Potential Use of Plastic Waste Materials in Pavement Structures Applications

Submitted by

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“To the rocks of my life; My beloved wife, Arij, beautiful sons Elseddig & Omar,
my parents, dearest brother and sisters, for their eternal love, support and encouragement.
I have proudly taken hardship into my stride, and here I am today”

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Abstract

There have been concerns worldwide on the scarcity of quality traditional construction materials that are associated with the evolution in roads construction industry and growing traffic demands. This has shifted the focus on investigating viable untraditional materials. Plastic waste has recently gained global popularity in the road building industry. The incorporation of different types of plastic waste have been investigated on the performance of the subgrade soil, on stabilizing embankments as well as on the influences of different grades of bitumen binders and hot asphalt mixes (HMA). Notwithstanding the aforementioned studies, consideration of plastic waste resins in pavement structure has not been neither reported practiced nor even allowed in the province of Ontario. That might be ascribed to the lack of compelling evidences on the beneficial impacts and feasible research studies urging the decision makers and the industry to permit and implement part of the national produced polymers in the road infrastructure. Hence, this research study aims to focus on investigating the influence of three different recycled polymers (plastics) and in two different shapes onto the performance of pavement structure components. The focus is given to the underneath granular and subgrade materials currently in practice in the province of Ontario in accordance with Ontario provincial standard specifications (OPSS).

Relatively light densities of plastic flakes and pellets resulted in lowering the maximum dry densities of the investigated soil. The types, shape and the adding rate of plastic were also reflected either by an increase or a decrease on the strength of the soil as measured by the CBR test and the resilient modulus. The impact of incorporation of plastic led to a raise in the permeability of the soil samples. Results of examining the subgrade imply that partial replacement of subgrade soil material with plastic waste may prove useful in road subgrade applications. The results revealed a decline in the optimum moisture content and its corresponding maximum dry density for all types of plastic. The results also showed a noticeable improvement in strength despite variation with the shape of utilized plastic. The permeability was reduced, which is favorable to protect pavement subject to moisture. All in all, the pellets brought higher improvements compared to the ground and flake forms. Properties of fresh as well as hardened concrete (i.e., density, workability, strength, permeability, etc.) were evaluated in the final stage of the study. Assorted types of recycled plastic utilized with a 30MPa concrete mixture and based on the findings; few conclusions were made. Results showed more workable concrete mixtures that contain flakes over that contain pellets. Mixtures with plastic waste have easy workability and are suitable for use in pavement or road applications. This plastic aggregates-induced increase in air content is beneficial for the freeze-thaw resistance of the concrete pavement materials. The addition of plastic waste aggregates decreased the compressive, splitting and flexural strength of the concrete pavement samples. Additionally, adding plastic leads to the coarsening of the pore structure of the pavement concrete material and deteriorates the durability properties of the concrete, such as water absorption, sorptivity, and permeability. Outcomes are important in assessing the feasibility of using recycled plastic in producing light weight concrete mixture for various pavement application. The outcomes of the study could possibly have a significant impact on the road industry by providing recyclable source to be utilized in the road structure together with maintaining the sustainability to the environment and the quality of life.

Keywords: Asphalt pavements – Granular base, Sub-base and subgrade – Recycling – Plastic Waste

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Chapter 1

HMA	Hot mix asphalt
ASTM	American standard of testing and materials
OPSS	Ontario provincial standard specifications
LDPE	Low-density polyethylene plastic
HDPE	High-density polyethylene plastic
PET	Polyethylene terephthalate plastic
PP	Polypropylene, also known as polypropene plastic
MC	Moisture content
SG	Specific gravity
MDD	Maximum dry density
OMC	Optimum moisture content
CBR	California bearing ratio
Mr	Resilient modulus
LS	Ministry of Transportation, Ontario Test Method LS
UCS	Unconfined compressive strength

Chapter 2

AASHTO	American Society of State Highway and Transport Officials
USCS	Unified soil classification system
JPCP	Jointed plain concrete pavement
JRCP	Jointed reinforced concrete pavement
CRCP	Continuously reinforced concrete pavement
PCP	Pre-stressed concrete pavement
CBP	Concrete block pavement
PVC	Polyvinyl chloride plastic
PS	Polystyrene plastic
EP	Epoxide plastic
PF	Phenol-formaldehyde plastic
PUR	Polyurethane plastic

PTFE	Polytetrafluoroethylene plastic
UP	Unsaturated polyester plastic
Chapter 3	
AASHTO	American Society of State Highway and Transport Officials
RAS	Recycled asphalt shingles
RCG	Recycled crushed glass
E*	The dynamic modulus of asphalt pavement
SMA	Stone mastic asphalt
OGFC	Open-graded frictional course
PG	Performance graded bitumen
ACWC	Asphalt concrete wearing course
ACBC	Asphalt concrete binder course
RTFOT	Rolling thin film oven tests
PAV	Pressurized aging vessel
RAP	Reclaimed asphalt pavement
VMA	Voids in mineral aggregates
VTM	Voids in total mixture
CR	Crumbed rubber
PEN	Penetration graded bitumen
δ	Phase angle of shear modulus
AC-10	Asphalt cement of 1000 poises viscosity at 60°C
AC-20	Asphalt cement of 2000 poises viscosity at 60°C
TSR	Tensile strength ratio
ITS	Indirect tensile strength
LA	Los Angeles abrasion value test
SHRP	Strategic Highway Research Program
FWD	Falling weight deflectometer
CIPR	Cold in place recycling
SPT	Simple performance tester
CCM	Recycled crushed concrete masonry
CWM	Ceramic waste material
MTO	Ministry of transportation Ontario
DCP	Ceramic waste material

CPIA	Canadian Plastics Industry Association
WRP	Waste recycled product
EN Standards	European Standards
RPP	Recycled plastic pins
Chapter 4	
RPP	Recycled plastic pins
OLRT	Ottawa Light-Rail transit project
CU	Consolidated undrained triaxial test
PI	Plasticity index
LL	Liquid limit
C	Cohesion of soil
Θ	Internal friction angle
CG	Crushed glass
FS	Foundry sand
MW	Mine waste
PW	Pumice waste
NCEAS	National Center for Ecological Analysis and Synthesis
W.C	Water content
N.P	None plastic
MEPDG	Mechanistic-Empirical Pavement Design Guide
GA	Granular materials type A
GB	Granular material type B
k	Permeability
AR	Aspect ratio
MOR	Modulus of rupture
S	Sorptivity
RILEM	International Union of Laboratories & Experts in Construction Materials, Systems & Structures

1 Chapter 1: General Introduction

1.1 Introduction

The rate of consumption of secondary materials in the construction industry in the province of Ontario increased from 4% in the 1990s to roughly 7% between 2000 and 2009, and this rate is projected to climb to 8% over the next 15 years. The overall figures for aggregate consumption represent about 81% of an average 186 million tonnes per year over the next 2 decades. The road industry takes nearly 50% of this total amount, and studies and national reports have indicated an expected surge of 13% by the end of 2020 in comparison to the average annual usage of 164 million tonnes over the past 20 years (Altus Group Economic Consulting, 2009). However, there are still constraints within the road industry that hinder the use of more recycled and alternatives materials in road works, as the preference in the province is still for traditional materials rather than unconventional materials (Altus Group Economic Consulting, 2009). Therefore, efforts still need to be made to circumvent that trend for primary materials from pits and quarries and accentuate the potential usage of secondary sources. Lack of practical experience, insufficient positive evidence, and insufficient performance data are all reasons for this tendency.

Plastic waste is a valuable recyclable material that requires further investigation for use in road pavement structures, and with a present consumption rate of 7% of secondary materials in construction projects, plastic waste has not yet been sufficiently considered (Altus Group Economic Consulting, 2009). Since its inception in 1994 under Ontario Regulation 101/94 (CM Consulting, 2016), the Blue Box program has not specifically provided guidance for sorting plastic waste based on type and composition, and polymer waste is collected and transferred to a recovery center where it is sorted, processed, baled, and distributed locally and to overseas retailers for uses other than in construction. Nevertheless, efforts in countries such as the USA (Loehr et al., 2000), Netherlands (The Mind Unleashed, 2016), India (Choudhary et al. 2010; Vasudevan et al. 2012; Jha et al. 2014; Sarang et al. 2014; Salem, 2014; Ur- Rahman & Abdulrahman, n.d.; Chavan, 2013; Arora, 2015), Nigeria (Sojobi et al., 2016), Malaysia (Maniruzzaman et al., 2016), and Argentina (Angelone et al. 2015) have gone further in this regard. Hence, experience in investigating the use of polymer waste in road construction is not new. However, studies on the potential use of plastic materials in the road construction have mainly focused on the use of plastic waste in the surfacing layer of flexible pavement. The outcomes of those studies have provided evidence that polymer waste can influence the physical properties of virgin binders in hot mix asphalt (HMA) applications, as well as the mechanistic performance of HMA.

However, very few studies have been performed on stabilizing subgrade soil with plastic waste. Furthermore, a lack of studies investigating the incorporation of plastic waste with granular base and Sub-base materials has been realized, and the focus of the majority of the literature has been on exploiting the polymers as bonding agents, either to modify the physical properties of asphalt binders or replacing the binders altogether (Karmakar & Roy, 2016; Maniruzzaman et al., 2016; Ball, 2016; Lastra-González et al., 2016). Incorporating polymers from plastic waste into pavement structure would be of great benefit from a sustainability point of view, and beneficial outcomes would underpin endeavours to minimize the gasses dispersing into the air due to the incineration of waste at landfills.

Exploiting plastic waste in pavement structures would assist in reducing the amount of plastic waste that is not being sufficiently used. The era of favoring conventional materials in road construction has already ceased to exist, and a revolution in consuming secondary materials in pavement has recently begun.

Minimal research has been conducted on the exploitation of plastic waste and shortages of virgin materials, and thus this provides the rationale underscoring this research study proposal. Moreover, the gaps concerning the potential of using plastic waste in pavement construction at the local level was another motive to undertake this research, and the scarcity of raw materials for road construction, availability of plastic waste from various industries, and cost of using virgin materials all form the basis for this study. Justification would then be reinforced for the potential use of such waste in pavement structures for different classes of roads in accordance with Ontario Ministry of Transportation (MTO) regulations.

1.2 Problem statement

In an attempt to use more unconventional materials in road pavement structures, several studies have been conducted (Ur- Rahman & Abdulrahman, n.d.; Loehr et al., 2000; Choudhary et al., 2010; Vasudevan et al., 2012; Chavan, 2013; Salem, 2014; Sarang et al. 2014; Musa & Haron, 2014; Angelone et al., 2015; Arora, 2015; Sojobi et al., 2016; Karmakar & Roy, 2016; Maniruzzaman et al., 2016; Lastra-González et al., 2016). However, most of these studies have given more attention to the enhancement of the properties of surface hot mix asphalt mixtures. The City of Vancouver has taken the lead in such initiatives in Canada by producing what is technically known as a plastic derived wax mixture (B. D. P. Ball, 2016). The impact of incorporating plastic waste into asphalt binders has also been examined, and the outcomes are promising. As well, soil stabilization with different plastic types and shapes in embankments structures were studied and proved to be effective in strengthening the overall shear resistance.

In view of the foregoing facts and taking into account the increasing annual amounts of plastic waste rise (Moore Recycling Associates, 2016), there is a significant need for an intensive investigation which explores the influence of utilizing plastic waste in miscellaneous ways to improve the performance of granular materials. Granular base, Sub-base , and subgrade materials, as per the Ontario Provincial Standard Specifications (OPSS), have been selected for this effort, and the 15% of annual plastic waste exported abroad (CM Consulting, 2016) could be exploited to improve these materials. Such exploitation would be of great benefit economically and would create a significant boost in the performance of pavement structures for different classes of roads. Furthermore, the adverse impact from such imperishable materials on the environment would be diminished. It is expected that this research will help expand the view on considering various types of plastic waste in pavement construction in Ontario.

1.3 Research Objectives

The main objective of this research is to explore the possibility of using plastic waste in road pavement structures. The investigation would aid in providing a stronger foundation for the road building industry in relation to utilizing more recyclable materials, i.e., plastic waste, with conventional materials in pavement structures. In addition, the research would help protect natural resources as well as decrease potential threats to the environment caused by the accumulation of plastic waste at curb sides and in landfills.

The specific objectives would be attained through conducting an investigation on the use of different plastic waste polymers in different layers of road structures. The American Standard for Testing and Materials (ASTM) and Ontario Provincial Standards Specifications (OPSS) would be the references for the investigation and to achieve the following sub-objectives:

- Assessing the influence of various types of recycled plastic types and shapes at different contents on the geotechnical properties of the natural subgrade material.
- Assessing the behaviour of the base-course and Sub-base properties regarding the incorporation of different plastic waste types, contents, and shapes into these materials.
- Assessing the impact of such utilization on the properties of fresh and hardened concrete materials used in surfacing layers in rigid pavement applications.

1.4 Research approach and methodology

Figure 1.1 below illustrates the approach and methodology adopted for the core objectives of this research, and the materials used are shown in Figure 1.2.

The experimental work comprised two phases. Phase One looked at the geotechnical and physical performance of the subgrade, sub-base, and base-course materials, whereas the second phase investigated the impact on the physical, hydraulic and mechanical properties of a concrete surfacing material (Figure 1.1).

Different materials for a conventional pavement structure (i.e., surfacing, base, Sub-base , and sub-grade layers) were selected to achieve the research objectives (Figure 1.2). They were selected in such a way that would reflect the influence of various plastic waste on different material gradation compositions. These gradations were a well-graded granular A base-course material, a dense-graded granular B Type II Sub-base material, and natural Ottawa subgrade soil (Figure 1.2).

