concrete paving blocks were mainly tight-jointed. This technology did not enter North America because the main focus of the pavement industry was on asphalt and concrete technology. Nevertheless, in the 1970’s the equipment used to produce this kind of pavement was introduced to North America. Most porous concrete block pavers use an open-jointed system (Ferguson, 2005).

Recycled rubber tires are mixed with asphalt binders to produce an asphalt-rubber pavement mixture. These recycled granular materials are also mixed with unbound coarse aggregate and a type of polyurethane binder to form a porous pavement system called Flexipave®. According to EPA (2003), U.S generates about 280 million scrap tires per year. Recycling of rubber dates back more than a hundred years to a time when rubber was a scarce commodity. The recycling of rubber drastically decreased over the years mostly because of the discovery of synthetic rubber from less expensive imported oil (RubberPavementAssociation, 2005). This discovery in pavement technology opened doors to more discoveries in this field as scientists sort to produce more environmental friendly pavement surfaces. Recently, a new pavement technology has been introduced which uses recycled glass obtained from crushed bottles and other glassware. This type of pavement was proposed in the early twenty-first century and is called Filterpave®. The edge of the recycled glass is rounded by a tumbling process to remove its sharp cutting edges and a polyurethane binder is used to bind these glass aggregate pieces into a pavement structure.

Problem Statement

According to Kevern (2008), most of pervious concrete installations are located in places in the U.S. which do not experience freeze-thaw cycles, and have favorable environmental and weather conditions. However, these areas witness other types of pavement durability failures. The common types of pavement distresses observed are raveling and surface rutting. Ravelling occurs as a result of dislodgement of aggregate particles, while rutting may be due to inadequate compaction of the flexible pavement layer during construction.

The strength of a pervious pavement system does not only depend on compressive and flexural properties but also on the strength parameters of the supporting underlying subgrade. The serviceability requirement of the entire pavement system also needs to be analyzed. As a result of its porous nature (no fines) to achieve high permeability, the compressive strength and flexural strength are both low, when compared to conventional concrete and asphalt pavements.

The maximum traffic load and volume that these pervious pavements can carry and still maintain their structural integrity needs to be evaluated even though its acknowledged that these pavements must only carry light vehicular loads. Most literature on strength testing has focused on pervious concrete. This research would critically evaluate the strength parameters for each of the six pervious pavement systems and establish the allowable traffic load and volume to provide some degree of confidence with strength and durability of pervious pavements.

Advantages and Disadvantages

Pervious pavements advantages far outnumber its disadvantages. This innovation in pavement technology makes better use of land use by eradicating the need for retention basins, swales and other traditional stormwater management devices (NRMCA, 2005) . It is cost effective and decreases pollutants from runoff. In general, pervious pavements replenish the groundwater and decreases the stormwater runoff and flooding over the area. Furthermore, it lessens evaporative emissions from parking lots and thermal pollutants.

These systems also have some limitations. In areas experiencing freeze thaw cycles, pervious pavements are easily affected by plowing because this process disintegrates the aggregate particles and can also damage the pavers. When supported by heavy clay soil subgrade, the voids easily get clogged thereby reducing its permeability properties (Kevern, 2008). In addition, the compressive strength of pervious concrete is low when compared to impervious (conventional) concrete because of the lack of fines, pore spaces and weaker bond strength between the aggregates. (Yang, et al., 2003). The mode of failure of these pavements is by excessive raveling, thereby creating surface rutting and loose particles which obviously reduces permeability.

Applications

Though this technology has being used as load bearing walls in homes (Ghafoori, et al., 1995a), its primary use is found in pavements. Pervious pavements are among the Best Management Practices (BMPs) recommended by Environmental Protection Agency (EPA) for managing stormwater runoff. According to Mulligan (2005), the limitation of pervious pavements is its lack of durability under heavy loads. It is only known to withstand light vehicular load and volume. This attribute limits its use to residential driveways, low traffic roads, fire lanes, emergency access roads, parking areas, sidewalks, road shoulders and vehicle cross-overs; boat launching ramps, pool decks and patios; greenhouses.

Research Objective

The broad objectives of this research study are:

- Identify strength parameters of pervious pavement systems.
- Compare the mechanical properties of pervious pavements with their conventional counterparts - Portland cement concrete (PCC) and asphalt concrete.
- Evaluate the in situ (field) pavement responses of flexible and rigid pavements by means of the Falling Weight Deflectometer (FWD) Test.
- Compare the deflections of pervious pavements with conventional pavements.
- Evaluate the minimum thickness of rigid pervious pavements by means of the AASHTO design method for rigid pavements.

- Evaluate the required structural number of flexible pervious pavements by means of the AASHTO design method for flexible pavements.

- Evaluate the stiffness parameters of various pervious pavements, stresses and strains, and compare these parameters to asphalt concrete and conventional PCC pavements of similar thickness.

Significance of the Research

The research plan is to collect information from existing literature, perform strength testing to obtain strength parameters of six pervious pavement systems using both laboratory and field tests. The typical failure mode of these pavements is by excessive raveling, thereby creating surface rutting and loose particles which obviously reduces the permeability of the system (Kevern, 2008) and leads to clogging. However, this phenomenon is not within the scope of this project. The research will be used to as a guideline by the FDOT to understand the strength properties and thickness design of different pervious pavements systems. Also, it gives individuals and organizations a better knowledge of the best pervious pavement system to use, even in the absence of pervious concrete.

Thesis Outline

A brief introduction of the history, advantages, disadvantages, and uses of pervious pavements is discussed in chapter 1. The importance and objective of this research is also emphasized. Chapter 2 contains a review of past research from existing literature on pervious pavements. Data collection, analysis and results of previous studies conducted on compressive strength, flexural strength and falling weight deflectometer (FWD) of pervious pavements are highlighted.

An in-depth study of the various components of the pervious pavement systems such as aggregates, binder, cement and the interaction effects on the strength properties of the pavements are discussed in chapter 3.

Chapter 3 outlines the methods used in conducting experiments, both in the laboratory and the field. The laboratory experiments conducted were tests to determine porosity, compressive strength and flexural properties. While the field test was carried out by the falling weight deflectometer (FWD) to determine the in-situ elastic moduli of the pavement systems. These testing procedures were explained in such a manner that it can easily be duplicated by another researcher.

Results from the various tests conducted are presented in Chapter 4. The relationships between the compressive strength and the unit weight are presented in various plots. The FWD data collected from the field experiment and the calculated results of the pavement responses of the pervious pavements are discussed. Furthermore, the layer elastic modulus of each layer in the different pervious pavement systems is obtained using a backcalculation program. A discussion of the stresses and strains obtained from the KENPAVE analysis is also presented. Lastly, pavement thickness design tables are shown by making use of the FWD data obtained from the field testing.

In chapter 5, conclusions are drawn from the experiments performed in this research with respect to pavement loading, thicknesses, stress-strain responses and layer elastic moduli. Recommendations for future research in the use of pervious pavements and their corresponding strength properties are also provided.

LITERATURE REVIEW

Introduction

The traditional stormwater management systems are designed to collect and distribute stormwater to nearby surface water bodies. The growing recognition of the disadvantages of traditional stormwater management has led to the studies on pervious pavement systems (Booth, et al., 1999). According to Scholz & Grabowiecki (2007), the general principle of pervious pavement systems is to collect, treat and infiltrate freely any surface runoff in order to recharge the groundwater. Most States and municipal governments must comply with the Federal Clean Water Act (Kloss, 2006) and solutions to excess stormwater runoff and related pollution issues are being sought.

Regulations are promulgated by these agencies to control the amount of impervious surfaces allowed for development. Pervious pavement surfaces are potential solution to comply with these regulations and also maximize land use. Though, various pervious paving systems are in the test stage, historically, only pervious concrete and pervious asphalt systems have generated the most interest among engineers and government agencies. Committees have been formed to develop a document concerning these pavements (like ACI 522), but it is apparent that a long term evaluation of their hydraulic and structural performance is still needed.

Pervious pavement systems have been in use for more than 20 years in various applications. Parking lots, streets, and local roads with minimal heavy truck traffic are places where it has been used. It allows water to filter through to replenish the water table due to its 15-40% air voids and may also have beneficial impacts on the water quality. This literature review includes a summary of the major findings in the field of pervious pavements.

Types of Pervious Pavements

The six types of pervious pavements reviewed in this research are (a) pervious concrete, (b) pervious asphalt, (c) recycled rubber pavement, (d) recycled glass pavement, (e) pervious aggregate and (f) pervious pavers.

Pervious Concrete

Pervious concrete (PC) has been used in Florida for over 30 years. Portland cement pervious concrete (PCPC) is composed of open-graded aggregate, Portland cement, water and any admixture. There are little or no fine aggregate in the mixture. The cement paste binds the uniformly graded coarse aggregate, which creates an interconnected void structure. Kevern (2008) stated that most of the pervious concrete mixes in the U.S. had relatively high porosity (15%-35%) and low strength, while European mixtures had lower porosity (15%-20%) and higher strength. Some highways make use of it mainly as surface course to reduce traffic noise and improve skid resistance (Beeldens, 2001). Researchers have carried out experiments on this pavement type to ascertain its durability properties by altering the aggregate size, quantity of cement and water.

Recently, validation of pervious concrete has been enhanced most notably the formation of the American Concrete Institute (ACI) committee 522 on pervious concrete and subsequent release of its March 2010 report on pervious concrete (ACI, 2010). In addition, the formation of the Association for the Standardization of Testing and Materials (ASTM) subcommittee 09.49 on pervious concrete, various reports of research sponsored by the National Ready Mixed Concrete Association (NRMCA) research and education foundation, and the Portland Cement Association (PCA) education foundation.

Prior Research at the University of Central Florida

A joint research initiative between FDOT, NRMCA research and education foundation, Florida Department of Environmental Protection (DEP) and Rinker materials led to the publication of a series of study on the performance of Portland cement pervious pavement. These three research reports are titled, "Hydraulic Performance Assessment of Pervious Concrete Pavements for Stormwater Management Credit" (Wanielista, et al., 2007), "Construction and Maintenance Assessment of Pervious Concrete Pavements" (Chopra, et al., 2007a), and "Compressive Strength of Pervious Concrete Pavements (Chopra, et al., 2007b) were based on the research performed at the University of Central Florida.

In the first study (Chopra, et al., 2007a), the construction and maintenance performance assessment compared field hydraulic performance of sites located in Florida, Georgia and South Carolina to the laboratory performance as determined by an embedded single ring infiltrometer. The objective of this study was to evaluate the clogging potential of existing pervious concrete systems and in addition analyze the effect of rehabilitation techniques on infiltration. Furthermore, the report provided installation specifications for the construction of pervious concrete in different geographical locations. Rehabilitation techniques performed were pressure washing, surface vacuuming or a combination of both. It was revealed from the results that permeability increased by 200% when a maintenance schedule was implemented. The quality of installation of this type of pervious pavement was discussed and the use of certified experienced pervious concrete contractors was recommended (Chopra, et al., 2007a).

The hydraulic performance assessment report (Wanielista, et al., 2007) laid emphasis on the infiltration potential of thirty (30) cored samples extracted from eight parking lot sites in Florida, Georgia and South Carolina. The embedded-single ring infiltrometer was used to measure the infiltration rates. For proper simulation of the hydrologic and hydraulic functions of pervious concrete system, a mass model was developed. Laboratory permeability was generally found to be lower than that measured in the field. Nevertheless, it recommended stormwater credit be granted for infiltration for pervious concrete pavement (Wanielista, et al., 2007).

This final research report (Chopra, et al., 2007b) is based on results obtained from compressive strength testing of core samples previously extracted for permeability testing. Thirty-two (32) test cylinders were tested to provide representative samples of different Aggregate – Cement (A/C) ratio and Water – Cement (W/C) ratios. The pervious concrete mixture was made from ⅜-inch aggregate and Type I Portland Cement. The thirty two samples were split into eight separate batches with four different A/C ratios and two methods of compaction (Standard Proctor (ASTM, 1991) and Modified Proctor (ASTM, 2002)). Pervious concrete with different mix proportions was tested that resulting in an average strength of 11.7 MPa (1,700 psi). The mode of failure, raveling, was clearly seen at the entrance and exits of various sites leading to the recommendation of limiting pervious concrete installation where repetitive loading occurs. It was observed that higher aggregate to cement ratios decreased strength while high water-to-cement ratios decrease porosity. A recommendation was made to low traffic loading applications (Chopra, et al., 2007b).

In addition, a report on the performance assessment of a pervious concrete pavement used as a shoulder for an Interstate rest area parking lot was released in 2007 (Chopra, et al., 2007c). The dimension of the shoulder was 90 feet long and 10 feet wide. The depth of pervious concrete used was 10 inches to accommodate truck parking loads. A 12 inch deep reservoir made up of select pollution control materials was used beneath the pervious concrete. This interstate shoulder was effectively monitored over a one year period for wear and stormwater quality. According to Chopra *et al* (2007c), the shoulder was monitored for traffic counts recording about 500 axles per week. It was noted that there was no significant wear even when 500 axles per week loads were experienced. In addition, the water quality through the PC system was found to be equivalent to rainwater.

Other Pervious concrete literature

Ghafoori, et al. (1995b) performed laboratory study of compacted pervious concrete in which it is used as a pavement material. This research investigated the effects of compaction energy, consolidation techniques, mix ratios, curing types and testing conditions on the physical and engineering properties of pervious concrete. The study noted that with proper proportioning and compaction, the compressive strength of 28-day pervious concrete could reach 20.7 MPa (3,000 psi) or greater.

Ghafoori, et al. (1995c) suggested the use of the two popular methods for pavement thickness design for pervious concrete. These methods are American Association of State Highway and Transportation Officials (AASHTO) design guide and Portland Cement Association (PCA) design procedure. He further stated that the AASHTO method is based on empirical regression equations created from AASHO road tests conducted in the late 1950s and early 1960s.

The study determined the thickness requirements of pervious concrete pavements based on the engineering properties produced in the laboratory and also different traffic conditions and subgrade characteristics. The application of reliability concept is an essential instrument of the AASHTO method. Traffic in this method is strictly based on cumulative expected 80 kN equivalent single axle loads (ESAL) for the design life of the pavement. Some factors that were put into consideration when using this design method are namely: serviceability, flexural strength and modulus of elasticity, load transfer coefficient, reliability, modulus of subgrade reaction.

Huang et al (2006) researched the effects of aggregate gradations on the permeability and mechanical properties of pervious concrete. The study characterized pervious concrete made from three aggregate gradations by carrying out laboratory tests. The evaluation of the mechanical properties of the pervious concrete was done through two properties: compressive strength and split tensile strength. This study concluded that aggregate gradation significantly affects the strength and permeability of pervious concrete mixtures. The materials used to produce the pervious concrete were Type I cement, three (3) gradations of gravel and a controlled amount of water. The aggregate sizes used are 4.75 mm (No. 4), 9.5 mm (3/8 inch) and 12.5 mm (1/2 inch). No fine aggregate or chemical admixture was added. The mix proportion for each aggregate gradation was 1:4.5:0.35 (cement: coarse aggregate: water). The mixing of the materials was done in a concrete mixer for about 10mins and thereafter poured into cylindrical PVC molds. Compaction of the cylindrical samples was carried out manually.

The researchers performed strength tests after 7 days of curing. The compressive strength test was conducted on three samples in accordance with ASTM C39. The size of the cylinders used was 152.2 mm x 404.4 mm (6 inch x 12 inch). As shown in

Table 1: Compressive strength of pervious concrete at 7-day

Aggregate Size in Mixtures (inch)	Compressive Strength at 7-Day (MPa)
½	7.2460 ±0.6754
3/8	3.3983 ±0.3341
No. 4	2.4230 ±0.0205

Research Engineers at Minnesota Department of Transportation

Rohne & Izevbekhai (2009) performed research on pervious concrete test cell at MnROAD facility. The test cell was subjected to daily traffic loads of 80 kip 5-axle semi trailer, twice a day, four days a week and also 102 kip 5-axle semi-trailer two times a day. Pavement performance was evaluated by comparing Falling Weight Deflectometer (FWD) deflection to those of conventional concrete pavements with similar thickness., the compressive strength increased as aggregate size increased. Failure occurred at the bond between the cement paste and the aggregate. They concluded that one reason for higher strength of pervious concrete made from larger aggregate sizes is that the binding agent (cement paste) is more between larger aggregate sizes.

Table 1: Compressive strength of pervious concrete at 7-day (Huang, et al., 2006)

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The size of the pervious concrete driveway section is 60-ft by 16-ft and it is bound on all sides by 2-ft wide concrete border. This study examines the performance of this pavement after three years of testing. Cylinders were cast on the placement site and compressive strength test was carried after 7 and 28 days curing. In addition, beam samples were cast on the field and flexural strength was done after 7 and 28 days curing period. Furthermore, cylindrical cores and core beams were cut from the test cell and tested after 28 days of casting for compressive strength and flexural strength. There was no standard method of rodding and placement of the cylindrical samples to effectively simulate the compactive energy of mechanical compactors or as done in the standard method of concrete cylinder preparation, ASTM C 31. Table 2 shows the results of different strength test conducted and rheological properties of the pervious concrete.

Table 2: MnROAD Cell 64 Mechanical and Rheological Concrete Properties

Parameter	Age	Range
Flexural Strength	7-day	250 psi
	28-day	540 psi
Compressive Strength	7-day	1231-3000 psi
	28-day	3000-4500 psi
Elastic Modulus	28-day	1200 ksi
Porosity	28-day	18-20%
Compression (Core)	28-day	5517-6045 psi
Flexure Beam cut from slab	29-day	500-580 psi

The results from this study show that the FWD deflection values for pervious concrete was higher than that of normal concrete. The maximum deflection for 6, 9 and

15-kips load was 78.8, 118.2 and 200-mils respectively. Normal PCC Cell 53, which has similar layer thickness as Cell 64, has a maximum FWD deflection of 98.9 mils when the 15-kip load is applied. While the maximum FWD deflection for TH 100 (normal PCC) is 39.4–mils when a load of 15-kip is applied. In summary, the deflection recorded by the Pervious concrete section (Cell 64) is about 2 to 5 times greater than those of normal concrete pavements. Calculated elastic moduli values ranged from 725 – 2900 ksi (5.00 – 19.99 N/mm^2). Rohne & Izevbekhai (2009) stated that a typical elastic moduli of conventional concrete (2000 – 6000 ksi) can be compared to the upper limit of the calculated E value. The elastic moduli values were obtained from elastic theory.

Pervious Asphalt

Water has often been described as the “enemy” of asphalt (Cahill, et al., 2003). Runoff from impervious surfaces finds their way into dense asphalt surface and erodes it. Therefore immense effort has being taken to prevent this occurrence. Pervious asphalt (PA) is an effective way of curbing this problem. Pervious asphalt, otherwise known as porous asphalt, is a well known pavement material for stormwater management purposes. This type of pavement is made up of asphalt cement (binder) and coarse aggregates. It is different from dense asphalt concrete because of its use of single sized aggregates. Like most pervious pavements, it has little or no fine aggregates in its mixture.

According to Cahill, et al., (2003), porous asphalt does not usually require additives or proprietary ingredients, even though it has been observed that polymers or fibers help to improve its durability and shear strength. Like most pervious pavements, this type of pavement is mostly used as parking lots, driveways, walkways.

Nevertheless, the major issue with porous asphalt is that of clogging (Ferguson, 2005). Clogging is normally caused by the asphalt binder. In some cases, the binder is too fluid or the bond between the binder and the single sized aggregates is weak, thereby making the binder gradually drain downwards from the surface through the pore space resulting into a clogging layer inside the pavement structure. This phenomenon mostly occurs in hot regions like Florida. The permeability of this pavement is adversely affected and also unbound surface particles are easily seen.

Pervious Pavers systems

This type of pavement system is made up of Permeable pavers (PP) as the top surface, limestone rock of two different sizes which acts as both the bedding and the base courses, and compacted subgrade. These permeable pavers can be interlocking blocks, open celled pavers, open grids. According to Smith (2006), ICPI studied ten sites and the observed infiltration rate ranges from 1.5 inches per hour to 780 in./hr. Clogging by fines (sand or aggregates) were responsible for the lower infiltration rates.

He further highlighted the data required to design a permeable paver. Firstly, the total area and percentage impervious surface draining into this pavement needs to be known. Secondly, the design storm with its return period and intensity in inches or millimeters per hour will be required. Thirdly, by means of the design storm, the volume of runoff or peak flow to be captured, exfiltrated, or released is required. Finally, the vehicular load expressed in 18-kip Equivalent Single Axle load (ESAL) over the design period of the pavement need to be known.

Recycled rubber tire pavement

Recycled or shredded tire chips are used in civil engineering applications as replacements for some construction materials such as crushed rock or gravel (RubberPavementAssociation, 2005). Currently, the largest market for recycled rubber tires is the molded products sector, where it is combined with urethane binders. Recycled rubber tire pavements are used for low load applications. This advancement in pavement technology is ideal for driveways, parking lots, walkways, sidewalks, golf cart paths, courtyards, nature trails etc.

Due to the porous nature, recycled rubber tire pavement is being used to decrease the amount of runoff water and also to improve and control stormwater quality and quantity. This pavement is made from recycled, ground up automobile tires, coarse aggregates and some additives. These materials are bound together by means of a binding agent known as XFP75 (urethane).

Recently the manufacturers introduced a new improved product. The binding agent urethane was improved so as to hold recycled passenger tires and aggregates more effectively. The new binder is called XFP95 (polyurethane).

According to the manufacturers, it is easily installed over a minimum of 4 inches (100 mm) of well compacted single-sized aggregates or crushed concrete. In addition, it could be installed over concrete or asphalt pavements. It could be installed in temperatures ranges of 45°F – 95°F. But it is clearly advised that when curing this pavement the temperature should not fall below 35°F. After installation, it is ready for use after 24 hours. It comes in various colors as requested by the customer. Porosity ranges from 50% - 60% (Flexi-Pave, 2005).

Recycled glass pavements

Recycled glass pervious system is a natural Low Impact Development (LID) BMP for managing stormwater. It is an architectural aesthetic porous pavement system and easily adsorbs hydrocarbons leakage from vehicles. Biological processes are then introduced to reduce these leakages to harmless by-products thereby reducing pollution of the groundwater.

This system is comprised of a 4” layer of specially-treated 100% post-consumer recycled glass, 20 - 30% granite and a polymer binder (urethane). It is hard surfaced and has a riding surface similar to concrete. It is relatively a new type of pervious pavement.

The literature from the manufacturer represents the mechanical and rheological properties of this pavement (Presto-Geosystems, 2009). Table 3 shows the summary of the properties of recycled glass pervious pavement.

Table 3: Summary of the mechanical and rheological properties of Recycled glass pavement (Presto-Geosystems, 2009)

Properties	Values
Aggregate Material	100% Post Consumer Recycled Glass
Binder	Polyurethane
Flexural Modulus	75,000 psi (515mPa)
Flexural Strength (per ASTM C78)	500 psi (3435 kPa)
Compressive Strength at Yield (per ASTM C39)	800 psi (5500kPa) – 7 days 1,000 psi (8240 kPa) – 28 days
Porosity	0.40 – 0.47

Pervious Aggregate

It is otherwise known as porous aggregate. This pavement system is comprised of a 4” layer of 1/8” - 1/4” rock and two component polymer binder (polyurethane) coating to reinforce the gravels. No further details are available from the manufacturer (Presto Geosystems) as yet.

Types of Strength Testing

There are two types of tests, laboratory and field testing. Laboratory strength testing in pavement encompasses compressive strength, flexural strength, split-tensile strength, rutting resistance, abrasion resistance.

Laboratory Testing

Laboratory testing of pervious pavements for strength testing was limited to mainly compressive and flexural strength testing. These two tests were performed in accordance with ASTM standards for conventional pavements because there are no strength testing standards for pervious pavements.

Flexural Strength

Flexural strength is determined in accordance with ASTM C 78-02 (ASTM, 2004b) which uses a 6 in. X 6 in. x 20 in. (152.4 mm x 152.4 mm x 508 mm) beam with a three point loading. The resulting property obtained from this test method is the flexural strength expressed as the modulus of rupture. The modulus of rupture is given by the following equation:

$$R = \frac{PL}{bd^2} \tag{1}$$

Where P = Applied load L = length of specimen

b = Width of beam d = Depth of beam

According to Crouch, et al. (2003), this equation used is obtained from elastic theory under the assumption that concrete exhibits elastic behavior up to the point of failure.

The test specimens should be smooth, free of scars or holes and their sides were at right angles with the top and bottom. The correlation between compressive strength and unit weight were analyzed.

<u>Compressive strength</u>

Compressive strength test was performed in accordance with ASTM C39 (ASTM, 2004a). It is the ability of a pavement to withstand axially directed compressive forces. This cylindrical specimens used for compressive strength testing were consistent with the ASTM recommended sizes. Compressive strength testing was conducted using a 1MN UTM machine using duplicate samples for each group. The testing rate was performed at a rate of 35 psi/s. Though the standard test method used here is meant for testing concrete, it is still a viable choice for testing rigid pervious pavements since no standard test method has been approved yet for this application. Recycled rubber tire pavement has a high recovery property due to rubber present in its mixture. The binder is the weak material holding together the coarse aggregates and other materials in the pervious pavements mixture. This test was conducted to estimate the amount of compressive force the binder can withstand before failure.

Field Testing

Six (6) pervious pavement test beds were tested in the field Recalling, these pervious pavements consist of pervious concrete, pervious pavers, pervious asphalt, recycled rubber tire pavement, recycled glass pavement, pervious aggregate. The falling weight deflectometer was used for field performance testing.

Falling Weight Deflectometer

The falling weight deflectometer (FWD) is non-destructive testing equipment used for the evaluation of the structural condition of pavements. It is made up of a trailer mounted falling weight system, which is capable of loading a pavement in such a way that simulates wheel/traffic loads, in both magnitude and duration. FWD testing usually involves measuring the mechanical properties of a flexible pavement layer at low strain levels (Goktepe, et al., 2006).

Impulse load is generated by dropping a mass (ranging from 6.7 – 156 KN) from a particular height. The mass is raised hydraulically and is then released by an electrical signal and dropped with a buffer system on a 12-inch (300-mm) diameter rigid steel plate. A set of springs between the falling mass and hit racket positioned above the load cell buffers the impact by decelerating the mass. A thin, neoprene pad rests between the plate and the pavement surface thereby allowing for an even load distribution (Choubane, et al., 2003).

When the weight is dropped, the impulse load generated enters the pavement system, thereby creating body and surface waves. According to Choubane, et al. (2003), the resulting vertical velocity of the surface of the pavement is picked up through a series of sensors (one of the sensors is located directly over the point of loading) located along the centerline of the trailer. Afterwards, these signals are used to obtain the maximum deflection from each geophone through analog integrations. Deflection-time trace is generated by the single analog integration of a signal. The deflection responses are mainly recorded by the data acquisition system located in the tow vehicle. Deflection is measured in “mils,” which is one-thousandths of an inch.

The FWD equipment is mostly used for flexible pavements. Over the years, engineers have used the FWD deflection basins to determine rehabilitation strategies for pavements and pavement system capability under estimated traffic loads.

The advantage of the FWD is that the impact load can easily be varied and it readily simulates actual traffic load. The disadvantage of FWD is that time efficiency is reduced during the measuring process as a result of constant stops at test point (Goktepe, et al., 2006).

Back-Calculation Program

According to (Turkiyyah, 2004), the traditional method for interpreting the FWD data is to back calculate structural pavement properties which entails extracting the peak deflection from each displacement trace of the sensors (deflection basin) and matching it, through an iterative optimization method, to the calculated deflections of an equivalent pavement response model with synthetic moduli (Goktepe, et al., 2006). Iterations are continually performed until a close match between the measured and calculated/predicted deflection values are attained.

There are three main back-calculation techniques, such as, static, dynamic and adaptive. The predicted or calculated deflection can either be static or dynamic. Every static back-calculation technique uses an iterative optimization process, and therefore the forward pavement response can be calculated by means of either layered elastic theory or finite element method (FEM) for either linear or non linear material behaviors (Goktepe, et al., 2006).

The dynamic pavement response depends on the elastic moduli thicknesses, damping ratios (β), Poisson's ratios (μ), mass densities (ρ). The values of these parameters (damping ratios, Poisson's ratios and mass densities) are usually known. Therefore, the unknown parameters are usually the thickness of the pavement layers and the complex moduli (G*) which are functions of the angular frequency (ω) and some material properties (Goktepe, et al., 2006).

In dynamic back-calculation analysis, deflection data are measured in time domain or frequency domain depending on the type of loading applied. Fourier transform is used to transform the time domain data to frequency domain. Nonlinear material behavior is not considered in dynamic back-calculation analysis because of the complexity of its analysis. Therefore, in most algorithms, material behaviors are considered to be linear (Goktepe, et al., 2006).

The third back-calculation method is known as adaptive back-calculation. It is less known compared to the other techniques. It was developed by (Meier, et al., 1993). It combines the forward and backward model into a single step by means of a supervised learning algorithm (Goktepe, et al., 2006).

Back-calculation of layer moduli of pavement layers is an application of Non-destructive testing (NDT). It involves measuring the deflection basin and varying moduli values until the best fit between the calculated and measured deflection is reached. This is a known method presently used for pavement evaluation. According to Huang (2004), there is presently no backcalcualtion method that will give reasonable moduli values for every measured deflection basin.

The Modulus 6.0 microcomputer program (Liu, et al., 2001) is one of the available programs that backcalculates layer moduli. This software is used by most DOTs here in U.S. The Texas Transportation Institute (TTI) developed this computer program and it can be used to analyze 2, 3 or 4 layered structures. A linear-elastic program called WESLEA is then utilized to produce a deflection basin database by assuming various modulus ratios. Huang (2004) states further that a search routine fits calculated deflection basins and measured deflection basin. Finally, after mathematical reductions and substitutions, the modulus can be expressed as:

$$E_n = \frac{q_a f_i \sum_{i=1}^{s} \left(\frac{f_i}{f_i \omega_i^m}\right)^2}{\sum_{i=1}^{s} \left(\frac{f_i}{f_i \omega_i^m}\right)} \tag{2}$$

Where f_i are functions generated from the database

q is contact pressure

ω_i^m is measured deflection at sensor i a is the contact radius

Table 4: Default range of moduli and Poisson ratio

Table 4 provides typical default ranges for the values of the elastic modulus and Poisson’s ratios for various materials used as layers in the design of pavements.

Table 4: Default range of moduli and Poisson ratio (William, 1999)

Material Type	Moduli range (MPa)	Poisson’s ratio
PCC (Portland Cement Concrete)	6890-68900	0.15
Asphalt concrete (cold>hot)	1378-17225	0.25-0.35
Unstabilized crushed stone or gravel base course (well drained)	69-1100	0.35-0.40
Unstabilized crushed stone or gravel base course (poorly drained)	69-690	0.40-0.42
Asphalt treated base	69-620	0.35
Sand base	35-550	0.35
Sand Subbase	35-550	0.35
Cement stabilized base and subbase	3445-17225	0.25-0.35
Lime stabilized base and subbase	35-1378	0.25-0.35
Subgrade soil cohesive clay	21-28	0.42-0.45
Subgrade soil fine-grained sands	170-205	0.42-0.45
Cement stabilized soil and bedrock	689-6890	0.20
Lime stabilized soil	689-2756	0.25

Determination of Layer Coefficients and Structural Number

The layer coefficient (a_i) and structural number (SN) can be estimated from the deflection data obtained from FWD testing. According to (AASHTO, 1993), the effective structural number SN_{eff} is evaluated by using linear elastic (Burmister) model which depends on a two layer structure. SN_{eff} is determined first before the layer coefficients of the different pavement layer. The effective total structural number can be expressed as:

$$SN_{eff} = 0.0045 h_p \sqrt[3]{E_p} \qquad (3)$$

Where:

h_p =total thickness of all pavement layers above the subgrade, inches

E_p = effective modulus of pavement layers above the subgrade, psi

Also, it must be noted that E_p is the average elastic modulus for all the material above the subgrade. SN_{eff} is calculated at each layer interface. The difference in the value of SN_{eff} of adjacent layers gives the SN. Therefore the layer coefficient can be determined by dividing the SN of the material layer by the thickness of the layer instead of assuming values.

Pavement Design

Pavement thickness design is an important aspect of this project. But before we tackle this subject matter more light should be shed on the types of pavements. There are three main types of pavement. The pavement types are:

- Flexible Pavement
- Rigid Pavement
- Composite Pavement

Flexible pavements are pavements that are made by bituminous concrete and aggregate materials. This type of pavement is frequently being used in U.S for constructing our highways and roads. In the present work, PA and recycled tires may be assumed to behave like flexible pavements.

Rigid Pavements are pavement types constructed by means of Portland cement concrete (PCC) and positioned on top of a granular material layer. (MHWA, 2006). PCC pavements are usually plain and jointed, jointed reinforced, continuous reinforced and prestressed concrete. Pervious concrete, pervious pavers and recycled glass pavement maybe assumed to behave like rigid pavement. Composite Pavements are pavements which have HMA layers placed on Portland cement concrete base. The PCC layer provides a strong support for the HMA.

There is a need to conduct research on the in situ strength parameters, damage analysis, reliability factors and standard surface layer thickness of the different pervious pavement system. Firstly, the stress, strain and displacement at various points in the layered system is analyzed (with some input values obtained from the FWD data), using a computer program known as KENPAVE (Huang, 2004) . Thereafter, manual calculations of both flexible and rigid pavements are done using equations. Next, FPS-19W, a mechanistic-empirical design software designed by the Texas Transportation institute is used to carry out the design of the pavement. Pavement design cannot be performed without clearly defining some terms which will be used in the design process.

KENPAVE computer program (Huang, 2004)

This is a mechanistic-empirical approach to conduct forward calculation pavement analysis. Like every other mechanistic-empirical design method, its operation relies on the mechanics of materials that relates input (such as wheel loads, layer thickness) to pavement responses (such as stress, strain and displacement) (Huang, 2004). Huang (2004), further states that the essence of mechanistic procedures are mostly to improve reliability of design, prediction of distress types and the possibility of extrapolating from given limited laboratory and field data.

Pervious Pavement Design

Design usually involves determination of the thickness of the layers in a pavement system to meet certain requirements. The thickness of the top layer is designed using a mechanistic-empirical approach. The terms listed below are some of the terms which will be used in this pervious pavement design procedure.

Surface course

This is the topmost layer of a pavement. It is also known as wearing course. Sometimes, an additional friction course is placed on top of this layer. Different types of asphaltic concrete, hot mix asphalts or Portland cement concrete are used for this layer. The purpose of this layer is to transmit the traffic loads to the base course.

Base course

This layer is usually below the surface layer and it is made up of various materials. It supports the surface course and distributes traffic loads to the layers below, subbase (if necessary) and finally the subgrade (FDOT, 2008). The materials placed in this layer are mainly open graded stabilized or untreated aggregates, crushed rock, crushed slag, hot mix asphalt, concrete (Huang, 2004).

Subbase course:

This material layer is placed below the base course and above the subgrade. Like the base course, it is made up of granular materials such as gravel, crushed stone, or a combination of these materials (Mass Highway, 2006). By including more fines, it can also act as a filter between the base course and subgrade.

Subgrade

This is the final layer in the pavement system. The surface of the subgrade layer is usually compacted to a recommended or specified density. This helps to improve the strength and carrying capacity of the soil layer and prevent failures.

Traffic Loading and Volume

Traffic volume and loading are one of the most essential parameter in pavement design. Traffic data is collected with the aid of Vehicle Classification counts. The magnitude and configuration of the load and the number of load repetitions should be added when considering traffic loads. According to Huang (2004), the three types of loads to consider are fixed traffic, fixed vehicle and variable traffic and vehicle.

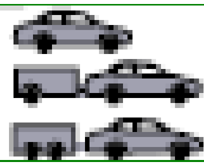
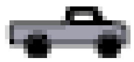
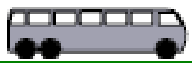
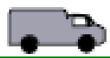
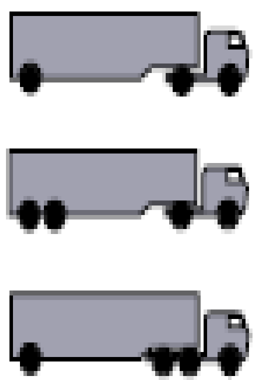
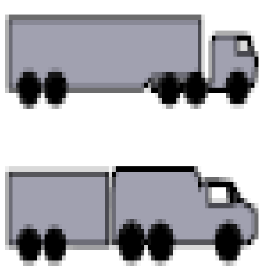
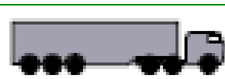
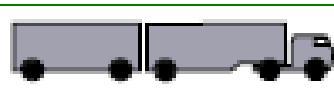
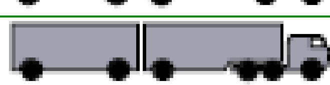
In the fixed traffic procedure, the determination of the pavement thickness is governed by the single wheel load and the number of load repetitions is not taken as a variable. This procedure can easily be seen in the design of highways or airport pavements which are subjected to heavy wheel loads.

When using the fixed vehicle method, the pavement thickness depends on the number of load repetitions of a vehicle or axle load. This standard axle load is 18 kip (80 kN) single axle load.

Lastly, variable traffic and vehicle procedure as discussed by Huang (2004) involves the individual consideration of the traffic and vehicle. This eliminates the reliance on an equivalent factor to be applied to axle loads.

AASHTO (AASHTO, 1993) came up with a classification system in which vehicles are classified into four classes namely: passenger cars, buses, trucks and recreational vehicles. FHWA further subdivided these four classes into fifteen vehicle types based on their number of axle. This classification is shown in Table 5 below.

Table 5: FHWA Vehicle Classification Scheme “F” (FDOT, 2002)

CLASS GROUP		DESCRIPTION	NO. OF AXLES
1		MOTORCYCLES	2
2		ALL CARS CARS W/ 1-AXLE TRAILER CARS W/ 1-AXLE TRAILER	2 3 4
3		PICK-UPS & VANS 1 & 2 AXLE TRAILERS	2, 3 & 4
4		BUSES	2 & 3
5		2-AXLE, SINGLE UNIT	2
6		3-AXLE, SINGLE UNIT	3
7		4-AXLE, SINGLE UNIT	4
8		2-AXLE, TRACTOR, 1-AXLE TRAILER (2S1) 2-AXLE, TRACTOR, 2-AXLE TRAILER (2S2) 3-AXLE, TRACTOR, 1-AXLE TRAILER (3S1)	3 4 4
9		3-AXLE, TRACTOR, 2-AXLE TRAILER (3S2) 3-AXLE, TRUCK W/ 2-AXLE TRAILER	5 5
10		TRACTOR W/ SINGLE TRAILER	6 & 7
11		5-AXLE MULTI-TRAILER	5
12		6-AXLE MULTI-TRAILER	6
13	ANY 7 OR MORE AXLE		7 or more
14	NOT USED		
15	UNKNOWN VEHICLE TYPE		

(FDOT, 2002)

Equivalent Single Axle Load (ESAL)

This is a unit of measurement used for pavement thickness design. ESAL is a way of converting different vehicular traffic by means of an equivalent axle load factor (EALF) into an equivalent single-axle, 18-kip load. The equivalent effect of all axle loads is obtained by multiplying the number of passes of the different axle loads by EALF. Therefore, ESAL is described as the sum total of the equivalent effects of axle loads (single or multiple) that occurs in its design period.