As shown in Figure 1.2, the selected plastic waste included low-density polyethylene (LDPE), high-density polyethylene (HDPE), polyethylene terephthalate (PETE), and polypropylene, also known as polypropene or (PP). They were supplied by Canada Fibers Ltd. and Enviroplast Canada. They were all in shredded-flake and granulated-pellet shapes except for the PETE, which was supplied in a ground form. This would presumably facilitate a better understanding of the interaction between these various shapes with the soil granules and aggregate particles. The flaked, pelleted, and ground plastic were added at various rates according to the material, and the results were then analyzed and discussed in terms the impacts on pavement designs. Conclusions are then drawn, and recommendations are provided.

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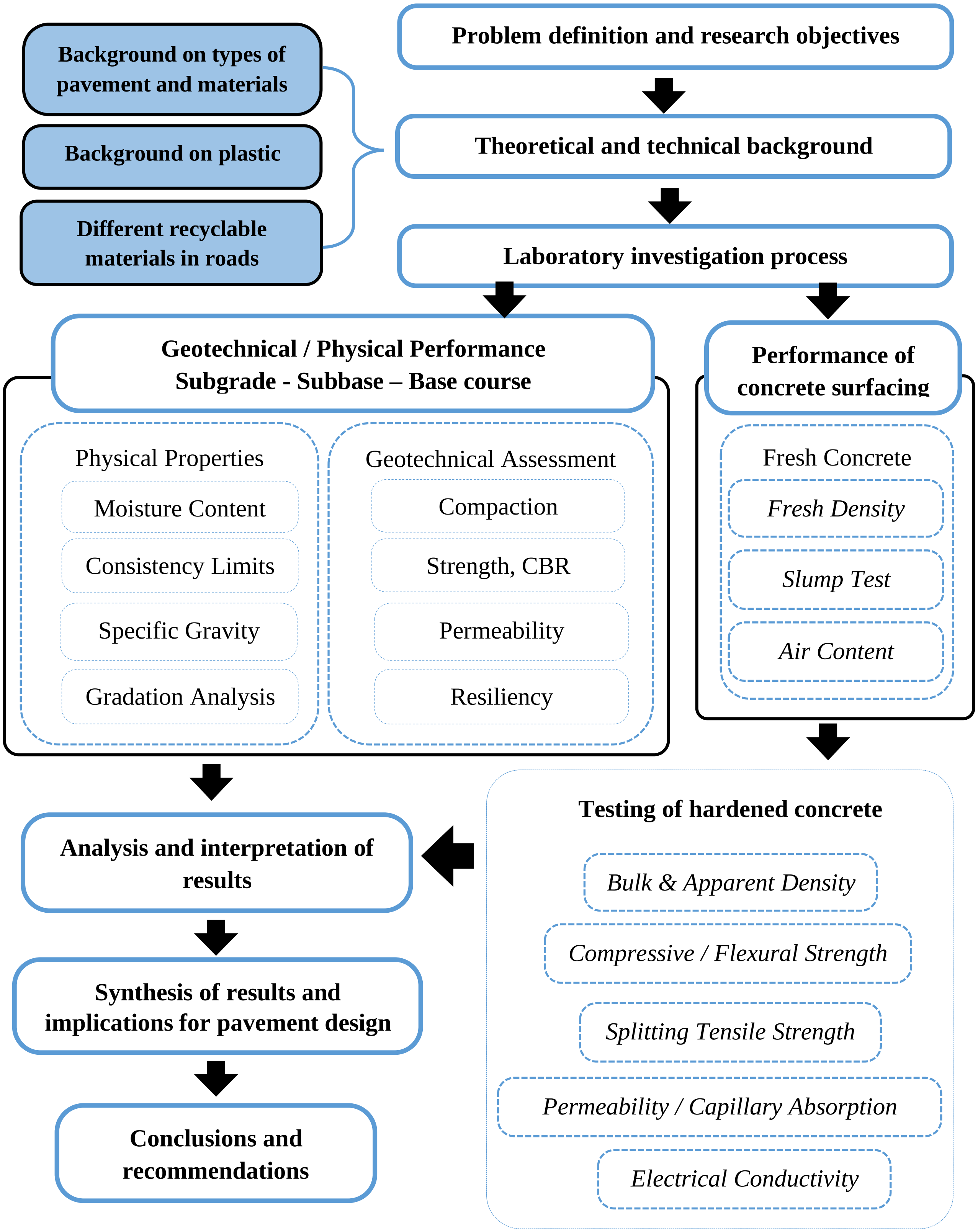

Figure 1.1 Research approach and methodology

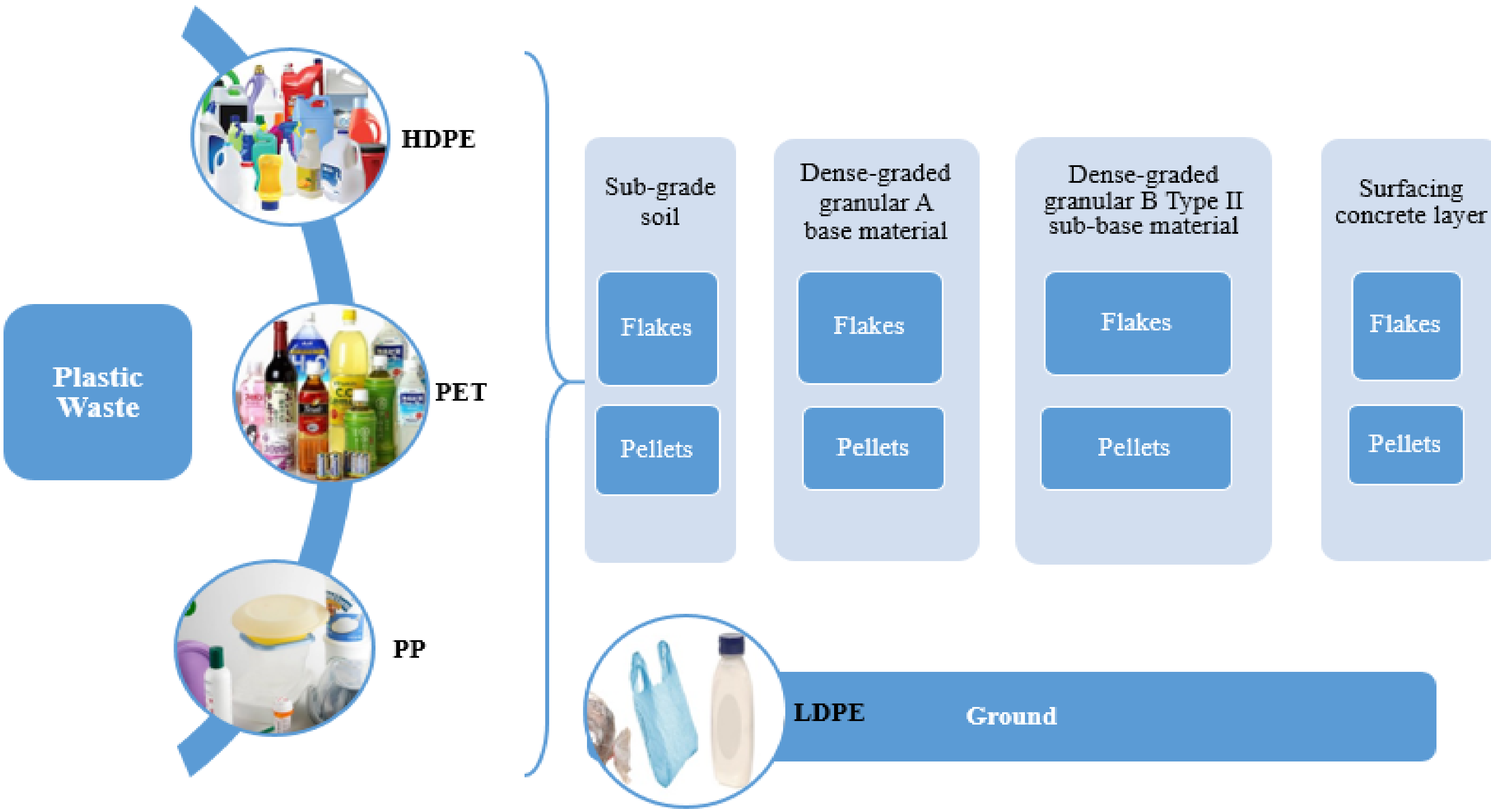

Figure 1.2. Breakdown of the selected plastic waste materials used in this research

Extensive laboratory testing was conducted in order to analyze the interactions of the utilized plastic waste with the pavement materials and to assess their performance. The main laboratory experiments for the purpose of attaining this goal are listed in Table 1-1.

Table 1-1. List of experiments and their references to standards

Experiment	Purpose	Standard Procedure
Sieve or gradation analysis of soil material	Determining the gradation of the subgrade soil	LS-702 ASTM D-5444
Sieve or gradation analysis of granular material	Determining the gradation of granular materials	LS-602 ASTM D-5444
Determination of moisture content of soil and rocks	Computing the amount of moisture in the soil or granular material	ASTM D-2216
Liquid limit, plastic limit, and plasticity of soils	Assessing the soil consistency	ASTM D-4318
Optimum moisture content (OMC) and maximum dry density (MDD)	Determining the densities of the compacted soils at the required optimum water content	ASTM D-698 ASTM D-1557
California bearing ratio (CBR)	Evaluating the strength of the subgrade soil and granular materials	AASHTO T-193
Unconfined compressive strength (UCS)	Evaluating the bonding strength of the mixture by measuring the compressive strength of cohesive soil and admixtures	ASTM D-2166
Determining the resilient modulus of soil and aggregates	Evaluating the strength of materials by their resistance to deformation	Analytical models

Slump test of concrete	Determining the workability of concrete	ASTM C-143
Bulk density of fresh concrete	Assessing the density of freshly mixed concrete	ASTM C567 standard
Bulk density of hardened concrete	Assessing the density of pre-mixed concrete	ASTM C642 standard
Compressive strength of concrete	Determining the strength of the mixture over time to find the optimum compressive strength	ASTM C-39
Splitting tensile strength test	To measure the tensile strength of concrete mixtures	ASTM 496
Fracture or bending strength	Determining flexural strength using simple beam with third-point loading	ASTM C-78
Permeability	Determining rate at which water is transported through the concrete matrix under pressure	ASTM D5084
Sorptivity test	Determining the porosity of the concrete	No standard procedure
Electrical conductivity testing	Determining the material's ability to conduct electricity	No standard procedure

The obtained results will be analyzed, discussed, and synthesized before evaluating the performance against plastic-free mixtures, and subsequently, conclusions will be drawn from the observations regarding the comparison with the performance of traditional granular and subgrade materials in pavement structures in accordance with the referenced standards.

1.5 Tasks and organization of the research proposal

This research is organized in seven chapters, and Figure 1.3 shows the structure of the fundamental tasks to be executed. It is also worth mentioning that Chapter Four is structured in a technical article format and contains three technical articles. Each article is composed of an abstract, an introduction, the materials used, and approach employed, results and discussion, conclusions, and lastly, a list of references. Therefore, redundant information may be present in the text.

Chapter 1 is a general introduction to this proposed research. It entails the identified problem statement, research objectives, approach deployed, and a thorough explanation of the methodology used to execute the proposed work.

Chapter 2 gives background information on pavement types and the materials compositions of pavement structures. This chapter also provides a brief description of the different types and forms of the plastics investigated.

Chapter 3 presents a literature review of various recyclable materials used in the road industry and concludes with a background on the practice of using plastic waste materials in pavement structures.

Chapter 4 presents three referred-to technical papers and contains those papers. First, it focuses on the geotechnical characteristics of plastic waste materials in pavement subgrade applications, and then on the geotechnical characteristics of plastic waste materials in pavement base and sub-base applications. Lastly, it sheds light on the impact of adding plastic waste materials in concrete pavement-surfacing applications.

Chapter 5 contains the synthesis of the results and their implications for pavement design applications.

Chapter 6 is dedicated to summarizing all the essential accomplishments and drawing the conclusions out from the conducted research. The recommendations for future work are also compiled towards the end of this chapter.

Chapter 7 is the appendix and it contains some pictures taken during carrying out the study and raw data from multiple tests conducted in the research.

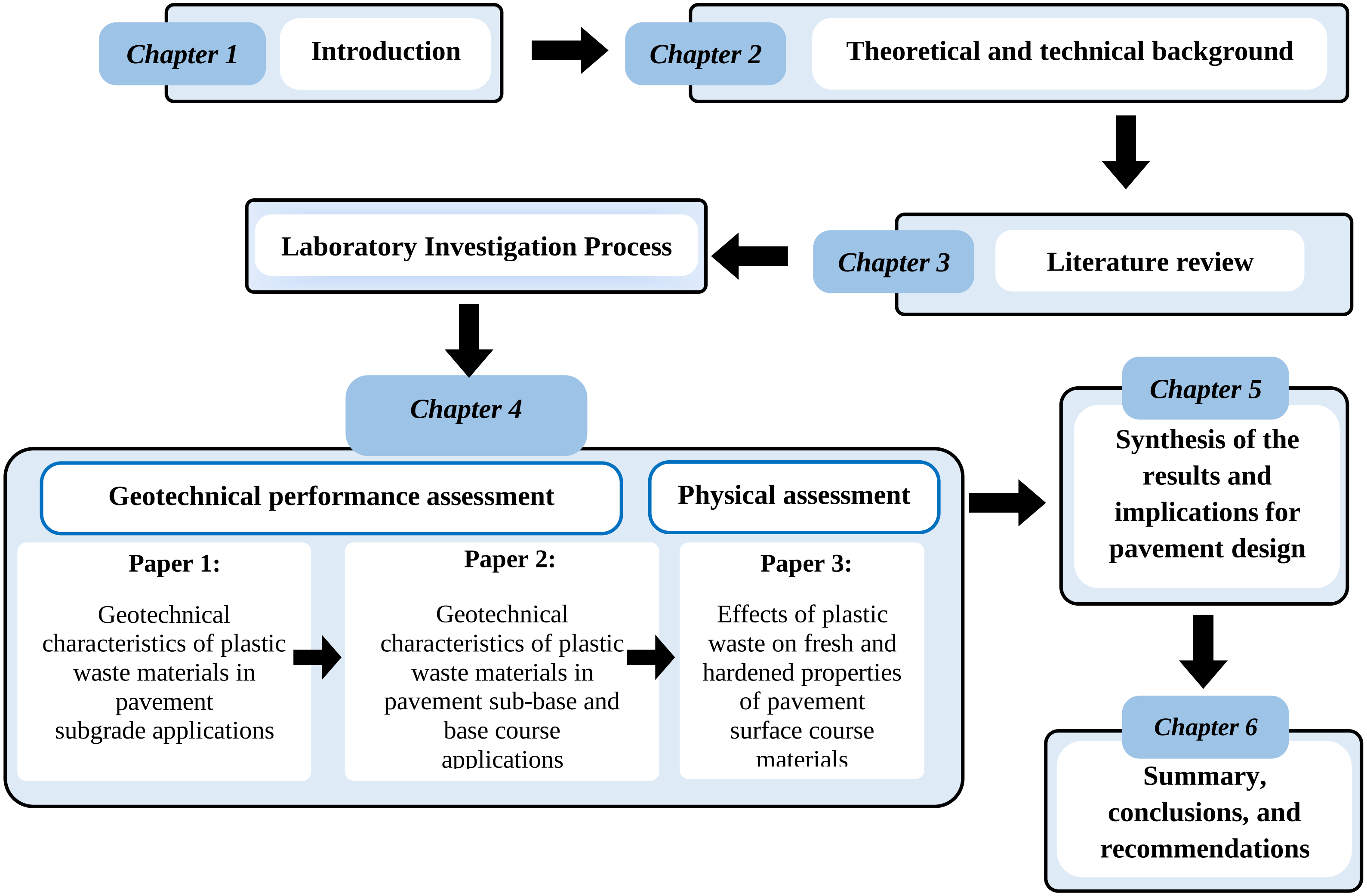

Figure 1.3. Thesis tasks and organization

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2 Chapter 2: Theoritical and Technical Background

2.1 Introduction

This chapter is dedicated to highlighting the most prevalent types of pavement structures and materials used in road construction. Also included is a description of the mechanisms that transfer loads to underlying layers in most paving. This is then followed by definitions of the plastic materials as well as their compositions and types.

2.2 Background on Pavement Structures

Roads are constructed for the mobility, safety, and comfort of drivers. They are built to sustain an anticipated level of traffic under various weather conditions while providing a smooth surface and efficient drainage (Christopher et al., 2006). This is attainable through comprehensive design approaches. Historically, road pavements are structurally either flexible or rigid, and are distinguished by the composition of the pavement body and the mechanism for load distribution over the base layer. These structures have evolved over time, and the materials have changed with changes in traffic demand and prevailing conditions. The most common types of road pavement structures, and their functions and compositions are discussed in the following.

2.2.1 Basic Composition of pavement structure

Regardless of the pavement structure type, the typical layers of the structure are generally similar, and entail natural or modified subgrade soil, sub-base and base-course layers, and final surfacing layer (Christopher et al., 2006). The top surfacing layer is what generally distinguishes the type of pavement. The basic structure is shown in Figure 2.1 below.

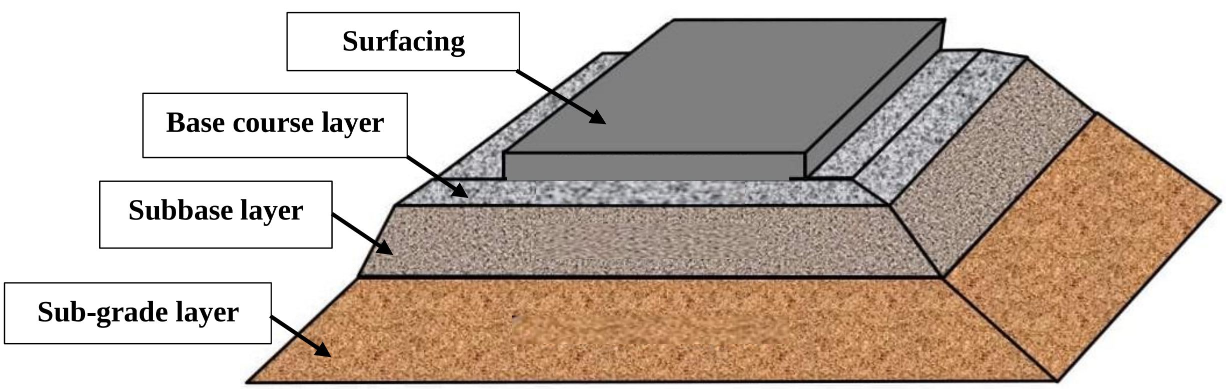

Figure 2.1. Pavement components

2.2.1.1 Surface course

The topmost layer is subjected directly to the traffic loads, provides protection for the layers beneath, and provides a comfortable riding surface. It is a relatively thin component of one or two layers contingent upon the type of pavement (Christopher et al., 2006). In flexible pavement, for instance, it is a combination of a wearing and binder course called the asphalt concrete course. The wearing course is thinner and of higher quality materials in order to sustain the wearing and compression imposed from traffic (Mathew, 2006)**,** and is made of an asphalt mixture that is richer in bitumen than the binder course. It also functions as a sealant against water on the surface, preventing seepage downwards, and provides resistance to skidding (Mathew, 2006).

2.2.1.2 Base-course

The cornerstones of the overall performance of road pavement is the quality of the layers beneath the surfacing layer. The layers beneath the surface play a significant role in prolonging the service life of any pavement. The base-course layer is made of a high quality material that helps distribute an applied load from the surface to the underneath layers safely (Mathew, 2006). The rationale behind using high quality material in top layers emanates from the fact that more sever stresses are subjected on pavement structures from the top (Delatte, 2008), and results in a substantial backbone for the structural capacity of the pavement structure (Christopher et al., 2006). This layer is usually made of a granular aggregate material; however, it can also be a mixture of granular aggregate treated with asphalt cement of a lesser quality than that of the surface mixture. High quality material is made up of unbound crushed aggregates, a mixture of unbound gravel and sand (Christopher et al., 2006), or any other approved secondary material such as recycled concrete aggregates or granular aggregates with reclaimed asphalt concrete or crushed glass. In addition, properties of materials of impaired quality can still be altered to suit a particular base-course application. Modifications may be made by using lime, steel slag, fly ash, etc. (Christopher et al., 2006).

2.2.1.3 Sub-base-course

This layer supplements the base-course layer in carrying the load and distributing it to the subgrade. The main intent of using a sub-base layer is to reduce the cost over using a base-course all the way between the top surfacing layer and the subgrade layer. It is a mixture of granular material of different sizes or a granular material treated with lime, cement, or another stabilizer. In concrete pavement, the base and sub-base layers are most often stabilized with bitumen or Portland cement and could be of different grading forms (i.e., dense, gap, or open-graded) (Delatte, 2008). This layer aims also at counteracting the detrimental effect of thawing and deterring the popping up of fine particles through joints to the surface in concrete pavements (Delatte, 2008).

2.2.1.4 Subgrade layer

The underlying layer of pavement structures is the formation layer on which the pavement structure rests, and it is technically called the subgrade layer. It supports the road structure and helps resist the stresses imposed by the traffic loads on the surface. It can be natural in-situ soil, mechanically modified or stabilized soil, or "borrowed" soil from pits nearby. The geotechnical properties of these materials are crucial in determining the overall depth of the pavement structure. A weak subgrade will require a thicker structure than strong soil with a high strength resistance. It is of great importance to understand the geotechnical engineering properties of these materials and the structure on top when aiming for a long-lasting pavement structure.

In order to assess the quality of the subgrade soil to be used, the soil has to be initially classified in order to rate its potential use. There are two main procedures used worldwide, which are the Unified Classification Approach, invented by the American Standards for Testing and Materials (ASTM), and the American Society of State Highway and Transport Officials (AASHTO) approach. The AASHTO approach is commonly used for highways, and the ASTM approach is used for other classes of pavement (Delatte, 2008).

In the Unified system, soil is divided into four groups: gravel, sand, silt, and clay, as described in Table 2-1 and Table 2.2 below. In the AASHTO procedure, soil is classified into seven major groups, A1 to A7, where A1 is the most favorable and A7 the least favorable. This classification is associated with certain physical and geotechnical properties such as the grain size distribution and consistency limits (Schaefer et al., 2008).

Table 2-1 Classification chart of soil according to AASHTO

General classification	General materials (≤ 35% passing 0.075mm)							Silt-clay materials (>35% passing 0.075mm)			
Group classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve analysis, % passing											
2.00mm (No.10)	50 max.										
0.425mm (No.40)	30 max.	50 max.	51 min.								
0.075mm (No.200)	15 max.	25 max.	10 max.	35 max.	35 max.	35 max.	35 max.	36 min.	36 min.	36 min.	36 min.
Liquid limit	6 max.		N.P	40 max.	40 min.	40 max.	41 min.	40 max.	41 min.	40 max.	40 min
Plastic limit				10 max.	10 max.	11 min.	11 min.	10 max.	10 max.	11 min.	11 min.
General Rating Of Subgrade	Excellent to good							Fair to poor			

Table 2-2. A comparison between AASHTO and Unified classification systems of soils

Soil group in AASHTO system	Comparable soil groups in USCS		
	Most probable	Possible	possible but improbably
A-1-a	GW, GP	SW, SP	GM, SM
A-1-b	SW, SP, GM, SM	GP	-
A-3	SP	-	SW, GP
A-2-4	GM, SM	GC, SC	GW, GP, SW, SP
A-2-5	GM, SM	-	GW, GP, SW, SP
A-2-6	GC, SM	GM, SM	GW, GP, SW, SP
A-2-7	GM, GC, SM, SC	-	GW, GP, SW, SP
A-4	ML. OL	CL, SM, SC	GM, GC
A-5	OH, MH, ML. OL	-	SM, GM
A-6	CL	ML, OL, SC	GC, CM
A-7-5	OH, MH	ML, OL, CH	GM, CM, GC, SC
A-7-6	CH, CL	ML, OL, SC	OH, MH, GC, SM

One crucial aspect of the soil for use in the subgrade is its frost-heave resistance. The penetration of water from the bottom and accumulation of snow on the surface cause water to seep into the soil during thaw periods, and once freezing occurs, ice crystals form in the voids between the soil particles. If freezing temperatures persist, the crystals cause stresses within these voids and lead to what is known as frost-heave action. The characteristics of frost-susceptible soils include but are not limited to having a fine portion exceeding 3% of particles passing a #10 sieve (Delatte, 2008).

2.2.2 Types of pavement

Highway pavement is typically composed of a multilayered structure that sustains the traffic load (Mathew, 2006). Based on numerous factors, it can be a double- or single-layer system that is laid over the subgrade layer. Depending on the available construction materials, design strategy, and anticipated weather conditions, pavement structures may vary significantly.

According to the structural performance desired, pavement structures are traditionally classified into two types: flexible and rigid (Mathew, 2006); However, in recent years, there has been a need to develop new structures to accommodate the tremendous increase in the numbers of vehicles and to account for changing weather conditions. For instance, composite pavement and concrete block pavement have been widely used in certain circumstances. The fundamental differences between these types are the manner of distribution of traffic load and the composition of the structure (Mathew, 2006). The most common three types are elaborated more in the following paragraphs.

2.2.2.1 Flexible asphalt pavement

Flexible pavement is typically a multi-layered structure laid on top of the formation subgrade layer. The topmost layer is made of a hot mix asphalt (HMA), which is a mixture of aggregates of different graded sizes heated and coated by a bonding agent. Bitumen is the most common type of binding agent used in flexible pavement. Asphalt cement can be liquefied through heating to cover the aggregates in the HMA, and compaction takes place on-site. The road is then opened to traffic once the mat cools down (Y. H. Huang, 2004)**.**

Beneath the surfacing layer are the base and sub-base layers. These two layers are usually made of granular materials of acceptable quality ("Types of Pavements - Flexible Pavements and Rigid Pavements," n.d.), and function primarily as load-carrying and -distributing layers (Kang et al., 2011). The layer above the subgrade is the sub-base-course, which is very often a mixture of unbound aggregate. The base layer is most frequently made of an unbound aggregate mixture with a quality superior to that of the sub-base material. The nature of the load's intensity, which is applied by the traffic on the surface layer of the pavement structure, inevitably requires the top layer to be of the appropriate quality. Thus, the lower the layer, the less the required properties for sustaining the imposed traffic load and resulting stresses (Huang, 2004).