The accumulated 18-kip (80kN) ESAL values during the design period are important for the pavement thickness design of new construction and reconstructed roads.

$$\mathrm{ESAL} = \sum_{i=1}^{m} F_i n_i \tag{4}$$

where m is the number of axle load groups and n_i is the number of passes of the ith axle load group during projected design period

$$\mathrm{ESAL} = (\mathrm{ADT})_0(T)(T_f)(G)(D)(L)(365)(Y) \tag{5}$$

where ADT is the Average Daily Truck Traffic

G is Growth factor

D is Directional distribution factor.

L is Lane distribution factor.

Y is Design period in years

Truck factor, T_f is the number of 18-kip (80 kN) single axle load applications per truck.

It can be expressed as:

$$T_f = (\sum_{i=1}^{m} p_i F_i)(A) \tag{6}$$

Average Daily Traffic (ADT)

Traffic is a vital entity in pavement design. According to AASHTO (1986), the ADT can be defined as the total volume of traffic at a given time period divided by the number of days in that time period. The average daily truck traffic (ADTT) is usually a percentage of the average daily traffic.

The Annual Average Daily traffic (AADT) is another important factor used in design. AADT can simply be defined as the total traffic volume on a section of a highway for one year, divided by the number of days in the year. This traffic volume could be obtained by adjusting a short term traffic count with weekly and monthly factors (AASHTO, 1986). In the absence of actual traffic data, the distribution of ADTT on different classes of highways in United States can be obtained.

Truck Factor

Single truck factor can be implemented on all trucks or different truck factors can be applied to various classes of trucks. Table 6 shows the computed truck factors for trucks with five or more axles.

Table 6: Computation of Truck Factors with Five or More Axles

Axle load (lb)	EALF	Number of axles	ESAL
	Single Axles		
Under 3000	0.0002	0	0.000
3000-6999	0.0050	1	0.005
7000-7999	0.0320	6	0.192
8000-11,999	0.0870	144	12.528
12,000-15,999	0.3600	16	5.760
16,000-29,999	5.3890	1	5.389
	Tandem Axles		
Under 6000	0.0100	0	0.000
6000-11,999	0.0100	14	0.140
12,000-17,999	0.0440	21	0.924
18,000-23,999	0.1480	44	6.512
24,000-29,999	0.4260	42	17.892
30,000-32,000	0.7530	44	33.132
32,001-32,500	0.8850	21	18.585
32,501-33,999	1.0020	101	101.202
34,000-35,999	1.2300	43	52.890
ESALs for all trucks weighed			255.151

Source: (Huang, 2004)

Growth Factors

According to Asphalt institute (1981a) and AASHTO design guide (AASHTO, 1986), total growth factor can be estimated by using the traffic over the design period of the pavement. Total growth factor is defined as growth factor multiplied by design period. It can be expressed as such:

$$\text{Total growth factor} = (G)(Y) = \frac{(1+r)^{Y} - 1}{r} \tag{7}$$

Total growth factor equation is shown in the Table 7 for different design periods and annual growth rate.

Table 7: Total Growth Factor

Design Period (years)	Annual growth rate (%)							
	No growth	2	4	5	6	7	8	10
1	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
2	2.0	2.02	2.04	2.05	2.06	2.07	2.08	2.10
3	3.0	3.06	3.12	3.15	3.18	3.21	3.25	3.31
4	4.0	4.12	4.25	4.31	4.37	4.44	4.51	4.64
5	5.0	5.20	5.42	5.53	5.64	5.75	5.87	6.11
6	6.0	6.31	6.63	6.80	6.98	7.15	7.34	7.72
7	7.0	7.43	7.90	8.14	8.39	8.65	8.92	9.49
8	8.0	8.58	9.21	9.55	9.90	10.26	10.64	11.44
9	9.0	9.75	10.58	11.03	11.49	11.98	12.49	13.58
10	10.0	10.95	12.01	12.58	13.18	13.82	14.49	15.94
11	11.0	12.17	13.49	14.21	14.97	15.78	16.65	18.53
12	12.0	13.41	15.03	15.92	16.87	17.89	18.98	21.38
13	13.0	14.68	16.63	17.71	18.88	20.14	21.50	24.52
14	14.0	15.97	18.29	19.16	21.01	22.55	24.21	27.97
15	15.0	17.29	20.02	21.58	23.28	25.13	27.15	31.77
16	16.0	18.64	21.82	23.66	25.67	27.89	30.32	35.95
17	17.0	20.01	23.70	25.84	28.21	30.84	33.75	40.55
18	18.0	21.41	25.65	28.13	30.91	34.00	37.45	45.60
19	19.0	22.84	27.67	30.54	33.76	37.38	41.45	51.16
20	20.0	24.30	29.78	33.06	36.79	41.00	45.76	57.28
25	25.0	32.03	41.65	47.73	54.86	63.25	73.11	98.35
30	30.0	40.57	56.08	66.44	79.06	94.46	113.28	164.49
35	35.0	49.99	73.65	90.32	111.43	138.24	172.32	271.02

Source: (Asphalt-Institute, 1981)

Lane distribution Factor

The design lane in a two-lane highway is the lane in either direction. While in a multilane highway, the design lane is usually the outside lane (Huang, 2004). This factor helps convert directional trucks to design lane trucks (FDOT, 2002). Asphalt institute (1981) provided the percentage of total truck traffic in design lane and this is shown in Table 8.

Table 8: Percentage of Total Truck Traffic in Design Lane

Number of traffic lanes in two directions	Percentage of trucks in design lane
2	50
4	45 (35 – 48)[a]
6 or more	40 (25 – 48)[a]

[a] Probable range

Source: (Asphalt-Institute, 1981)

This percentage is based on the total traffic. The AASHTO design manual (AASHTO, 1986) suggests the lane distribution factor (Table 9) whose values are strictly based on traffic in one direction.

Table 9: Lane Distribution Factor

Number of lanes in each direction	Percentage of 18-kip ESAL in design lane
1	100
2	80 – 100
3	60 – 80
4	50 – 75

Source: (AASHTO, 1993)

Flexible Pavement Design

The 1993 AASHTO design guide design equations were directly obtained from the in-depth American Association of State Highway Officials (AASHO) Road test at Ottawa, Illinois (Huang, 2004). This provided the basis for estimating the required pavement thickness (FDOT, 2008). Different models were created that tried to show the relationship between vehicular loading, pavement performance, pavement structure and strength of roadbed soils.

There are three different methods of designing a flexible pavement. These methods are: Calibrated Mechanistic design (also known as mechanistic-empirical process), Asphalt Institute method and AASHTO method. For this project we will base our design on the AASHTO method which is mostly used by State department of transportation across the U.S.

AASHTO METHOD

The main goal of this method is to develop a model and set of equations to obtain the structural number which is required for the estimation of the pavement layer thickness. The structural number (SN) is a measure of the structural strength of the pavement sections. This parameter is based on the layer type and thickness of the layer. The design variables to consider when using this method are discussed below:

Time Constraints

Correct estimation of analysis period is important. Therefore it is encouraged that the analysis period should be greater than the performance period. Performance Period describes the time frame at which the initial pavement structure remains durable before requiring rehabilitation. In other words, it can be defined as the time elapsed when a newly constructed, reconstructed or resurfaced pavement structure dilapidates from its initial serviceability condition to its terminal serviceability (Huang, 2004). Some factors that affect the choice of the performance period include; the level and type of maintenance applied, life cycle costs, classification of the pavement. Alternatively, the analysis period is the time period used as the design life of the pavement. It could be the same as the selected performance period. Longer analysis periods should be considered because they are required for the estimation of alternative long-term strategies based on life cycle cost (Huang, 2004). Guidelines to estimate the length of analysis period is shown in Table 10 below.

Table 10: Guidelines for Length of Analysis Period

Highway conditions	Analysis period (years)
High-volume urban	30-50
High-volume rural	20-50
Low-volume paved	15-25
Low-volume aggregate surface	10-20

Source: (AASHTO, 1993)

Accumulated 18-kip ESAL

As previously stated pavement design is based on the cumulative expected 18-kip (80kN) equivalent single axle load (ESAL).

Reliability (%R):

Reliability use in pavement design helps in achieving a level of certainty in the design procedure. This is the probability of assuring the Design Engineer that various design alternatives will endure throughout the analysis period. Factors to consider when selecting the level of reliability to be used in design include volume of traffic, problems in early rehabilitation especially if the actual traffic load is greater than it was projected. AASHTO (1986) provided the table below to estimate the level of reliability. It is important to note that the results were obtained from a survey of an AASHTO Pavement Design Task Force. The design engineer is very flexible in choosing the reliability value that best suits the project. The reliability values are not directly entered into the AASHTO Flexible Pavement Design. Instead Standard Normal Deviate (Z_R), which is a converted value, is used in its place. Table 11 shows the suggested reliability levels for different classifications.

Table 11: Suggested levels of reliability (%) for various functional classifications

	Recommended level of reliability	
Functional Classification	Urban	Rural
Interstate and other freeways	85-99.9	80-99.9
Principal arterials	80-99	75-95
Collectors	80-95	75-95
Local	50-80	50-80

Source: (AASHTO, 1993)

Standard Normal Deviate (Z_R)

This is a normal deviate that corresponds to a given reliability (%R) value. It is the logarithmic form of the reliability values for ease of calculation. Table 12 shows the standard normal deviate value (Z_R) for the corresponding reliability (%) value.

Table 12: Standard Normal Deviates for Various Levels of Reliability

Reliability (%)	Standard normal deviate (Z_R)	Reliability (%)	Standard normal deviate (Z_R)
50	0.000	93	-1.476
60	-0.253	94	-1.555
70	-0.524	95	-1.645
75	-0.674	96	-1.751
80	-0.841	97	-1.881
85	-1.037	98	-2.054
90	-1.282	99	-2.327
91	-1.340	99.9	-3.090
92	-1.405	99.99	-3.750

Source: (Huang, 2004)

Resilient Modulus (M_R)

All materials experience some measure of deformation (strain) and are also subjected to stress (load per unit area). Failure occurs when the stress exceeds the strength of the material. Resilient modulus of a material is an estimate of its modulus of elasticity. Modulus of elasticity is ratio of stress to strain for a slowly applied load while resilient modulus is the ratio of stress to strain for a rapidly applied load (WSDOT, 2004). Therefore M_R can be defined as elastic modulus based on recoverable strain under repetitive applied loads (Huang, 2004). Using the FWD equipment, the elastic modulus of the pavement layers in different sections will be obtained by using a simple formulation specified by AASHTO (1993) but modified to suit Florida conditions. Boussinesq's theory of a concentrated load applied on an elastic half-space. This could be expressed as:

$$M_R = \frac{0.24 * P}{D_{36} * r} \tag{8}$$

where:

M_R=Resilient Modulus of subgrade (psi)

P = Applied load (lb)

r = Radial distance at which the deflection is measured (inches)

D_{36} = Deflection measured at that radial distance r (inches)

Finally, Holzschuher, et al (2007) redefined the resilient modulus (M_R) as the mean value plus two standard deviations. Equation (9) was revised to (Holzschuher, et al., 2007):

$$M_R = \frac{0.24 * P}{(D_{36} + 2 * \sigma_{D36}) * r} \quad (9)$$

These M_R values will be used in the pavement and layer thickness design in the present work. There are some constants used in the AASHTO Design equation and are listed below (Huang, 2004).

Present Serviceability Index (PSI):

PSI can be defined as the ability of a roadway to serve the anticipated traffic. This is a measure of serviceability and is usually rated on a scale of 0 to 5. On this scale, zero (0) means the existing roadway is poor and impossible to drive on and 5 being the best condition for driving.

Initial Serviceability Index (PI)

This index is a function of the type of pavement. It could be referred to as the quality of construction of a newly constructed roadway. From the AASHO Road test, a typical value of 4.2 is assumed for flexible pavements and 4.5 for rigid pavements.

Terminal Serviceability (PT)

This is the lowest index that the condition of a roadway will attain before being considered for reconstruction or rehabilitation. A typical value of 2.5 is assumed for major highways with high traffic, while 2.0 is for highways with low traffic volume. For this project we will assume a PT value of 2.0.

Change in Serviceability (ΔPSI)

After the assumptions of the P_I and PT values the ΔPSI will need to be calculated and entered into the AASHTO design equation. The ΔPSI is the difference between the initial and terminal serviceability.

Standard Deviation (So)

This is a constant statistical value used to account for variations in predicted traffic loadings and construction process. S_o value of 0.45 is used in this project to take care of these errors. The unknowns which are calculated are the Required Structural Number (SN) and the layer thickness. These are important terms that will further provide an insight into the strength properties of pervious pavements.

Structural number (SN)

According to FDOT (2008), the SN value is a weighted thickness in inches, which is estimated from traffic load data and the stiffness of roadbed soils. In addition, this parameter depends on layer thicknesses, layer coefficients and drainage coefficients. The layer coefficient, denoted by a_i is the ability of a unit thickness of layer to act as a structural entity of the pavement.

The original AASHTO design equation obtained from the AASHO Road Test was modified to take into consideration other regions in U.S. The modified AASHTO Flexible Design equation can be presented as follows:

$$\log W_{18} = Z_R S_o + 9.36\log(SN + 1) - 0.20 + \frac{\log[\Delta PSI/(4.2 - 1.5)]}{0.4 + 1094/(SN + 1)^{5.19}} + 2.32\log M_R - 8.07 \tag{10}$$

Where W_{18} = 18 kip ESAL load

M_R = Resilient Modulus

ΔPSI = Change in Serviceability

Z_R = Standard Normal Deviate

SN = Structural Number

S_o = Standard Deviation

The Structural Number can now be expressed as:

$$SN = a_1 D_1 + a_2 D_2 + a_3 D_3 \tag{11}$$

where a_1, a_2, a_3 are the layer coefficients for the different layers such as surface, base and subbase and D_1, D_2, D_3 are the depth of the pavement layers. This SN expression was later modified to take into consideration local precipitation and drainage conditions. The modified SN equation is:

$$SN = a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3 \tag{12}$$

where m_2 is the drainage coefficient of base course

m_3 is the drainage coefficient of subbase course.

For the purpose of this research, the drainage coefficients, m_2 and $m_{3,}$ will be taken as 1.

Minimum Layer Thickness selection

After the determination of the required structural number (SN), a set of layer thicknesses are selected that provide a SN value greater than the required SN. Cost implications must be considered when choosing the set of layer thicknesses. For the top layer, the minimum layer is computed as:

$$D_1 \geq \frac{SN_1}{a_1} \tag{13}$$

For the next layer, that is base course, the layer thickness will be:

$$D_2 \geq \frac{SN_2 - a_1 D_1}{a_2 m_2} \tag{14}$$

The third layer thickness can be obtained as such:

$$D_3 \geq \frac{SN_3 - a_1 D_1 - a_2 D_2 m_2}{a_3 m_3} \tag{15}$$

This design procedure is used for flexible pavement design for porous asphalt and recycled tire pavement.

Rigid Pavement Design

This pavement type has three layer systems; surface course, base or subbase course and the subgrade. The surface layer is constructed of Portland cement concrete and the base or subbase layer is composed of granular materials. This pavement type should be analyzed by means of plate theory rather than layered theory (Huang, 2004). Plate theory is a simplified form of layered theory which takes the PCC slab as a medium thick plate with a plane before and after bending. Huang (2004) further stated that the reason for the choice of plate theory instead of layered theory is because the concrete slab has a higher stiffness than HMA and it has a wider load distribution range.

Initially, rigid pavements were constructed with the PCC slabs being placed over the subgrade but with increase in traffic loads and volume, pumping occurred more frequently, thereby resulting in the introduction of base course. The granular base layer helps to decrease the critical stress in the PCC slab and also control pumping action. Pumping is referred to as the removal of water and subgrade soil through joints and cracks caused by the downward movement of the concrete slab as a result of heavier traffic loads (Huang, 2004).

There are two (2) known methods for designing rigid pavements. The methods are Portland cement association (PCA) method and AASHTO method. But we will only focus on the AASHTO design method in this research as it is most commonly used in transportation applications.

AASHTO Method

AASHTO (1993) produced empirical equations obtained from the same AASHO Road Test in Illinois, like those done for flexible pavements. This method focuses on calculating pavement thickness only. Some design variables are common to both flexible and rigid pavement. Other new parameters to consider when using this method are described below

Modulus of Subgrade Reaction (k)

This is a property of the subgrade soil used to design the thickness of rigid pavement instead of the resilient modulus. As mentioned by FDOT (2009), this is a roadbed hypothetical elastic sping support for the PCC slab.

Modulus of Elasticity of Concrete

This is also known as the Young's modulus or the ratio of stress to strain. The elastic modulus of concrete can be obtained by using the relationship recommended by American Concrete Institute (ACI) (Huang, 2004) .

$$E_c = 57{,}000(f_c^{'})^{0.5} \tag{16}$$

Where

E_c is the concrete elastic modulus in psi

f'_c is Compressive strength of concrete in psi (From ASTM C39)

Modulus of Rupture of Concrete (S'_c)

The modulus of rupture of concrete is the average value of 28-day flexural strength obtained by means of third point loading. This test is conducted in accordance with ASTM C78. The value used in the current design is 500 psi.

Load Transfer Coefficient (J)

The load transfer factor (J) is the ability of a rigid pavement (concrete) to transfer load across its joints and cracks. Table 13 shows the recommended load transfer coefficient for various pavement types and design conditions.

Table 13: Recommended Load transfer coefficient for various pavement types and design conditions

Type of shoulder	Asphalt		Tied PCC	
Load transfer devices	Yes	No	Yes	No
JPCP and JRCP	3.2	3.8-4.4	2.5-3.1	3.6-4.2
CRCP	2.9-3.2	N/A	2.3-2.9	N/A

Source: (AASHTO, 1993)

Drainage coefficient (C_D)

This factor is defined as the ability of a pavement to drain out over a period of 1 hour to 72 hours (FDOT, 2009) . Table 14 shows recommended values of the drainage coefficients for rigid pavements.

Table 14: Recommended values of drainage coefficients C_d for Rigid Pavements

Quality of drainage		Percentage of time pavement structure is exposed to moisture levels approaching saturation			
		Less than			Greater than
Rating	Water removed within	1%	1-5%	5-25%	25%
Excellent	2 hours	1.25-1.20	1.20-1.15	1.15-1.10	1.10
Good	1 day	1.20-1.15	1.15-1.10	1.10-1.00	1.00
Fair	1 week	1.15-1.10	1.10-1.00	1.00-0.90	0.90
Poor	1 month	1.10-1.00	1.00-0.90	0.90-0.80	0.80
Very Poor	Never drain	1.00-0.90	0.90-0.80	0.80-0.70	0.70

Source: (AASHTO, 1993)

The required depth (D_R) of the concrete slab is estimated using this method by using traffic load data and subgrade strength. This shows a true reflection of the strength of the rigid pavement structure (FDOT, 2009).

Like the flexible pavements, the design equation for rigid pavement was developed from the AASHO Road Test and it was modified to take into consideration some conditions that were not considered earlier.

$$\log W_{18} = Z_R S_o + 7.35\log(D+1) - 0.06 + \frac{\log[\Delta PSI/(4.5-1.5)]}{1 + 1.624x10^{7}/(D+1)^{8.46}} + (4.22 - 0.32p_t)\log\left\{\frac{S_c C_d (D^{0.75} - 1.132)}{215.63J[D^{0.75} - 18.42/(E_c/k)^{0.25}]}\right\} \tag{17}$$

This leads to the design of pervious concrete, recycled glass, porous aggregate pavements. This design procedure will be used for these pavements.

METHODOLOGY

Introduction

This chapter details the procedures used in producing and testing the various pervious pavements. The pavements to be tested are pervious concrete, pervious asphalt, pervious pavers, recycled rubber tires, recycled glass pavements, pervious aggregate, regular asphalt concrete and conventional concrete.

Compressive strength and modulus of rupture can easily be evaluated by subjecting test samples to loadings until failure occurs. A correlation between cast in place pervious concrete and cored samples from existing parking lots will be developed.

Falling weight deflectometer (FWD) test procedures and subsequent calculation programs are discussed in this chapter. Elastic moduli, deflections, stress and strains of the different layers of each pavement are obtained from this analysis.

Traffic loadings and volumes are estimated for existing sites. Actual traffic counts for these sites are the most accurate form of data, but, unfortunately, time constraint does not permit the collection of this data. Transportation charts are used to make estimates of traffic volumes and loadings. The FPS 19 program with the aid of FWD data are used to determine the surface and base layer thicknesses required to carry projected traffic load. The values obtained from calculations using transportation charts are compared to those obtained from the FPS 19 program.

Cylinders and beams used for testing

Cylinders and beams used for compressive and flexural strength testing are made for one time use only. Pervious concrete samples were made from 3/8 inch aggregate, water and Type I Portland Cement. The pervious concrete mixture and the cylindrical samples for testing are in accordance with ASTM C31/C31M-03a.

The pervious concrete was placed in cylinders, then the surface was leveled, afterwards a 6mil thick polyethylene plastic covering was placed over each cylindrical sample for proper curing. Ten (10) eight inches depth and 4 inches diameter pervious concrete cylinders were cast. In addition, five (5) pervious concrete beams of 20" by 6" by 6", needed to carry out flexural strength test, were placed in beam molds and the same polyethylene material was used as a covering.

Curing was done to simulate external conditions. Visual inspection of the pervious concrete mix was used to measure the consistency since no standard method exists to measure the consistency of pervious concrete. Though, this research does not take into consideration the effect of the proprietary mix ratio on the strength parameters. The mix design of the concrete samples was provided by the manufacturers.

After seven days had elapsed, the cylindrical molds were removed from ten (10) pervious concrete samples and the beam molds were removed from the five (5) beam samples. These fifteen (15) pervious concrete samples were then wrapped with the 6 mil thick polyethylene plastic. Compressive strength test were conducted on three eight (8) by four (4) inches cylinders on the 7^{th} day after casting, while the remaining seven (7) cylinders and five (5) beams remained in the plastic confines for three more weeks. After

28 days of curing, the polyethylene plastic was removed from all the beams and the remaining cylinders and each sample was weighed.

Porosity and void ratio experiments and calculations were also performed on the seven (7) pervious concrete cylinders. The mix design, as provided by the manufacturer, for the test cell P.C sample is shown in Table 15 below

Table 15: Manufacturer's Pervious Concrete mix design

Material	Source	Desription	ASTM	Specific Gravity	Weight (lbs/cy)
Cement	American Cement	Type I Cement	C-150	3.15	650
Fine Agg.			C-33	2.63	0
Course Agg.	Martin Marietta Materials	# 89 Bahama Rock	C-33	2.40	2240
Water	Well		C-94	1.00	225
Admix 1					
Admix 2					
Admix 3					
NOTES:				Totals	3115
Designed Slump: 1.0"+/- 1.0"		Designed Unit Weight: 115.4 lbs/cu.ft.			
Designed Air:		Designed W/C Ratio: 0.35			

Seventeen (17) pervious concrete cored cylindrical samples with an average depth of 7.4 inches in depth and 3.7 inches in diameter were tested. These samples were cored from our research site in the SMART lab.

Flexi-pave samples with the old proprietary mix design were initially tested. Four (4) cylinders with two different aggregate gradations were tested. Two (2) samples each of HD 2000 with #89 granite and XFP75 urethane and two (2) samples of HD 2000 with #7 granite and XFP75 urethane. The dimension of the cylinders is 2" depth and 6" diameter. The 2 inch thickness is not a standard size for cylinders used in compressive strength test but it was used to replicate the actual thickness of the pavement on site. Six (6) beams, with three each for the different aggregate size, were tested but the results of the test carried out on these samples were not reported in this research. The new flexi-pave mix provided by the manufacturer of Flexi-Pave® was also tested. The difference between the old and new mix design lies in the binding agent used. For the new Flexi-Pave® samples, HDX 6000 Urethane was used. Six (6) cylinders of eight inches depth and 4 inches diameter were tested and flexural test on six (6) beams was also carried out. The results of these testing procedures for the new samples were reported.

The strength parameters of filter-pave® (Presto-Geosystems, 2009) samples were also evaluated. Five (5) 12in. x 6in. cylindrical cast in-situ samples with #89 granite and urethane binding agent and five (5) 8in. x 4in. cylindrical cast in-situ samples with #89 granite and urethane binding agent were used to carry out compressive strength testing. In addition, eight (8) 20in. x 6in. x 6in. beam cast in-situ samples with #89 granite and urethane binding agent were tested.

Pervious aggregate is comprised of 4” layer of 1/8” - 1/4” rock and two component polymer binder (polyurethane) coating to hold the gravels. Four (4) 12in. x 6in. cylindrical cast in-situ samples with #89 granite and urethane binding agent. Four (4) 8in. x 4in. cylindrical cast in-situ samples with #89 granite and urethane binding agent. One (1) 20in. x 6in. x 6in. beam cast in-situ samples with #89 granite and urethane binding agent.

Porosity and void ratio

Porosity and void ratio tests are conducted to obtain the amount of pore spaces in each cylindrical sample before they are tested for compression. The method used was that of weight of water displaced. This is in accordance with Archimedes principle and ASTM C29/29M-97. The volume of the cylindrical samples is calculated as V_T. A five-gallon bucket was filled with water up to a certain level and its initial depth was recorded as h_1. The cylinder was then gently placed in the container and then the final water level was recorded as h_2. The change in water level was recorded as ΔH. The volume of the solid displaced (V_s) was calculated with the aid of a dimensional mathematical equation developed for the five gallon container, as follows

$$V_s = 0.3904\left(\frac{\Delta H}{7.481}\right)12^3 \tag{18}$$

The volume of voids (V_V) is calculated by subtracting V_S from V_T. Subsequently, the void ratio (e) is determined by dividing V_V by V_S (V_V / V_S) and porosity (n) is calculated by dividing V_V by V_T (V_V / V_T).

Compressive strength testing

Compressive strength test was conducted in accordance with ASTM C 39. After 28 days the cylinders were crushed by means of a 1MN SATEC Universal Testing Machine. Neoprene cap was placed at the top and bottom of each cylinder before testing. This test was a stress based test, where each sample was loaded at a rate of 35 psi/sec until fail occurred. The data obtained was recorded as applied load (in pounds) and displacement (in inches).

Flexural strength testing

Flexural strength test was conducted in accordance with ASTM C78-02. This test was performed using the SATEC 1MN load cell. After the 28 day curing period, the beams were placed on a flexural attachment which has two nose load applying points and two bottom supports blocks. The loading rate was calculated using the formula:

$$r = \frac{S.b.d^2}{L} \tag{19}$$

This test was carried out with a loading rate of 4500lb/min still failure occurred. The modulus of rupture was measured from this flexural strength test.

Installed Pavement Systems

Twenty three (23) pervious pavement sections and one impervious pavement section with zero (0%) slope were built at the research academy known as the Stormwater Management Academy Research Testing Laboratory (SMARTL) at the University of Central Florida (UCF) to carry out various field testing. These porous pavements include porous asphalt, porous concrete, recycled glass pavements (Filterpave®), recycled rubber tire (Flexi-Pave®), permeable interlocking concrete pavers, and porous aggregate (Firmapave®). Some research tests being carried out include embedded ring infiltrometer test, water quality and the effect of rejuvenation using vacuum sweeping.

Each of the pervious pavement sections has one 4” PVC pipe (perforated at the top) in the subgrade and filter fabric between the base/subbase and the subgrade to collect water quality samples. All the installed pervious pavement sections are surrounded by concrete curbs. Figure 1 presents a plan view of all the pervious pavement sections installed at SMARTL

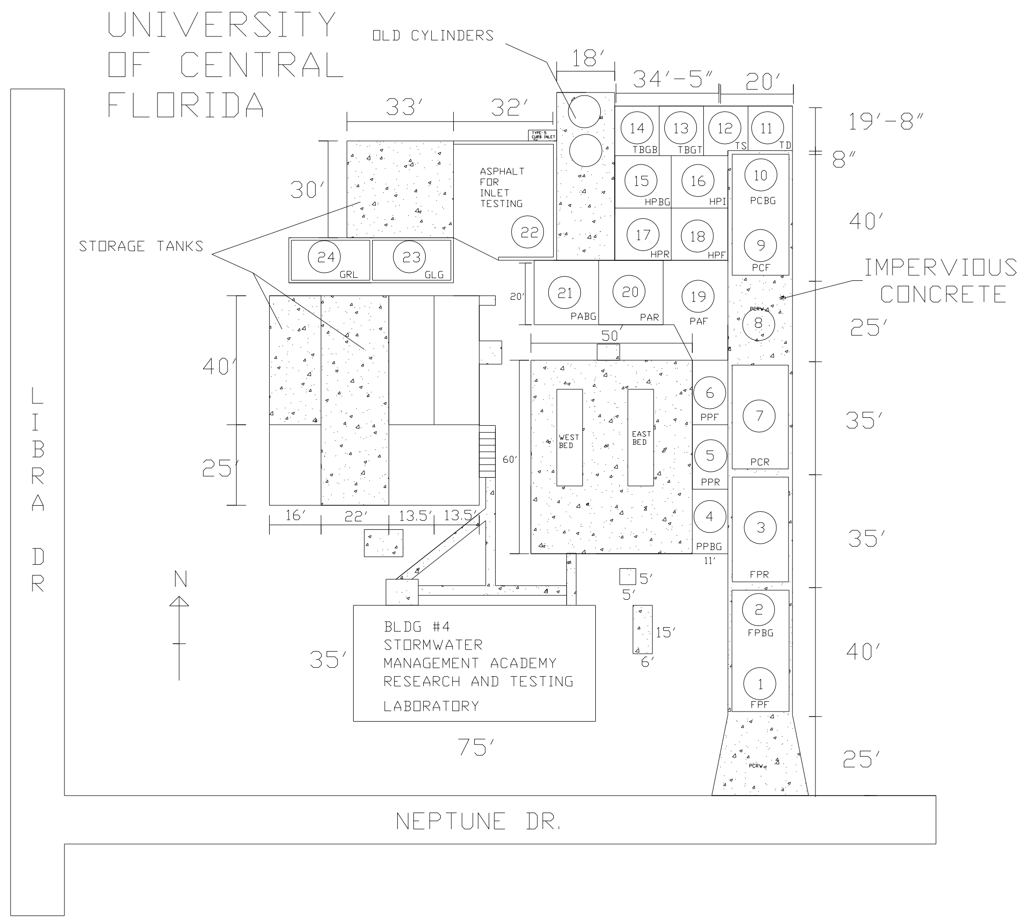


Figure 1: Plan view of pervious pavements sections at SMARTL

Pervious concrete

The thickness of the surface course for each of the three sections is 6”. Each section was named according to its use or constituents. The sections are Pervious Concrete rejuvenation (PCR), Pervious Concrete BOLD & GOLD™ (PCBG) and

Pervious Concrete Fill (PCF). The length of PCR is 35' and its width is 20'. PCBG and PCF have a combined length of 40' and a width of 20'.

In addition, ERIK (embedded ring infiltrometer kit) devices were buried in each section for infiltration rate monitoring and 4" perforated top PVC pipe were buried in the subgrade of each section for water quality sampling.

A pollution control media known as BOLD AND GOLD™ was used mainly to reduce phosphorus load (Hardin, 2006). These sections have only two layers above the compacted subgrade. The section that was used to study the rejuvenation potential of pervious concrete had a 10" layer of BOLD & GOLD™ with coarse sand as its base course. The subgrade was made up of well compacted (92-95% of Modified proctor) (ASTM, 2002) A-3 soils. A filter fabric material is placed between the base course and the subgrade to reduce the flow of some contaminants into the groundwater. Also, PCBG had 10" thick BOLD & GOLD™ as its base course. 10" fill sand was used as the base course of PCF.

The control impervious concrete section was cast in order to compare the results obtained from tests on conventional concrete with of the results from pervious concrete. The dimension of the impervious section is 25' long and 20' wide. The cross-section of PCR, conventional concrete control, PCF and PCBG are presented in Figures 2, 3, 4 and 5 respectively.

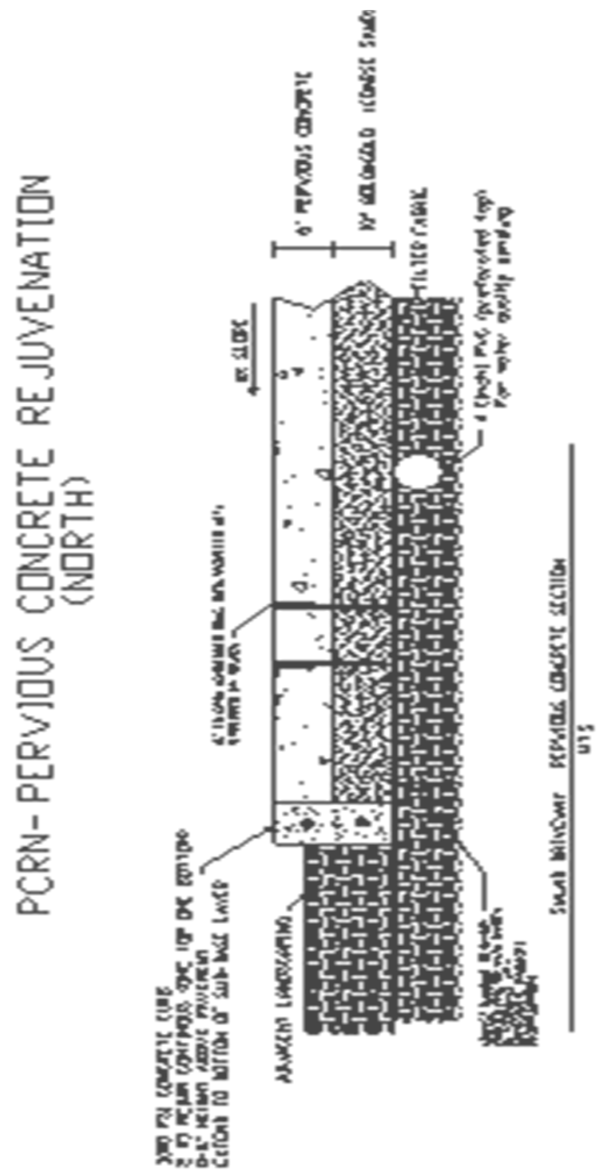


Figure 2: Cross-sectional view of Pervious Concrete Rejuvenation (PCR)

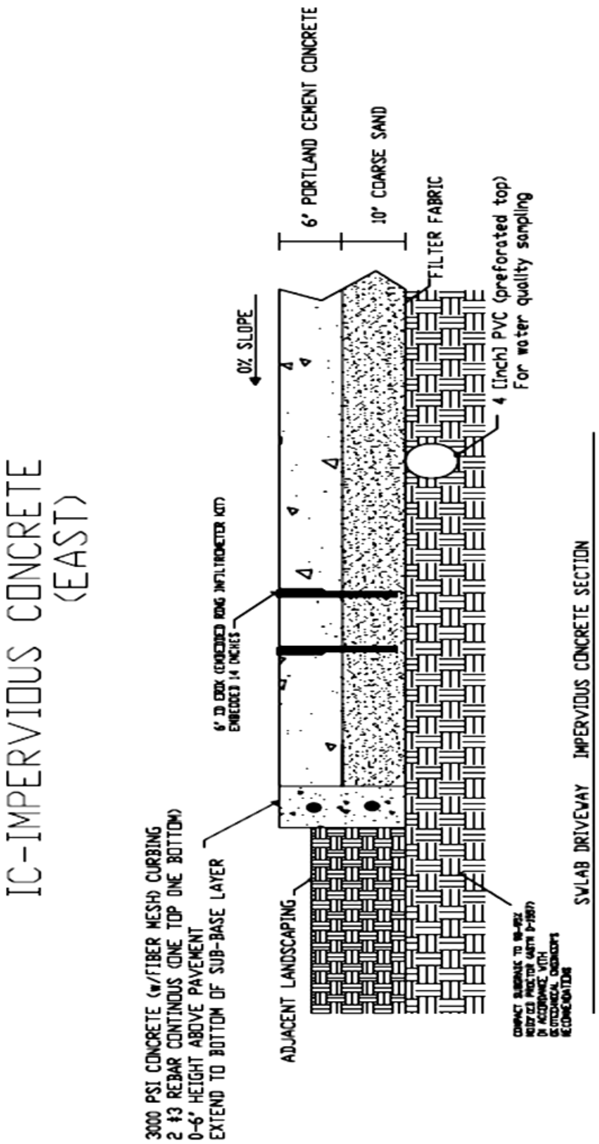


Figure 3: Cross-sectional view of Conventional Impervious Concrete

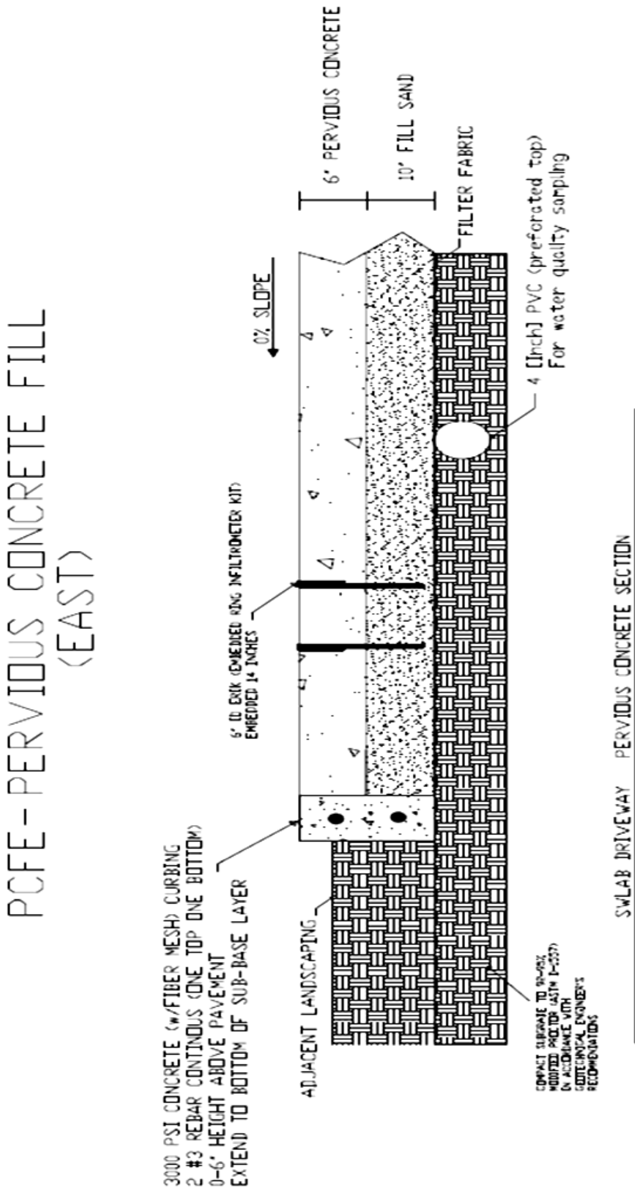


Figure 4: Cross-sectional view of Pervious Concrete Fill (PCF)

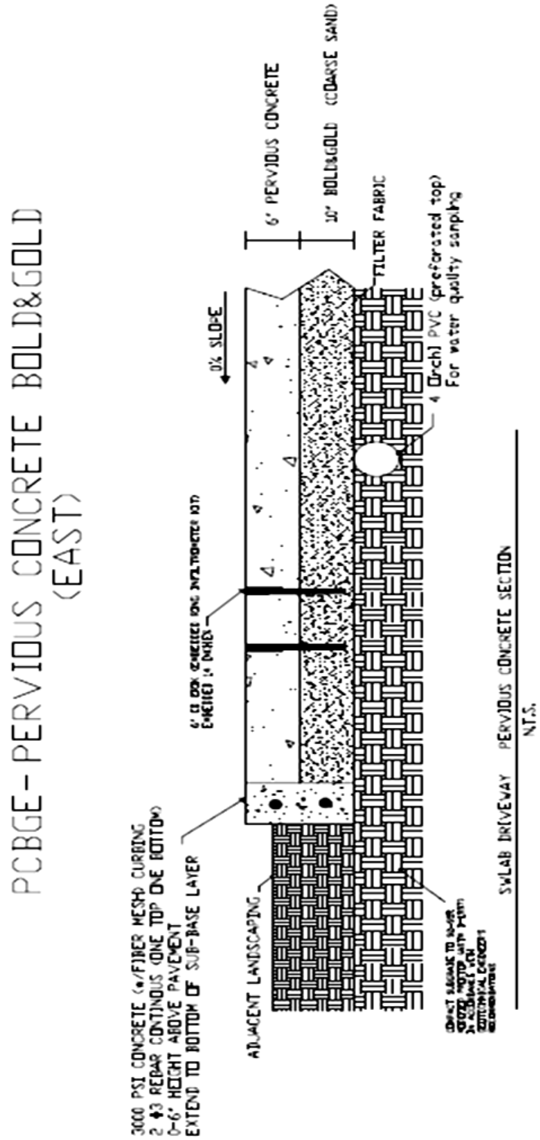


Figure 5: Cross-sectional view of Pervious Concrete Bold & Gold (PCBG)

Porous Asphalt

Three (3) sections of porous asphalt were constructed. Like pervious concrete, ERIK devices were installed vertically in each of the sections. They were laid side by side and have a common width of about 21’ and a length of about 23’. The names of these sections are Porous asphalt fill (PAF), Porous asphalt rejuvenation (PAR), Porous asphalt BOLD & GOLD™ (PABG).