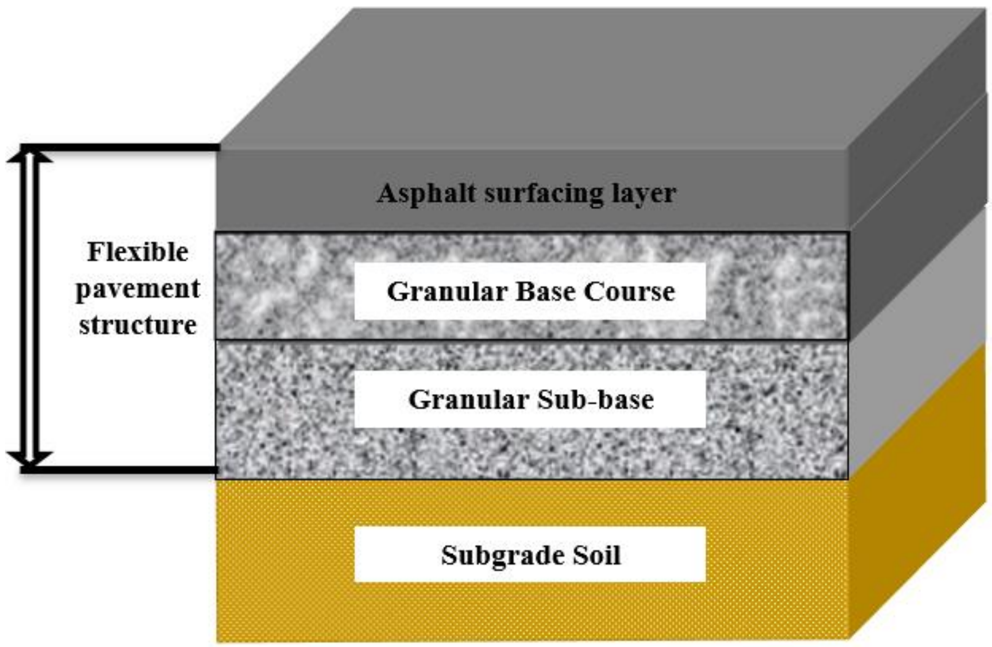

Figure 2.2. Structure of flexible pavements

In the construction field, the subgrade layer begins where the excavation stops, or at around one meter from the top level (Kang et al., 2011). It is a fact that the strength and stiffness properties of the layers beneath the flexible pavement structure have a very crucial impact on the deterioration of the structure and the performance degradation. Stagnant water within these layers adversely affects the structure as a whole and leads to detrimental effects that shorten the service life of the pavement (Kang et al., 2011).

2.2.2.2 Rigid pavement

Rigid pavement is essentially a Portland cement concrete slab placed on top of a single- or multi-layered structure. The single-layer structures are natural or stabilized subgrades, whereas a multi-layered structure may involve a base-course that is sometimes placed before laying the surface concrete slab. The presence of the base layer in concrete pavement is an optional practice to reduce the impact of moisture in areas with high water tables, and also helps prevent the pumping of fine particles through joints and cracks (Christopher et al., 2006). Portland cement is traditionally used as a binder in rigid pavement (Maniruzzaman et al., 2016), and is renowned for the high elastic modulus of concrete slabs. Such pavements have a higher stiffness than flexible pavements due to the high modulus of elasticity of the concrete slab. One distinguishing feature of rigid pavements over flexible pavements its ability to transfer traffic load on the surface to the bottom layer over a larger area. Any little deformation underneath a subgrade layer can be overcome by a highly rigid slab, unlike in flexible pavement ("Types of Pavements - Flexible Pavements and Rigid Pavements," n.d.)

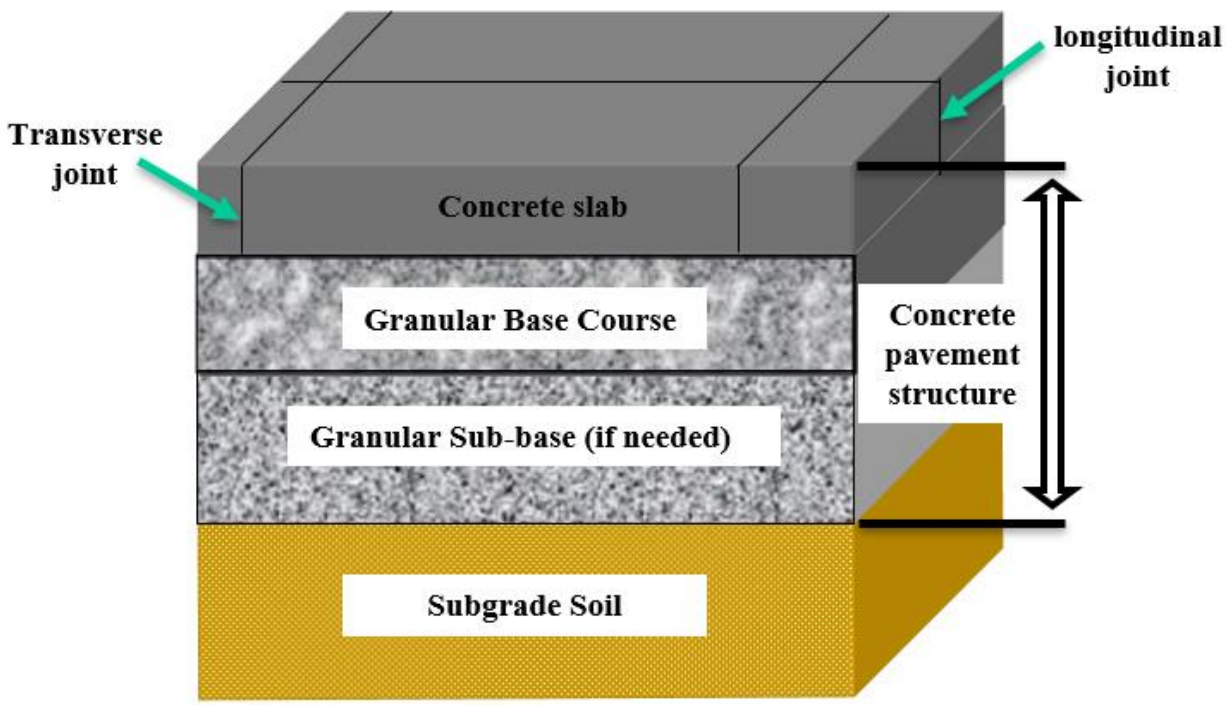

Figure 2.3. Structure of rigid pavements

Rigid pavement is constructed in several ways. It can either be jointed plain (JPCP), jointed reinforced (JRCP), continuous reinforced (CRCP), or pre-stressed concrete (PCP). JPCP is a non-reinforced concrete slab five to ten meters wide and relies on interlocking dowel bars at the joints or the aggregates component in order to transfer the load between adjacent slabs. JRCP is a jointed reinforced concrete pavement which is constructed in longer spans that range from ten to thirty meters. The reinforcement aims to hold the slab together in the case of cracking. It also uses dowel bars to help transfer the applied loads. In contrast, CRCP contains fully reinforced slabs with no joints that acts as complete entities in order to counteract the traffic stress imposed on the surface of the slab (Mathew, 2006). Reinforcement function is only to extend the spacing between joints (Christopher et al., 2006).

As with flexible pavement, rigid pavement is also used in streets, parking lots, highways, airport runways etc. However, it is preferred over flexible pavement when considering lower maintenance costs and longer life.

Table 2-3. The basic differences between flexible and rigid pavement structures

	Flexible	Rigid
1.	Multi-layered structure with a high-quality material on top.	- Usually a concrete slab laying on a natural subgrade or an optional base-course layer.
2.	Asphalt cement is the bonding agent that binds the aggregate particles together.	- Portland cement is the additive that holds the particles together in the concrete mortar and aggregates.
3.	Deformation in the underneath layers will be reflected on the surface.	- A highly rigid slab will absorb deformations in the subgrade without any repercussions.
4.	Its stability depends upon the aggregate interlock, particle friction, and cohesion.	- Its structural strength is provided by the slab itself through its beam action.
5.	Subgrade strength and material properties have a significant impact on the design.	- The most significant feature that affects the pavement design is the flexural strength of the concrete slab.

6.	Traffic load is very intense at the surface and is transmitted through different layers to the subgrade layer.	- It acts like a beam in transmitting the load over a large area to the subgrade layer.
7.	Deformation due to stresses can be recovered or cause a permanent deformation and corrugation on the surface.	- Excessive deformations may result in cracking under contraction and expansion.
8.	Can be constructed on stages contingent upon the available budget and projected traffic.	- Stage construction is not always viable with rigid pavement.

2.2.2.3 Composite pavement structure

Composite pavement structures are a combination of the two fundamental types of structures; flexible and rigid (Christopher et al., 2006). Composite pavement is a flexible pavement structure covered with a rigid concrete slab rather than an asphalt layer. It is efficient in sustaining the adverse effects of traffic loads and weather conditions; nonetheless, it is very costly and therefore is rarely implemented (Mathew, 2006). Figure 2.4 presents the cross-sections of two different composite pavements (Christopher et al., 2006).

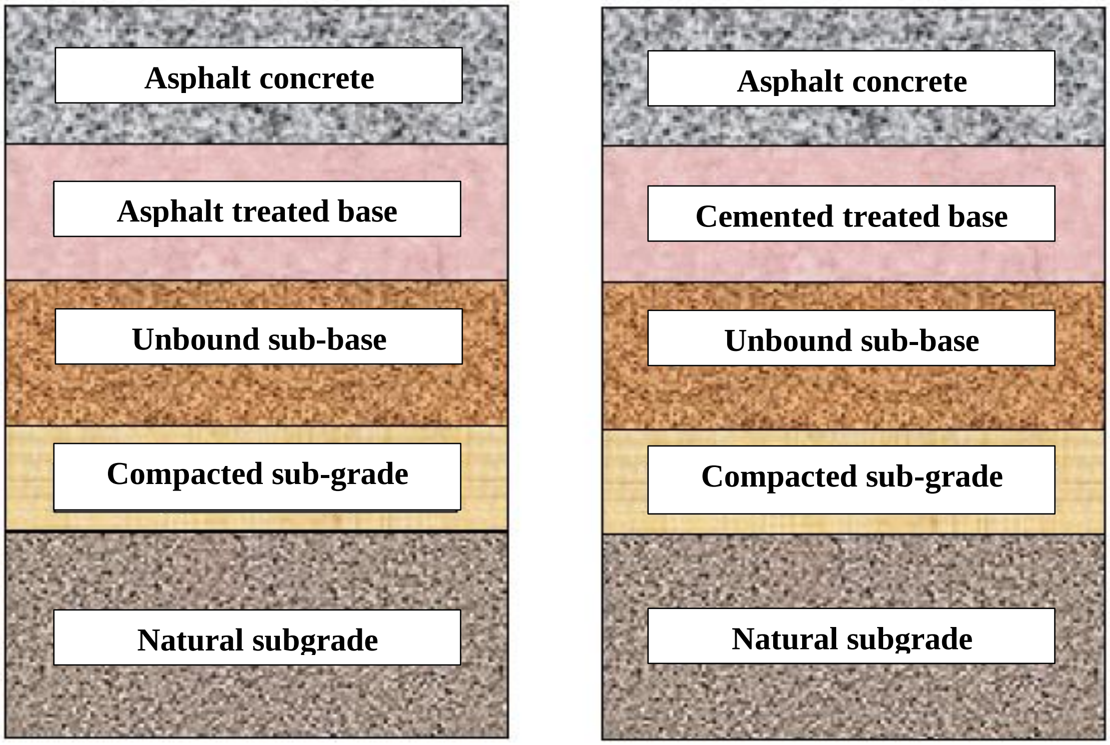

Figure 2.4. Types of composite pavement structure

2.2.2.4 Concrete block pavement (CBP)

Concrete block pavement (CBP) is another pavement type that has gained popularity because of its attractive appearance and satisfactory performance in different applications. It is a segmented pavement structures formed with different shapes of concrete blocks. The composition of this pavement structure is illustrated in Figure 2.5 (Concrete Manufacturers Association, 2009). The flexibility in using these shaped blocks creates more options for implementing this type of pavement in many applications. For instances, residential streets, intersections, toll plazas, pedestrian walkways, parking lots, patios, and alleyways are some applications

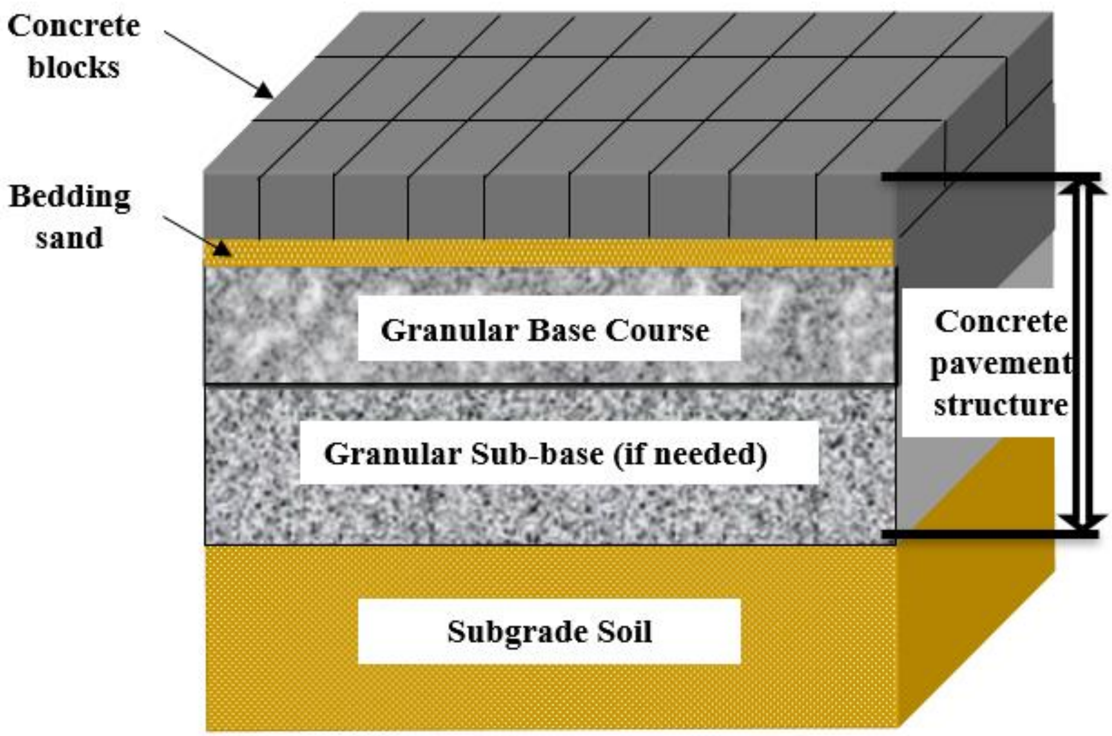

Figure 2.5. Concrete block pavement composition

Figure 2.6 next shows the usage of CBP in various applications, including: (a) a shopping mall parking lot, (b) swimming pool decking, and (c) a driveway application (Concrete Manufacturers Association, 2009)

(a)

(b)

(c)

Figure 2.6. Different applications of the CBP; (a) Shopping Centre Parking lots (b) Swimming pool surrounding (c) Alleys in residential areas (Concrete Manufacturers Association, 2009)

There are three principal aspects that need to be taken into account when selecting the appearance of CBP. These include the shape of the block, the colour, and the pattern. It is worth mentioning that the shape governs the pattern selected for the CBP. Figure 2.9 shows various CBP patterns and shapes (Concrete Manufacturers Association, 2009).

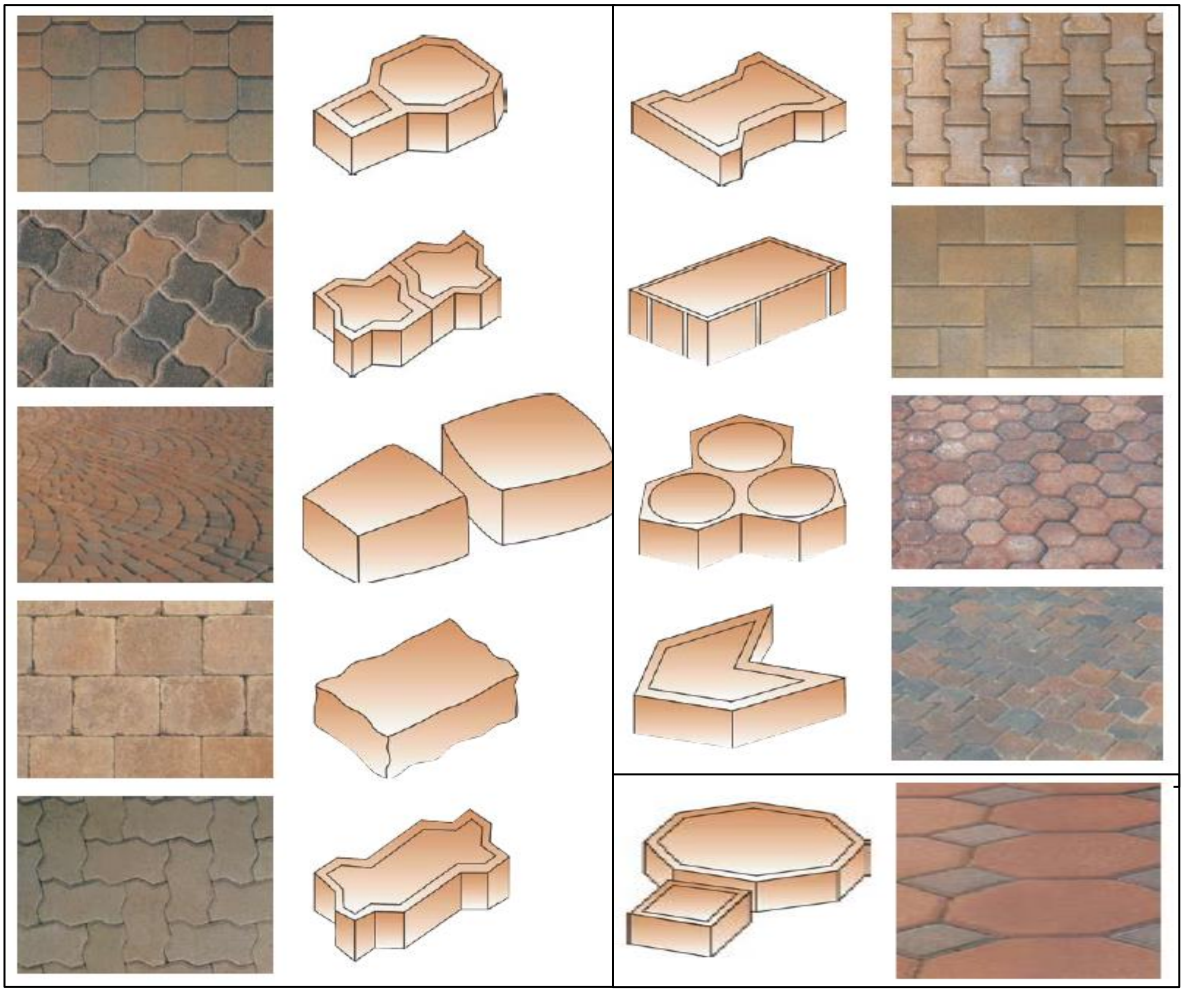

Figure 2.7. Different patterns and block shapes of CBP (Concrete Manufacturers Association, 2009)

Laying a course material or a bedding sand layer in CBP is done to free the area of any materials that may harm the blocks. In general, materials passing the sieve sizes tabulated below are used. However, when there is enough engineering evidences exist for any other gradation envelop, other sand still can be used (Concrete Manufacturers Association, 2009).

Table 2-4. Grading requirement for sand bedding in CBP pavements

Nominal maximum aggregate size, (mm)	Amount passing through, (%)
9.52	100
7.5	95 - 100
2.36	80 - 100
1.18	50 - 85
0.60	25 - 60
0.30	10 - 30
0.15	5 -15
0.075	0 - 10

2.2.2.5 Unpaved Road Structure

This also is known as a naturally surfaced road. They are subjected to a very light traffic but require special attention due to dust and environmental factors. The structural capacity of unpaved roads comes from either the granular materials laid or the natural subgrade material (Christopher et al., 2006). One issue that drivers on this type of roads encounter is when rain falls and traffic causes mud and poor driving conditions.

2.2.3 Load transfer mechanism

Pavement structures are constructed to alleviate and transfer the loads imposed on the surface by traffic to the bottom layer of the structure. Each type of pavement has its own characteristics in distributing and reducing the traffic stress. Traffic loads are intense at the surface and decrease through the layers to a level that the subgrade formation layer can withstand. The mechanisms of load transmission of two common pavement structures are described in the following.

Flexible pavement uses the contact points between aggregate particles to transmit applied loads to the underneath subgrade soil. The load is transferred to the bottom over a wide area of the structure in a reverse proportion, i.e., intensity decreases with depth (Mathew, 2006).

Rigid pavement is more capable of distributing the load over a larger area than flexible pavement. Most of the applied load is carried by the concrete slab in rigid pavement, whereas all layers participate in load-carrying in flexible pavements. Due to the fact that most of the load is carried by the concrete slab in rigid pavement, there is less concern over using stiff materials beneath the slab (Delatte, 2008). The reduced load reaches the subgrade at its lowest level in both pavement types; however, the magnitude of the load is higher in flexible pavement due to concentration of the load over a smaller as opposed to rigid pavements.

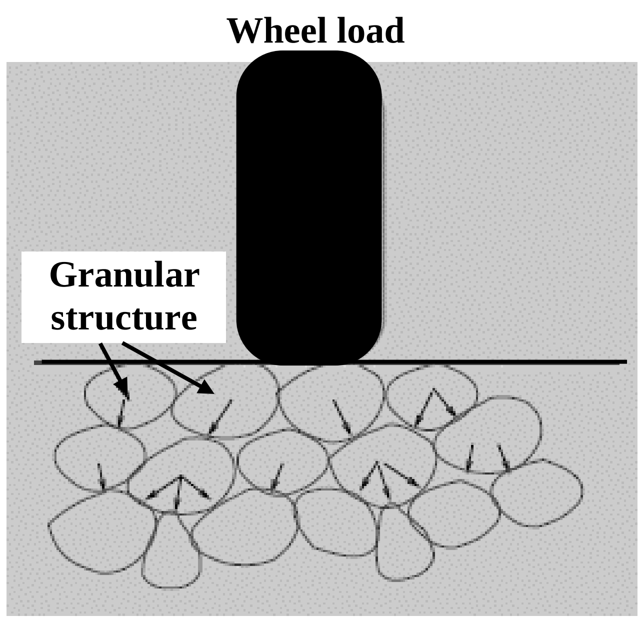

Figure 2.8. Transferring the applied load through contact points between particles in flexible pavement

Figure 2.9 shows the intensity of traffic load over both types of pavement as well as how it is transferred to the bottom subgrade layer.

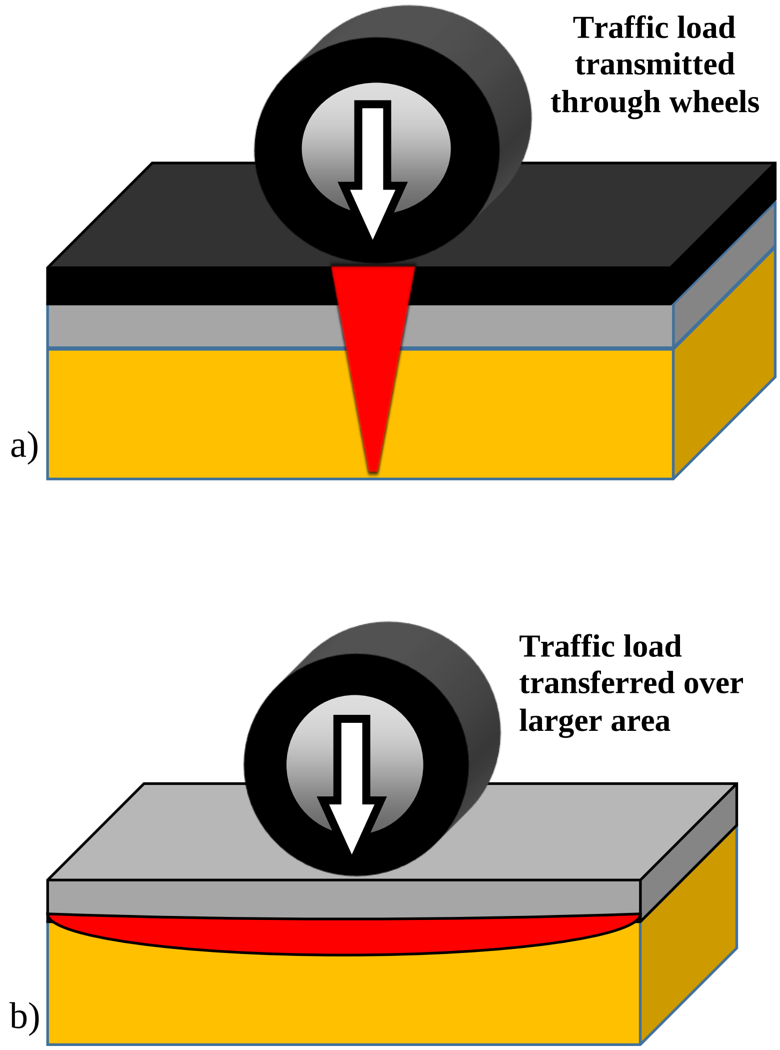

Figure 2.9. Intensity of traffic load for a) flexible and b) rigid pavement structures

2.3 Background on Plastic Materials

Plastic is defined as an organic lightweight material (Punthutaecha et al., 2007) with recognized thermal and electrical insulation characteristic (Thompson et al., 2009). It is regularly encountered in various aspects of our daily lives in a wide spectrum of applications (Punthutaecha et al., 2007). Plastic is deemed a strong and durable material that does not decompose rapidly. Organic compounds that plastic is made from include coal, cellulose from plants and trees, and crude oil (PlasticsEurope, n.d.). It is used in miscellaneous synthetic or semi-synthetic applications that range from clothing to items such as soft drink bottles, garbage bags, shopping bags, milk pouches, soap dispensers, plastic wraps, straws, plastic utensils, and food containers (PlasticsEurope, n.d.).

The term "plastic" refers to the plasticity of the composition of the substance itself which can be cast, compressed, or extruded into different shapes depending the desired end-use (PlasticsEurope, n.d.). Plastic comes in two types, thermoplastic or thermosetting (Punthutaecha et al., 2007). The difference between the types is their ability to reform under heat. While thermoplastics can be repeatedly reformed by heating, the thermosetting types cannot be reformed after solidification (PlasticsEurope, n.d.). The solidification characteristics of thermosetting plastics is what makes them useful for more durable products (American Chemistry Council, 2005). The American Chemical Council created an analogy comparing thermosetting plastic to a cooked egg, stating that the same way a cooked egg cannot be reversed into its raw state, once thermosetting plastic has solidified, this cannot be undone (American Chemistry Council Inc., n.d.). The melting characteristics of thermoplastics when subjected to heating give them the advantage of being recyclable.

During crude oil's refining process, it is separated into substances with different molecular structures, and simply put, these molecules are composed of carbon and hydrogen. In the polymerization process, monomers are bonded together in a chain to form a polymer whose properties are contingent on the type of monomers utilized (PlasticsEurope, n.d.). Thus, all polymers fall under the broad umbrella term of 'plastic' (American Chemistry Council Inc., 2011).

Examples of plastic resins include polyethylene, polypropylene, polystyrene, and polyvinylchloride (that is also called interchangeably vinyl), and these substances can have some impurities (Vasudevan et al., 2012). Table 2-5 displays the different types of thermoplastics and thermosetting plastics, along with their acronyms in brackets.

Table 2-5. Types of plastics (PlasticsEurope, n.d.)

Thermoplastics	Thermosetting plastics
Polyethylene (PE)	Epoxide (EP)
Polyethylene terephthalate (PET)	Phenol-formaldehyde (PF)
Polyvinyl chloride (PVC)	Polyurethane (PUR)
Polypropylene (PP)	Polytetrafluoroethylene (PTFE)
Polystyrene (PS)	Unsaturated polyester resins (UP)

Each of these types of plastic is assigned a unique mark to identify it for recycling. This mark is a quasi-triangle shape comprising three arrows surrounding a number which is unique for each type. These marks are displayed below in Figure 2.11

The use of plastic resins is associated with the types of plastic and their properties and applications, as shown in Figure 2.12.