PAF and PAR have the same layer constituents, but PAR is dedicated to rejuvenation studies. The surface course is made of 4” porous asphalt. Furthermore, 4” deep No. 57 limestone was used as base course and 8” thick well compacted A-3 soil fill was used as the subbase above the subgrade. While PABG had the same thickness and material of surface course and base course, but it had 8” thick subbase composed of BOLD & GOLD™ and fill sand. The cross-sections of PAF, PAR, PABG and conventional asphalt are presented in Figures 6, 7, 8 and 9 respectively.

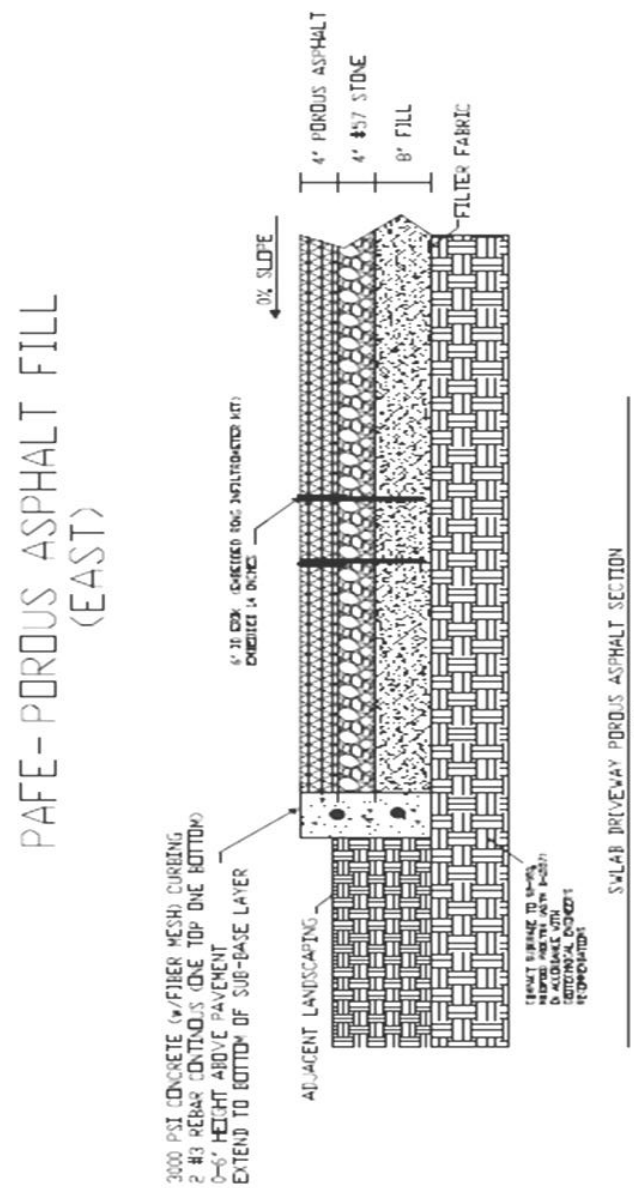


Figure 6: Cross-sectional view of Pervious Asphalt Fill (PAF)

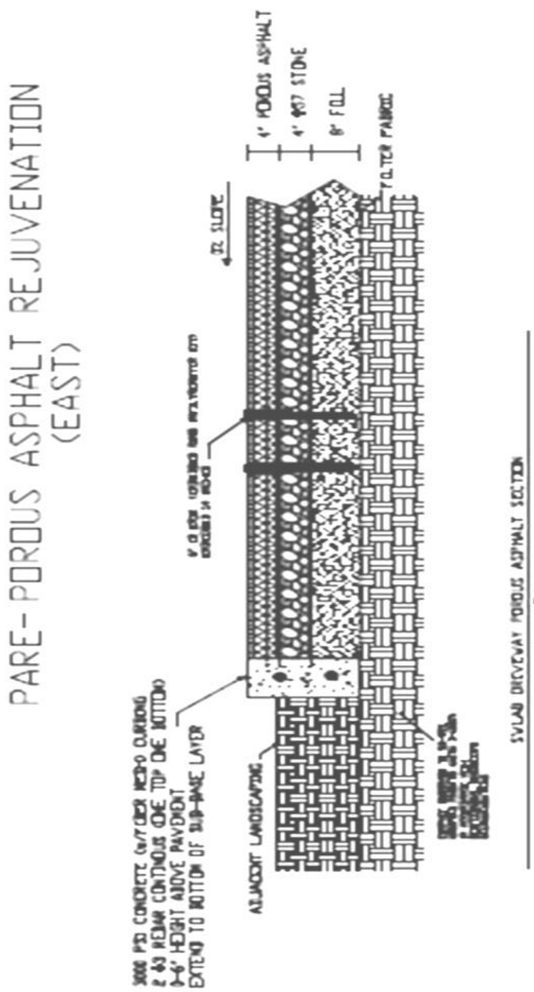


Figure 7: Cross-sectional view of Pervious Asphalt Rejuvenation (PAR)

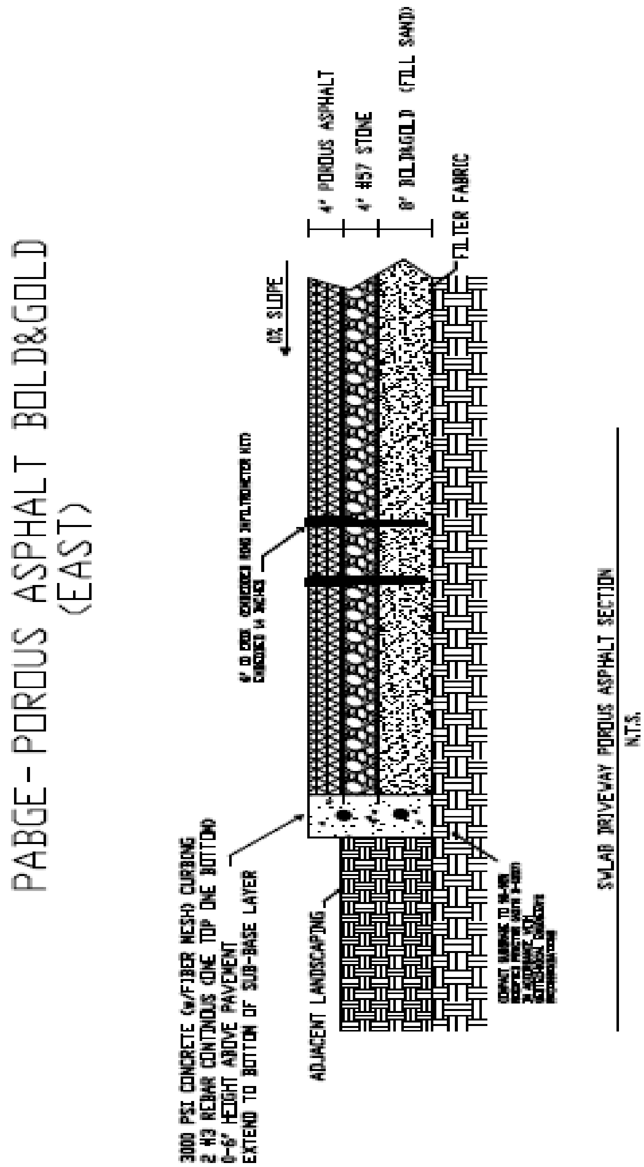


Figure 8: Cross-sectional view of Pervious Asphalt Bold & Gold (PABG)

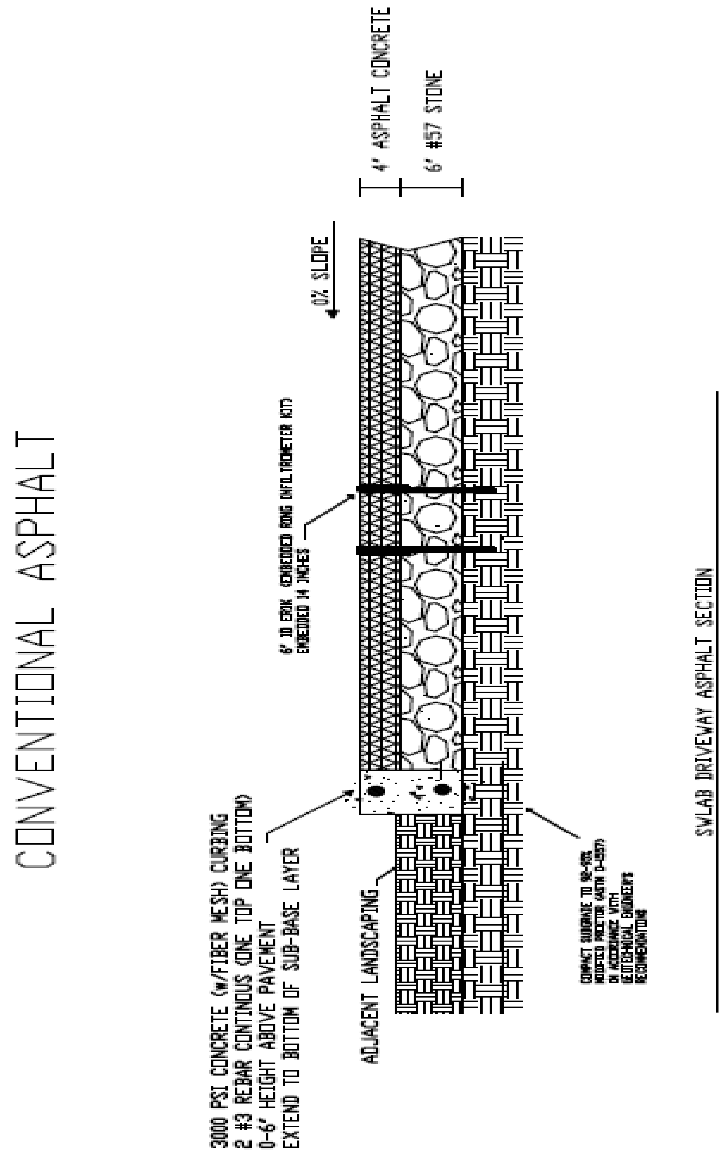


Figure 9: Cross-sectional view of Conventional Asphalt inlet

Permeable Pavers

At the academy field laboratory, we have installed three different types of interlocking concrete pavers. Each of them was made by different manufacturers and they each differ from each other in structure, shape and joint type. The earliest installed permeable paver sections have three sections with dimensions of 20’ long and 12’ wide. These three permeable interlocking concrete pavement systems are named Permeable pavers BOLD & GOLD™ (PPBG), permeable pavers rejuvenation (PPR) and permeable pavers fill (PPF).

The surface course of PPBG has a depth of 3” whose joints are filled with #89 limestone (Figure 10). The bedding course has 2” depth of #89 limestone, 4” of #57 limestone and 5” of #4 limestones as base and subbase. A 2” BOLD & GOLD ™ media with coarse sand were laid above the compacted subgrade. Both PPR (Figure 11) & PPF (Figure 12) have the same layer orientation. The surface course has a 3” concrete paver (with #89 limestone), 2” #89 limestone bedding course, 4” #57 granite and 7” #4 granite base and subbase course.

The other manufacturer of permeable paver used the aquaflow system. Each concrete aquaflow paver are 100mm wide (4”) X 200mm long (8”) X 80mm thick (3.13”). Four sections of this product were constructed in the academy. They are labeled HPBG (H-Paver BOLD & GOLD™) (Figure 13) and HPI (H-Paver with pollution control fabric called Inbitex) (Figure14), HPR (H-Paver rejuvenation) (Figure 15), HPF (H-Paver fill). These sections occupy an area of about 480 sqft. HPR and HPR have the same nomenclature.

The depth of the surface course is about 3”, the bedding course of #89 limestone is 2” deep, 4” #57 limestone and 7” #4 limestone are provided as the base and subbase course. HPI has the same layer arrangement as HPF and HPR, just except that Inbitex is placed between the bedding course and the base. HPBG has about 3” deep concrete paver as surface layer, 2” bedding course of #89 limestone; 4” #57 limestone, 5” #4 limestone and 2” BOLD & GOLD™ (with coarse sand) as base and subbase course.

The third manufacturer of the interlocking concrete paver is called TREMRON, and they installed four systems namely; TBGB (T-paver BOLD & GOLD™ bottom) (Figure 19), TBGT (T-paver BOLD & GOLD™ top) (Figure 18), TS (T-shallow) (Figure16), TD (T-deep) (Figure 17). The dimension of the concrete paver is 9.6” long x 5” wide x 3.13” thick. These sections covered about 1180 sq ft. The main issue with the paver was the manner in which it was laid. The contact area between the pavers was small. Wide gaps (filled with #89 limestone) were evident between paver. This aggregate washed out easily during rain events or when vacuumed.

TBGB system consist of about 3.13” deep concrete paver, 2” aggregate bedding course of #89 limestone, a layer of non woven filter fabric, 2” layer of drainage cell with BOLD & GOLD, another layer of non woven filter fabric, open-spaced plastic mini rain tank of 9.5” depth as support, then another layer of the filter fabric, compacted A-3 soil as subgrade. TGBT system is similar to that of TBGB except that the 3” deep BOLD & GOLD media is laid below the mini rain tank. Furthermore, the TS system is a shallow layered system with shallow ERIK pipes. It has about 3.13” of concrete paver, 2” thick #89 aggregate bedding, layer of 4-oz non woven geotextile filter fabric, 2” drainage cell, and filter fabric layer above the compacted subgrade.

Finally, the TD system is a deep layered system with deep ERIK pipes. This section is being used to carry out rejuvenation monitoring. It has the same layer orientation as the TS section, but unlike the TS, below the second layer filter fabric, 10” layer of Fine aggregate (sand with < 5% fines) with 2" drainage cells at 16" spacing as vertical drains, layer of geotextile fabric, 9.5" mini rain tank, another layer of geotextile fabric above the compacted subgrade.

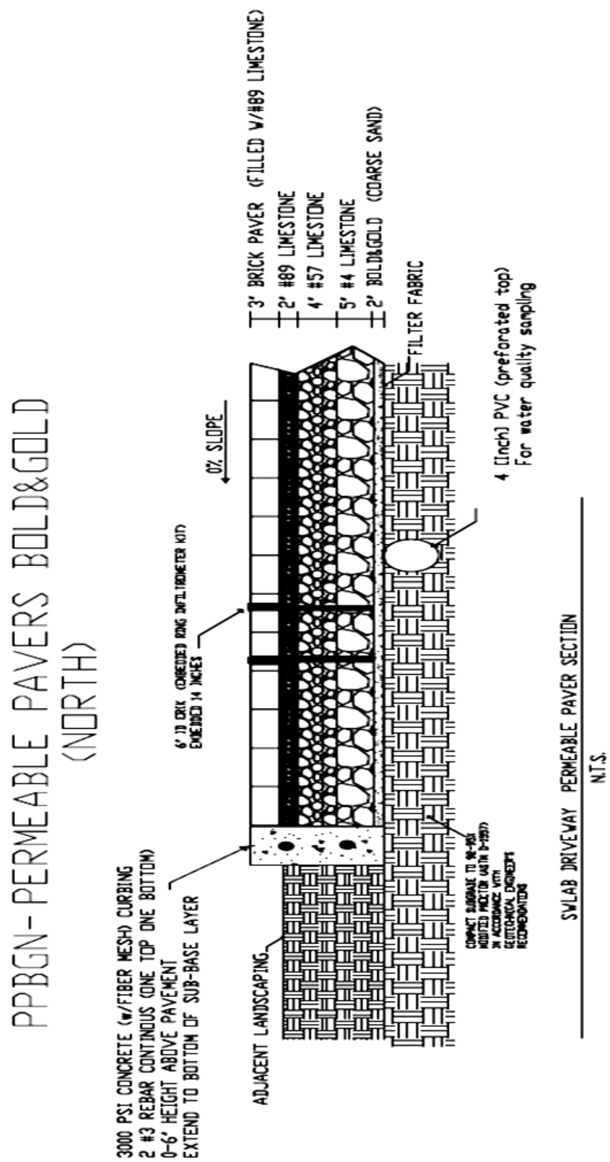


Figure 10: Cross-sectional view of Permeable Pavers Bold & Gold (PPBG)

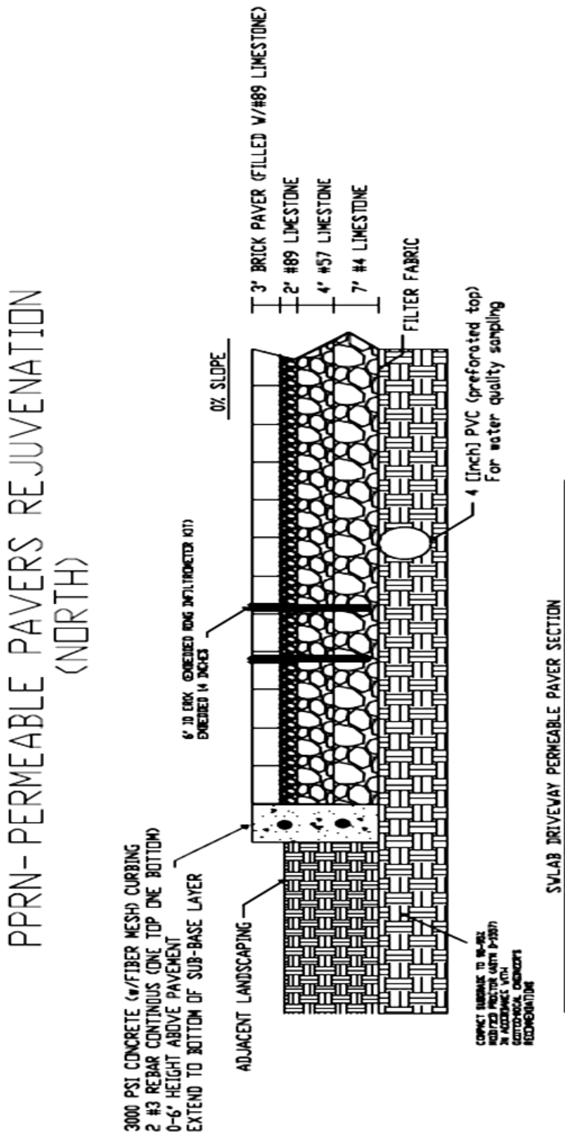


Figure 11: Cross-sectional view of Permeable Pavers Rejuvenation (PPR)

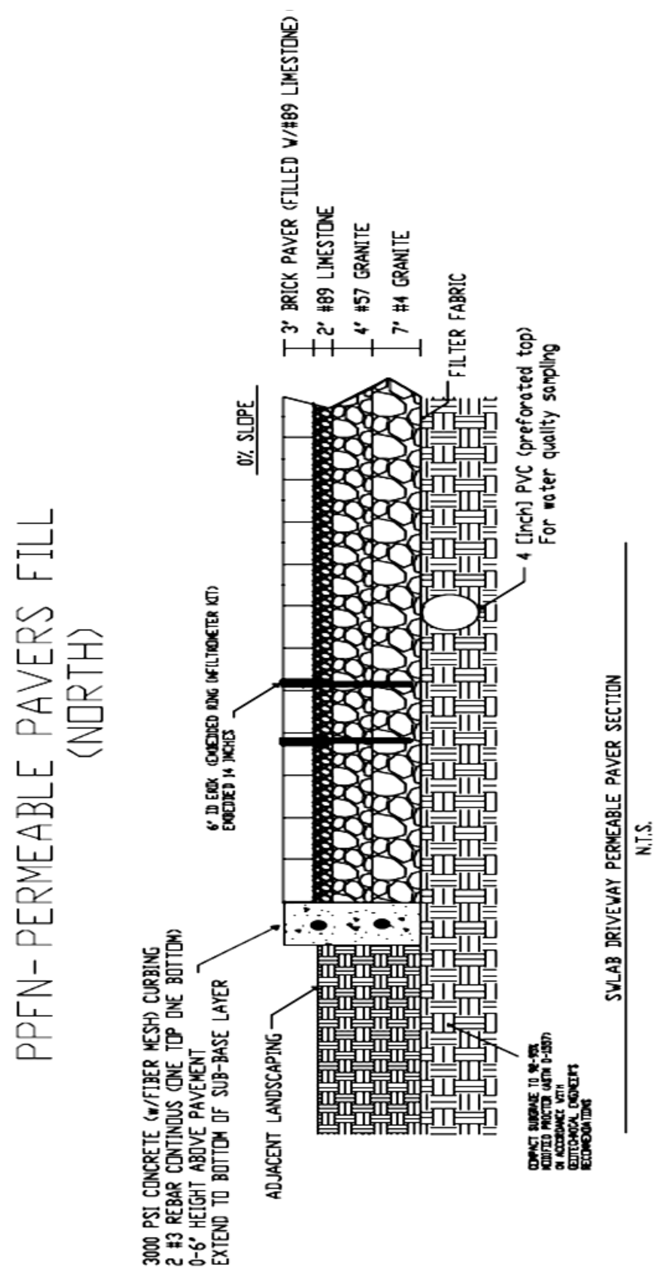


Figure 12: Cross-sectional view of Permeable Pavers Fill (PPF)

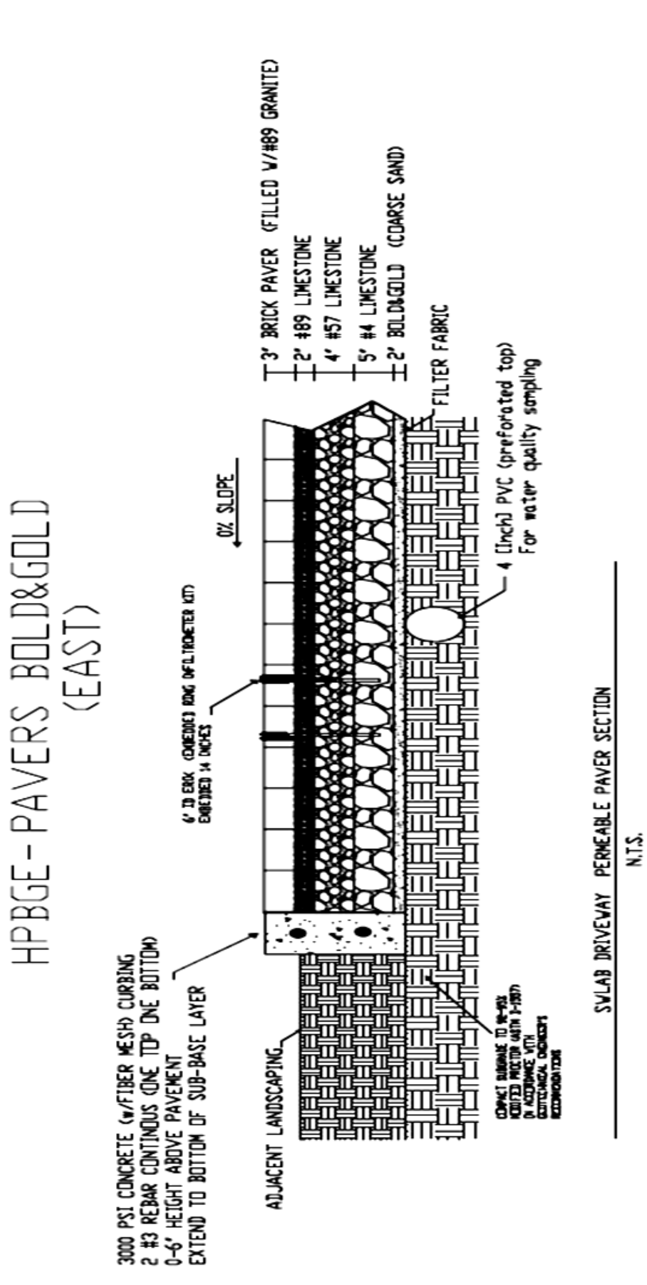


Figure 13: Cross-sectional view of Hanson Pavers Bold & Gold (HPBG)

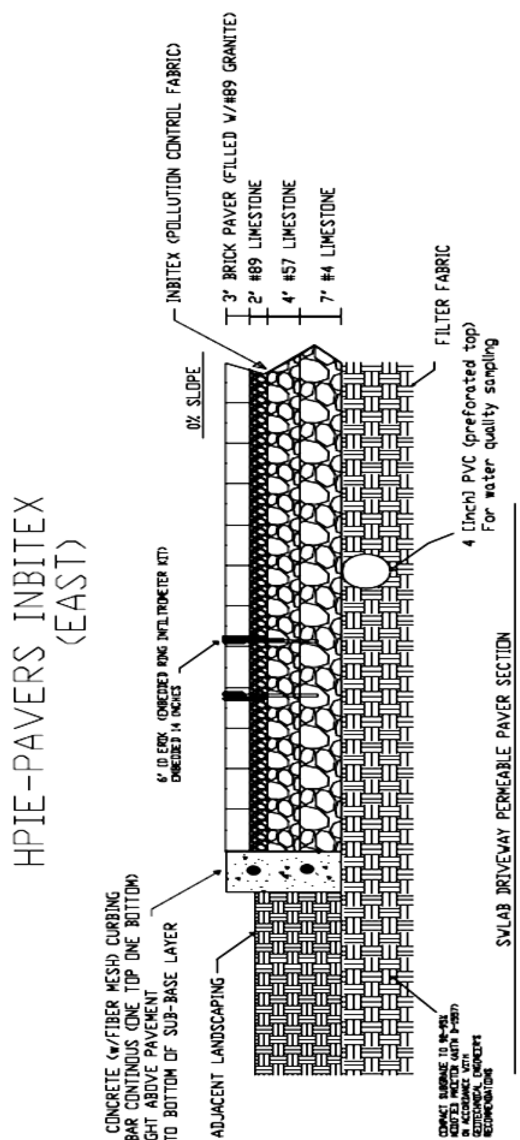


Figure 14: Cross-sectional view of Hanson Pavers Inbitex (HPI)

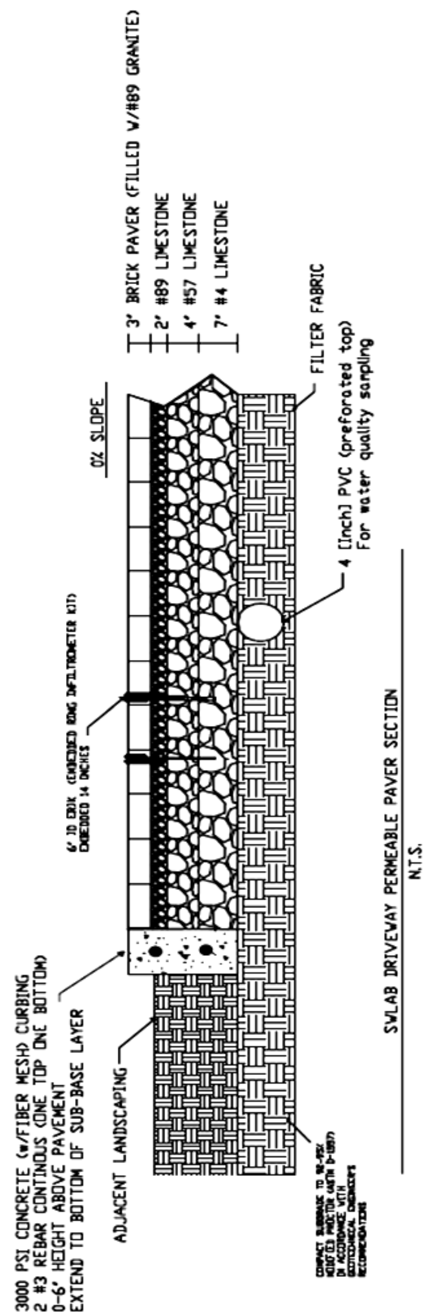


Figure 15: Cross-sectional view of Hanson Pavers Rejuvenation (HPR)

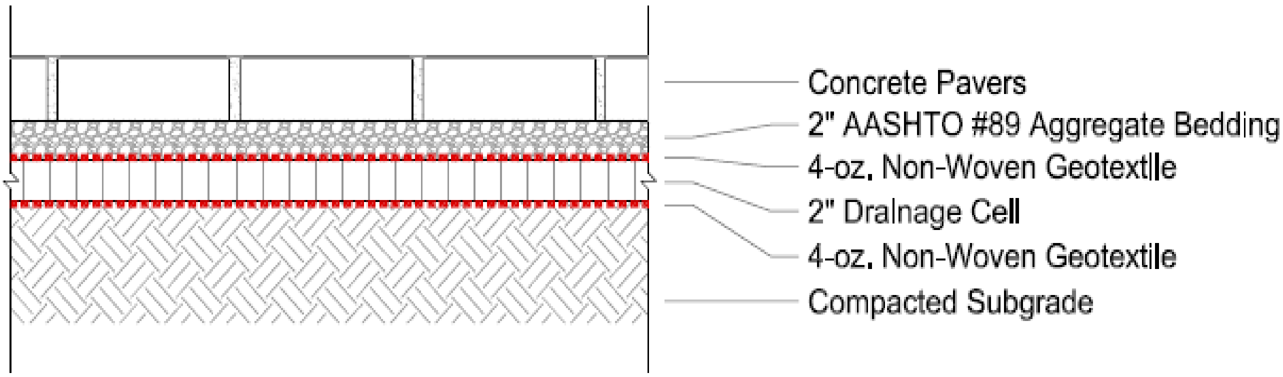


Figure 16: Cross-sectional view of Tremron Shallow (TS)

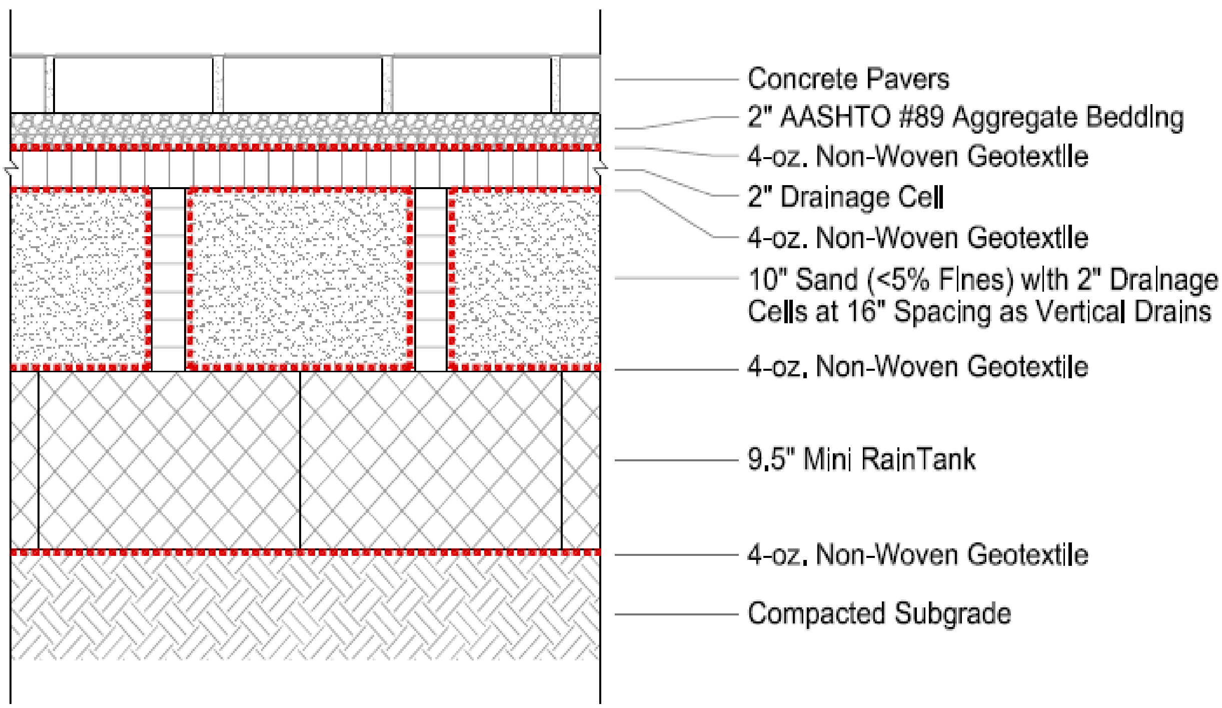


Figure 17: Cross-sectional view of Tremron Deep (TD)

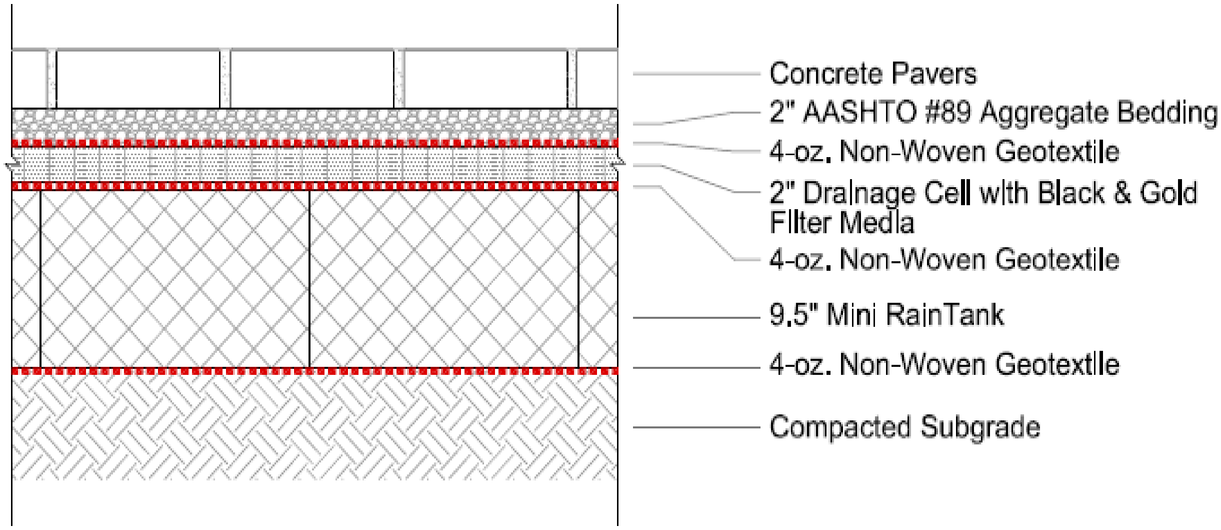


Figure 18: Cross-sectional view of Tremron Black & Gold Top (TBGT)

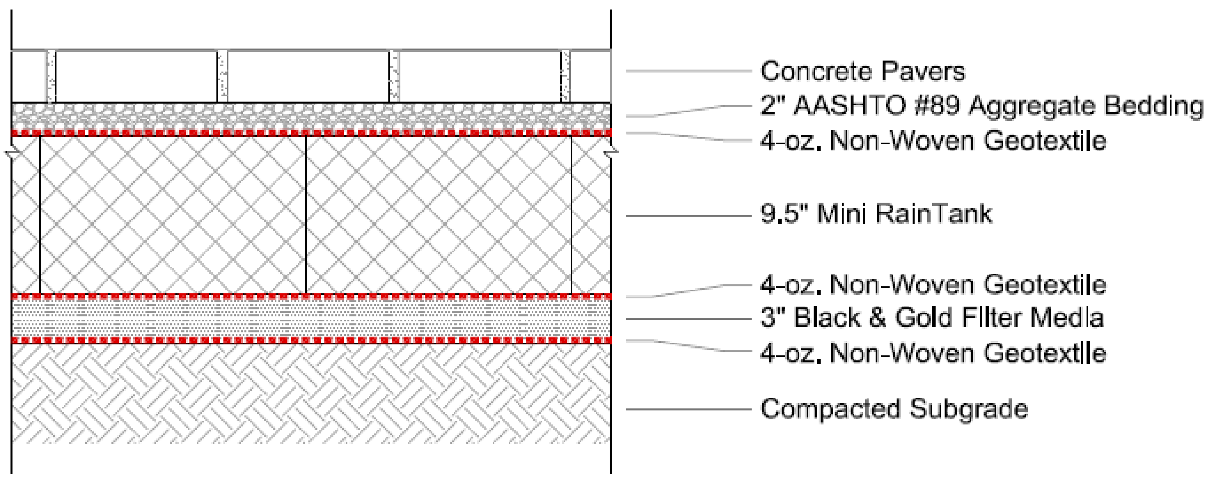


Figure 19: Cross-sectional view of Tremron Black & Gold Bottom (TBGB)

Recycled rubber tire pavement (Flexi-Pave®)

This pavement can be seen at the entrance of the driveway. Three pavement systems were installed. These include FPF (FP-fill), FPBG (FP-BOLD & GOLD) and FPR (FP-rejuvenation). The area occupied by the FPF (Figure 20) and FPBG (Figure 21) sections is about 800 sq ft, while FPR section is installed over an area of 700 sq ft. The FPF and FPR have the same layer materials. The surface course is 2” recycled rubber tire pavement and it is installed over a 4” deep compacted #57 aggregate (limestone), 10” thick fill sand represents the subbase layer. While, the FPBG system has a 10” deep fill sand and BOLD & GOLD filter media subbase layer.