Figure 2.10 Bulgarian reservoir cleaning process for plastic waste
(Rochman et al., 2013)

Figure 2.11. Classes of plastic

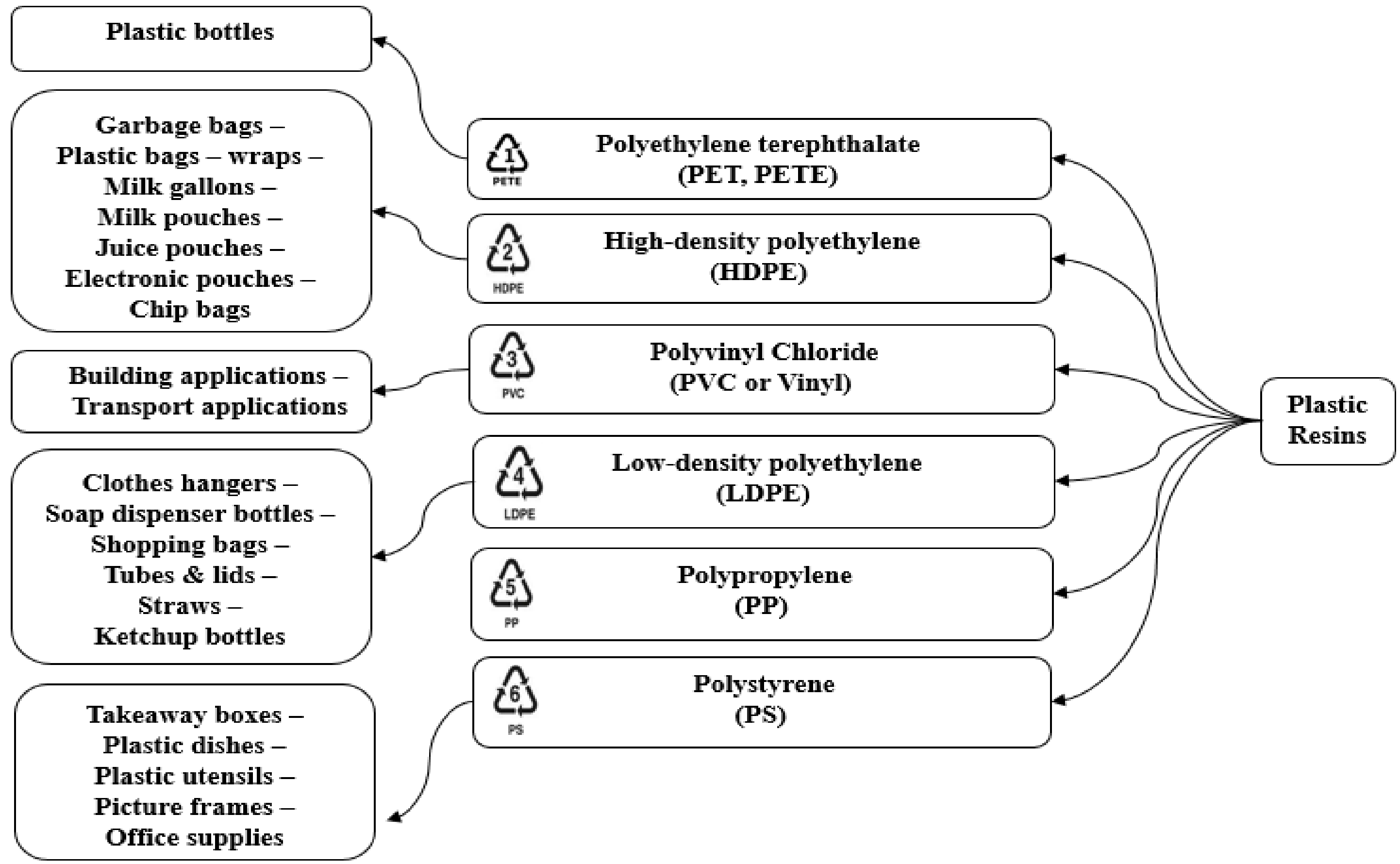

Figure 2.12. Different plastic resin uses

Polyethylene (PE) is a thermoplastic used in the manufacturing of a wide spectrum of items and is classified based on the weight of the molecules as either low-density, high-density, or linear low-density. Polymerization of the monomer ethane yields polyethylene, which is where the name polyethylene originates. HDPE is stronger than LDPE from the perspective of tensile strength (American Chemistry Council, 2005), and LDPE and HDPE are the main polyethylene compounds in plastic bags as well as bottles, tubes, garbage bags, milk and juice pouches, plastic wraps, and boat components.

Polypropylene (PP) is another type of thermoplastic which is used in many food applications, such as in ketchup bottles and drinking straws. PP is a fatigue-resistant material and has an average modulus of elasticity between those of LDPE and HDPE. The melting temperature of PP is above 160°C, which makes it highly resistant to heat (American Chemistry Council, 2005).

Polystyrene (PS) is used for plastic utensils, dishes, and disposable food boxes. Styrene is the monomer that yields polystyrene. This resin is known for its low strength and heat resistance as well as being less durable than other types of resins. Foamed polystyrene (also called expanded polystyrene) is made of 90% air and 5% polystyrene, and the high void ratio makes it lightweight and less thermally conductive. This type of PS is utilized in applications, such as coffee cups, office supplies, and takeaway food containers (American Chemistry Council, 2005).

There are several approaches to processing thermoplastic waste so that it can have multiple end-use applications after being recycled. Once the plastic waste is collected, it is then baled (Figure 2.13) and shipped to recycling facilities to undergo procedures including sorting, washing, shredding, and eventually extruding. The most common approach for preparing recycled plastic for commercialization is what is referred to as extrusion. Extrusion is similar to the process of grinding meat in a grinder, with the exception that the shaft is heated. It is the process of converting shredded plastic into films and sheets in a heated chamber. The temperature in the chamber along with the revolving screw cause the plastic to melt and pass through an opening (American Chemistry Council Inc., n.d.). As a result, large molecules are broken down into smaller molecules or monomers (American Chemistry Council, 2005) which are then joined together to form polymers with different properties (American Chemistry Council, 2005). Thus, it can be molded into various shapes or pellets through heating in the extrusion chamber (American Chemistry Council Inc., n.d.). Figure 2.14 shows an example of plastic pellets produced by extrusion. Recycled plastic is utilized in similar applications in the commercial market. These applications include the manufacturing of pouches, bottles, toys, cups, tubs, lids, etc.

Figure 2.13. Bales of collected plastic bottles

(University of Bath, New way of recycling plant-based plastics instead of letting them rot in landfill 2020)

Figure 2.14. Examples of plastic pellets produced by Enviroplast Inc. in Montreal, Canada

(Thermo Fisher Scientific Inc., n.d.)

Plastic recycling businesses have grown in recent years, and recycling facilities may be found across the globe, including in Canada, where recycling is booming. Further, between 1975 and 2012, there was an great increase in globally produced plastic materials, which hit 280 million tonnes (Jambeck et al., 2015). It has been reported that nearly 50% of this plastic is either sent to landfills or recycled for commercial reuse (Rochman et al., 2013). Moreover, alleviating the amount of uncontrolled plastic waste through more viable options would help lessen the repercussions to the environment (Jambeck et al., 2015; Abukhettala and Fall, 2021).

According to the Plastic Ocean Foundation, there is eight million of the 3 billion tons of plastic produced every year that end up after use in oceans, causing vast pollution and harm (Plastic Oceans Foundation, 2017). Another report indicated that in 2010, between 5-13 million metric ends up in the oceans (Jambeck et al., 2015).

Without proper and efficient waste management, the rate of ocean pollution will become a tragic by 2025, (Jambeck et al., 2015). The Plastic Ocean Foundation is among the most dedicated organizations endeavoring to raise awareness regarding the fact that there needs to be a better approach to discarding plastic material than merely dumping it in the ocean (Plastic Oceans Foundation, 2017). Plastic waste is considered a solid waste in the United States and most European countries as a result of the potential harm this waste has on animals, their habitats, and people (Rochman et al., 2013).

Furthermore, utilizing plastic waste in road construction work could open up many doors that would be of great benefit. This aligns with protecting the environment, as the plastic waste would no longer end up in landfills or the ocean. In addition, emphasizing the threat of plastic waste would encourage more research on the exploitation of this valuable material (Rochman et al., 2013)**.** In order for the outcomes of any studies exploring the use of plastic waste to be fully implemented for everyone's best interests, combined proactive actions are required not only by industry, but also by the public and decision-makers (Thompson et al., 2009).

2.4 Conclusions

This chapter covered the different types of pavement structure based on the application and the available materials as well as the required and sought level of comfort. Each type has its pros and cons in terms of absorbing the applied traffic load and transferring it to the underneath subgrade layer through the structure. It also briefly provides background on plastic types and its industrial classifications based on the resins incorporated in its manufacturing. The chapter highlighted the worldwide production of the plastic materials and bring the attention to the global risk signs associated with the disorder of disposing this waste into several locations including seas and oceans. It was mentioned that the wrongly handling of the waste and the lack of options to deal with it have led to such an issue.

The potential investigation of benefiting from the plastic waste is further examined in the following chapters. It will be incorporated with a certain type of subgrade soil and examined its geotechnical properties related to pavement application. Subsequently, the addition of assorted plastic waste with sub-base and base granular materials would be then investigated and discussed in paper two. The final stage is to investigate what plastic can bring to the properties of fresh and hardened concrete mixture intended for use in pavement surfacing layer application. The outcomes of the three articles are then analyzed and summarized.

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3 Chapter 3: Literature Review on Recycled Materials in Road Construction

3.1 Introduction

The construction and the maintenance of road consume huge amounts of natural materials (e.g., quarried aggregates, natural soils). So, road construction and maintenance place a significant burden on non-renewable natural resources, such as quarried aggregate sources. The warning signs of depletion of traditional or natural road construction materials have also gained awareness in many countries, including Canada. For example, in Ontario, a recent report on the state of natural resources was indicative of intensive consumption of natural aggregates, pointing to an urgent need to orient the market towards secondary and tertiary materials or unconvential materials (LVM-JEGEL, 2009). So, limited financial resources, endangered natural resources (depletion of natural aggregates), increasing environmental concerns, and provisions and stricter regulations that control the excavation and processing of rock have made it essential for researchers and practitioners to examine available unconventional materials (e.g., waste materials, waste by-products) to assess their potential incorporation in road construction (De Rezende et al., 2005). Unconventional materials in the road building industry are defined as materials that do not possess the same properties as traditional materials. Moreover, the alarming rate of increase of waste production and the problems associated with the management of these wastes are a problematic issue for governmental institutions in several parts of the world. International cooperation is a key factor in protecting the environment through diminishing greenhouse gas emissions, limiting environmental impacts of the generated waste and preserving natural resources via the effective exploitation of waste materials in road industry (Bassani et al., 2009).

Many pieces of research have proven the success of reusing recycled waste materials in pavement or road structures and shed the light on the gains from their inclusion in pavement construction. Different recyclable waste materials have been incorporated in various road construction projects all over the globe, displaying the opportunity not only for budgetary savings, but also for protecting the environment (Figure 3.1). Examples of the materials used include, asphalt roofing shingles (RAS), reclaimed asphalt pavement (RAP), crumbed rubber tires, recycled crushed glass (RCG), steel slag, foundry sand, fly ash, coal bottom ash, pond ash, oil sand, pumice stones and crushed concrete aggregates (RCA).

In the following sections, a state-of-the-art review of use of waste materials in road construction is provided. The main types of waste materials used in or proposed for road construction, their characteristics, the recycling methodology and previous studies on recycling these materials into road construction materials are presented and discussed. Moreover, the current and future challenges or concerns for use of such materials in road constructions are also discussed. Another objective of this review is also to understand or learn about the progress of the research on recycling plastic waste into road construction materials compared to research on recycling non-plastic materials into these materials. This will enable to assess and identify the gap or need of research on recycling plastic waste into road construction materials.

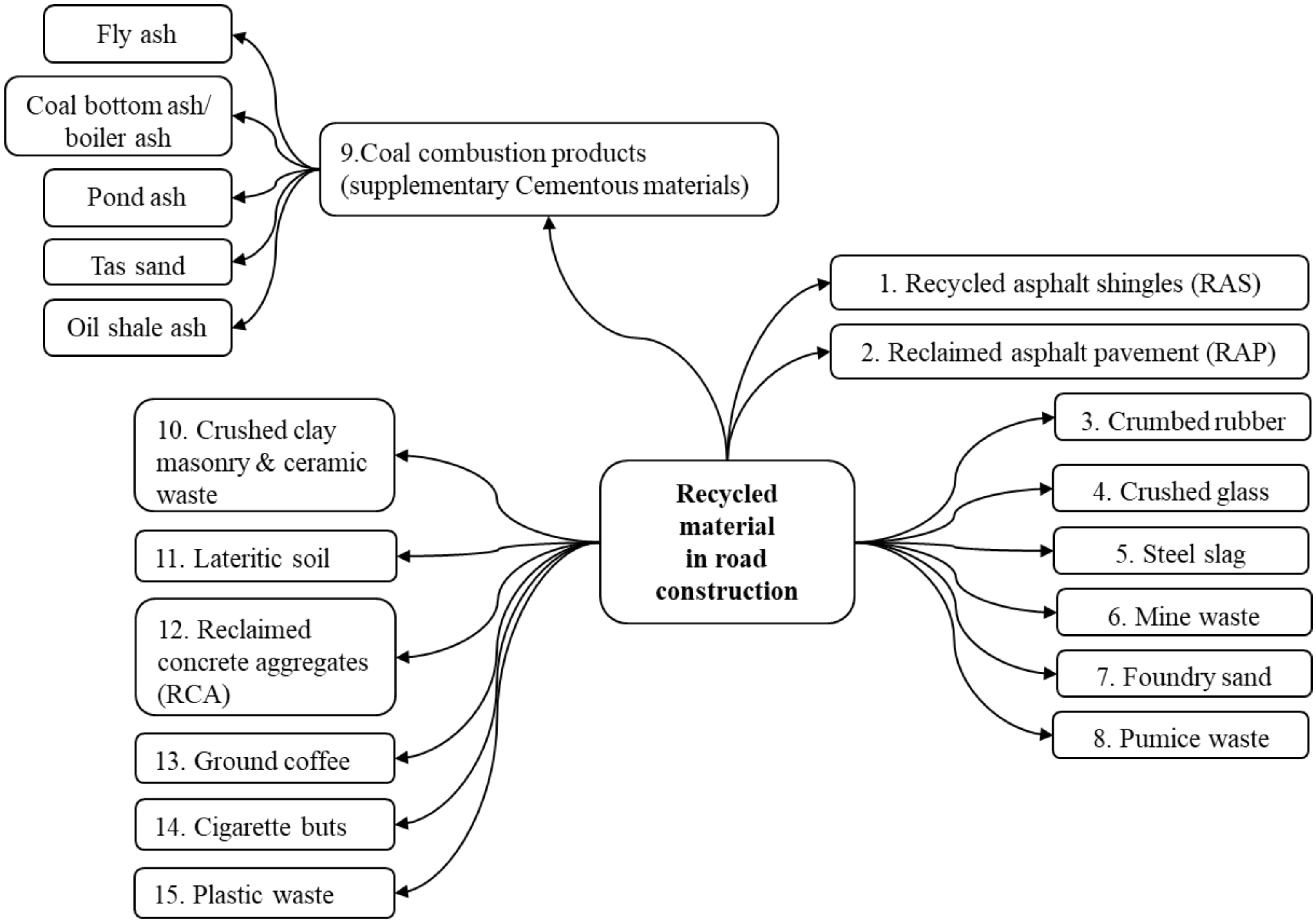

Figure 3.1. Diagram showing various recyclable materials in road construction

3.2 Types of Recycled Materials in Roads: A review

3.2.1 Recycled Asphalt Shingles (RAS)

Shingles are typically used for roof coverings in commercial and residential buildings, and are periodically replaced due to environmental deterioration (Wang et al., 2014). They can be either fiberglass or organic. Both types of shingles are composed of varying rates of mineral fibers, mineral fillers, and hard rock granules (tiny ceramic-coated granules). The content of asphalt cement in recycled asphalt shingles ranges from 19% to 31% (Cascione et al., 2015) with an overall average of 30% by total mass (Yang et al., 2014). Fiberglass shingles have an asphalt cement rate about 15% less than that of organic shingles, which makes fiberglass shingles cheaper and preferable than organic ones (Mohammad et al., 2014).

Fluctuation in the crude oil industry and unpredictable increases in the market price of virgin bitumen have spurred interest in looking for a substitute that is rich in bitumen and can be used in road construction. Asphalt shingles have great potential and have been proven to have acceptable performance as a potential substitute for asphalt cement in road construction (Abbas et al., 2013; Barry et al., 2013) with the aim of reducing the amount of virgin binder and associated costs. Therefore, efforts have been made over the last few decades to benefit from the asphalt shingles as a less expensive source of bitumen for pavement structures (Abbas et al., 2013). The economic benefit of incorporating recycled shingles in asphalt works was highlighted by Alvergue et al. (2014), where a reduction of 9.4% in the total required rate of natural bitumen for asphalt mixtures was reported. Additionally, a reduction of 7.7 to 8.0% for the dry and wet process was respectively reported (Mohammad et al., 2014).

There are two types of recycled asphalt shingles: manufacture scrap shingles and tear-off scrap shingles. These are most interchangeably called post-manufacturing shingles, or pre-consumer and post-consumer asphalt shingles, respectively. Post-manufacturing or pre-consumer shingles are obtained after manufacturing when shingles are rejected and cuts as scrap, while tear-off shingles are obtained from construction and demolition activities (Abbas et al., 2013) when shingles have deteriorated due to environmental aging (Barry et al., 2013) and roof coverings are replaced with new asphalt shingle membranes (Abbas et al., 2013). Post-manufacturing shingles are characterized by their low asphalt content compared to that of tear-off shingles. Using different sources of recycled shingles is an important practice in order to ensure the proper asphalt content in the shingles for proper designs and implementation

(Cascione et al., 2015). According to Alvergue et al., (2014) and Mohammad et al., (2014), asphalt shingle waste accounts for nearly 15% of the annual 170 million tons of construction and demolition waste, as estimated by the US Environment Protection Agency. Canada is claimed to produce about 1.5 million tonnes of shingle waste per annum (Yang et al., 2014), with about 0.3 million tonnes in Ontario alone (LVM-JEGEL, 2009).

The process of preparing recycled shingles for pavement applications consists of five steps. It begins by collecting high-quality RAS and conducting polarized light microscopy tests to assess the amount of asbestos in the shingles, which should not exceed 1%. Then, cleaning of the RAS from any debris takes place, where impurities such as nails, woods, and plastic are removed. Subsequently, the shingles are pulverised or ground into uniform sizes from 2.35 to 12.5mm (Mohammad et al., 2014). Some studies have indicated that typical ground sizes are 9.5mm (Barry et al., 2013) or 12.5mm (LVM-JEGEL, 2009), while others have noted that it could have a nominal maximum size of 1.19mm (Barry et al., 2013). Lastly, ground plastic is screened for any unwanted sizes in the mix (Mohammad et al., 2014).

Results of researches on the impact of adding recycled asphalt shingles (RAS) on the mechanistic properties of asphalt mixtures revealed that asphalt content varies with the type of shingles being considered for recycling. Conducting a binder recovery test using a centrifugal device showed that the asphalt cement (AC) content of pre-consumer shingles ranges from 14.6 to 18.1%; however, the rate and the range for different samples of RAS incorporated in the same project were much higher, at 20.5 to 36.7%. It was concluded that caution should be exercised when collecting post-consumer shingles, and sorting each source of post-consumer shingles separately would be more useful when designing RAS modified mixes (Cascione et al., 2015). The asphalt cement content in the utilized RAS is what should be considered rather than the percentage of RAS as a recycled material. The range of asphalt cement content varied from 14.6 to 36.7% in the RAS from the different sources used in the study, which was deemed to be considerably influential (Cascione et al., 2015). Hot Mix Asphalt (HMA) properties improved significantly with the inclusion of shingles in the mix at a predetermined rate, and tensile strength, rutting resistance, and resistance to moisture damage have been reported to be enhanced. However, the resistance to fatigue and thermal cracking has to be given more attention (Xiao & Amirkhanian, 2009).

The use of recycled asphalt shingles is limited by the percentage of 5% of the total weight of the mixture from a performance perspective, as it is made of an air-blown asphalt binder that stands out for its high viscosity compared to its virgin counterpart used in HMA (Abbas et al., 2013). Over 15 states in the US have extended the rate to 7.5% and limited the minimum allowable percentage to 2% in the dry approach, where RAS is mixed with the aggregate blend before the virgin bitumen is added to the mixture (Abbas et al., 2013). Still more research needs to be done in order to assess the extended use of RAS rate in asphalt pavement (Yang et al., 2014).

Aside from the dry approach of adding RAS to HMA mixtures, another approach to adding recycled asphalt shingles to original asphalt binders was developed in recent years, and is known as the wet approach (Abbas et al., 2013), which can reduce the concerns over improper and insufficient interaction between virgin asphalt cement and RAS bitumen resulting from aging and stiffening. The method was derived from the wet approach of adding scrap rubber and plastic waste to virgin asphalt cement and aims to preserve the original grade of binder used for the particular asphalt mixture. In this approach, the RAS is mixed with the virgin bitumen at a high temperature, close to the mixing temperature for conventional hot mix production, and then mixed with the aggregates to produce the hot mix asphalt. In most cases, the rate for adding RAS in the dry approach is expressed as the rate with respect to the total weight of the mixture; however, in the wet approach, the rate for adding the RAS is with respect to the weight of the asphalt binder (Mohammad et al., 2014). The preparation process for RAS in the wet approach includes four steps. First, the RAS is cleaned of any debris. Then, the RAS is pulverized to a uniform fine size where greater than 80% of it should pass the number 200 sieve size (Mohammad et al., 2014). The literature shows that the fine shingle quantity retained from the sieve size 0.297mm (#50) is used in HMA applications (Barry et al., 2013). Next, the pulverized RAS is thoroughly mixed with the original binder for a period of 30 minutes at a temperature of 180 to 200ºC. Finally, the modified binder is added to the hot aggregates to produce HMA (Mohammad et al., 2014).

The dynamic modulus (E*) values are lower for HMA with post-manufacturing shingles than for that with tear-off shingles. This indicates a higher rut resistance for tear-off-shingle-modified asphalt mixtures. Tear-off-shingle-modified mixes possess higher flow numbers than post-manufacturing mixtures which is also another indicator of better resistance to deformation than tear-off-modified asphalt mixtures. Unlike post-manufactured shingle mixes, tear-off RAS mixtures have proved to be ineffective in resisting low temperature cracking due to the effect of aging and oxidation the binder has gone through as a roof cover. An asphalt mixture that bears a higher tensile strain at failure is claimed to be more crack-resistant than that with a lower tensile strain value (Cascione et al., 2015). To conquer thermal cracking deficiency, it is suggested to use less-stiff asphalt cement when RAS is used (Moon et al., 2014). Some researchers have indicated that post-manufactured shingles have little influence of the stripping resistance of an asphalt mixture as compared to tear-off-modified mixtures (Moon et al., 2014).

Improved resistance to permanent deformation of stone mastic asphalt (SMA) was attained with the inclusion of RAS. Dynamic modulus testing is deemed very useful in evaluating the influence of using RAS and RAP on the performance of hot-mix asphalt mixtures. It is always favorable in order for asphalt pavement to resist repeated permanent deformation and fatigue- cracking failure to have a high dynamic modulus when tested at a high temperature and a low modulus when tested at lower temperatures (Yang et al., 2014). High dynamic modulus means less induces strain by the mixture, which consequently means better performance prediction (Cascione et al., 2015). It has been reported that as the percentage of RAS increases, dynamic modulus improves, which supports the fact that RAS mixtures are rut-resistant (Abbas et al., 2013). The modulus was higher for mixes in hot weather conditions and heavy traffic loading than for temperate weather and less traffic loading. An added amount of 5% RAS showed satisfactory fatigue resistance for the SMA (Cascione et al., 2015). Nonetheless, when creep strength testing was performed to examine the rutting resistance of asphalt mixtures modified with a rate of 5 to 10% of RAS as compared to conventional mixtures, the rutting resistance of the investigated mixtures declined (Mohammad et al., 2014).

Similarly, the impact of RAS on an open-graded frictional course (OGFC) has also been investigated. Asphalt cement grades of PG64-22 and PG76-22 were used with a recycled shingles rate of 5.5%. Rutting resistance was evaluated by means of asphalt pavement analyser, which demonstrated that a modified-RAS mixture is more rut-resistance than a non-modified mixture. Interestingly, the lower grade binder PG64-22 modified by RAS exhibited a higher resistance than the stiffer binder PG76-22. Tensile strength as measured by indirect tensile strength testing was greater for the RAS modified mixture. Visual observations indicated that RAS mixtures are also more moisture-damage resistant than a control mixture. Also, dynamic modulus was much higher for RAS mixtures when compared with the conventional non-modified mixtures. Modified RAS OGFC mixtures were more resistant to fatigue failure than their conventional counterparts. The complex modulus results showcased that RAS mixtures exhibited higher stiffness at the low and intermediate temperatures of 4ºC and 20ºC, respectively (Wang et al., 2014). Using RAS with an open-graded frictional course would be beneficial in terms of potential utilization with lower binder grades to get the required stiffness, and thus cost reduction is attained. This was also noted by Yang et al. (2014), where RAS was found to be efficient in meeting specification requirements when the grade of the binder is reduced by 6ºC. It is in fact recommended to do so not only for RAS, but also for RAP-modified mixtures where performance is expected to be satisfactory (Yang et al., 2014)**.** Since RAS-modified mixtures demonstrated unfavorable performance at the lower temperature, the study suggested further investigation on this issue for OGFC mixtures (Wang et al., 2014).

A rate of 5% of either post-manufacturing or tear-off shingles has been reported to be sufficient in providing an asphalt mixture that complies with specification requirements (Abbas et al., 2013). However, only 3% of recycled shingles is needed to be added to a RAP-modified mix to produce a more rutting-, fatigue- and thermal crack-resistant RAP asphalt mixture (Yang et al., 2014). It has been noted that the inclusion of RAS into asphalt mixtures at varying rates of 5% to 10% causes tensile strength to decline as the rate of RAS increases (Abbas et al., 2013;Moon *et al.*, 2014; Mohammad *et al.*, 2014), and resistance to thermal cracking also drops (Abbas et al., 2013). HMA is believed to exhibit more durability when RAS is used (Abbas et al., 2013), and this was also concluded when a rate of 5-10% of asphalt shingles was used (Mohammad et al., 2014).

When two asphalt cement types, PG58-28 and PG 52-34, were tested by a three-point bending test, it was concluded that the softer binder had a lower critical cracking temperature than the stiffer binder, which exhibited higher thermal stress (Moon et al., 2014). Asphalt cement PG64-22 and PG76-22 used with 5.5% RAS used in an open-grade frictional course and the lower-grade binder PG64-22 modified by RAS interestingly exhibited higher resistance than the stiffer binder PG76-22 (Wang et al., 2014).