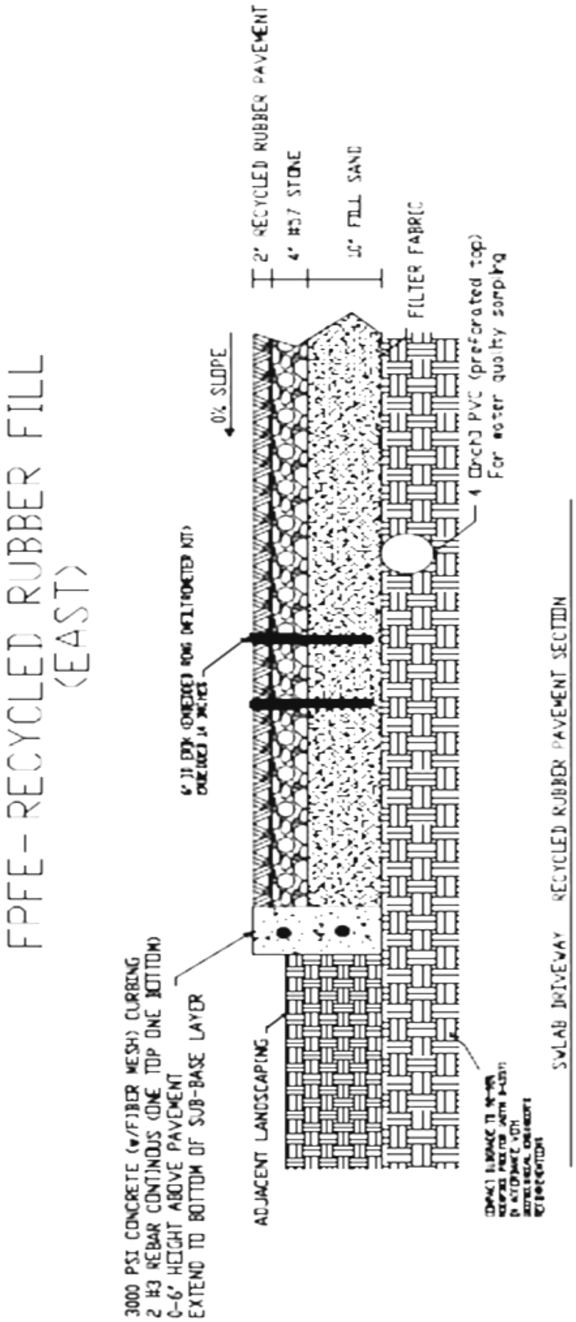


Figure 20: Cross-sectional view of Recycled Rubber Fill (FPF)

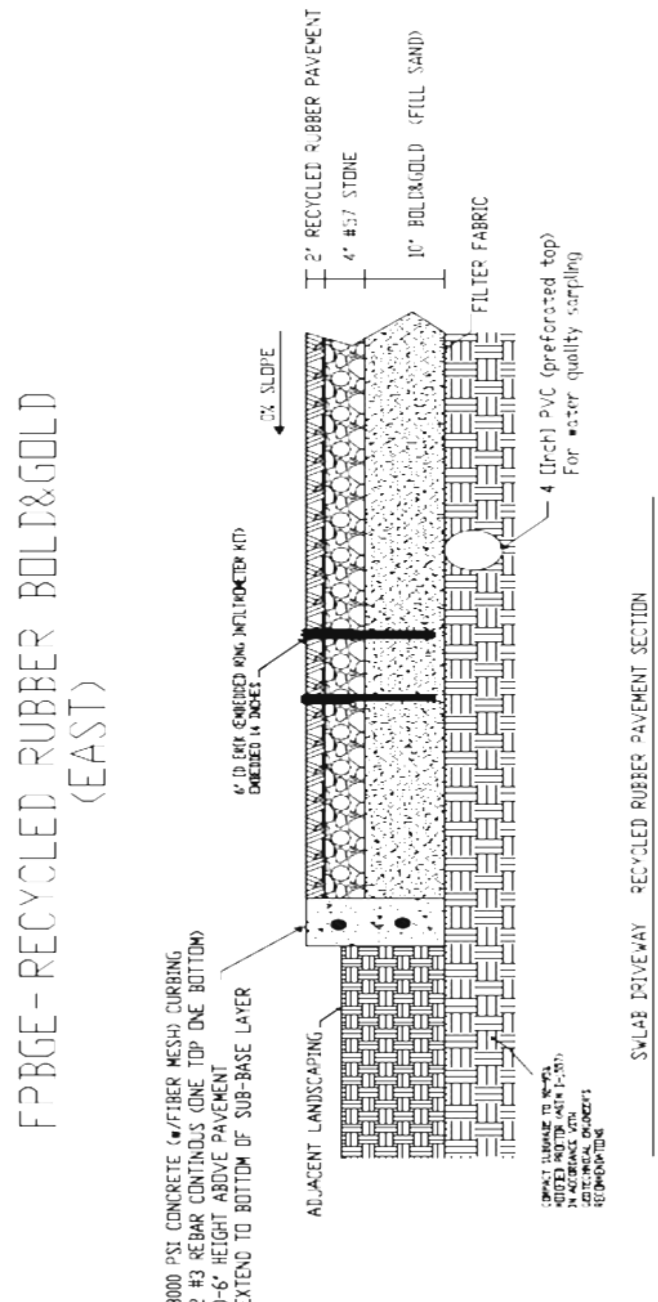


Figure 21: Cross-sectional view of Recycled Rubber Bold & Gold (FPBG)

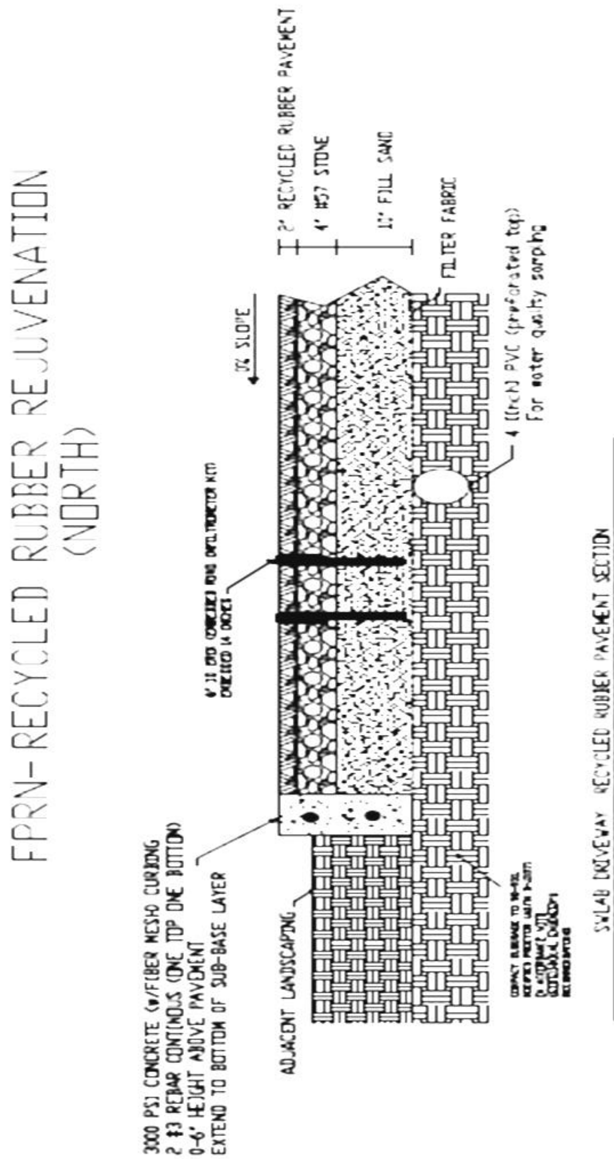


Figure 22: Cross-sectional view of Recycled Rubber Rejuvenation (FPR)

Recycled glass pavement (Filterpave®)

A recycled glass pavement system was also installed in the field. This is a new pervious pavement system that is still under study and being revised by the manufacturer (Presto-Geosystems, 2009). It has a hardened surface and can be considered to be a rigid pavement. The size of the section is 24” long x 12” wide. 3” deep recycled glass pavement is the surface course and it is being supported by 4” thick No. 57 aggregate (base) and 5” Fill sand (subbase) (Figure 23).

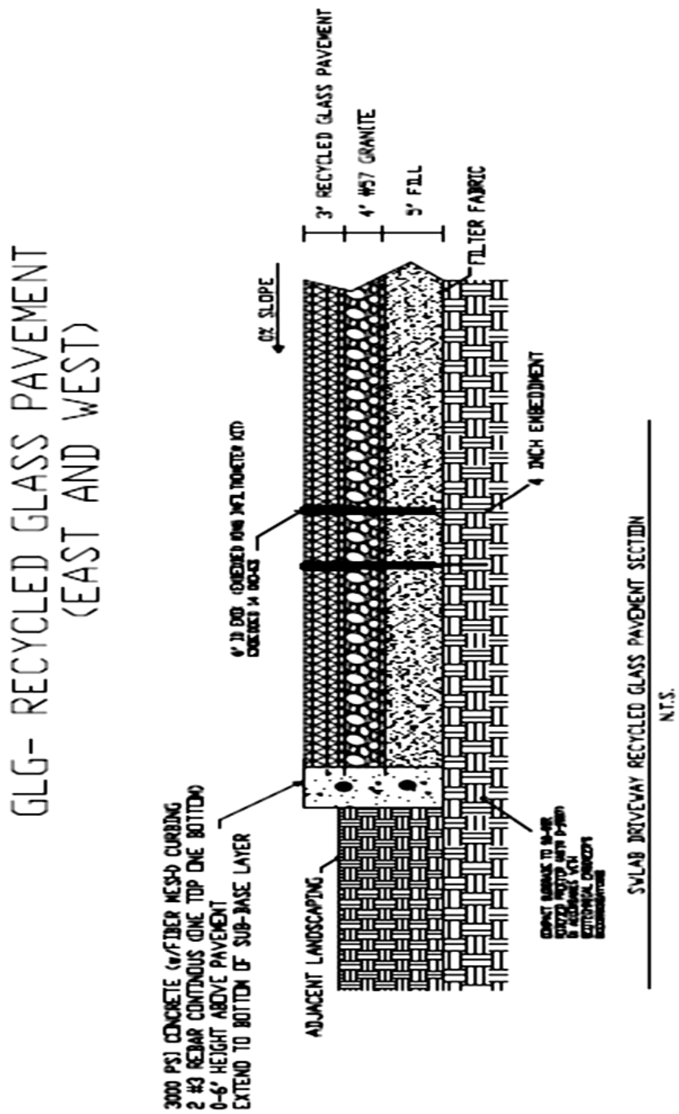


Figure 23: Cross-sectional view of Recycled glass pavement (Filterpave®)

Porous aggregate (Firmapave®)

The same manufacturer as recycled glass aggregate also produced this type of pavement. It is also hard surfaced with stabilized granite aggregate. The dimension of the section is 24” long by 12” wide. This pavement system has the same layer orientation and material like that of the recycled rubber tire pavement (Figure 24).

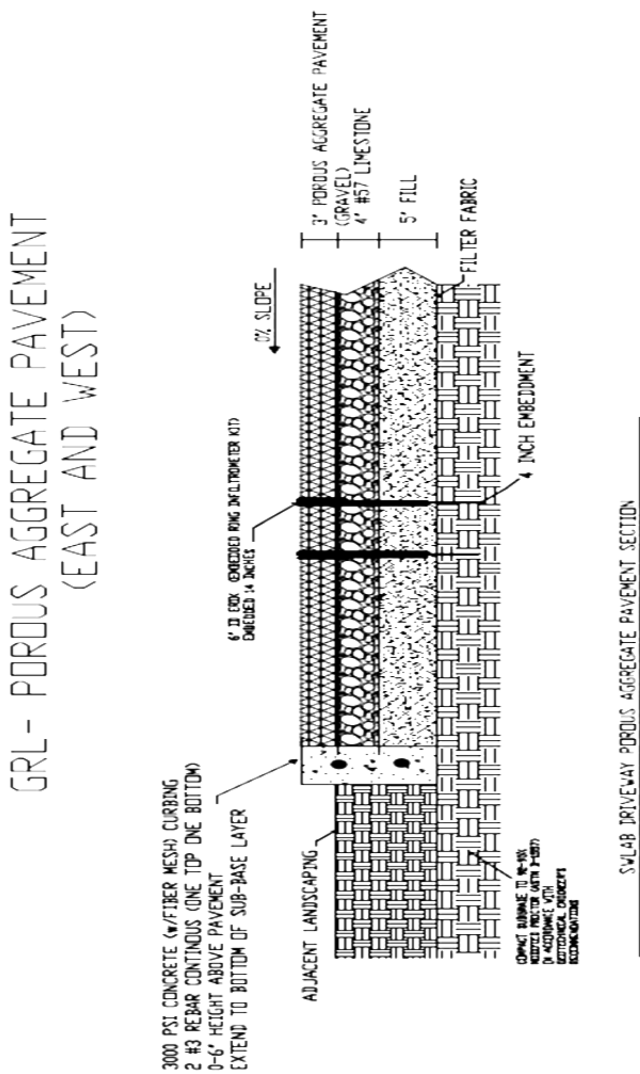


Figure 24: Cross-sectional view of Porous aggregate pavement

Falling Weight Deflectometer (FWD) Test

FWD test was used to find the deflection and relate it to the performance of pervious pavements subjected to traffic loads. Field test was conducted in accordance with ASTM D4694. The FWD tests were performed with FDOT Dynatest equipment (Figure 25) on each of the twenty four (24) pervious pavement driveway sections at the field site. Three points on each cell were marked and FWD impact hammer was dropped at these points. A total of nine (9) load drops consisting of three drops 6, 9 and 12 kips were performed on each cell. A 15-kip impact load was not used because it may have caused damage to the pavement structure, especially the permeable pavers. The data were collected and processed using a mobile computer which recorded the displacement response at different locations (Figure 26). The FWD deflection data on pervious concrete and porous asphalt cells were compared to that obtained from conventional PCC pavement cell and conventional asphalt respectively.

Figure 25: FWD testing on a pavement

Figure 26: Computer system for Data Collection

Back-calculation and Pavement Thickness Design program

Modulus 6.0 (Liu, et al., 2001) was used to perform back-calculation analyses to obtain in-situ layer moduli, properties of the deflection basins, depth to bedrock and to obtain the estimated remaining life of the pervious pavements from damage analysis. This program, which was developed at the Texas Transportation Institute (TTI) for the Texas DOT (TxDOT), performs linear analysis. According to (Ameri, et al., 2009), this program makes use of WESLEA program, which is based on multilayer linear elasto – static theory, as a forward routine. Furthermore, it interpolates within the measured deflection bowls to find the minimum possible error between the field data and the calculated deflection bowls. Therefore, the optimum solution is the solution with minimum errors.

The elastic modulus value generated for asphalt surface layer is based on a temperature of 77F, while that generated for the base layer is based on a thickness of 10 in. Therefore, temperature correction factors must be applied to the elastic moduli values obtained from the backcalculation analysis as shown in Equation (20). These corrected moduli values will be used in the pavement thickness design.

$$\text{Temperature correction factors, } CF = \frac{T^{2.81}}{200{,}004} \tag{20}$$

The limitations of back-calculation are that the base moduli are not constant. Also, this program is not be able to predict stiffness values for asphalt surface layer with thickness < 3 or base layer with thickness < 6 (Ameri, et al., 2009).

The inputs into the program are the number of pavement layers (maximum of four), the average temperature of the surface, thickness of each layer, material type of each layers, and Poisson's ratio of each layer. The expected outputs are E – values for all the pavement layers and the depth to stiff layer.

The Flexible pavement system – windows version (FPS 19W) (Liu, et al., 2006) program is used to determine the surface and base layer thicknesses to carry expected traffic loads. This is a mechanistic-empirical design process that makes use of performance model, cumulative 18-kip ESAL load. It works well for both new construction as well as reconstruction. It is used for flexible pavement design and overlay thickness design. This program makes use of back-calculated modulus values from FWD testing. These moduli values are completely different from the resilient modulus values used in AASHTO design procedure. Environmental data such as number of freeze thaw cycles and average temperature are also taken into account in this case.

FPS 19W uses reliability or confidence levels to take into consideration variability in construction, traffic loading growth and in-situ stiffness of subgrade. Input parameters includes, initial present serviceability index (initial PSI), final PSI. This program is not used to design heavily stabilized, concrete pavements.

Porous asphalt pavement was designed with this program. The estimated ESAL traffic load was taken as 0.41 million ESAL. The design period is twenty (20) years. The elastic moduli used for the different pavement layer was obtained from backcalculation analysis. This program provides possible pavement layer thickness design and cost analysis of the project.

RESULTS AND DISCUSSIONS

Introduction

In this chapter, the results of the laboratory and field tests are discussed. Relationships between the compressive strength, flexural strength, porosity are presented. In addition, a statistical analysis of the strength parameters is provided. The results of the back-calculation and forward calculations of each pervious pavement section are tabulated. The stress, strain and displacement of each layer of the pavement as determined from the KENPAVE program are also presented. Comparisons of the minimum thickness design of the flexible pavements using the AASHTO Method hand calculation and FPS 19W program are provided.

Porosity, Unit weight and Compressive Strength

As discussed in the previous chapter, tests were conducted to evaluate the porosity and compressive strength of the cylindrical pavement samples. The dry unit weight was also obtained for the different pervious pavement sections.

Pervious Concrete

Cored and cast-in situ pervious concrete cylinders were tested. The average depth of the core sample was 7.4” while the width was 3.7”, so a correction factor was implemented when calculating the compressive strength. Samples C1 – C7 cylinders were cored from the pervious concrete driveway installed in 2005 while samples M1 –

M10 were cored from PC section in the storage area which was installed in 2009. Table 16 below shows a summary of the laboratory tests performed in this research.

The compressive strength values ranged from 988 – 2429 psi. It may be noted that sample C4 had very low compressive strength and high porosity. This abnormality shows that these pavements tend to be non-homogenous.

Table 16 Porosity and Compressive strength of Cored pervious concrete cylinders.

Sample	Maximum Load at failure (lbf)	Compressive strength (psi)	Unit weight (lb/ft^3)	Porosity, n	Void ratio, e
C1	18758	1698.38	114.161	0.193	0.24
C2	26818	2428.14	121.720	0.101	0.11
C3	18072	1636.27	110.582	0.128	0.15
C4	6150	556.83	98.247	0.298	0.42
C5	19700	1783.67	116.912	0.103	0.11
C6	21598	1955.51	116.899	0.076	0.08
C7	22227	2012.47	113.181	0.131	0.15
M1	16082	1456.09	109.519	0.165	0.20
M2	18989	1719.29	111.396	0.265	0.36
M3	14300	1294.74	109.519	0.320	0.47
M4	14522	1314.84	114.281	0.201	0.25
M5	20414	1848.31	110.199	0.201	0.25
M6	15712	1422.59	113.357	0.230	0.30
M7	24437	2212.56	114.281	0.201	0.25
M8	20477	1854.02	111.257	0.093	0.10
M9	10902	987.08	104.977	0.298	0.42
M10	20248	1833.28	107.700	0.240	0.32

C – Pavement section 7 - 9
M – Pervious concrete section at storage area

This cylindrical concrete samples were obtained from two different production process, mix design and age. A statistical check on the results for the porosity and void ratio are shown in the Table 17. One (1σ) and two (2σ) standard deviations were used to determine the accuracy of the data. It was found that only about 59% of the porosity data passed the 1σ (less than 67%) test while about 100% passed the 2σ test. This shows that the data provided were not within acceptable range as shown by the 1σ test.

Table 17: Statistical Data for Porosity

Sample	Average Void ratio, e	Average Porosity, n	Standard deviation of porosity, σ	(n-2σ, n+2σ)	Proportion within 2σ	Coefficient of variation, CV
C1 – C7	0.18	0.147	0.076	(-0.005, 0.299)	1	0.52
M1 – M10	0.29	0.221	0.066	(0.089, 0.353)	1	0.30
C1 – M10	0.25	0.191	0.078	(0.113, 0.268)	0.59	0.41 (1 σ)
C1 – M10	0.25	0.191	0.078	(0.035, 0.347)	1	0.41

From the statistical analysis shown in Table 18, 76% of the data passed the 1σ test (greater than the 67%). This shows that the compressive strength values are within acceptable range.

Table 18: Statistical Data for Compressive strength

Sample	Average Compressive strength (psi)	Standard deviation, σ	Range	Proportion within 2σ	Coefficient of variation, CV
C1 – C7	1724.47	578.33	(567.80, 2881.13)	1	0.34
M1 – M10	1594.28	360.88	(872.53, 2316.04)	1	0.23
C1 – M10	1647.89	450.60	(1197.28, 2098.49)	0.76	0.27 (1σ).

A statistical analysis was done by means of MINITAB statistical software. The normal probability plot in Figure 27, shows that the third assumption of residual analysis, which states that the probability distribution is normal, is not violated. There is no great departure from normality. Therefore it can be said that the probability distribution is normal. The plot only has moderate departures which have minimal effect on the validity of the statistical tests. The relative frequency histogram shows that the data are approximately normal judging by its similar shape with the normal curve plot (mound shape). The plot of residual versus time order shows that that there is no pattern in the distribution. This shows that there is no visible correlation between the random errors of the different observations.

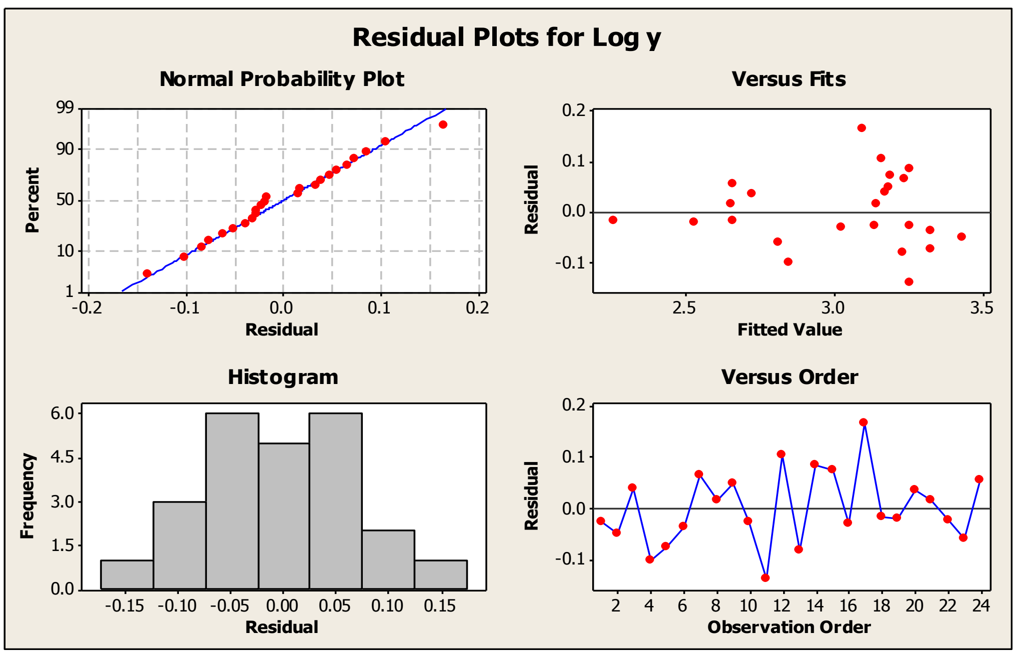


Figure 27: Statistical plot for the cored pervious concrete.

From the statistical analysis, the independent variables, porosity and unit weight are highly correlated with the compressive strength. From Figure 28, it can be seen that the porosity of the cored sample increases as its compressive strength decreases and vice versa. Therefore, porosity has a high negative correlation with the compressive strength. On the contrary, the unit weight of the sample is positively correlated to the compressive strength.

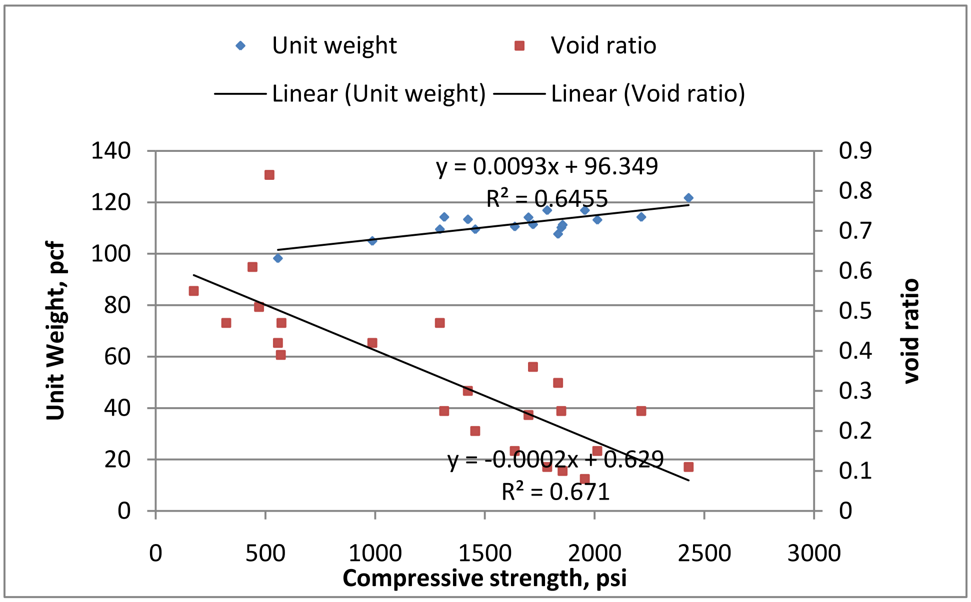


Figure 28: Relationship between Compressive strength, Porosity and Unit weight

The relationship between the estimated compressive strength and the actual compressive strength is shown in Figure 29 . The estimated compressive strength is obtained from the regression equation from the statistical analysis using MINITAB.

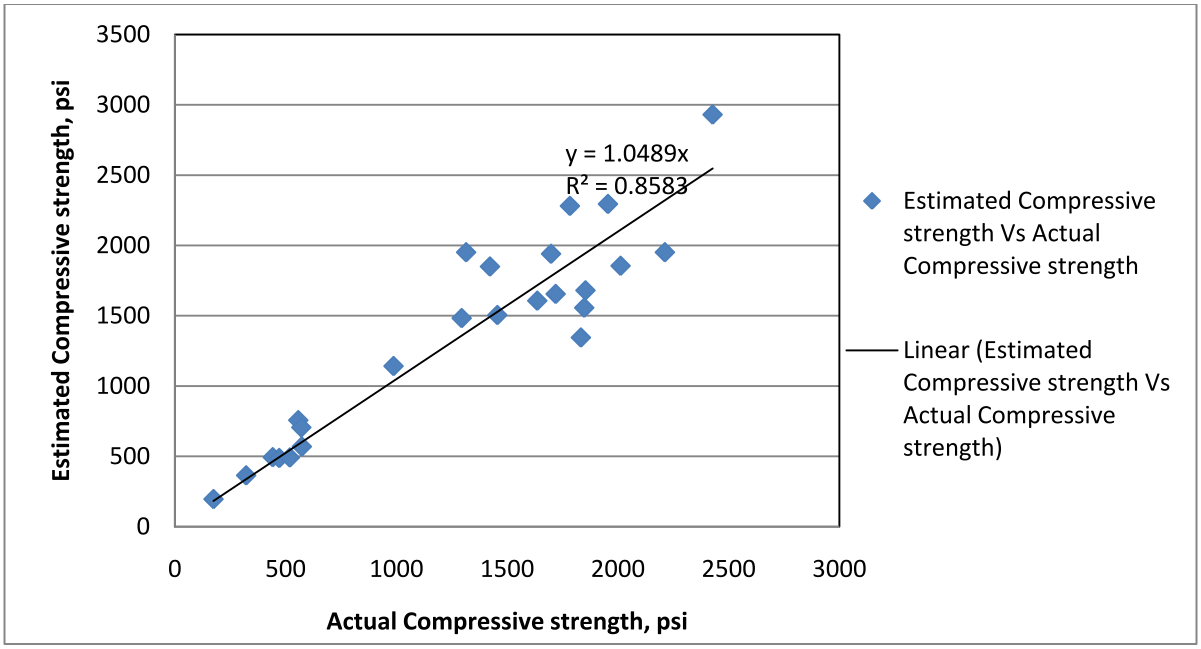


Figure 29: Relationship between the estimated compressive strength and actual compressive strength for the cored P.C samples

28-day cast in-situ P.C cylinders of about 8 in. X 4 in. diameter size was tested. Table 19 shows the 28-day P.C laboratory test performed on the test cylinders. The compressive strength values range from 364 – 1100 psi. The unit weight ranges from 93 – 105 pcf, while porosity ranges from 0.25 – 0.38.

Table 19: Porosity and Compressive strength data of 28-day pervious concrete

Sample	Size (in)	Maximum Load (lbf)	Compressive strength (psi)	Unit weight (lb/ft^3)	Porosity, n	Void ratio, e
PC 6	8x4	6743	536.59	96.149	0.32	0.47
PC 7	“	10577	841.69	104.728	0.25	0.34
PC 8	“	5396	429.40	95.927	0.31	0.45
PC 9	“	7893	628.10	102.150	0.26	0.35
PC 10	“	13814	1099.28	103.847	0.25	0.34
PC 11	“	4564	363.19	92.684	0.38	0.61
PC 12	8x4	13682	1088.78	104.769	0.26	0.35

The average porosity of the 8 x 4 samples is 0.29 as shown in the Table 20. The 2σ test shows that the porosity values fall within the acceptable limits.

Table 20: Statistical data for Porosity

Sample	Average void ratio, e	Average Porosity, n	Standard deviation, σ	(n-2σ, n+2σ)	Proportion within 2σ	Coefficient of variation, CV
PC6 – PC10	0.42	0.29	0.05	(0.20, 0.39)	1	0.16

The compressive strength range of the 8 x 4 samples is 364 – 1100 psi. Table 21 shows the average compressive strength of the two sample types and the 2σ test shows that the compressive strength values fall within acceptable limits. The average compressive strength of 8 x 4 samples is 712.43 psi.

Table 21: Statistical data for Compressive strength

Sample	Average Compressive strength (psi)	Standard deviation, σ	Range	Proportion within 2σ	Coefficient of variation, CV
PC6 – PC12	712.43	302.24	(107.95, 1316.92)	1	0.42

Figure 30 show different statistical plots aimed at proving normality in the data. The shape of the relative frequency histogram is not similar to normal curve. It shows non-normality. The residual in time order plot shows no apparent correlation between the errors. The plot of residual versus the fitted values shows that the variance of the probability distribution is unequal and constant. A random pattern is seen in this plot. In conclusion, the plots show moderate to high departures from normality of the data.

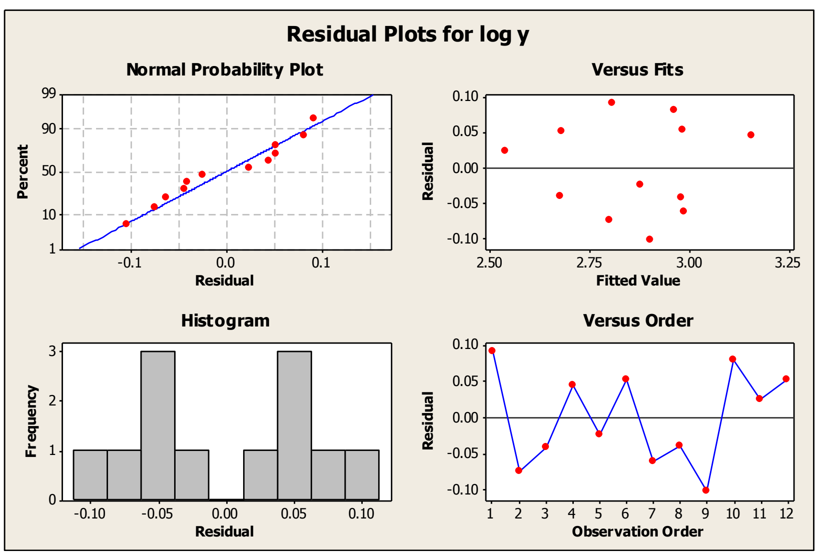


Figure 30: Statistical plot for the 28-day pervious concrete

Figure 31 shows the relationship between the compressive strength and unit weight. Increase in unit weight leads to corresponding increase in the compressive strength. The low compressive strength might be attributed to poor mix design or fabrication by the manufacturer. It can also be said that the number of samples tested may not have being adequate to reach a desirable conclusion. Furthermore, at failure the aggregate particles disintegrated suggesting that the cement paste binding the particles together is weak.

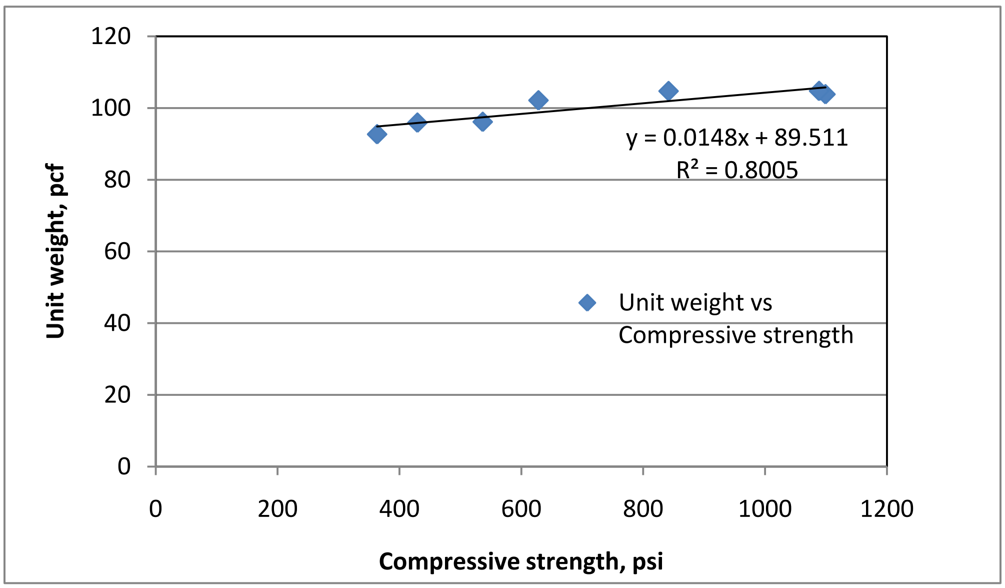


Figure 31: Relationship between Compressive strength and unit weight for 28-day PC

Figure 32 shows the plot of the estimated and the actual compressive strength of the 28-day PC tested. It has an R-square value of about 82 %.

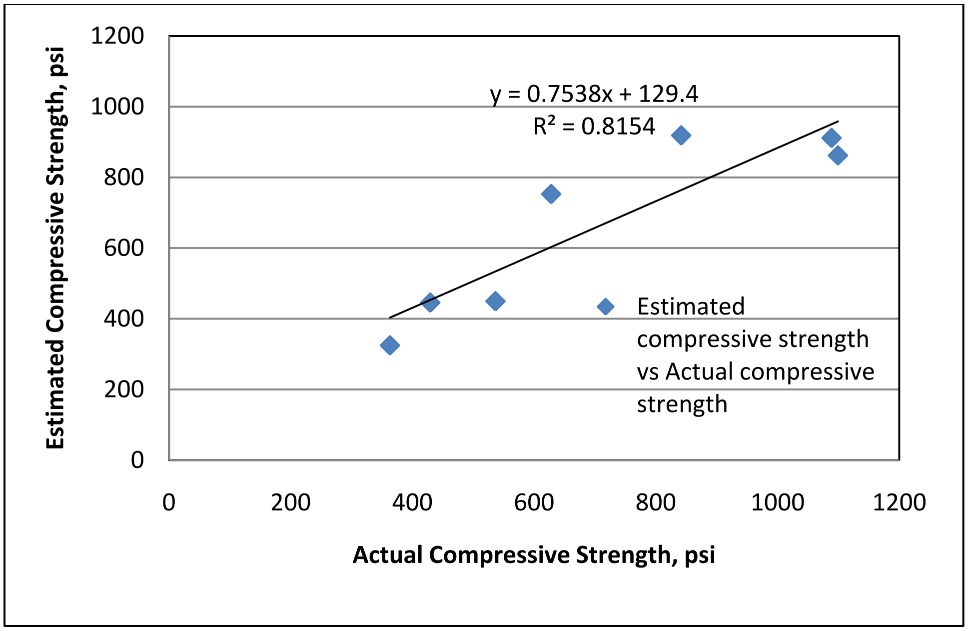


Figure 32: Estimated Compressive strength vs Actual Compressive strength of 28-day PC

Recycled Rubber Tire Pavement (Flexi-pave®)

This pavement was tested for porosity and compressive strength. As result of its flexibility and its ability to return to its previous shape after the application of load or deformation, the compressive strength may not have being the most desired testing process. The sample sizes were 8" x 4". The average porosity of the sample is 0.53, while its average compressive strength is 115.4 psi.

Table 22 presents the laboratory test conducted on the 8 x 4 cylinders. These representative samples were prepared by the manufacturer. The compressive strength ranges from 108 – 129 psi. The porosity is 0.53 while the unit weight ranges from 57 – 59 psi.

Table 22: Porosity and void ratio data of recycled rubber tire pavement (Flexi-pave®)

Sample	Maximum Load (lbf)	Compressive strength (psi)	Unit weight (lb/ft^3)	Porosity, n	Void ratio, e
A2	1449	119.0	56.76	0.53	1.14
B2	1312	107.75	56.33	0.53	1.12
C2	1373	112.76	55.88	0.53	1.14
D2	1568	128.77	58.08	0.53	1.12
E2	1379	113.29	55.88	0.53	1.14
F2	1351	110.95	56.76	0.52	1.10

The average void ratio and porosity for these samples are 1.12 and 0.53 respectively and are shown in Table 23. The 2σ test shows that all the void ratio values fall within the specified range.

Table 23: Statistical Data for Porosity

Sample	Average Void ratio, e	Average Porosity, n	Standard deviation, σ	Range (n - 2σ,n +2σ)	Proportion within 2σ	Coefficient of variation, CV
A2 – F2	1.12	0.53	0.0033	(0.52, 0.54)	1	0.006

Table 24 shows that the average compressive strength of these 8 x 4 cylinders is 115.41 psi. All the compressive strength values are within the range in the 2σ test. The compressive strength is low but unlike other pervious pavements it can still withstand more applied load even after failure because of its high flexibility.

Table 24: Statistical Data for Compressive strength

Sample	Average Compressive strength (psi)	Standard deviation, σ	Range	Proportion within 2σ	Coefficient of variation, CV
A2 – F2	115.41	7.506	(100.40, 130.42)	1	0.065

First it is important to note that the number of samples tested was low. These six samples are not enough to draw a very accurate conclusion based on this test. The plots in Figure 33 show a moderate deviation from normality. The relative frequency histogram shows that the data are quite normal. An outlier can be seen in the plot of residual against the fitted value. This may have being as a result of abnormalities in the mixing process.

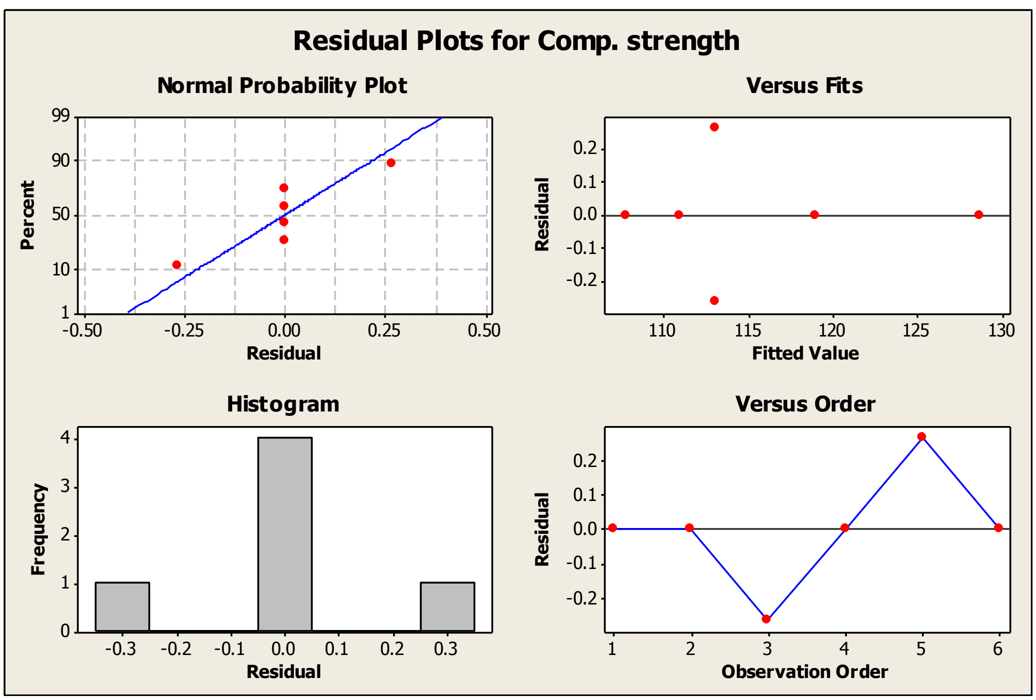


Figure 33: Statistical plot for Recycled rubber pavement (Flexi-pave®)

Figure 34 shows that the unit weight once more is a vital variable that affects the outcome of the compressive strength. As the unit weight increases, the compressive strength of the sample increases. The unit weight is also dependent on the volume of the sample and aggregate size in the mixture. Failure occurs when the load applied breaks the binding agent holding the aggregate.

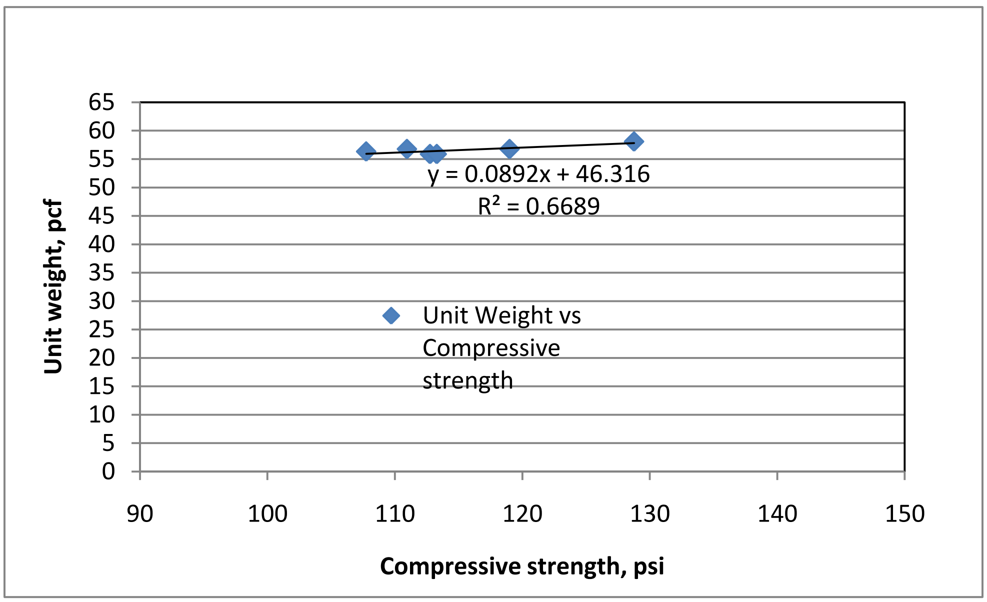


Figure 34: Relationship between compressive strength and unit weight of Flexi-pave®

Also at the instance of failure, from visual observations, it is seen that the crack is not very visible. The elasticity of the sample allows it to return to its initial position upon application of the load. It can however be said that the sample can still accommodate more load even after failure. Figure 35 shows the high correlation between the estimated compressive strength and actual compressive strength. This shows that the compressive strength of the entire sample falls within the same range. Therefore the estimated and actual compressive strength of the samples are equal.

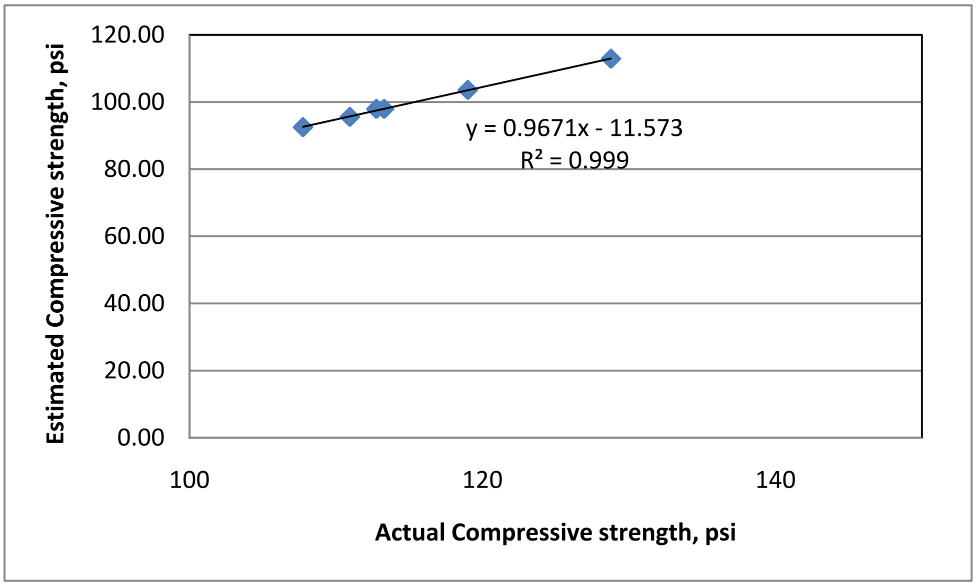


Figure 35: Estimated Compressive Strength vs Actual Compressive strength (Flexi-pave®)

Recycled glass pavements (Filterpave®)

Unlike the recycled rubber pavement, these samples were collected on site while it was being poured in the field. Therefore, some irregularities and discrepancies may have occurred in its batch mixing, compaction and fabrication. Table 25 presents the results from the different laboratory test conducted on these samples. Compressive strength value is found to be 479 – 616 psi for 12” x 6” samples and 1127 – 1179 psi for 8” x 4” samples. The unit weight for the 12” x 6” and 8” x 4” cylinders ranges from 93 – 95 pcf and 99 – 101 pcf respectively. The porosity of the larger sample ranges from 0.31

– 0.39 while the smaller one ranges from 0.41 – 0.50. The average compressive strength of the 12” x 6” diameter and 8’ x 4” diameter cylinder is 538.3 psi and 1155.65 psi respectively.

Table 25: Recycled glass pavements

Sample	Size (in.)	Maximum Load (lbf)	Compressive strength (psi)	Unit weight (lb/ft^3)	Porosity, n	Void ratio, e
A	12x6	14548	514.53	94.24	0.31	0.44
B	12x6	15383	544.06	94.51	0.38	0.60
C	12x6	13530	478.53	92.20	0.38	0.60
D	12x6	17416	615.97	94.49	0.39	0.64
G	8x4	14606	1162.31	100.55	0.41	0.71
H	8x4	13714	1126.25	100.12	0.41	0.71
J	8x4	14808	1178.38	98.84	0.5	1.00

Table 26 shows the average porosity of the 12” x 6” diameter cylinder is 0.37 while that of the 8” x 4” diameter cylinder is 0.44 and the average void ratio of the 12” x 6” and 8” x 4” cylinders are 0.57 and 0.81 respectively.

Table 26: Statistical data for Porosity

Sampl e	Average Void ratio, e	Average Porosity, n	Standard deviation, σ	(n-2σ, n+2σ)	Proportion within 2σ	Coefficient of variation, CV
A – D	0.57	0.37	0.04	(0.29,0.45)	1	0.11
G – J	0.81	0.44	0.05	(0.34,0.54)	1	0.11

Table 27 further highlights the average value of the compressive strength. The average compressive strength of the 12” x 6” diameter and 8’ x 4” diameter cylinder is 538.3 psi and 1155.65 psi respectively. The compressive strength value for the 8 x 4 cylinder is far greater than that specified by the manufacturer (1000 psi).

Table 27: Statistical Data for Compressive Strength

Sample	Average Compressive strength (psi)	Standard deviation, σ	Range	Proportion within 2σ	Coefficient of variation, CV
A – D	538.3	58.32	(421.63, 654.91)	1	0.108
G – J	1155.65	26.7	(1102.25,1209.04)	1	0.023

The normality plot in Figure 36 indicates that there is a great departure from normality. In other words, there is an indication that the data collected have dependent errors, and outliers thereby suggesting that the independent variables may not be adequate to predict the dependent variable (compressive strength). From the statistical analysis, it can be seen that the compressive strength increase the unit weight increases.

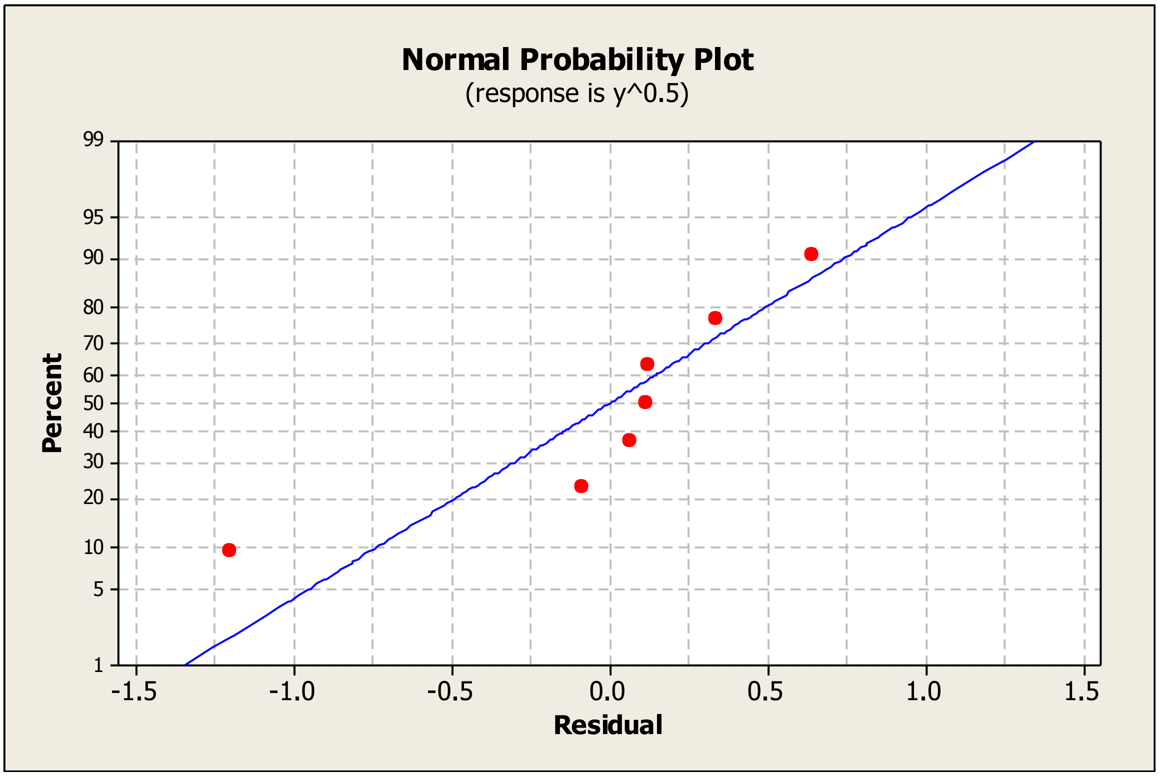


Figure 36: Normal Probability Plot

As the unit weight increases the compressive strength of this pavement type increases as presented in Figure 37. Therefore, a benchmark unit weight should be incorporated so as to obtain the desired compressive strength. After 28 days of curing, a hard binder layer could be seen on the outside of the sample. This might have aided in increasing the compressive strength.

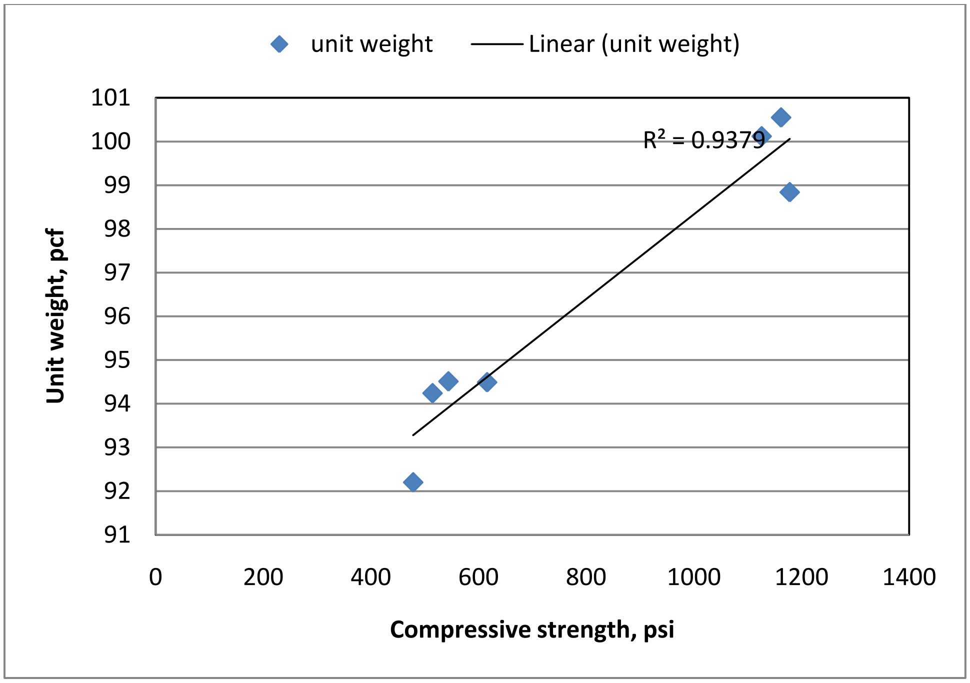


Figure 37: Relationship between the Compressive strength and unit weight (Filterpave®)

Figure 38 is evidence that there is a need to always estimate the compressive strength given some independent variables. The estimated and actual compressive strength are almost the same. This shows that the test samples are representative samples.

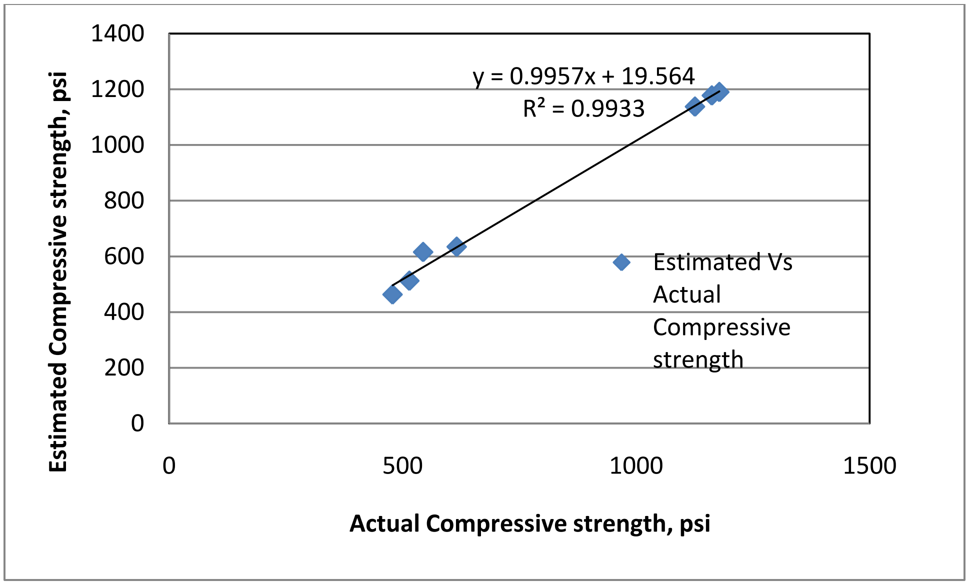


Figure 38: Relationship between the estimated compressive strength and actual compressive strength

Pervious aggregate (Firmapave®)

Pervious aggregate pavement can be considered as a rigid pavement. The strength of the of the binding agent was low judging from the results obtained from the strength test. The various laboratory strength results are presented in

Table 28. The compressive strength values of 12” x 6” diameter cylinders ranges from 73 – 159 psi while that of the 8” x 4” diameter cylinders ranges from 190 – 218 psi. The unit weight of the two sample types ranges from 89 – 93 pcf. The porosity of the 12” x 6” diameter and the 8” x 4” diameter samples is between 0.37 – 0.40 and 0.54 – 0.56 respectively.

Table 28: Pervious aggregate pavements

Sample	Size (in)	Maximum Load (lbf)	Compressive strength (psi)	Unit weight (lb/ft^3)	Porosity, n	Void ratio, e
A1	12 x 6	2035	75.07	91.96	0.38	0.62
B1	12 x 6	2017	72.85	89.81	0.38	0.62
C1	12 x 6	2178	80.34	91.68	0.37	0.57
D1	12 x 6	4300	158.62	88.43	0.40	0.66
E1	8 x 4	2722	216.61	92.44	0.54	1.16
F1	8 x 4	2734	217.56	90.33	0.55	1.24
G1	8 x 4	2378	189.24	88.18	0.55	1.21
H1	8 x 4	2706	215.34	89.60	0.56	1.25

Table 29 presents the average porosity and void ratio values. A 2σ test was performed on the porosity and compressive strength results. The average porosity values are 0.38 and 0.55 for the 12” x 6” diameter and 8” x 4” diameter sizes respectively. The 2σ test for the porosity test show that porosity results fall within acceptable limits.

Table 29: Statistical data for Porosity

Sample	Average Void ratio, e	Average Porosity, n	Standard deviation, σ	(n-2σ, n+2σ)	Proportion within 2 σ	Coefficient of variation, CV
A1 – D1	0.62	0.38	0.014	(0.35, 0.41)	1	0.04
E1 – H1	1.22	0.55	0.010	(0.53, 0.57)	1	0.02

For the 12” x 6” diameter and 8” x 4” diameter sizes, the average compressive strength is 96.72 psi and 209.69 psi respectively and it is shown in Table 30. The statistical test show that all the samples fall within acceptable range. The low compressive strength values at failure may be as a result of the weak binding strength between the aggregate particles. In addition, the method of sampling or preparation and batch mixing may have caused discrepancies in the results obtained. The failure mode during compression test was observed to be by shear as well as cone and shear.

Table 30: Statistical Data for Compressive strength

Sample	Average Compressive strength (psi)	Standard deviation, σ	Range	Proportion within 2σ	Coefficient of variation, CV
A1 – D1	96.72	41.387	(13.95, 179.49)	1	0.43
E1 – H1	209.69	13.665	(182.36, 237.02)	1	0.065

Flexural Strength Laboratory Testing

The main aim of these tests was to obtain the ability of each beam sample to resist bending. Only the 28-day P.C, recycled rubber tire pavement, recycled glass pavement and pervious aggregate beam samples were tested. The modulus of rupture obtained from this test will subsequently be used in the rigid pavement design.

Pervious Concrete

28-day Pervious Concrete (PC) beams which were cast on site were tested for flexure. Failure occurred at the middle third section of the beam. Once again, the errors may have occurred as a result of batch mixing, fabrication, sampling method and compaction. This test is very sensitive to mix design, moisture content, sample preparation, handling and curing process (ASTM, 2004b).

Flexural strength values for pervious concrete as discussed in some literature ranges from 450 – 620 psi. The flexural strength range of conventional concrete is between 500 – 800 psi. Table 31 shows that the modulus of rupture ranges from 198 – 279 psi. The lower values obtained in the current study may be attributed to factors such as weaker bonding agent (cement paste) used and improper mix design.

Table 31: Flexural strength test of 28-day cast in-situ pervious concrete

Sample	Maximum load at failure, P (lbf)	Modulus of Rupture, M.R (psi)
B1	2003	197.265
B2	2699	256.010
B3	2493	243.021
B4	2680	256.517
B5	2797	278.054

The 2σ test in Table 32 shows that the modulus of rupture values falls within acceptable range. The average modulus of rupture of the beams was 246.17 psi. This value is almost half of that specified in some literature.

Table 32: Statistical data for Modulus of rupture (M.R)

Sample	Average Modulus of rupture (psi)	Standard Deviation, σ	Range	Proportion within 2σ	Coefficient of variation, CV
B1 – B5	246.17	30.09	(185.99, 306.36)	1	0.12

Recycled rubber tire pavement (Flexi-pave®)

The flexural strength is the preferred strength test on this type of pavement because in compression it has the ability to return to its original position after deformation. Visible diagonal cracks were observed at the middle third of the beam under flexural behavior. For this pavement type, it appears that this test actually measures the strength of the polyurethane binder in bending. Table 33 presents the modulus of rupture for each corresponding sample. The range of M.R is between 164 – 186 psi.

Table 33: Flexural strength of new recycled rubber tire pavement

Sample	Maximum load at failure, P (lbf)	Modulus of Rupture, M.R (psi)
G2	2153	178.94
H2	2011	184.99
I2	2074	178.26
J2	1751	163.46
K2	2026	180.14
L2	2037	178.53

The statistical analysis of the flexural strength results in Table 34 shows that all the results obtained fall within acceptable range. The average modulus of rupture of these samples is 177.39 psi.

Table 34: Statistical Data for Modulus of Rupture

Sample	Average Modulus of Rupture (psi)	Standard Deviation, σ	Range	Proportion within 2σ	Coefficient of variation, CV
G2 – L2	177.39	7.26	(12.86, 191.91)	1	0.041

Recycled glass pavement (Filterpave®)

As mentioned before, the beam samples collected showed some irregularities and variations which can be clearly seen in the results. The errors may have being as a result of preparation of samples, handling and inadequate curing. Therefore some values had to be discarded as such. Failure occurred at the middle third of the beam. The values for sample N, O, Q and R were discarded as they were visibly different from the samples that looked like acceptable Filterpave® sections. This test results are shown in Table 35.

Table 35: Flexural strength test of recycled glass pavement

Sample	Maximum load at failure, P (lbf)	Modulus of Rupture, M.R (psi)
K	6839	669.25
L	4972	426.71
M	5679	509.42
P	4756	425.45

From the statistical analysis done in the Table 36, the mean value of the beam samples were obtained. The average modulus of rupture of the beam samples is 508 psi. This value is greater than that specified by the manufacturer (500 psi). All the modulus of rupture values fall within acceptable range.

Table 36: Statistical Data for Modulus of Rupture

Sample	Average Modulus of Rupture (psi)	Standard Deviation, σ	Range	Proportion within 2σ	Coefficient of variation, CV
K – R	507.707	114.636	(278.435,736.979)	1	0.226

Pervious Aggregate (Firmapave®)

Only one sample could be collected on site and the modulus of rupture result is presented in Table 37. Therefore no precise conclusion can be drawn from this result. Failure occurred at the middle third of the beam.

Table 37: Flexural strength test of pervious aggregate

Sample	Maximum Load at failure, P (lbf)	Modulus of Rupture, M.R (psi)
I1	862	85.63

The comparison between the compressive strength test and flexural strength conducted in this research and values obtained from past literature is summarized in Table 38. From (NRMCA, 2005), the compressive strength range of PC is in the range of 500 – 4000 psi. But typically it is 2,000 – 2,500 psi. The flexural strength of PC is in the range of 150 – 550 psi (NRMCA, 2005). But the compressive and flexural strength test conducted on recycled glass pavements is greater than that specified by the manufacturer. The compressive strength of cored pervious concrete cylinders obtained from three field locations were in the range of 1643 – 2495 psi (Crouch, et al., 2006)

Table 38 Comparison between the strength laboratory test and literature

Pavement Type	Compressive strength (psi)		Flexural strength (psi)	
	Test	Literature	Test	Literature
Cored Pervious Concrete (8x4)	1725	1643 - 2500	-	-

28-day Pervious concrete (8x4)	365 - 1100	500 – 4000 2000 (typical)	247	150 - 550
Recycled glass pavement (Filterpave®	1160	1000	508	500

FWD Backcalculation Analysis

As previously stated, backcalculation of the moduli values was done by means of the software Modulus 6.0. For a clearer analysis, each pavement type will be discussed for each load application and the result of the resilient moduli and the measured deflection will be summarized in a table. This analysis treats the pavement system as a deflection basin. This terminology will be employed henceforth in the discussion on the results.

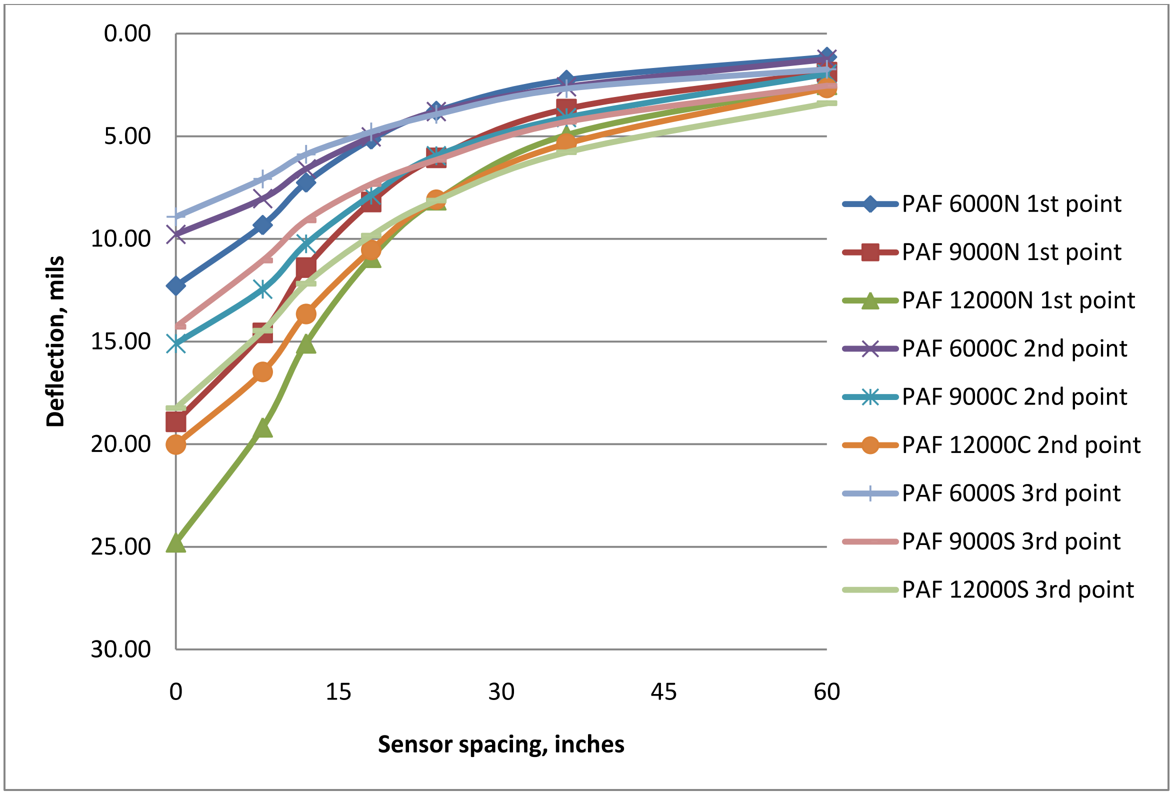


Figure 39 Deflection Basin of Porous Asphalt Fill (PAF)

Table 39 shows the comparison between the backcalculated moduli for the 3 porous asphalt types and the conventional asphalt pavement in the field. It is observed here that the elastic moduli range from 535 – 1002 ksi for porous asphalt while the elastic modulus of the conventional asphalt is 904 ksi.

Table 39: Backcalculation Moduli for P.A and Conventional Asphalt for 6000 lb load

Pavement	PAF	PAR	PABG	Asphalt Inlet	Neptune Drive
$E_{surface}$ 6000 (ksi)	709.4	1001.6	534.2	903.7	111.5
E_{base} 6000(ksi)	72.6	64.1	50	74.6	13.2
$E_{subbase}$ 6000(ksi)	37.6	63.2	36	0	0
$E_{subgrade}$ 6000(ksi)	16.5	13.2	12.3	10.7	20.9
Abs error/sens (%)	0.76	1.14	0.59	1.4	3.06

For an impact load of about 9000 lb the backcalculated elastic moduli range of porous asphalt is between 485 – 1028 ksi and that of conventional asphalt is about 794 ksi as shown in Table 40.

Table 40 Backcalculation moduli for PA and conventional asphalt for 9000 lb load

Pavement	PAF	PAR	PABG	Asphalt Inlet	Neptune Drive
$E_{surface}$ 9000(ksi)	721.4	1027	484.1	793.1	148.5
E_{base} 9000(ksi)	45.1	64.8	75.1	77.9	11.5
$E_{subbase}$ 9000(ksi)	57.1	49.4	27.9	0	0
$E_{subgrade}$ 9000(ksi)	15.6	12.8	12.1	10.8	19.8
Abs error/sens (%)	0.85	0.65	0.45	1.3	3.68

In Table 41 the backcalculated elastic moduli for the pervious asphalt ranges from 461 – 987 ksi while the conventional asphalt is about 851 ksi when an impact load of 12000 lb is applied on the pavement.

Table 41 Backcalculation moduli for PA and conventional asphalt for 12000 lb load

Pavement	PAF	PAR	PABG	Asphalt Inlet	Neptune Drive
$E_{surface}$ 12000(ksi)	692.2	986.1	460.1	849.5	178.1
E_{base} 12000(ksi)	59.8	60.8	76.9	75	10.3
$E_{subbase}$ 12000(ksi)	35.2	59.8	25	0	0
$E_{subgrade}$ 12000(ksi)	15.1	12.3	11.7	10.5	19.3
Abs error/sens (%)	0.55	0.72	0.56	1.36	3.99

This table summarizes the backcalculated in-situ moduli values. As previously discussed, three points were tested on every pavement section and three load applications 6000 lb, 9000 lb and 12000 lb) were impacted at every point. The average surface layer modulus value of PAF is 707.7 ksi, that of PAR is 1004.9 ksi and PABG is 492.8 ksi. Conventional Asphalt roadway on Neptune drive had an average elastic modulus value of 184.3 ksi while the asphalt inlet asphalt concrete surface had a modulus value of 849.5 ksi. The low modulus value of Neptune drive can be attributed to the numerous alligator cracking and rutting visible on this layer.

The FWD deflections obtained from a representative pervious asphalt section was compared to that of a conventional asphalt surface. This comparison of the pavement response at the seven sensor locations for the two pavement surfaces is shown in Table 42. The deflection of conventional asphalt is greater than that of porous asphalt. This shows that when the load is dropped on porous asphalt surface, the response in each sensor is not that of the pavement system but instead it is the rebound displacement when rubber loading plate rebounds from the flexible pavement surface.

Table 42 Comparison between FWD deflections of PA and conventional asphalt

Porous Asphalt							
Load (lb)	Sensor spacing (in.)						
	0	8	12	18	24	36	60
6000	10.33	8.15	6.58	4.99	3.83	2.51	1.38
9000	16.10	12.69	10.25	7.80	6.05	4.02	2.13
12000	21.01	16.71	13.64	10.43	8.11	5.36	2.85
Conventional Asphalt							
Load (lb)	Sensor spacing (in.)						
	0	8	12	18	24	36	60
6000	22.15	13.03	7.92	4.88	3.23	1.89	1.02
9000	31.37	19.36	12.25	7.57	4.94	2.73	1.53
12000	41.06	26.13	16.92	10.58	6.78	3.62	2.14

The falling weight deflectometer (FWD) deflection basins for the different impact load applied on the surface of the pervious asphalt is shown in Figure 40. The greater impact load (12000 lb) produced more deflections.

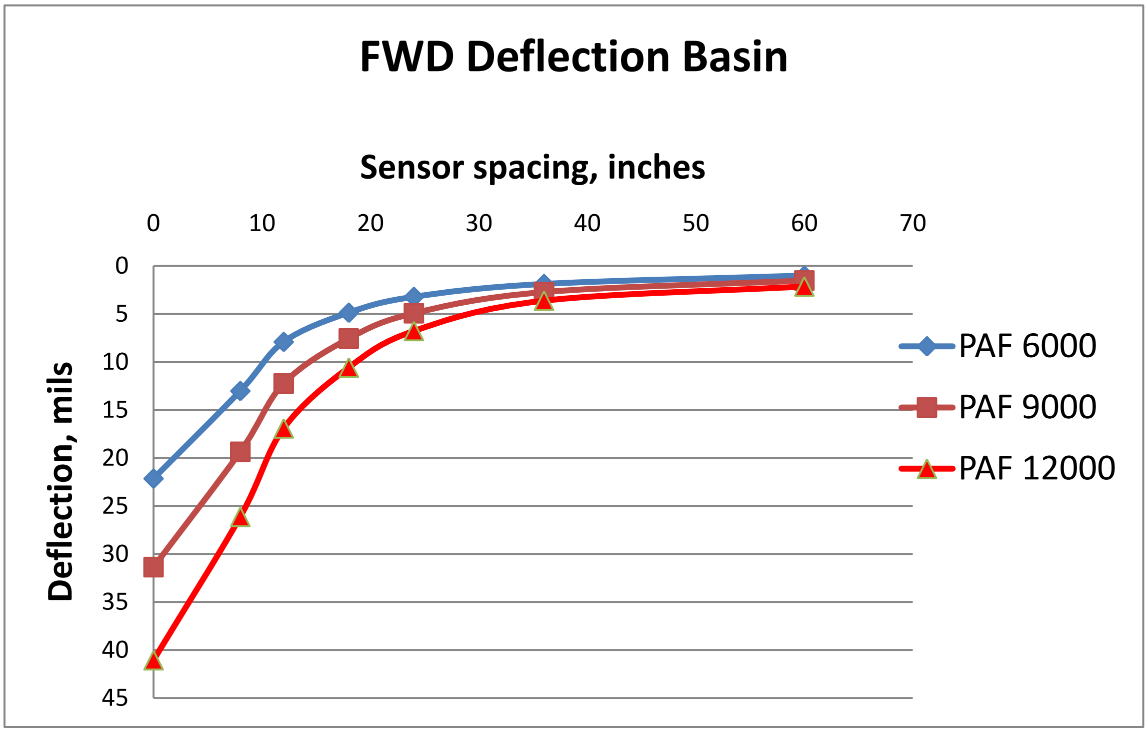


Figure 40 FWD Deflection basins for porous asphalt

The falling weight deflectometer (FWD) deflection basins for the various impact load applied on the surface of the conventional asphalt is shown in Figure 41. Like most pavements, the greater impact load (12000 lb) produced more deflections.

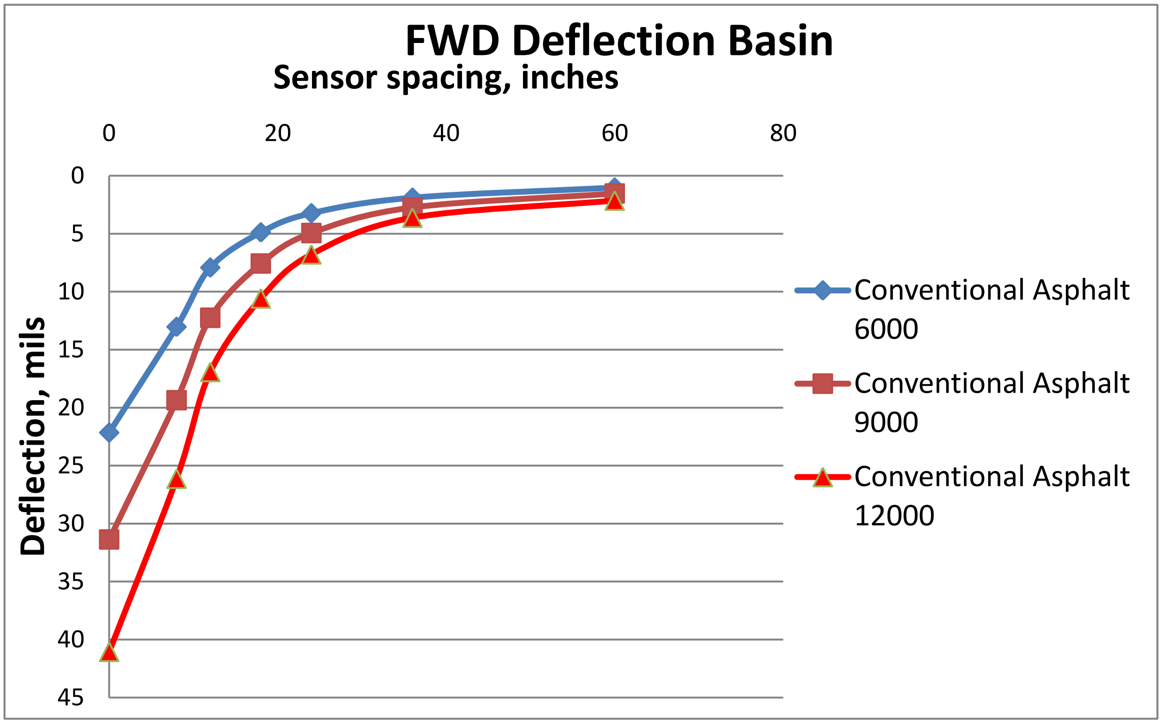


Figure 41 FWD deflection basins for conventional asphalt

Meanwhile, for rigid pervious pavement surfaces, the FWD deflection basin was compared to that of conventional concrete surface as shown in Table 43. As expected, the pervious concrete FWD deflections were greater than that of conventional concrete because its surface has pore spaces and it is not as rigid as the conventional concrete.

Table 43 Comparison between the pervious concrete and conventional concrete

Pervious Concrete							
Load (lb)	Sensor spacing (in.)						
	0	8	12	18	24	36	60
6000	15.76	13.49	12.17	10.24	8.71	5.94	2.53
9000	22.66	19.53	17.69	15.05	12.72	8.62	3.63
12000	30.30	26.11	23.74	20.14	17.10	11.61	4.90

Conventional Concrete							
Load (lb)	Sensor spacing (in.)						
	0	8	12	18	24	36	60
6000	3.95	3.65	3.46	3.17	2.85	2.19	1.29
9000	5.88	5.48	5.19	4.74	4.29	3.32	1.96
12000	7.33	6.81	6.43	5.88	5.32	4.14	2.45

The FWD deflection basin for the pervious concrete is shown in Figure 42. The FWD deflection from the load of 12000 lb is greater than that of 6000 lb and 9000 lb. This deflection basin is not as parabolic as that of flexible pavements.

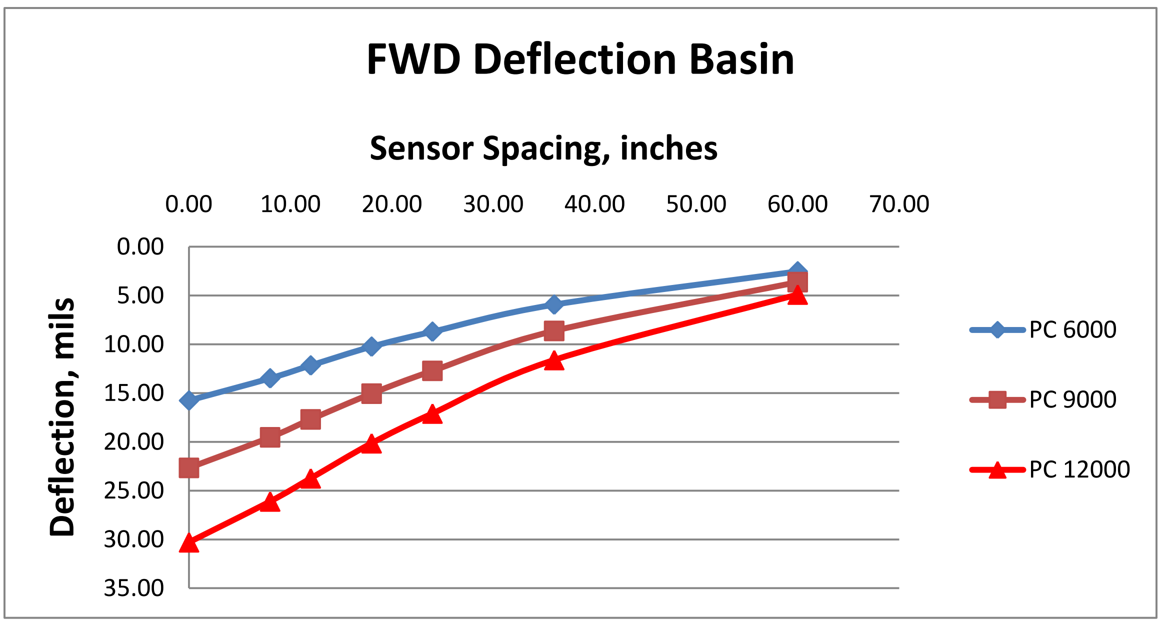


Figure 42 FWD deflection basins for Pervious concrete

The FWD deflection basin for the conventional concrete is shown in Figure 43. The FWD deflection from the load of 12000 lb is greater than that of 6000 lb and 9000 lb. This concrete slab had no reinforcement installed. This deflection basin is not as parabolic as that of conventional asphalt because of its rigidity.

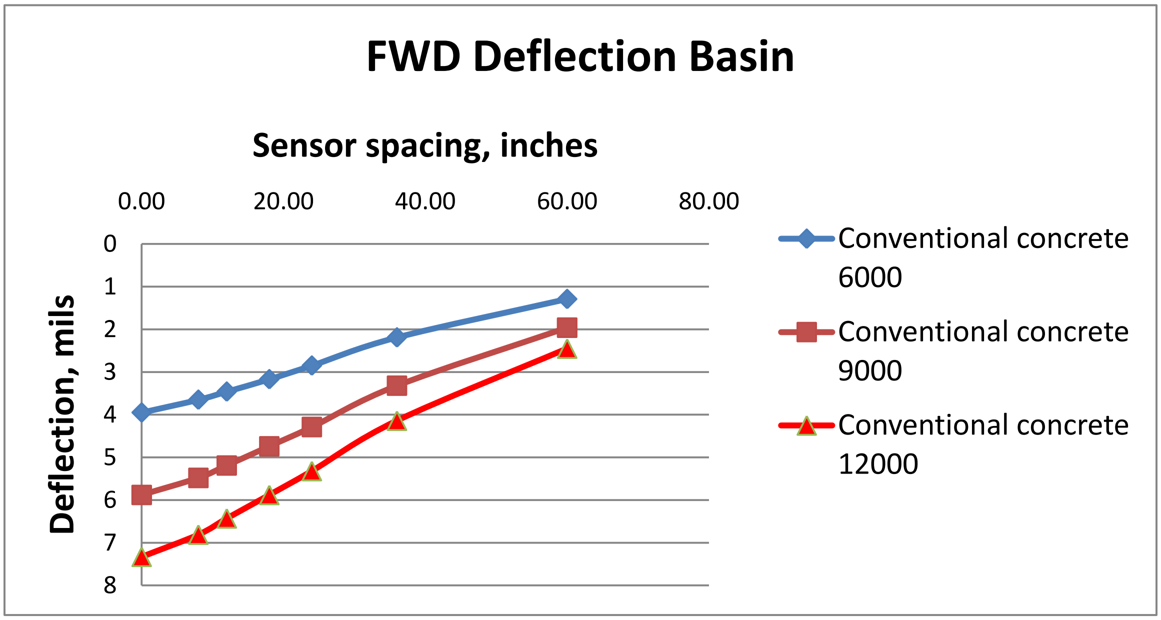


Figure 43 FWD deflection basin of conventional concrete

Table 44 compares the backcalculated surface elastic moduli for the various pervious pavements with value stated in past literature. The in-situ elastic modulus for porous asphalt ranges from 300 – 1100 ksi. Conventional asphalt from this study falls within 100 – 1500 ksi and from literature 100 – 1500 ksi (Liu, et al., 2001). Flexipave® elastic modulus was between 20 – 230 ksi because of the flexibility of this pavement, the FWD deflection reading was erroneous. The deflections especially at the point of load application surpassed the maximum allowable deflection value by the FWD equipment used (129 mils). FWD should not be used for determining the modulus of Flexipave® type of pavements.

The in-situ elastic modulus of pervious concrete ranges from 740 – 1350 ksi compared to 725 – 2900 ksi published in literature (Rohne, et al., 2009). The conventional concrete resilient modulus ranges from 3000 – 7700 ksi. Modulus 6.0 does not give precise result when used to calculate the elastic moduli of rigid pavements. The elastic modulus of porous aggregate and recycled glass pavement is 150 and 850 ksi respectively.

Table 44 Comparison of backcalculated in-situ elastic moduli

Pavement Type	Backcalculated Elastic Moduli (ksi)	
	Test	Literature
Porous Asphalt	300 - 1100	-
Conventional Asphalt	100 - 1500	100 - 1500
Flexi-pave®	20 - 230	-
Porous Aggregate	150	-
Recycled glass (Filterpave®)	850	-
Pervious Concrete	740 – 1350	725 - 2900
Conventional Concrete	3000 - 7700	2000 - 6000

Pavement Layer Thickness Design

The flexible pavements analyzed in the SMART laboratory are Porous Asphalt and Flexipave®. The backcalculated moduli values and traffic data were used in this program to design the layer thickness. Given equivalent single axle load (ESAL) and resilient moduli (M_R), the required structural number for 90% and 95% reliability are shown in Table 45 and Table 46, respectively. At a given degree of certainty, as the resilient modulus increases for a given traffic load the required structural number (SN_r) increases, indicating that the strength of the pavement system increases as the resilient modulus increases.

Table 45: Required Structural Number for 90% Reliability level

	Resilient Modulus range, M_R (psi)									
ESAL	4,000	6,000	8,000	10,000	12,000	14,000	15,000	16000	17000	18000
100,000	2.9	2.5	2.3	2.1	2.0	1.8	1.8	1.7	1.7	1.7
150,000	3.1	2.7	2.4	2.2	2.1	2.0	1.9	1.9	1.8	1.8
200,000	3.2	2.8	2.5	2.3	2.2	2.1	2.0	2.0	1.9	1.9
250,000	3.4	2.9	2.6	2.4	2.3	2.1	2.1	2.0	2.0	1.9
300,000	3.5	3.0	2.7	2.5	2.3	2.2	2.1	2.1	2.0	2.0
350,000	3.5	3.1	2.8	2.6	2.4	2.3	2.2	2.2	2.1	2.1
400,000	3.6	3.1	2.8	2.6	2.4	2.3	2.3	2.2	2.1	2.1
450,000	3.7	3.2	2.9	2.7	2.5	2.4	2.3	2.2	2.2	2.1
500,000	3.7	3.2	2.9	2.7	2.5	2.4	2.3	2.3	2.2	2.2
550,000	3.8	3.3	3.0	2.7	2.6	2.4	2.4	2.3	2.3	2.2
600,000	3.8	3.3	3.0	2.8	2.6	2.5	2.4	2.3	2.3	2.2
650,000	3.9	3.4	3.0	2.8	2.6	2.5	2.4	2.4	2.3	2.3
700,000	3.9	3.4	3.1	2.8	2.7	2.5	2.5	2.4	2.3	2.3
750,000	3.9	3.4	3.1	2.9	2.7	2.5	2.5	2.4	2.4	2.3
800,000	4.0	3.5	3.1	2.9	2.7	2.6	2.5	2.5	2.4	2.3
850,000	4.0	3.5	3.2	2.9	2.7	2.6	2.5	2.5	2.4	2.4
900,000	4.0	3.5	3.2	3.0	2.8	2.6	2.6	2.5	2.4	2.4
1,000,000	4.1	3.6	3.2	3.0	2.8	2.7	2.6	2.5	2.5	2.4
1,500,000	4.4	3.8	3.5	3.2	3.0	2.8	2.8	2.7	2.6	2.6
2,000,000	4.5	4.0	3.6	3.3	3.1	3.0	2.9	2.8	2.8	2.7

Table 46 shows the required structural number obtained from the AASHTO empirical equation for a given 95% reliability level. At a given degree of certainty (95%), as the resilient modulus increases for a given traffic load the required structural number (SN_r) increases, indicating that the strength of the pavement system increases as the resilient modulus increases.

Table 46: Required Structural Number for 95% Reliability level

	Resilient Modulus range, M_R (psi)									
ESAL	4,000	6,000	8,000	10,000	12,000	14,000	15,000	16000	17000	18000
100,000	3.1	2.7	2.4	2.2	2.1	2.0	1.9	1.9	1.8	1.8
150,000	3.3	2.9	2.6	2.4	2.2	2.1	2.0	2.0	1.9	1.9
200,000	3.4	3.0	2.7	2.5	2.3	2.2	2.1	2.1	2.0	2.0
250,000	3.6	3.1	2.8	2.6	2.4	2.3	2.2	2.2	2.1	2.1
300,000	3.7	3.2	2.9	2.7	2.5	2.3	2.3	2.2	2.2	2.1
350,000	3.7	3.3	2.9	2.7	2.5	2.4	2.3	2.3	2.2	2.2
400,000	3.8	3.3	3.0	2.8	2.6	2.5	2.4	2.3	2.3	2.2
450,000	3.9	3.4	3.0	2.8	2.6	2.5	2.4	2.4	2.3	2.3
500,000	3.9	3.4	3.1	2.9	2.7	2.5	2.5	2.4	2.4	2.3
550,000	4.0	3.5	3.1	2.9	2.7	2.6	2.5	2.5	2.4	2.4
600,000	4.0	3.5	3.2	2.9	2.8	2.6	2.6	2.5	2.4	2.4
650,000	4.1	3.6	3.2	3.0	2.8	2.6	2.6	2.5	2.5	2.4
700,000	4.1	3.6	3.3	3.0	2.8	2.7	2.6	2.6	2.5	2.4
750,000	4.2	3.6	3.3	3.1	2.9	2.7	2.6	2.6	2.5	2.5
800,000	4.2	3.7	3.3	3.1	2.9	2.7	2.7	2.6	2.5	2.5
850,000	4.2	3.7	3.4	3.1	2.9	2.8	2.7	2.6	2.6	2.5
900,000	4.3	3.7	3.4	3.1	2.9	2.8	2.7	2.7	2.6	2.5
1,000,000	4.3	3.8	3.4	3.2	3.0	2.8	2.8	2.7	2.6	2.6
1,500,000	4.6	4.0	3.7	3.4	3.2	3.0	2.9	2.9	2.8	2.8
2,000,000	4.8	4.2	3.8	3.5	3.3	3.1	3.1	3.0	2.9	2.9

For a reliability level of 90% and at a given traffic load, as the modulus of subgrade reaction increases, the minimum thickness of the rigid pervious pavement decreases as shown in Table 47. For a given modulus of subgrade reaction, the minimum slab thickness increases as the traffic load increases.

Table 47: Minimum Thickness in inches for 90% Level or Reliability

	Modulus of Subgrade Reaction, k (psi/in)									
ESAL	50	70	100	120	150	200	250	300	360	400
100,000	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
150,000	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
200,000	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
250,000	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
300,000	6.2	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
350,000	6.4	6.2	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
400,000	6.6	6.4	6.1	6.0	6.0	6.0	6.0	6.0	6.0	6.0
450,000	6.8	6.5	6.2	6.0	6.0	6.0	6.0	6.0	6.0	6.0
500,000	6.9	6.7	6.4	6.2	6.0	6.0	6.0	6.0	6.0	6.0
550,000	7.0	6.8	6.5	6.3	6.1	6.0	6.0	6.0	6.0	6.0
600,000	7.2	6.9	6.7	6.5	6.2	6.0	6.0	6.0	6.0	6.0
650,000	7.3	7.1	6.8	6.6	6.3	6.0	6.0	6.0	6.0	6.0
700,000	7.4	7.2	6.9	6.7	6.5	6.0	6.0	6.0	6.0	6.0
750,000	7.5	7.3	7.0	6.8	6.6	6.2	6.0	6.0	6.0	6.0
800,000	7.5	7.3	7.1	6.9	6.7	6.3	6.0	6.0	6.0	6.0
850,000	7.6	7.4	7.2	7.0	6.8	6.4	6.0	6.0	6.0	6.0
900,000	7.7	7.5	7.3	7.1	6.9	6.5	6.1	6.0	6.0	6.0
1,000,000	7.9	7.7	7.4	7.3	7.0	6.7	6.3	6.0	6.0	6.0
1,500,000	8.4	8.2	8.0	7.9	7.7	7.4	7.1	6.7	6.2	6.0
2,000,000	8.8	8.7	8.4	8.3	8.1	7.8	7.6	7.3	6.9	6.6

In Table 48, for a reliability level of 95% and at a given traffic load, as the modulus of subgrade reaction increases, the minimum thickness of the rigid pervious pavement decreases. For a given modulus of subgrade reaction, the minimum slab thickness increases as the traffic load increases.

Table 48: Minimum Thickness in inches for 95% Level or Reliability

	Modulus of Subgrade Reaction, k (psi/in)									
ESAL	50	70	100	120	150	200	250	300	360	400
100,000	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
150,000	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
200,000	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
250,000	6.3	6.1	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
300,000	6.5	6.3	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
350,000	6.8	6.5	6.2	6.0	6.0	6.0	6.0	6.0	6.0	6.0
400,000	6.9	6.7	6.4	6.2	6.0	6.0	6.0	6.0	6.0	6.0
450,000	7.1	6.9	6.6	6.4	6.1	6.0	6.0	6.0	6.0	6.0
500,000	7.2	7.0	6.7	6.6	6.3	6.0	6.0	6.0	6.0	6.0
550,000	7.4	7.2	6.9	6.7	6.5	6.1	6.0	6.0	6.0	6.0
600,000	7.5	7.3	7.0	6.8	6.6	6.2	6.0	6.0	6.0	6.0
650,000	7.6	7.4	7.1	7.0	6.7	6.3	6.0	6.0	6.0	6.0
700,000	7.7	7.5	7.2	7.1	6.9	6.5	6.1	6.0	6.0	6.0
750,000	7.8	7.6	7.3	7.2	7.0	6.6	6.2	6.0	6.0	6.0
800,000	7.9	7.7	7.4	7.3	7.1	6.7	6.3	6.0	6.0	6.0
850,000	8.0	7.8	7.5	7.4	7.2	6.8	6.4	6.0	6.0	6.0
900,000	8.0	7.9	7.6	7.5	7.3	6.9	6.6	6.1	6.0	6.0
1,000,000	8.2	8.0	7.8	7.6	7.4	7.1	6.8	6.4	6.0	6.0
1,500,000	8.8	8.6	8.4	8.2	8.1	7.8	7.5	7.2	6.8	6.5
2,000,000	9.2	9.0	8.8	8.7	8.5	8.2	8.0	7.7	7.4	7.2

Temperature corrector was applied to the base modulus. Four (4) optimized designs were suggested in the output. The suggested maximum surface depth is 3 inch. The cost estimation of each design is evaluated. A summary of these designs can be seen below.

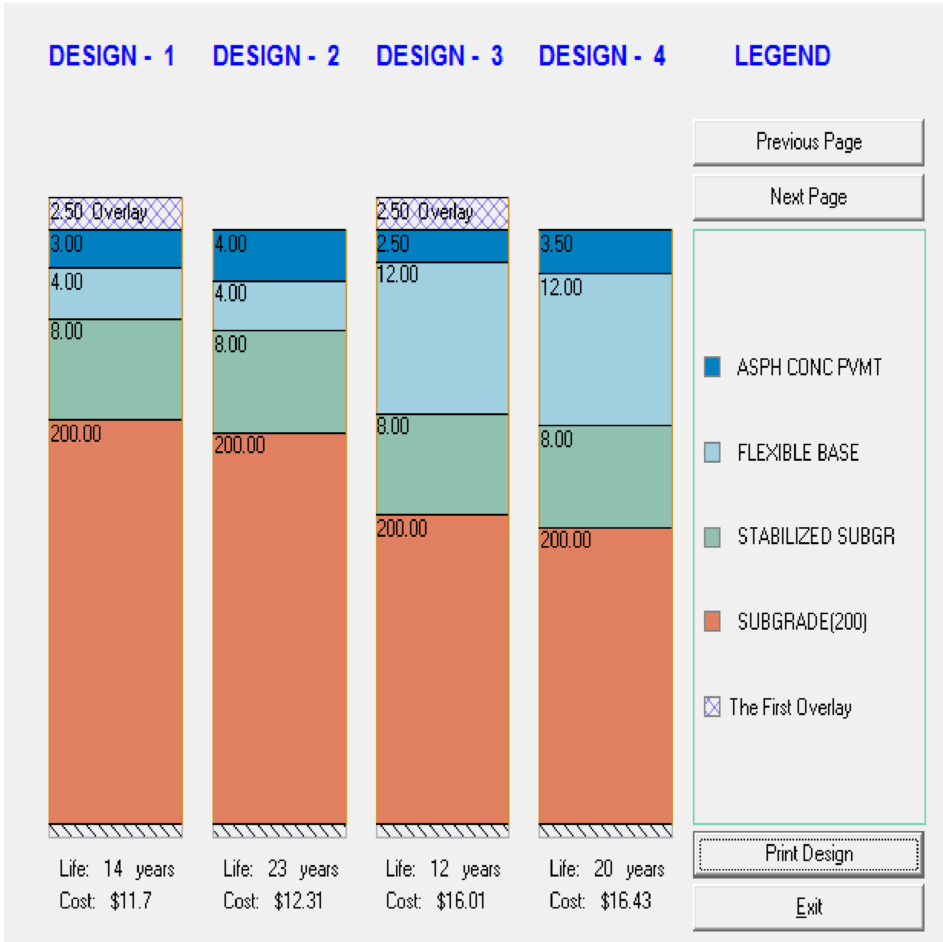


Figure 44: Pavement layer Plot

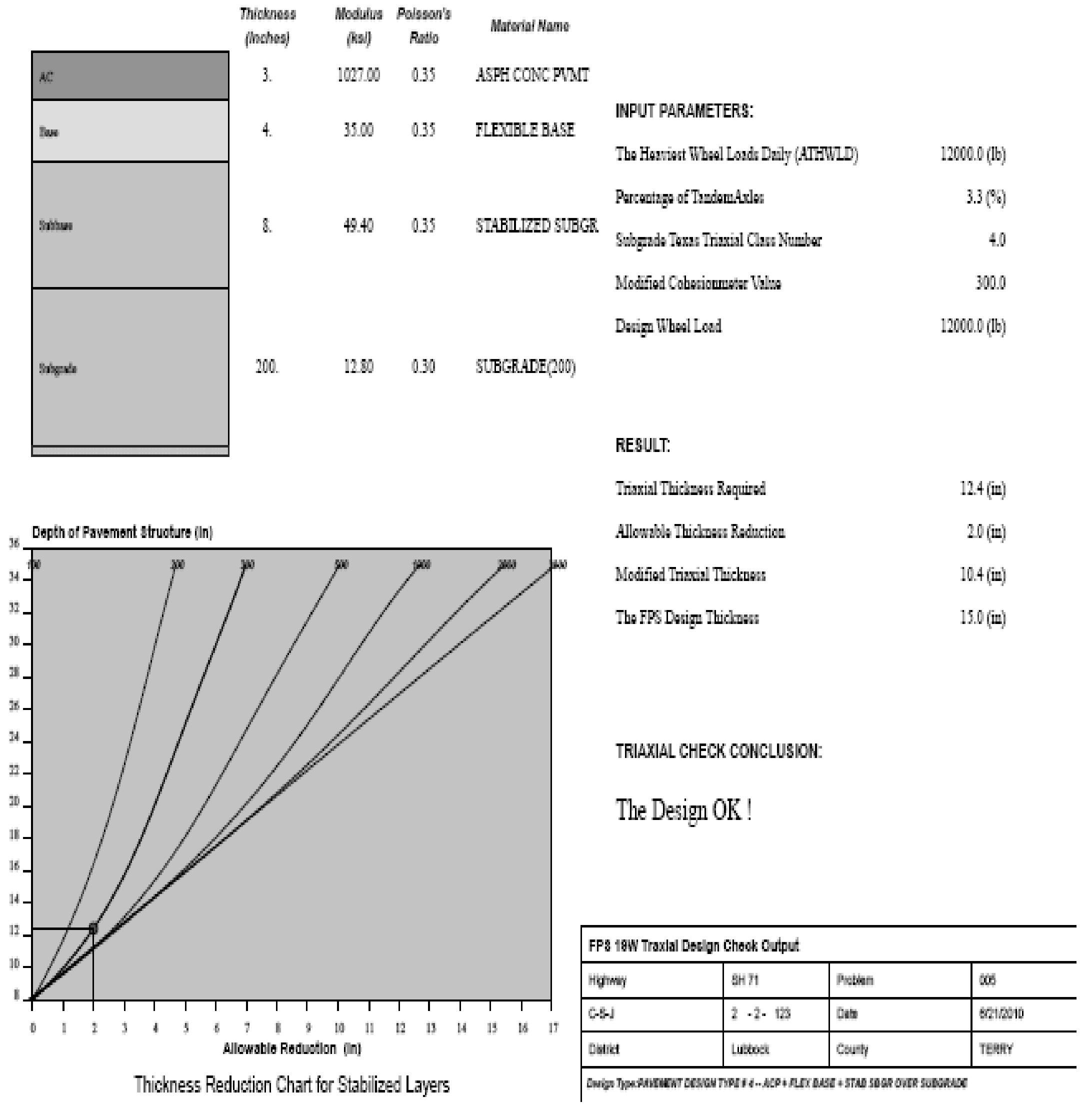


Figure 45: Texas Triaxial Design check (TTC)

The mechanistic design check makes use of fatigue analysis to determine the number of repetitive loads for crack to start from the bottom up to the top surface. In addition, it performed a rutting check by initially assuming a100 psi tire with 9000 lb load. The design is satisfactory if the design life (in millions) is less than the crack life and rut life (in millions).

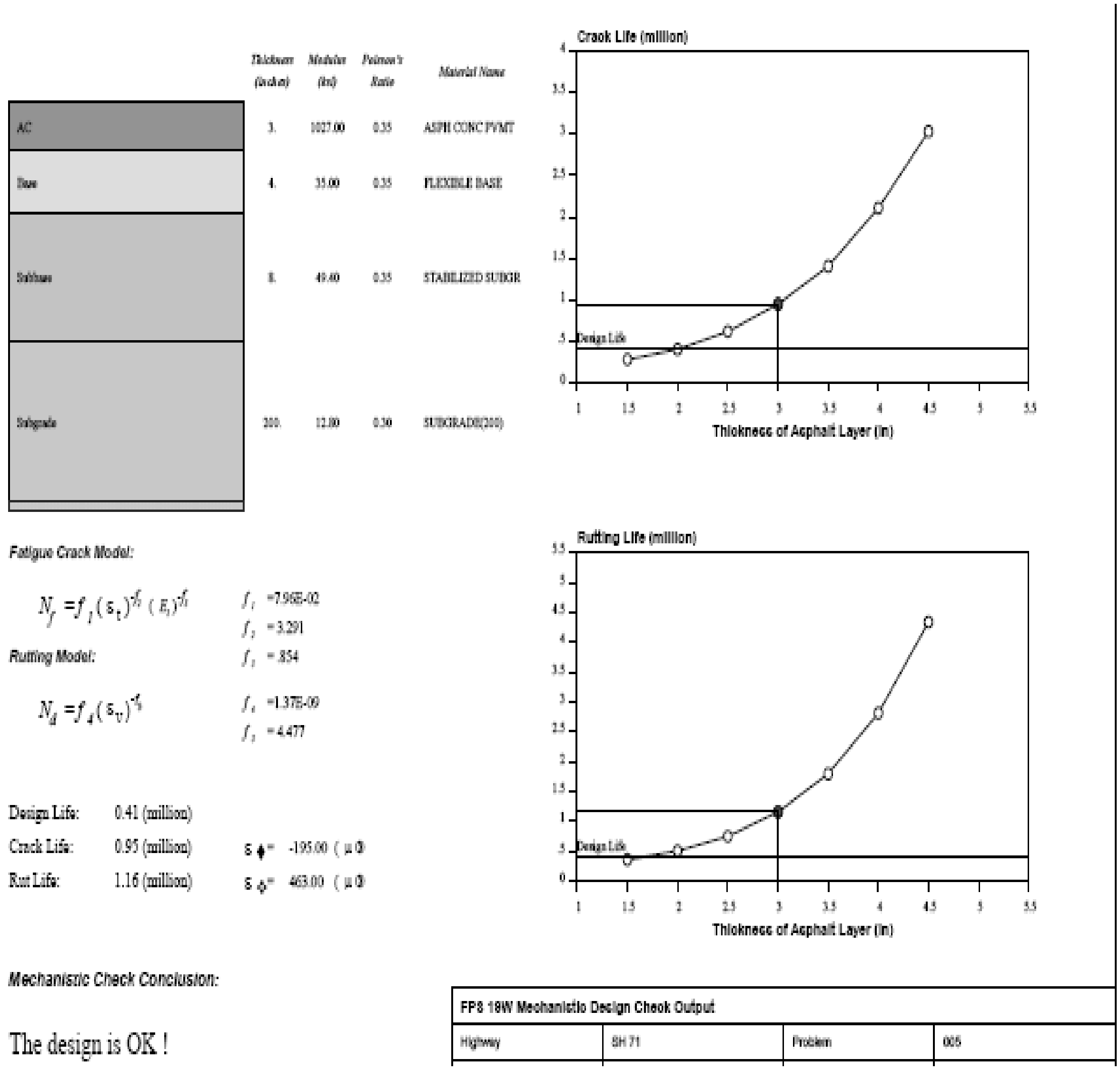
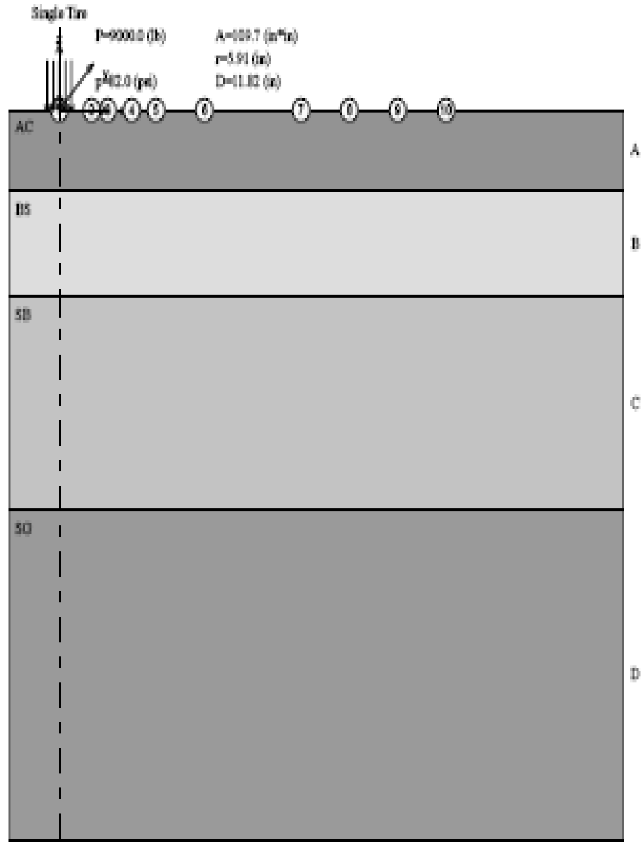


Figure 46: Mechanistic Design check

Furthermore, a single tire with FWD load of 9000 lb will be used and data from FWD testing will be used to calculate the displacements in the sensor points after the design life of the pavement.



Sensor points	Location (in.)	Projected Displacement at the design life (mils)
Sensor 1	0	17.70
Sensor 2	8	13.50
Sensor 3	12	10.90
Sensor 4	18	8.20
Sensor 5	24	6.45
Sensor 6	36	4.29
Sensor 7	60	2.19

Figure 47: FWD load points and projected design life deflections

<u>KENPAVE Program</u>

Displacement obtained from KENLAYER program (Huang, 2004) was compared with that from the Falling weight deflectometer (FWD) data measured at each sensor point. The pavement layers were assumed to be linearly elastic in the KENLAYER software just like the MODULUS 6.0 software. The displacement values obtained from KENPAVE varied from the FWD test because KENPAVE builds an empirical pavement model and analyzes based on the some parameters such as the backcalculated elastic moduli from FWD testing. In the summary tables shown of the three porous asphalt sections analyzed the highest variation in displacement values was noticed at the last sensor (60 inches from the load). The smallest variation occurs at the second sensor point (8 inches from the load).

Table 49: Comparison of KENPAVE and FWD Deflections for PAR

For 6000 lb load							
Sensor spacing	0	8	12	18	24	36	60
δ (KENPAVE)	11.29	8.78	7.71	6.5	5.19	3.85	2.38
δ (FWD Data)	10.1	8.21	7.04	5.72	4.55	3.22	1.69
% Variation	11.8	6.9	9.5	13.6	14.1	19.6	40.8
For 9000 lb load							
Sensor spacing	0	8	12	18	24	36	60
δ (KENPAVE)	17.21	13.49	11.85	9.96	7.82	5.79	3.55
δ (FWD Data)	15.03	12.3	10.53	8.53	6.93	4.76	2.49
% Variation	14.5	9.7	12.5	16.8	12.8	21.6	42.6
For 12000 lb load							
Sensor spacing	0	8	12	18	24	36	60
δ (KENPAVE)	22.67	17.73	15.6	13.19	10.44	7.84	4.87
δ (FWD Data)	20.53	16.77	14.39	11.64	9.39	6.5	3.39
% Variation	10.4	5.7	8.4	13.3	11.2	20.6	43.7

Table 50: Comparison of KENPAVE and FWD Deflections for PAF

For 6000 lb load							
Sensor spacing	0	8	12	18	24	36	60
δ (KENPAVE)	11.88	8.48	7.23	5.37	4.38	3.07	1.85
δ (FWD Data)	10.33	8.15	6.58	4.99	3.83	2.51	1.38
% variation	15	4.0	9.9	7.6	14.4	22.3	34.1
For 9000 lb load							
Sensor spacing	0	8	12	18	24	36	60
δ (KENPAVE)	17.96	12.8	10.89	8.18	6.67	4.79	2.94
δ (FWD Data)	16.1	12.69	10.25	7.8	6.05	4.02	2.13
% variation	11.6	0.9	6.2	4.9	10.2	19.2	38.0
For 12000 lb load							
Sensor spacing	0	8	12	18	24	36	60
δ (KENPAVE)	24.56	17.73	15.11	11.22	9.13	6.4	3.87
δ (FWD Data)	21.01	16.71	13.64	10.43	8.11	5.36	2.85
% variation	16.9	6.1	10.8	7.6	12.6	19.4	35.8

Table 51: Comparison of KENPAVE and FWD Deflections for PABG

For 6000 lb load							
Sensor spacing	0	8	12	18	24	36	60
δ (KENPAVE)	15.52	11.03	9.41	7.03	5.8	4.13	2.52
δ (FWD Data)	12.68	9.88	8.15	6.38	5	3.41	1.77
% Variation	22.4	11.6	15.5	10.2	16.0	21.1	42.4
For 9000 lb load							
Sensor spacing	0	8	12	18	24	36	60
δ (KENPAVE)	22.78	16.25	13.95	10.53	8.65	6.05	3.68
δ (FWD Data)	18.63	14.72	12.19	9.54	7.51	5.05	2.65
% Variation	22.3	10.4	14.4	10.4	15.2	19.8	38.9
For 12000 lb load							
Sensor spacing	0	8	12	18	24	36	60
δ (KENPAVE)	31.23	22.3	19.15	14.45	11.84	8.24	4.99
δ (FWD Data)	25.44	20.18	16.76	13.13	10.27	6.9	3.58
% Variation	22.8	10.5	14.3	10.1	15.3	19.4	39.4

This same comparison was also done for the recycled rubber pavement sections. On site while running the FWD test, it was observed that the displacement at the point of load application was above the accepted allowable displacement for the equipment used. This error was as a result of the large void spaces in this pavement section and its excessive flexibility. The FWD equipment only allows a maximum deflection of 129 mils. Table 46 – 48 present the results of the deflection from the KENPAVE program only under the point of impact. Sensor at 60 in has more error due to the assumption of linear elasticity when using KENPAVE.

Table 52: Comparison of KENPAVE and FWD Deflections for FPF

For 6000 lb load							
Sensor spacing	0	8	12	18	24	36	60
δ(Kenpave)	224.32	37.07	15.19	9.15	7.11	4.77	2.9
δ (FWD Data)	NA	48.67	14.29	8.79	6.14	4.11	1.52
% Variation	73.9	-23.8	6.3	4.1	15.8	16.1	90.8
For 9000 lb load							
Sensor spacing	0	8	12	18	24	36	60
δ (KENPAVE)	279.05	60.38	24.08	14.53	12.31	8.57	5.19
δ (FWD Data)	NA	63.38	16.03	11.62	7.96	5.6	2.44
% Variation	116.3	-4.7	50.2	25.0	54.6	53.0	112.7
For 12000 lb load							
Sensor spacing	0	8	12	18	24	36	60
δ (KENPAVE)	164.48	58.82	28.17	12.93	10.13	7.71	4.53
δ (FWD Data)	NA	88.34	28.92	15.91	9.49	6.03	4.71
% Variation	27.5	-33.4	-2.6	-18.7	6.7	27.9	-3.8

*NA Not Applicable

Table 53: Comparison of KENPAVE and FWD Deflections for FPR

For 6000 lb load							
Sensor spacing	0	8	12	18	24	36	60
δ (KENPAVE)	180.04	29.26	13.05	10.26	8.71	5.96	3.57
δ (FWD Data)	NA	34.88	11.84	9.59	6.83	4.43	2.76
% Variation	68.1	-16.1	10.2	7.0	27.5	34.5	29.3
For 9000 lb load							
Sensor spacing	0	8	12	18	24	36	60
δ (KENPAVE)	228.78	38.28	18.48	14.09	11.63	7.97	4.79
δ (FWD Data)	NA	46.05	17.57	13.84	9.3	5.58	3.09
% Variation	77.3	-16.9	5.2	1.8	25.1	42.8	55.0
For 12000 lb load							
Sensor spacing	0	8	12	18	24	36	60
δ (KENPAVE)	202.11	50.48	25.09	17.01	14.31	9.8	5.94
δ (FWD Data)	NA	58.93	23.29	18.63	12.02	7.32	4.43
% Variation	56.7	-14.3	7.7	-8.7	19.1	33.9	34.1

*NA Not Applicable

Table 54: Comparison of KENPAVE and FWD Deflections for FPBG

For 6000 lb load							
Sensor spacing	0	8	12	18	24	36	60
δ (KENPAVE)	188.29	59.38	30.29	14.23	9.16	5.96	3.71
δ (FWD Data)	NA	73.77	27.19	16.74	7.71	4.73	2.75
% Variation	46.0	-19.5	11.4	-15.0	18.8	26.0	34.9
For 9000 lb load							
Sensor spacing	0	8	12	18	24	36	60
δ (KENPAVE)	169.42	86.59	51.18	21.99	10.82	6.02	4.13
δ (FWD Data)	NA	111.71	43.41	22.3	8.85	5.88	4.2
% Variation	31.3	-22.5	17.9	-1.4	22.3	2.4	-1.7
For 12000 lb load							
Sensor spacing	0	8	12	18	24	36	60
δ (KENPAVE)	157.06	82.86	53.09	25.71	12.77	4.91	3.4
δ (FWD Data)	NA	116.71	56.24	26.66	8.57	5.74	3.46
% Variation	21.8	-29.0	-5.6	-3.6	49.0	-14.5	-1.7

*NA Not Applicable

This chapter presented the results of the laboratory and field testing on the different types of pervious pavement systems. The following chapter lists the conclusions from this study and presents recommendations for future research on this topic.

CONCLUSION AND RECOMMENDATIONS

A detailed analysis and testing study of the structural properties the existing pervious pavement driveways at the Stormwater Management Academy laboratory was conducted in this study. From this study, the in-situ modulus of elasticity for porous asphalt pavements was determined to range from 300 -1100 ksi. While that of conventional asphalt concrete ranges from 100 – 1500 ksi. Depending on the deterioration of the pavement, the conventional asphalt pavement has modulus that is greater by a factor of 1.5 – 3 times that of porous asphalt. Pervious concrete was found to have a range of modulus of elasticity of 740 – 1350 ksi. This is comparable to the elastic modulus value of 725 - 2900 ksi specified in literature. Typical elastic moduli for conventional concrete ranged from 2000 – 6000 ksi. The FWD results show that impervious concrete has a lower deflection than pervious concrete. The deflection of pervious concrete is greater than conventional concrete by a factor of 1.5 – 5 depending on the applied load.

Permeable pavers have in-place modulus of elasticity range of 45 – 320 ksi. These moduli values for permeable pavers depend on the type, orientation, shape, joint and size of the paver. Recycled rubber tire pavement has an in-situ resilient moduli range of 20 – 230 ksi. Lastly, the recycled glass and porous aggregate pervious pavements had moduli of about 850 ksi and 150 ksi respectively. Therefore, the recycled glass pavement elastic modulus falls within the range of the pervious concrete and is able to withstand heavier loads than the aggregate-based pavement.

From this study, the deflection value of the porous asphalt is less than that of conventional asphalt. This suggests that during the FWD test on the porous asphalt pavement section, the measured deflection was not the response of the pavement system under the load application. Instead it was the rebound displacement of the pavement surface. Therefore, falling weight deflectometer (FWD) test should not be used for testing porous asphalt and Flexi-pave® because of the high flexibility and rebound potential.

There are no exact mix designs for pervious pavements that will produce high mechanical properties. Laboratory testing and field testing performed on existing pavement systems is the best method of establishing a range of values which will lead to an acceptable design.

It should be emphasized that the use of pervious pavements should be limited to areas with low volume traffic. The accumulated 18 kip equivalent single axle load (ESAL) of approximately 412,000 was estimated as the load the pavement will be subjected to during its design life. The summary tables at different reliability levels in the previous chapter show the effect of traffic loading on the structural capacity of the pavement. For the flexible pavements, as the resilient moduli increases for a certain reliability level under a given traffic load, the structural strength of the pavement increases. In rigid pavements, at a given degree of certainty and traffic load, as the modulus of subgrade reaction increases the minimum thickness of the rigid pervious pavement decreases.

It is also observed that the unit weight is an important factor in determining the compressive strength of the pervious pavements. A correlation is provided in this study and can act as a guide in obtaining the resulting compressive strength based on a known value of unit weight. Some of the 28-day pervious concrete (PC) samples that were tested for compression were poorly mixed and probably were not properly compacted resulting in large void space. These were considered to be outliers. The average compressive strength of the cored samples was about 1725 psi which falls within that specified in literature (1643 – 2500) psi. The 28-day PC compressive strength was within the 365 – 1100 psi range as compared to a typical value of around 2000 psi in literature. The average compressive strength of the recycled glass pavement (Filterpave®) was found to be 1160 psi while the manufacturer specification reports it as being 1000 psi. Compressive strength is not a desired method of testing recycled rubber pavement.

The flexural strength of 28-day PC tested ranges from 198 – 280 psi, while it is reported as 150 – 550 psi in literature. The average compressive strength of Filterpave® is 508 psi which is greater than that shown in the manufacturer's specification.

The software program Modulus 6.0 did not give accurate moduli results for recycled rubber tire pavement because its surface thickness is only 2 inches and also because of the errors in the FWD deflection reading.

The FWD deflections for permeable pavers were not very consistent which resulted in significant errors. The surface area at the joints was not adequate. This pavement system is not a continuous unit, so when load is applied the responses in form of surface waves will be interrupted in the pavement system. The aggregates used to fill the void spaces in the paver easily wash out during a rain event, vacuuming or human activities leading to more discontinuities. Therefore, FWD testing procedure should not be performed on permeable pavers.

Rutting, cracking and massive clogging of porous asphalt affected the in-situ backcalculated moduli values. Although recycled glass pavement has a higher compressive strength compared to aggregate based pavements, which suggests that it can withstand heavier load just like pervious concrete it is found to be slippery especially when wet or under dew conditions. The recycled rubber tire pavement is found to be one of the best solutions besides pervious concrete. It is slip resistant, very flexible, with a high porosity even when it is loaded with sediments.

Curbing the sides of the pavements with conventional concrete curbs is very important because it acts as a support to the pavement system especially pavers and prevents erosion of the subbase. Maintenance vehicles such as vacuuming truck and garbage trucks can be driven on these pavements but the areas where impact loads are applied are easily damaged.

The tables for the various design factors provided in this study are to be used mainly as a guidelines to choose the factors for design. By selecting the right pavement layer material and assigning the subgrade moduli one can design the pervious system thickness under the estimated traffic loads.

Recommendations for Future Research

Future research endeavors should be undertaken to address some limitations of the present research. The aggregate size in the recycled glass pavement needs to be varied. This will help in discovering the effect of aggregate gradation, permeability and unit weight on the compressive strength. Either the mix design should be provided to the researcher or the manufacturer should provide representative samples. The rutting tendency of recycled rubber tire should be studied by means of the Hamburg test (Grzybowski, 2005). Pervious concrete strength need to be studied further by adding admixtures, sand and fibers so as to understand the relationship between these parameters.

The FWD impact load is actually a dynamic load. Therefore, a dynamic back-calculation analysis procedure should be done to compare the results obtained in this static back-calculation process. Parameters such as the loading frequency and time histories should be researched.

Permeable concrete paver systems have not yet being effectively studied as regards a holistic approach in evaluating their strength property. A better testing method should be researched which could test this pavement system as a whole unit. The joint orientation, layer thickness, rainfall data, traffic data should be used to analyze the design.

Ground Penetration Radar (GPR) and the FWD should be used in conjunction to provide information as regards pavement performance. The GPR is used to accurately estimate the thickness of concrete (especially if this information is not known), hot mix asphalt (HMA) and other composite pavements. It can be employed in the design, rehabilitation and management process.

Pervious pavements are limited to areas that are subjected to low traffic loading but can occasionally be used by tandem and tridem axle trucks. Strain gauges can be positioned in the pavements and its displacement recorded each time a truck of a certain load, similar to the effect of the impact hammer of the FWD, passes over it.

Accurate traffic data should be collected. Road tubes, permanent loop sensors and some other devices can be used to collect traffic data from existing sites. Collection of data should be carried out on every day of the week throughout the year.

The scope of the current study covers significant new grounds but like every successful research there is always room for improvement. Therefore, the implementation of the outlined future studies can lead to improvements in pervious pavement technology.

APPENDIX A
BACKCALCULATION ANALYSIS

ASSUMPTION:

Linear Elastic model

INPUTS:

Average Temperature

Load Applications – 6000lb, 9000 lb and 12000 lb

Recycled rubber tire pavement

BACKCALCULATION ANALYSIS			
Pavement	FPF	FPR	FPBG
$E_{surface}$ 6000 (ksi)	20.5	28.3	40.1
E_{base} 6000(ksi)	2.0	2.0	2.9
$E_{subbase}$ 6000(ksi)	6.4	22.9	2.5
$E_{subgrade}$ 6000(ksi)	10.7	8.8	7.8
Abs error/sens (%)	8.87	6.53	15.44
$E_{surface}$ 9000(ksi)	49.7	29.6	217.2
E_{base} 9000(ksi)	2.0	2.5	2.0
$E_{subbase}$ 9000(ksi)	14.9	18.7	2.5
$E_{subgrade}$ 9000(ksi)	11.2	9.4	10.1
Abs error/sens (%)	12.43	6.75	16.01
$E_{surface}$ 12000(ksi)	180.5	72.5	238.4
E_{base} 12000(ksi)	2.0	3.1	10
$E_{subbase}$ 12000(ksi)	32.2	21.4	2.1
$E_{subgrade}$ 12000(ksi)	13.1	10.1	16
Abs error/sens (%)	12.96	8.97	21.11
FATIGUE ANALYSIS			
Pavements	FPF	FPR	FPBG
Average temp.	83.3	88	89.3
Design SCI Mils (6000)	113.85	102.17	82.97
W7 (mils)	1.24	3.3	3.41
Layer Strength (6000)			
Upper (6000)	VP	VP	VP
Lower (6000)	VP	VP	VP
Subgrade (6000)	GD	PR	VP

Remaining Life 6000 (yrs)			
Rut (6000)	0-2	0-2	0-2
Crack (6000)	0-2	0-2	0-2
Design SCI Mils (9000)	66.78	82.05	18.26
W7 (mils)	1.68	2.43	3.75
Layer Strength 9000			
Upper (9000)	VP	VP	GD
Lower (9000)	VP	VP	VP
Subgrade (9000)	MD	VP	PR
Remaining Life 9000 (yrs)			
Rut (9000)	0-2	0-2	0-2
Crack (9000)	0-2	0-2	> 7
Design SCI Mils (12000)	29.68	51.67	9.3
W7 (mils)	3.22	2.74	2.25
Layer Strength 12000			
Upper (12000)	PR	VP	GD
Lower (12000)	VP	VP	VP
Subgrade (12000)	PR	VP	PR
Remaining Life 12000 (yrs)			
Rut (12000)	0-2	0-2	0-2
Crack (12000)	>2	0-2	>9

Permeable Paver

BACKCALCULATION ANALYSIS			
Pavement	PPBG	PPR	PPF
$E_{surface}$ 6000 (ksi)	95.8	99.1	253.7
E_{base} 6000(ksi)	11.1	18.7	6.1
$E_{subbase}$ 6000(ksi)	33.1	116.9	300
$E_{subgrade}$ 6000(ksi)	14.5	14.7	13
Abs error/sens (%)	3.93	5.74	6.6
$E_{surface}$ 9000(ksi)	119.2	121.9	303.7
E_{base} 9000(ksi)	9.8	11.6	7
$E_{subbase}$ 9000(ksi)	76.2	122.3	206.3
$E_{subgrade}$ 9000(ksi)	15	15	13.3
Abs error/sens (%)	3.22	5.59	5.93
$E_{surface}$ 12000(ksi)	126	149.5	378.7
E_{base} 12000(ksi)	11	9.2	5.1
$E_{subbase}$ 12000(ksi)	81.3	115.9	300
$E_{subgrade}$ 12000(ksi)	15.8	15.5	13.4
Abs error/sens (%)	3.55	5.26	5.98

FATIGUE ANALYSIS			
Pavements	PPBG	PPR	PPF
Average temp.	61.7	63.7	63
Design SCI Mils (6000)	22.53	19.65	13.77
W7 (mils)	2.0	1.62	1.86
Layer Strength (6000)			
Upper (6000)	PR	PR	MD
Lower (6000)	PR	PR	VP
Subgrade (6000)	PR	MD	PR
Remaining Life 6000 (yrs)			
Rut (6000)	>5	2-5	>5
Crack (6000)	<6	>5	>9
Design SCI Mils (9000)	21.82	18.43	12.57
W7 (mils)	1.99	1.6	1.89
Layer Strength 9000			
Upper (9000)	PR	PR	MD
Lower (9000)	PR	PR	VP
Subgrade (9000)	PR	MD	PR
Remaining Life 9000 (yrs)			
Rut (9000)	>5	>5	>8
Crack (9000)	<6	>5	> 9
Design SCI Mils (12000)	21.76	17.74	11.79
W7 (mils)	1.96	1.59	1.86
Layer Strength 12000			
Upper (12000)	PR	PR	MD
Lower (12000)	PR	PR	PR
Subgrade (12000)	PR	MD	PR
Remaining Life 12000 (yrs)			
Rut (12000)	>5	>5	>8
Crack (12000)	>5	>5	10+

Other Permeable Pavers

BACKCALCULATION ANALYSIS					
Pavement	HPBG	HPI	HPR	HPF	Porous Agg
$E_{surface}$ 6000 (ksi)	176.1	87.4	159.1	41.8	103.8
E_{base} 6000(ksi)	5.9	3.8	11.2	25.1	8.2
$E_{subbase}$ 6000(ksi)	24.3	249	28.7	14.4	10.1
$E_{subgrade}$ 6000(ksi)	11	9.1	10.3	14.3	10.4
Abs error/sens (%)	4.62	4.34	2.2	2.98	1.3
$E_{surface}$ 9000(ksi)	66.5	193	201	74.1	119.4
E_{base} 9000(ksi)	14	3.9	4.7	24.5	9.3
$E_{subbase}$ 9000(ksi)	14.2	189.3	408.8	18	10
$E_{subgrade}$ 9000(ksi)	12.3	10.9	9.9	14.2	10.4
Abs error/sens (%)	4.4	8.33	7.82	3.04	1.62
$E_{surface}$ 12000(ksi)	125.7	171.6	190.2	118.5	116.3
E_{base} 12000(ksi)	10.2	5.4	24.2	23.4	11.8
$E_{subbase}$ 12000(ksi)	28.3	490.9	42.2	28.1	7.1
$E_{subgrade}$ 12000(ksi)	12.5	11	10.1	14	10.7
Abs error/sens (%)	5.1	6.01	3.24	2.85	1.37
FATIGUE ANALYSIS					
Pavements	HPBG	HPI	HPR	HPF	Porous Agg
Average temp.	65.7	67.3	69	69.3	63.7
Design SCI Mils (6000)	27.36	45.26	17.88	22.07	27.13
W7 (mils)	2.33	2.03	2.36	1.94	2.17
Layer Strengths 6000					
Upper (6000)	VP	VP	PR	VP	VP
Lower (6000)	PR	VP	PR	MD	VP
Subgrade (6000)	VP	PR	PR	PR	PR
Remaining life 6000(yrs)					
Rut (6000)	0-2	0-2	0-2	0-2	0-2
Crack (6000)	3	0-2	<6	3	0-2
Design SCI Mils (9000)	31.74	15.31	45.01	19.73	25.43
W7 (mils)	2.22	2.27	2.41	1.91	2.22
Layer Strengths 9000					
Upper (9000)	VP	PR	PR	PR	VP
Lower (9000)	PR	VP	PR	MD	VP
Subgrade (9000)	VP	VP	VP	PR	VP
Remaining life 9000 (yrs)					
Rut (9000)	0-2	0-2	0-2	0-2	0-2
Crack (9000)	0-2	<9	>7	<5	0-2
Design SCI Mils (12000)	32.11	30.17	32.74	18.47	24.21
W7 (mils)	2.02	2.25	2.22	1.90	2.19

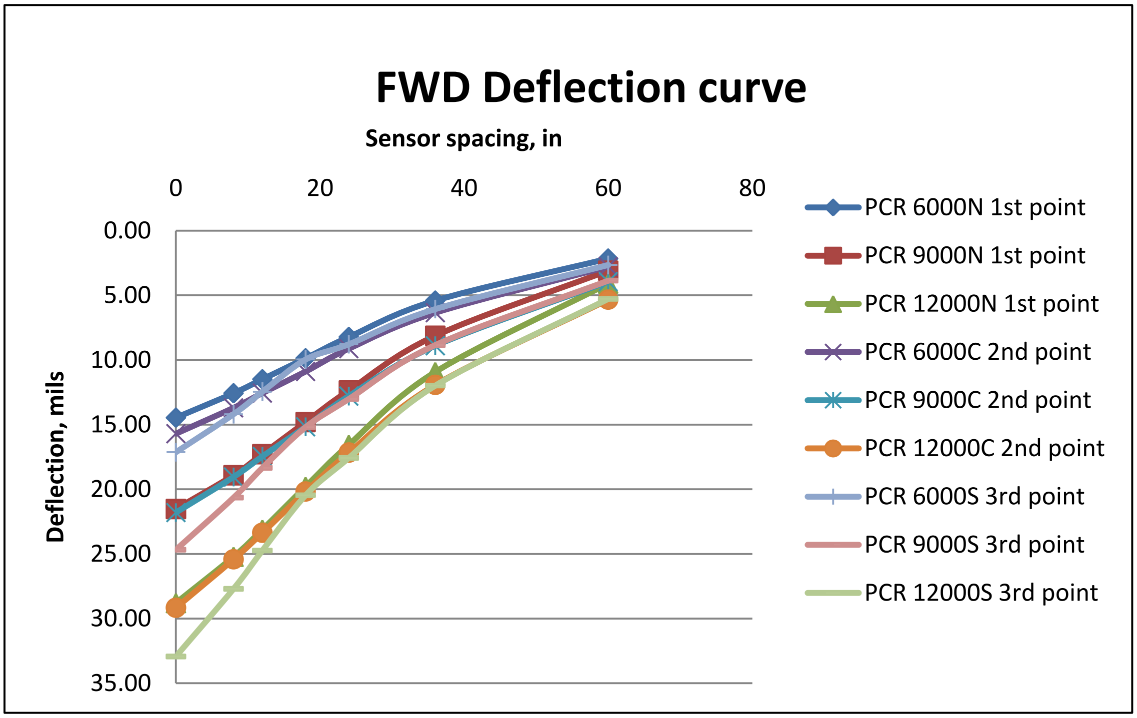
Layer Strengths 12000					
Upper (12000)	PR	PR	PR	PR	VP
Lower (12000)	PR	VP	PR	MD	VP
Subgrade (12000)	PR	VP	PR	PR	VP
Remaining life 12000(yrs)					
Rut (12000)	0-2	0-2	0-2	0-2	0-2
Crack (12000)	>4	>6	>7	2-5	0-2

DEFLECTION AND DEFLECTION BASINS

i) Pervious Concrete

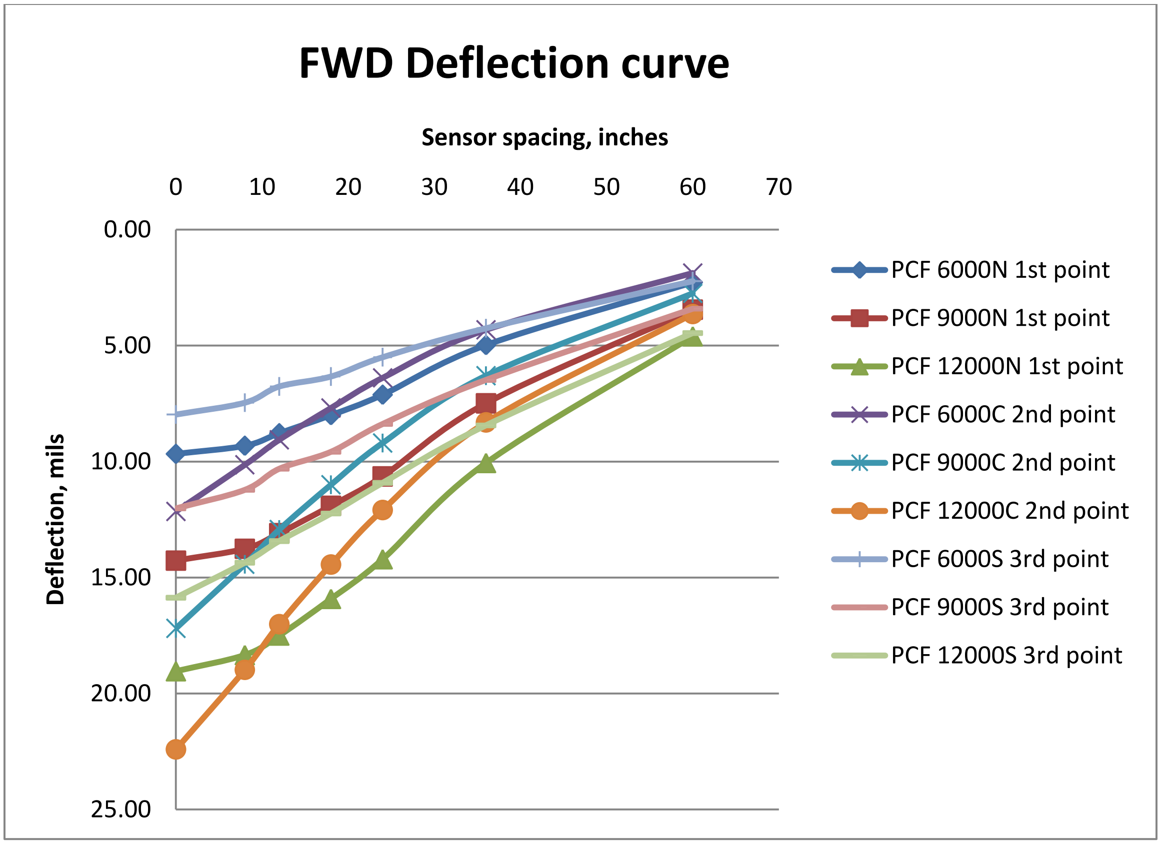
For PCR:

Load	0	8	12	18	24	36	60
6291	14.46	12.57	11.49	9.87	8.22	5.41	2.16
9270	21.52	18.91	17.28	14.80	12.36	8.13	3.09
11939	28.80	25.22	23.12	19.80	16.56	10.93	4.09
6471	15.70	13.70	12.56	10.89	9.12	6.34	2.77
9135	21.80	19.05	17.46	15.17	12.79	8.90	3.94
11852	29.16	25.41	23.36	20.17	17.16	11.92	5.33
6399	17.13	14.19	12.45	9.96	8.78	6.06	2.65
9212	24.67	20.64	18.34	15.19	13.01	8.84	3.86
11947	32.93	27.70	24.74	20.46	17.57	11.98	5.27



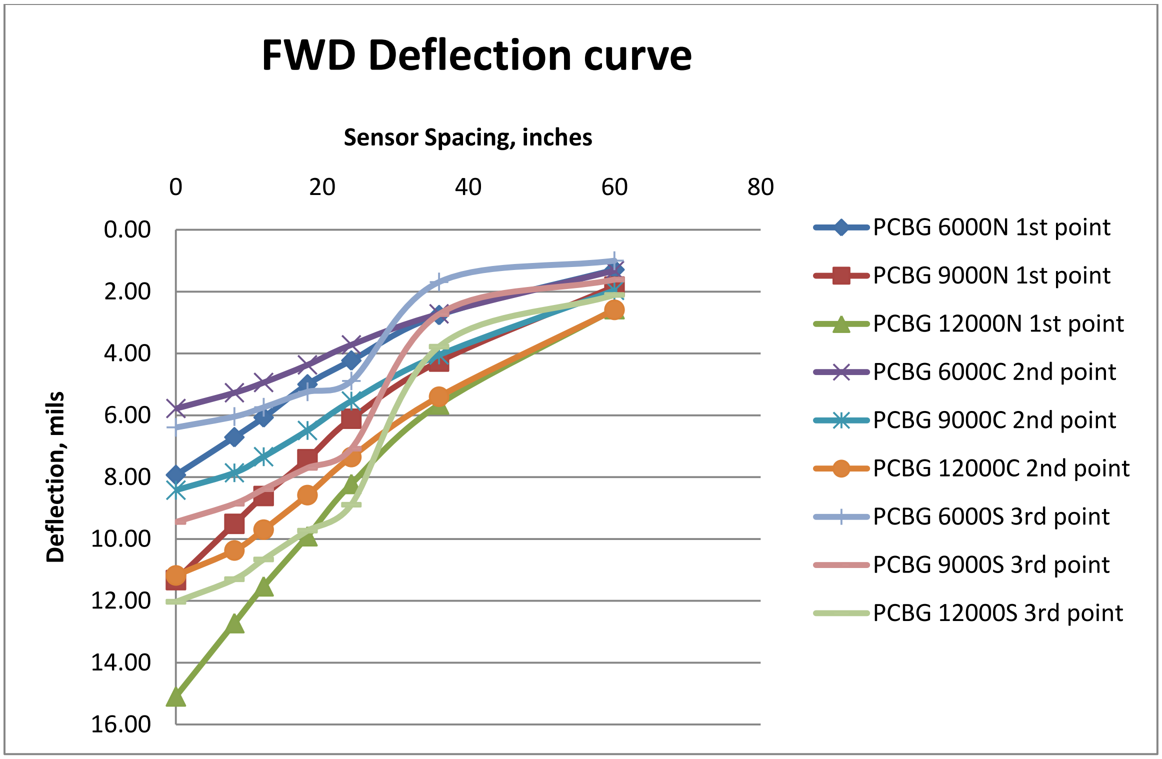
For PCF:

Load	0	8	12	18	24	36	60
6291	9.67	9.32	8.78	8.00	7.12	4.98	2.28
9143	14.27	13.76	13.09	11.91	10.64	7.49	3.45
11812	19.04	18.34	17.50	15.92	14.21	10.06	4.60
6387	12.15	10.14	9.06	7.69	6.39	4.32	1.87
9207	17.19	14.43	12.93	11.00	9.20	6.30	2.75
11836	22.41	18.98	17.01	14.44	12.09	8.31	3.64
6268	7.97	7.46	6.76	6.33	5.50	4.25	2.22
9366	12.02	11.22	10.31	9.58	8.38	6.48	3.41
11857	15.87	14.36	13.43	12.24	10.92	8.44	4.46



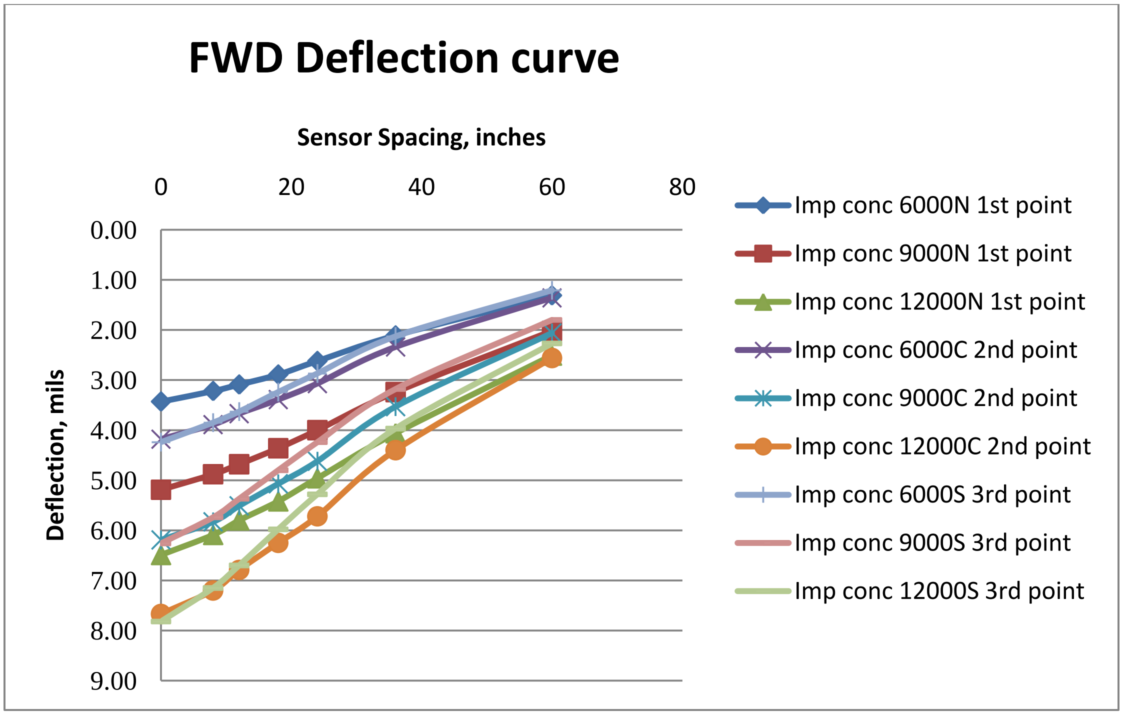
For PCBG:

Load	0	8	12	18	24	36	60
6347	7.93	6.71	6.07	5.00	4.23	2.76	1.29
9124	11.33	9.51	8.61	7.42	6.11	4.26	1.84
11889	15.09	12.72	11.53	9.91	8.22	5.65	2.57
6447	5.78	5.27	4.94	4.37	3.72	2.72	1.32
9262	8.42	7.86	7.34	6.49	5.54	4.05	1.96
11912	11.18	10.37	9.70	8.58	7.35	5.40	2.59
6442	6.39	6.04	5.74	5.24	4.89	1.69	1.00
9418	9.45	8.86	8.39	7.70	7.08	2.73	1.61
11844	12.03	11.30	10.66	9.73	8.89	3.78	2.11



For Impervious Concrete:

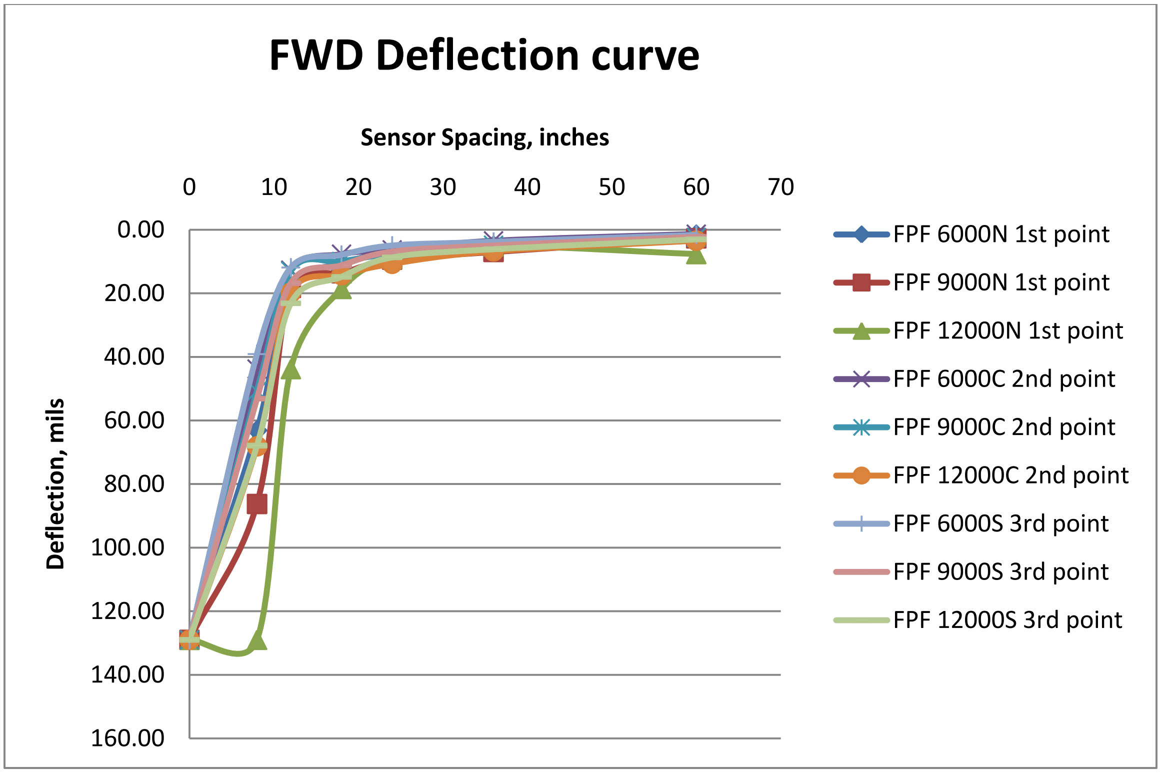
Load	0	8	12	18	24	36	60
6336	3.43	3.22	3.09	2.89	2.62	2.11	1.31
9588	5.19	4.88	4.68	4.36	4.00	3.24	2.02
11804	6.49	6.09	5.80	5.42	4.96	4.06	2.51
6411	4.18	3.89	3.67	3.39	3.07	2.34	1.36
9609	6.19	5.83	5.51	5.07	4.62	3.53	2.06
11725	7.67	7.20	6.79	6.25	5.72	4.40	2.56
6506	4.24	3.85	3.63	3.24	2.87	2.13	1.21
9577	6.26	5.74	5.37	4.80	4.24	3.18	1.80
11809	7.82	7.15	6.70	5.98	5.28	3.97	2.27



ii) Recycled rubber tire pavement

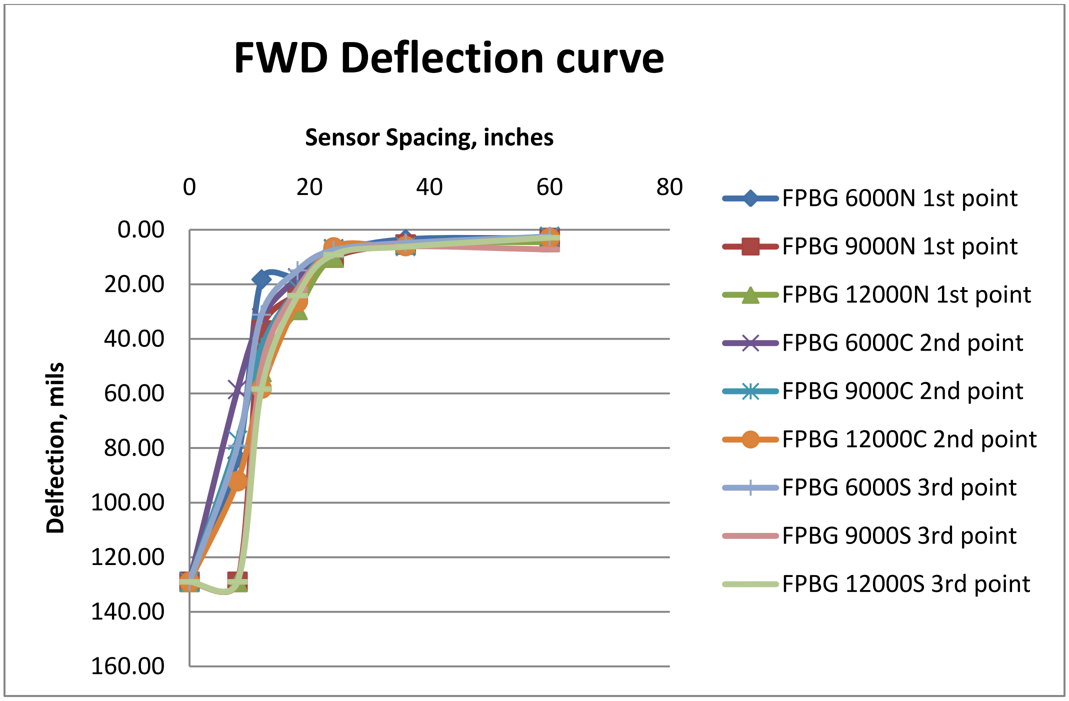
For FPF:

Load	0	8	12	18	24	36	60
6399	129.00	63.14	18.28	10.82	7.16	5.10	1.60
8651	129.00	86.26	18.29	13.69	9.40	7.00	2.69
12151	129.00	129.00	43.85	18.63	9.11	5.22	7.66
6260	129.00	43.78	12.72	7.68	6.32	3.48	1.35
8682	129.00	50.74	13.00	9.96	7.59	4.84	2.37
12373	129.00	68.13	19.78	14.29	10.70	6.76	3.44
6411	129.00	39.09	11.86	7.87	4.93	3.75	1.61
9124	129.00	53.14	16.81	11.20	6.89	4.97	2.26
12278	129.00	67.89	23.12	14.81	8.67	6.11	3.02



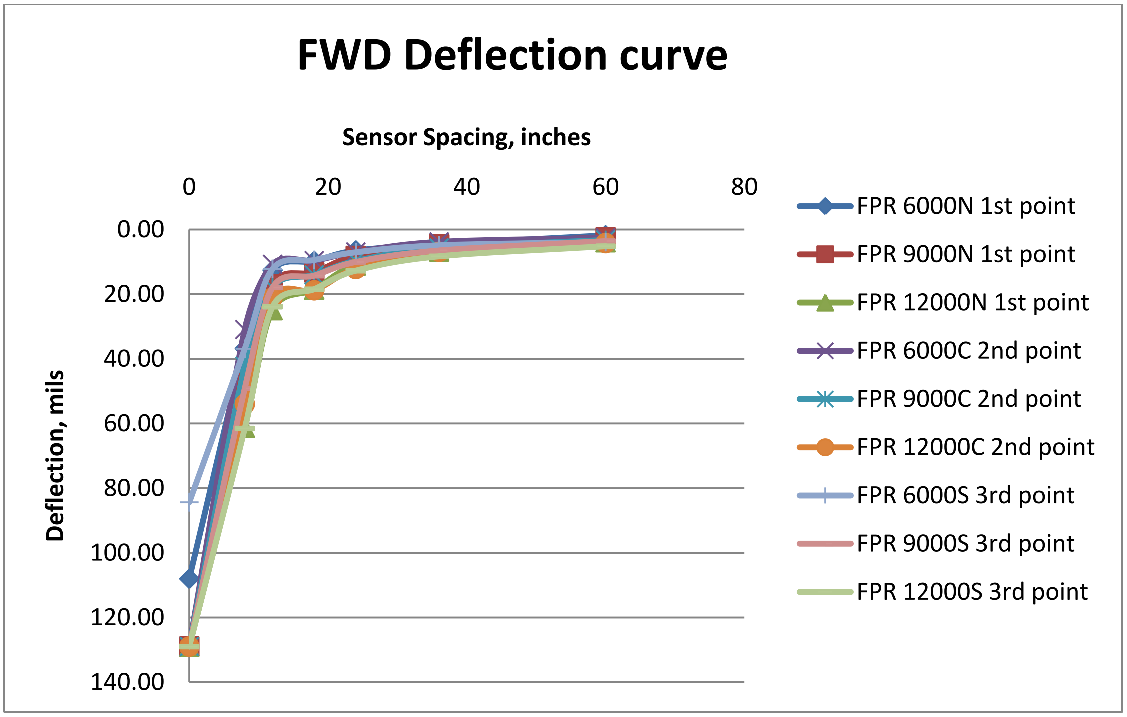
For FPBG:

Load	0	8	12	18	24	36	60
5891	129.00	83.64	18.21	17.73	8.72	3.48	3.12
8266	129.00	129.00	36.91	23.56	10.16	5.41	3.01
11563	129.00	129.00	52.16	29.38	10.11	5.03	4.50
6140	129.00	58.54	32.25	17.59	6.95	5.89	2.64
8520	129.00	77.13	43.00	21.53	7.22	6.15	2.36
11896	129.00	92.13	58.19	26.47	6.48	5.98	2.98
5867	129.00	79.13	31.12	14.91	7.46	4.83	2.49
9469	129.00	129.00	50.32	21.81	9.16	6.07	7.22
11221	129.00	129.00	58.38	24.12	9.13	6.20	2.91



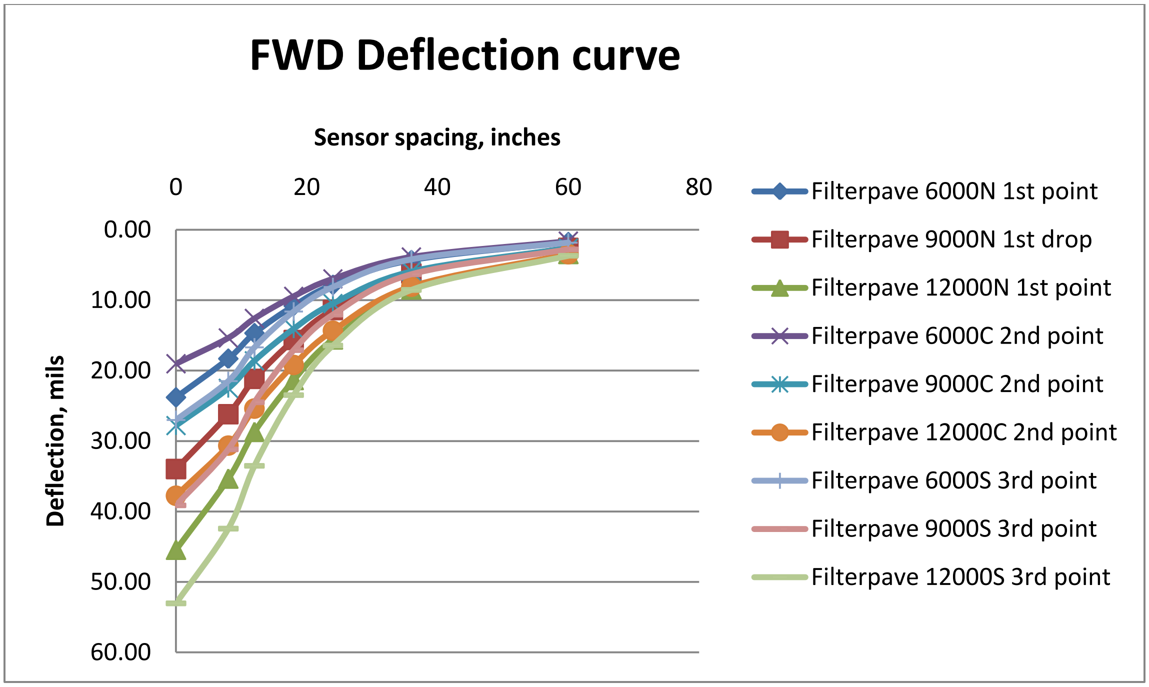
For FPR:

Load	0	8	12	18	24	36	60
6299	108.02	36.83	12.59	9.74	6.51	4.59	1.81
8822	129.00	46.69	16.88	13.26	8.19	4.56	2.46
12082	129.00	61.30	24.94	18.59	11.12	6.78	3.92
6399	129.00	30.93	10.74	9.61	6.96	3.84	2.61
9140	129.00	42.28	17.89	14.02	9.69	5.69	3.22
12381	129.00	53.96	21.04	18.81	12.20	6.87	4.17
6395	84.38	36.88	12.18	9.41	7.02	4.86	3.86
9370	129.00	49.18	17.94	14.25	10.03	6.48	3.58
12143	129.00	61.54	23.89	18.48	12.74	8.30	5.19



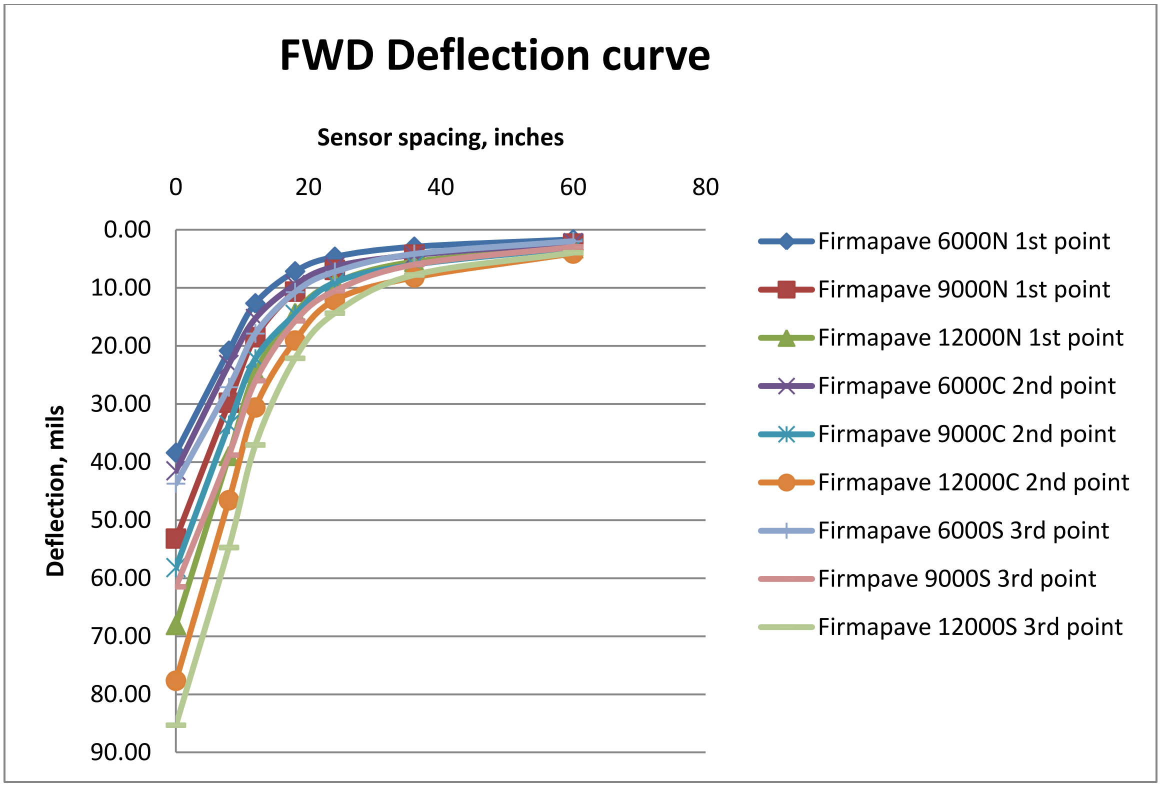
iii) Recycled glass pavement

Load	0	8	12	18	24	36	60
6363	23.79	18.30	14.68	10.80	7.83	4.31	1.76
9127	33.97	26.19	21.17	15.65	11.39	6.22	2.58
11936	45.43	35.33	28.69	21.33	15.57	8.48	3.44
6225	19.03	15.36	12.60	9.45	6.97	3.87	1.63
9135	27.80	22.51	18.60	14.06	10.48	5.92	2.60
12119	37.76	30.64	25.39	19.19	14.33	8.07	3.47
6236	27.00	21.48	16.70	11.61	8.14	4.20	1.83
9061	39.13	31.17	24.49	17.09	12.05	6.32	2.79
11939	53.04	42.43	33.51	23.47	16.43	8.54	3.69



iv) Porous Aggregate

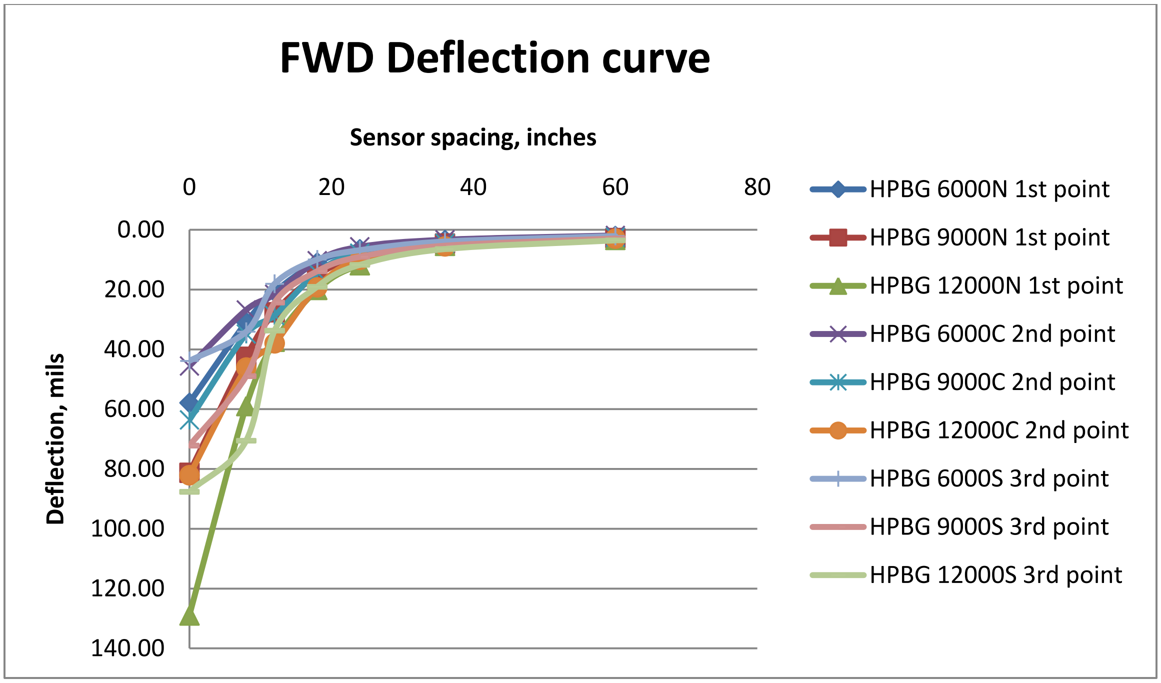
Load	0	8	12	18	24	36	60
6196	38.40	20.83	12.68	7.17	4.66	2.93	1.67
9056	53.18	29.78	18.50	10.65	6.90	4.27	2.39
12170	68.04	38.93	24.60	14.30	9.11	5.49	3.20
6082	41.51	23.18	15.28	9.46	6.35	4.36	2.20
8703	58.17	33.59	22.18	14.56	9.09	6.01	3.14
11844	77.65	46.58	30.61	19.07	12.13	8.21	4.10
6169	43.72	27.12	17.87	10.74	7.24	4.17	1.98
8767	61.49	38.80	26.00	15.70	10.50	5.98	2.95
11809	85.31	54.72	37.06	22.13	14.36	7.78	3.96



v) Other Permeable pavers:

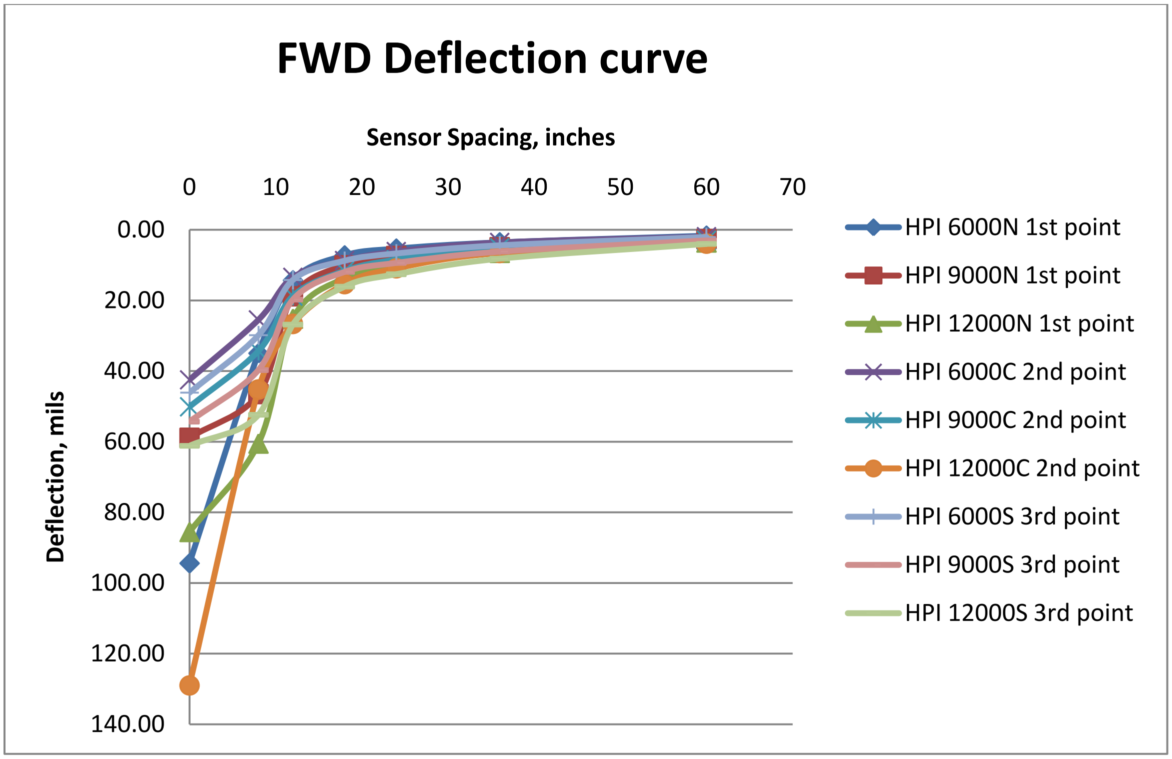
For HPBG:

Load	0	8	12	18	24	36	60
6371	57.85	30.95	20.80	10.96	6.55	3.46	2.11
9000	81.27	42.52	27.61	15.26	9.09	4.61	2.81
12032	129.00	58.96	37.45	20.11	11.86	5.37	3.39
6399	45.56	26.80	21.60	10.37	5.69	3.32	1.89
8997	63.59	34.87	28.66	14.31	7.81	4.50	2.47
12000	82.10	46.11	37.98	19.26	10.04	5.57	2.98
6323	43.86	34.09	18.07	9.93	6.81	3.90	2.09
8878	72.19	48.96	24.35	13.85	9.04	5.32	2.90
11984	87.64	70.54	33.74	19.06	11.76	6.46	3.56



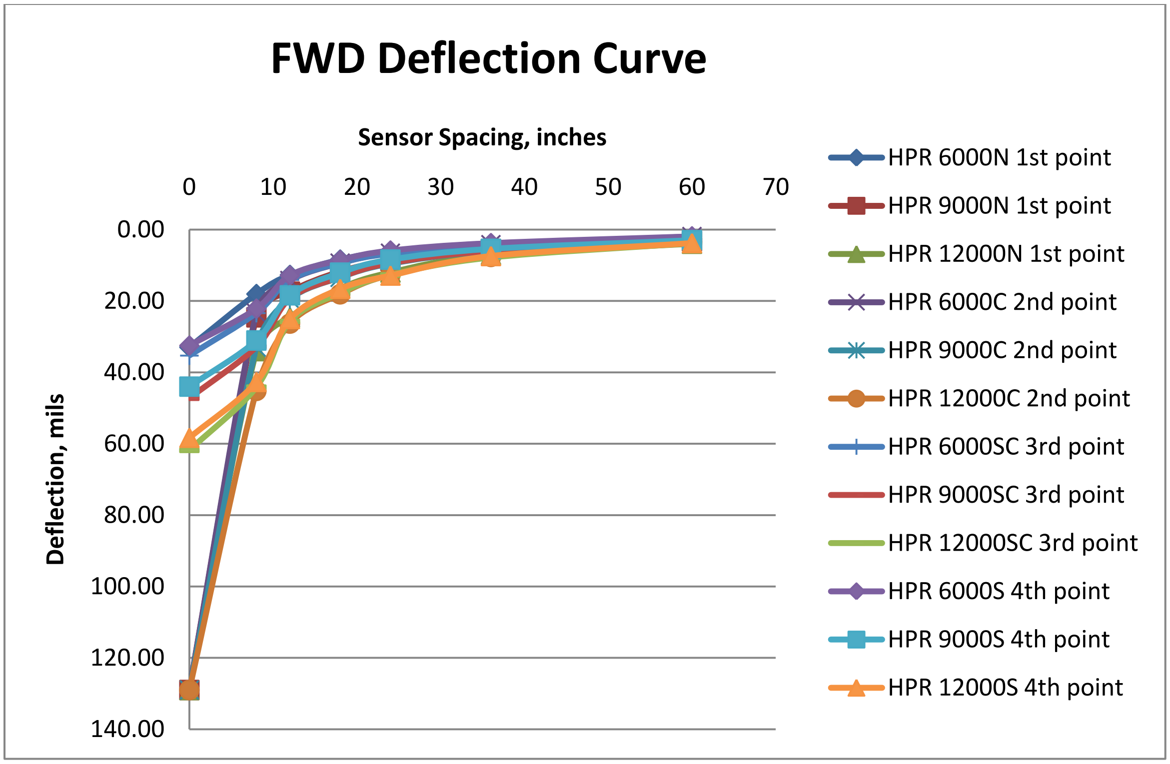
For HPI:

Load	0	8	12	18	24	36	60
6387	94.44	34.99	14.34	7.29	5.36	3.59	1.71
8897	58.89	46.52	18.85	10.05	7.44	4.94	2.65
12114	85.56	60.53	25.16	13.65	9.98	6.54	3.60
6156	42.48	25.51	13.48	8.72	6.17	3.61	1.97
8791	50.12	34.34	18.78	11.65	8.47	5.08	2.94
12130	129.00	45.24	26.66	15.38	10.91	6.55	3.92
6423	46.13	29.87	14.22	8.78	6.72	4.42	2.07
8806	54.33	39.72	19.76	11.93	9.37	6.19	3.00
12079	61.16	52.39	26.84	16.11	12.59	8.19	4.02



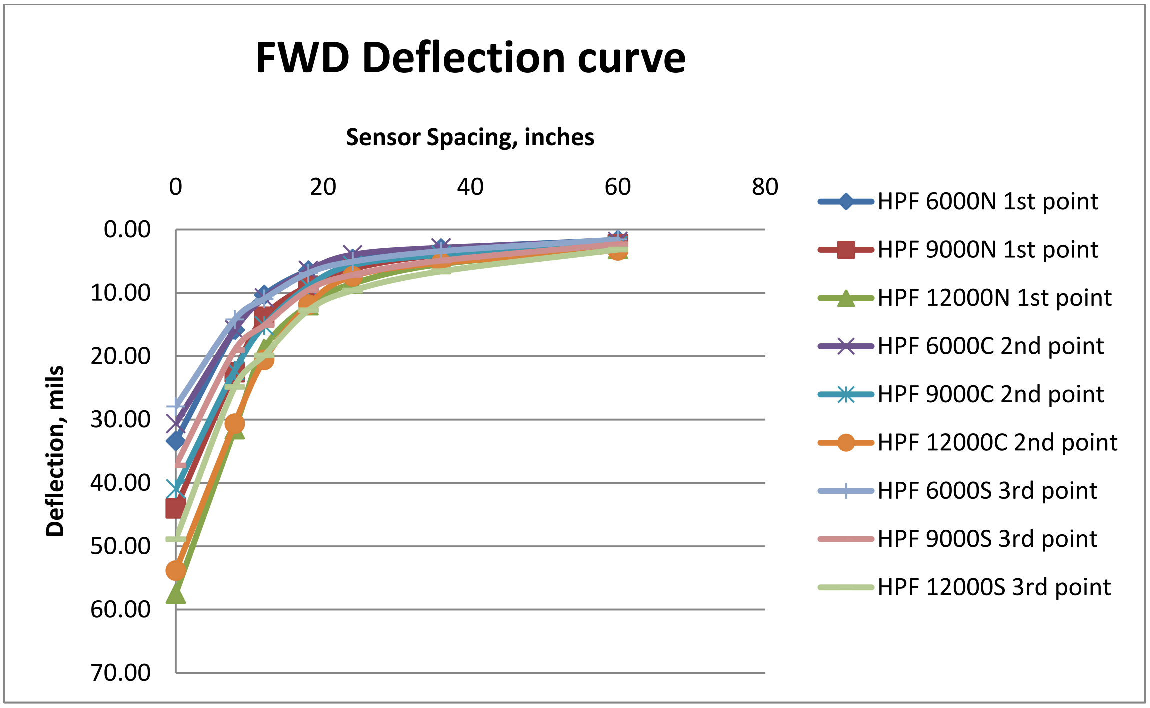
For HPR:

Load	0	8	12	18	24	36	60
6339	32.94	18.04	12.69	8.39	6.15	3.87	2.42
8889	129.00	24.56	17.39	11.74	8.76	5.42	3.20
11928	129.00	34.04	24.56	16.49	12.00	7.04	3.95
6320	129.00	23.39	14.25	9.39	6.59	4.21	1.94
8941	129.00	32.94	19.47	13.38	9.54	6.00	2.84
11908	129.00	45.20	26.35	18.11	12.70	7.66	3.66
6304	35.30	23.39	14.13	9.38	6.75	4.35	2.06
8973	47.26	32.83	19.33	13.35	9.40	5.98	2.95
12008	61.85	44.16	25.73	17.76	12.72	7.82	3.79
6403	32.50	22.15	12.88	8.54	5.89	3.81	1.91
9053	44.00	31.07	18.41	12.07	8.39	5.37	2.96
11920	58.34	42.68	25.04	16.60	12.89	7.42	3.80



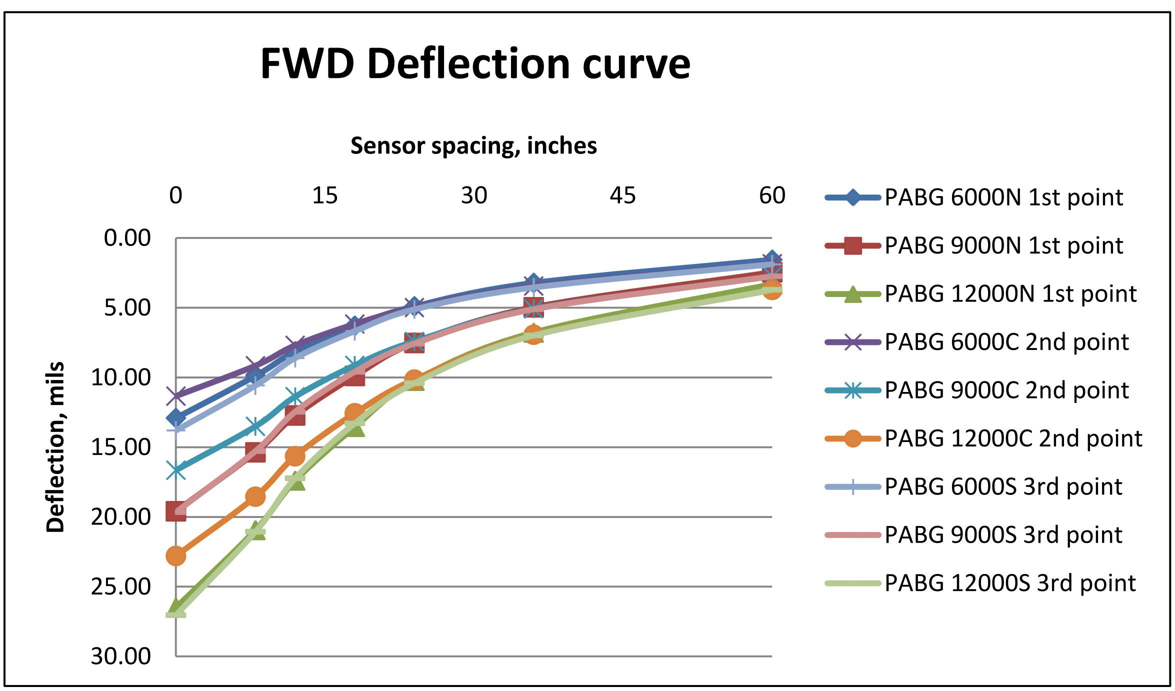
For HPF:

Load	0	8	12	18	24	36	60
6280	33.39	15.86	10.33	6.48	4.66	3.00	1.65
8894	44.02	22.52	13.77	8.87	6.19	4.03	2.32
11952	57.46	31.52	18.79	11.94	8.54	5.45	3.06
6196	30.62	15.82	10.78	6.45	3.97	2.88	1.88
8902	40.93	22.26	15.16	8.99	5.64	4.02	2.50
11928	53.83	30.65	20.58	11.86	7.38	5.27	3.30
6384	27.93	14.20	10.94	6.87	5.06	3.41	1.57
8945	37.25	19.02	15.11	9.54	7.16	4.91	2.31
11931	48.89	24.84	19.87	12.75	9.62	6.59	3.16

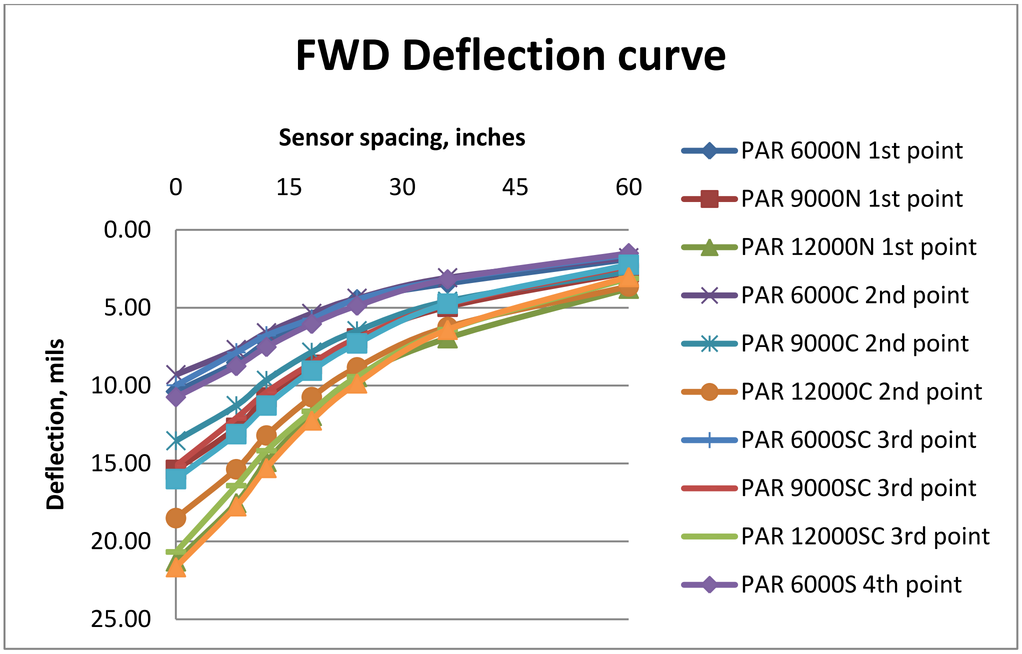


(i) Porous Asphalt Deflection Basins

For PABG



For PAR


FWD Deflection curve
Sensor spacing, inches
0
15
30
45
60
0.00
5.00
10.00
15.00
20.00
25.00
Deflection, mils
PAR 6000N 1st point
PAR 9000N 1st point
PAR 12000N 1st point
PAR 6000C 2nd point
PAR 9000C 2nd point
PAR 12000C 2nd point
PAR 6000SC 3rd point
PAR 9000SC 3rd point
PAR 12000SC 3rd point
PAR 6000S 4th point

APPENDIX B
FPS-19W FLEXIBLE PAVEMENT DESIGN

COMMENTS ABOUT THIS PROBLEM

Estimate Traffic at 0.4M
New Design Project

BASIC DESIGN CRITERIA

LENGTH OF THE ANALYSIS PERIOD (YEARS)	20.0
MINIMUM TIME TO FIRST OVERLAY (YEARS)	12.0
MINIMUM TIME BETWEEN OVERLAYS (YEARS)	8.0
MINIMUM SERVICEABILITY INDEX P2	3.0
DESIGN CONFIDENCE LEVEL (95.0%)	C
INTEREST RATE OR TIME VALUE OF MONEY (PERCENT)	7.0

PROGRAM CONTROLS AND CONSTRAINTS

NUMBER OF SUMMARY OUTPUT PAGES DESIRED (8 DESIGNS/PAGE)	3
MAX FUNDS AVAILABLE PER SQ.YD. FOR INITIAL DESIGN (DOLLARS)	99.00
MAXIMUM ALLOWED THICKNESS OF INITIAL CONSTRUCTION (INCHES)	69.0
ACCUMULATED MAX DEPTH OF ALL OVERLAYS (INCHES) (EXCLUDING LEVEL-UP)	6.0

TRAFFIC DATA

ADT AT BEGINNING OF ANALYSIS PERIOD (VEHICLES/DAY)	90.
ADT AT END OF TWENTY YEARS (VEHICLES/DAY)	120.
ONE-DIRECTION 20.-YEAR ACCUMULATED NO. OF EQUIVALENT 18-KSA	411000.
AVERAGE APPROACH SPEED TO THE OVERLAY ZONE(MPH)	30.0
AVERAGE SPEED THROUGH OVERLAY ZONE (OVERLAY DIRECTION) (MPH)	30.0
AVERAGE SPEED THROUGH OVERLAY ZONE (NON-OVERLAY DIRECTION) (MPH)	35.0
PROPORTION OF ADT ARRIVING EACH HOUR OF CONSTRUCTION (PERCENT)	6.0
PERCENT TRUCKS IN ADT	10.0

ENVIRONMENT AND SUBGRADE

DISTRICT TEMPERATURE CONSTANT	16.0
SWELLING PROBABILITY	0.00
POTENTIAL VERTICAL RISE (INCHES)	0.00
SWELLING RATE CONSTANT	0.00
SUBGRADE ELASTIC MODULUS	13000.00

CONSTRUCTION AND MAINTENANCE DATA

SERVICEABILITY INDEX OF THE INITIAL STRUCTURE	4.2
SERVICEABILITY INDEX P1 AFTER AN OVERLAY	4.0
MINIMUM OVERLAY THICKNESS (INCHES)	2.0
OVERLAY CONSTRUCTION TIME (HOURS/DAY)	12.0
ASPHALTIC CONCRETE COMPACTED DENSITY (TONS/C.Y.)	1.90
ASPHALTIC CONCRETE PRODUCTION RATE (TONS/HOUR)	200.0
WIDTH OF EACH LANE (FEET)	12.0
FIRST YEAR COST OF ROUTINE MAINTENANCE (DOLLARS/LANE-MILE)	50.00
ANNUAL INCREMENTAL INCREASE IN MAINTENANCE COST (DOLLARS/LANE-MILE)	100.00

DETOUR DESIGN FOR OVERLAYS

TRAFFIC MODEL USED DURING OVERLAYING	3
TOTAL NUMBER OF LANES OF THE FACILITY	4
NUMBER OF OPEN LANES IN RESTRICTED ZONE (OVERLAY DIRECTION)	1
NUMBER OF OPEN LANES IN RESTRICTED ZONE (NON-OVERLAY DIRECTION)	2
DISTANCE TRAFFIC IS SLOWED (OVERLAY DIRECTION) (MILES)	0.00
DISTANCE TRAFFIC IS SLOWED (NON-OVERLAY DIRECTION) (MILES)	0.00
DETOUR DISTANCE AROUND THE OVERLAY ZONE (MILES)	0.00

PAVING MATERIALS INFORMATION

LAYER	CODE	MATERIALS NAME	COST PER CY	E MODULUS	POISSON RATIO	MIN. DEPTH	MAX. DEPTH	SALVAGE PCT.
1	A	ASPH CONC PVMT	70.00	1027000.	0.35	1.50	6.00	30.00
2	B	FLEXIBLE BASE	28.00	35000.	0.35	4.00	12.00	75.00
3	C	STABILIZED SUBGR	8.00	49400.	0.35	8.00	8.00	90.00
4	D	SUBGRADE(200)	2.00	12800.	0.30	200.00	200.00	90.00

```
---------------------------------------------------------------------------------

     C. LEVEL C       SUMMARY OF THE BEST DESIGN STRATEGIES
                        IN ORDER OF INCREASING TOTAL COST
                         1      2      3      4
---------------------------------------------------------------------------------
MATERIAL ARRANGEMENT   ABC    ABC    ABC    ABC
INIT. CONST. COST     10.72  12.67  15.97  17.92
OVERLAY CONST. COST    1.89   0.00   2.16   0.00
USER COST              0.00   0.00   0.00   0.00
ROUTINE MAINT. COST    0.87   1.26   0.77   1.26
SALVAGE VALUE         -1.77  -1.62  -2.90  -2.75
---------------------------------------------------------------------------------
TOTAL COST            11.70  12.31  16.00  16.43
---------------------------------------------------------------------------------
NUMBER OF LAYERS        3      3      3      3
---------------------------------------------------------------------------------
LAYER DEPTH (INCHES)
     D(1)              3.00   4.00   2.50   3.50
     D(2)              4.00   4.00  12.00  12.00
     D(3)              8.00   8.00   8.00   8.00
---------------------------------------------------------------------------------
NO.OF PERF.PERIODS      2      1      2      1
---------------------------------------------------------------------------------
PERF. TIME (YEARS)
     T(1)              14.    23.    12.    20.
     T(2)              32.           29.
---------------------------------------------------------------------------------
OVERLAY POLICY(INCH)
(INCLUDING LEVEL-UP)
     O(1)              2.5           2.5
---------------------------------------------------------------------------------
SWELLING CLAY LOSS
 (SERVICEABILITY)
    SC(1)              0.00   0.00   0.00   0.00
    SC(2)              0.00          0.00
---------------------------------------------------------------------------------
```

APPENDIX C
AASHTO METHOD FOR FLEXIBLE PAVEMENT DESIGN

ASSUMPTIONS:

Existing pavement

Design period – 20 years

Light traffic roads such as Parking lots, residential driveways

45 vehicles per day

INPUTS:

Initial Serviceability, $P_I = 4.2$

Terminal Serviceability, $P_T = 2.5$

Reliability Level 90%

Overall Standard Deviation, $S_o = 0.45$

Calculation of Accumulated 18-kip Equivalent Single axle load

Assumptions: 3.3% trucks

4% Growth rate

$$AADT = (\frac{45 vehicles}{day})(2 \text{passes})(\frac{7 days}{week})(\frac{52 weeks}{year}) = 32{,}760 \frac{vehicles}{year}$$

Using equation (21),

$$\text{ESAL} = (32760)*(0.033)*(0.07)*(29.78)*(0.50)*365 = 411{,}285.5$$

Therefore take ESAL to be 411,286 (0.411million)

Estimation of Design Resilient Modulus (M_R):

Using FWD load application of 9000 lb

Pavement type = Porous Asphalt Fill (PAF)

From Forward calculation,

M_R = 13,427psi

Therefore take Roadbed Soil Resilient Modulus, M_R = 14,000psi

W_{18} =	411286	
M_R =	14000	psi
S_0 =	0.45	
Z_R =	-1.282	
ΔPSI =	2.2	

log W_{18} =	5.614		
SN =	2.319814	=	2.3
A =Z_R * S_0 =	-0.5769		
B = log(ΔPSI/2.7) =	-0.08894		
C = 2.32log(M_R) - 8.07 =	1.549017		
log W_{18} =	5.615		

Therefore the Required structural number is 2.3, take SN = 2.4

Computation of Minimum Thickness of layers

Assumption:

For Porous Asphalt, a_1 = 0.42;

For crushed stone base, a_2 = 0.14;

Granular subbase, a_3 = 0.10

$$SN_{calc} = (0.42*4) + (0.14*4) + (0.10*8) = 3.04$$

Recall that SN_{AASHTO} = 2.4

SN_{calc} > SN_{AASHTO}. Therefore, the pavement design is Okay.

AASHTO procedure to determine in-situ SN of a pavement structure by aid of data from FWD Deflection. The backcalculated moduli values are:

$E_{surface}$ = 721.4 ksi

E_{base} = 45.1 ksi

$E_{subbase}$ = 57.1 ksi

Average Pavement Modulus (above the subgrade), E_p = 274.5 ksi

D = 16 in.

$$SN_{eff} = (0.0045)(16)(\sqrt[3]{274500}) = 4.68$$

APPENDIX D
AASHTO METHOD FOR RIGID PAVEMENT DESIGN

Input:

Average Modulus of rupture of the previous concrete tested, S_c = 246.17 psi. From literature ((Rohne, et al., 2009), modulus of rupture of 28 days pervious concrete is assumed to be 540 psi. Therefore use S_c = 550 psi

Modulus of Elasticity of pervious concrete = 1,200 ksi

From Chart for estimating modulus of subgrade reaction (AASHTO, 1993),

k_∞ = 200 pci

W_{18} =	411286	
S_0 =	0.29	
Z_R =	-1.282	
ΔPSI =	1.7	
p_t=	2.8	
Sc=	550	
C_d=	1	
J=	3.2	
Ec=	1200000	psi
k=	200	pci

log W_{18} =	5.614144		
D =	4.397357	=	4.398
A =Z_R * S_0 =	-0.37178		
B = log(ΔPSI/3.0) =	-0.24667		
C = (4.22-(0.32*p_t)) =	3.324		
E=(Ec/k)	6000		
F = S_cC_d =	550		
log W_{18}=	5.61439		

Use Minimum thickness of Pervious Concrete of 6 inches

Minimum D = 6 inches

APPENDIX E

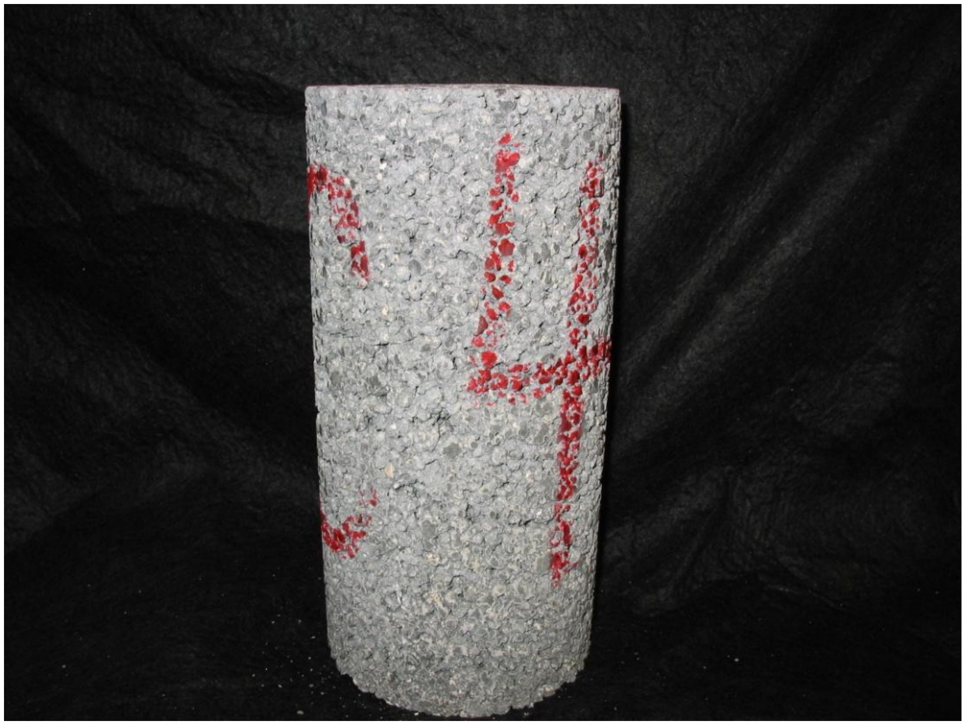
TEST CYLINDERS AND BEAMS AND FWD TEST PHOTOGRAPHS

Recycled glass cylindrical samples

Recycled rubber tire cylindrical sample

Porous Aggregate Cylindrical samples

Cored Pervious Concrete



Typical 28-day Pervious Concrete 12 X 6 Sample

Typical 28-day 8 X 4 PC Cylinder

FWD Test on Porous Aggregate

FWD Test on Recycled Glass Pavement

FWD Test on Porous Asphalt

FWD Test on Permeable Pavers

FWD Test on more permeable pavements

FWD Test on even more permeable pavers

FWD Test on Recycled Rubber tire pavement

FWD Test on Conventional Hot mix Asphalt road

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