There are several concerns over incorporating RAS and recycled-asphalt pavement (RAP) is asphalt pavements. These concerns stem from the aging and brittleness characteristics of such materials, which have been proved to have an adverse low-temperature impact on the performance of asphalt pavement (Moon et al., 2014). The combination of recycled-shingle RAS together with RAP has been examined and found to cause an increase in the lower temperature grade of the asphalt cement by 6ºC, which suggests the selection of a lower temperature binder grade when such a combination exists (Xiao & Amirkhanian, 2009). Modified asphalt mixtures with recycled-shingle RAS and reclaimed asphalt pavement RAP differ in the type and amount of asphalt binder involved. RAS involves an air-blown binder at a rate considerably higher, about 5 times than that of RAP (Mohammad et al., 2014). Asphalt binder from tear-off shingles is considered stiffer than the binder from pre-consumer shingles due to aging and environmental exposure issues (Abbas et al., 2013). Another concern with RAS-modified mixes is skidding resistance, which has blocked the utilization of RAS-modified mixtures in road surfacing layers (Yang et al., 2014). Using less viscous asphalt cement in hot asphalt mixtures modified with RAS is recommended to overcome the issue of low temperature cracking. Also, the high stiffness posed by the inclusion of RAS in these mixtures is a substantial issue when recycled shingles are added (Abbas et al., 2013).

Interestingly, using both RAS and RAP in HMA has been found to be effective is increasing high and low temperatures of the asphalt binder. A higher temperature grade is preferable to resist rutting, but an increase in a low-temperature grade would make the pavement more vulnerable to a low-temperature cracking. Therefore, to overcome the result of this change on performance at high and low temperatures, a softer asphalt binder is recommended. The results also indicated that on average, 3% of RAS would increase low-temperature by nearly 5.7ºC, whereas on average, 20% of RAP would bring the low-temperature grade to an increase of 6 ºC (Cascione et al., 2015).

In order to address the concern over the low-temperature performance of RAS-modified asphalt mixtures, different combined mixtures with RAS and RAP at varying rates were prepared. A total of 16 asphalt mixtures were mixed with two types of performance-graded bitumen, PG58-28 and PG52-34. From the three-point bending test, creep stiffness, m-value, thermal stress, and critical cracking temperature were determined, and comparison analysis was performed to interpret the obtained results. Since no strength testing was performing, a single asymptote procedure was followed to determine the critical cracking temperature (Moon et al., 2014). Creep resistance was not appreciably affected by the addition of RAS, and the referenced free-RAS mixture demonstrated a better relaxation ability than other modified RAS mixtures, as derived from the *m*-value results. Additionally, the study encouraged the combination of RAP and RAS together in asphalt mixtures. The study demonstrated that an amount of 25% of RAP used with 5% RAS result no observed changes (Moon et al., 2014). Therefore, an investigation should be performed in order to evaluate various combinations of RAP and RAS and their impact on the performance of HMA (Yang et al., 2014).

The high viscosity of recovered RAS binder has put an emphasis on further study of the impact of RAS on the performance of asphalt mixtures of different types. The extent of mixing recovered RAS binder with the original binder is still inconclusive as extracted bitumen from RAS-based asphalt mixture is a blend of original and recovered bitumen, and any characterization of the effective properties of this blend could be misleading and unreliable (Barry et al., 2013).

Characterization of aged physical properties of RAS binder were examined (Abbas et al., 2013). The soft binder of PG58-28 super-pave grade used with a battery of post-manufactured RAS binder rates of 0%, 5%, 7%, and 10% of the total weight of virgin binder used. Recovered RAS binder was cleaned of any impurities. Physical tests conducted on the recovered binder included rolling thin film oven tests (RTFOT) and pressurized aging vessel (PAV) tests to simulate the impact of short- and long-term aging on the asphalt binder during pumping, handling, storing, mixing with aggregates, transporting, and compacting. As the rate of RAS increased, the binder became stiffer and more viscous. If the RAS will be mixed with aggregates at the pug mill of the asphalt plant, as per the dry approach, the viscosity of the original binder for pumping, handling, and storing will not be affected; however, the flowability and workability of the produced asphalt mixture could be affected (Abbas et al., 2013). A bending beam rheometer test was also conducted as per AASHTO T313 to assess stiffness at low temperature, and creep stiffness and *m*-value (which represents the relaxation ability of the mixture) were measured. The results showed that the stiffness at low temperature was in compliance with the maximum set value by Super-pave; however, the m-value was much lower than the requirement and it was concluded an increase in RAS represents a detriment to the resistance to low-temperature cracking. The results indicated that RAS would have an impact on performance in the long term rather than in the short term. This was derived from the index values of the binders aged in the RTFOT and PAV respectively at incremental RAS content (Abbas et al., 2013). Research conducted in Canada also confirmed this finding (Yang et al., 2014).

With insufficient research on the long-term performance and construction requirements of RAS as a substitute in conventional HMA, a better understanding of RAS utilization is recommended (Cascione et al., 2015). RAS modified mixtures have been proved to provide acceptable performance for pavement in the short term. However, the long-term performance has been suggested to be evaluated in order to interpret the impact of aging, traffic, and environmental deterioration on the serviceability of RAS-based mixes (Yang et al., 2014). This needs to be done to ensure the acceptance by the industry that RAS-modified mixtures would be capable of providing as high-quality HMA as conventional mixtures (Cascione et al., 2015).

Cracks were observed in asphalt pavements modified with a high rate of RAS (Abbas et al., 2013). However, field observations of a major highway paved with a modified open-graded frictional course asphalt mixture exhibited an acceptable performance after a few years of service, and the pavement showed no indication of surface defects or failure over the 2 and a half years of observations. Nonetheless, the observations were made by visual inspection rather than a long-term performance evaluation procedure which should have been conducted for better assessment of the evolving pavement condition (Wang et al., 2014).

Having gone over literature relevant to using recycled shingles in asphalt pavement, it is noteworthy that practitioners and researchers have felt concerns using RAS-modified hot asphalt mixes in asphalt pavement surfacing. These concerns were also in relation to skid-resistance. Therefore, modified RAS mixtures have demonstrated an acceptable level of resistance to skidding, which may be a good indication for relieving concerns over the safety of road users. In a study in Canada, an 800m test segment was laid in 2009 using modified 1.5% RAS and 13.5% RAP, and a skidding evaluation was conducted in 2011 as per ASTM E-303 by using the Pendulum tester. All the pendulum number measurements were close to each other and to the acceptable skid number of 40, which indicated good skidding resistance. Thus, the authors suggested that RAS mixes can have appreciably good performance (Yang et al., 2014).

When asphalt shingles are used in such applications, environmental advantages include protecting the public health through abating the emissions from virgin binder production. Also, the overall amount of bitumen is preserved and thus less consumption is incurred (Mohammad et al., 2014). The reduction in fuel gas emissions is expected to be on an average of 11% when RAS is incorporated in pavement construction (Cascione et al., 2015).

3.2.2 Reclaimed Asphalt Pavement (RAP)

Recycling of reclaimed asphalt pavement (RAP) is deemed one of the successes in reusing waste materials in various industries due to the conservation of the required natural/processed aggregates as well as the asphalt cement contained in this reclaimed material (Kang et al., 2011). Asphalt pavements that undergo severe traffic and climate conditions without maintenance will eventually exhibit surface defects and unfavorable user-related conditions that will need to be eventually removed and recycled. Recycling can be executed partially or fully into new pavement or underneath pavement layers with the inclusion of new virgin bitumen, an additive, or a rejuvenator to restore the properties of the aged binder (Int et al., 2012). RAP is ranked by far the largest recyclable material in use in the North American region (Yang et al., 2014).

Several factors need to be understood for the better utilization of RAP and better performance of pavement structures. The literature and field experience with RAP have mentioned that having a clear picture of the properties of RAP is a key in safeguarding pavement construction and prolonging its service life (Khosla et al., 2012). The characteristics of RAP vary from one source to another according to the type and properties of the aggregates as well as the binder content and stiffness of the pavement, which justifies the need to give more attention to such factors when designing RAP-modified mixtures (Khosla et al., 2012).

The concentration of RAP in a new asphalt mixture is considered a pivotal factor influencing the performance of HMA. Technically, modification to the binder grade in the asphalt mixture, especially low temperature grades, does not need much concern when RAP is used at less than 20% of the total weight of the mixture, according to the Ontario Hot Mix Producers Association, OHMA. It has been reported that the performance of new asphalt pavement would be as acceptable as that composed of a low to intermediate rate of 25% RAP (Khosla et al., 2012). The impact of using a rate of more than 15-20% of RAP on the grade of binder was significantly higher than that when the rate is in the low range (< 15-20%) (Al-Qadi et al., 2007). It was indicated in the literature that between 15-25% of RAP in the mix needs a modification of reducing the grade on both low and high temperature by 6ºC (Khosla et al., 2012). However, when a combination of an average rate of 20% RAP along with 3% RAS is used, the low temperature grade of asphalt cement goes up by 6ºC, and a reduction in the low temperature performance grade of asphalt cement was suggested (Cascione et al., 2015). The performance of asphalt mixtures most likely varies between using virgin binder and RAP binder. Asphalt binders undergo aging and hardening during service, where long-term aging is related to field compaction, voids in the mix, and the water drainage quality. A stiff binder in RAP would increase the likelihood of getting an asphalt mixture with favorable resistance to rutting, fatigue, and low-temperature cracking (Khosla et al., 2012)

Researchers found that the grade of the virgin binder affects the rate of increase in the stiffness of a RAP binder, and it has crucial and critical influence on mix properties. Conclusions emphasized the importance of a precise measurement of a RAP grade before any inclusion into new pavement. Two sources of reclaimed asphalt pavement were used at a rate of 20% and 40%, with three performance-graded binders, PG 52-28, PG 58-28, and PG 64-22 (Khosla et al., 2012). The assessment process was carried out by testing the blended binder with a dynamic shear rheometer in three conditions: unaged, aged in rolling thin film over test (RTFOT), and aged in the pressure aging vessel (PAV). Stiffness of these tested binders were on the rise as softer binder exhibited a rise of 1.5- to 2-fold, and the increase was in the range of 1- to 1.5- fold for the stiffer binder (Khosla et al., 2012).

The usage of recycled asphalt pavement varies, from substitution of aggregates and binder in asphalt mixtures to an aggregate material for base and Sub-base and a stabilized CIPR (Cold in place recycling) base (Copeland, 2011). Low resilient modulus, low CBR value, and low compressive strengths are some properties that preclude incorporating RAP in CIPR work unless modified with some type of additives to enhance the impaired properties (Sireesh Saride et al., 2015). The use of a cold recycled mix has been limited to base-course applications in light of the insufficient strength of the mix. In some studies, this deficiency has been reflected by the splitting strength, which is as low as 0.4-0.6MPa compared to that of 0.8-1.6MPa for conventional HMA. This could be ascribed to the interaction between the aged and stiffed RAP binder and the aggregate materials. A study examined the impact of RAP material from 10-year old milled pavement treated with cement on the performance of a CIPR mixture and compared the performance with reference samples without reclaimed pavement. The aging impact could be discerned in the penetration, softening point, and ductility values of the tested recovered RAP binder. Triaxial shear and splitting strength tests were used to assess the cohesion strength and internal frictional angle, and residual strength respectively. The study revealed that the residual strength of the RAP mix has salient effects on CIPR properties. A reduction in splitting strength observed with an increase of RAP rate. That be attributed to the difficulty that the added asphalt cement encountered interacting with the coarse aggregates in the RAP. That was observed from the comparison made between the sieve analysis results of RAP and the aggregates from the extraction test. Samples were prepared in such a way that RAP-modified samples tested in the triaxial shear tester without exposure to temperature and to compare the cohesion strength with that of the recorded value of the unbound aggregate samples. It was shown than the binder around the milled aggregates in the RAP is what resulted in the higher cohesion of the CIPR mixture when compared to the unmodified samples (Khosla et al., 2012). It was strongly recommended to include new aggregates in CIPR mixtures to provide room for the binding agent to effectively coat the aggregate surfaces and improve the strength (Khosla et al., 2012), which it turn will limit the amount of RAP to be incorporated in the CIPR mixture. Another study in India concluded that the inclusion of RAP in a CIPR mixture would develop the resilient modulus and tensile strength of the material as well as resistance to deformation under loading, as reflected by the falling weight deflect graph test (Reddy et al., 2014).

One concern in incorporating RAP in new pavement construction is the effective amount of aged binder in RAP that interacts effectively with the new virgin binder. It was assessed that only 11% of the coated binder around aggregates in RAP effectively interact with the new aggregates (Huang et al., 2005). A study exploring the influence of aged RAP binder on the properties of PG 64-22 virgin binder in an asphalt mixture indicated a reduction in the amount of binder after dry-mixing with new aggregates, and it was shown that out of the 6.8% initial amount of binder extracted from RAP, mechanical mixing with new aggregates and the extraction of the binder resulted in a binder content of only 6% (Huang et al., 2005). The ignition method is preferred over the centrifugal method for extracting the asphalt binder from RAP mixes (Copeland, 2011). The authors concluded that the 0.8% was the rate at which new aggregates blended with aged binder, while the 6% remained as a thin film on the surface of old RAP aggregates (B. Huang et al., 2005). The literature indicates that sufficient heating and mixing time is crucial for the aged binder to be thoroughly blended with a new binder and to prevent the mixture from undergoing workability and compaction difficulties and to keep the VMA ratio within limits (Solanki et al., 2013).

RAP was also investigated as a base-course material over a lime-treated expansive clay subgrade, and the results from lab testing indicated a considerably high modulus and strength. Field monitoring using horizontal inclinometers showed that limited deformation occurred in the amount of 10mm after 2.5 years, which is less than the allowable value of 25mm. Pressure cell measurements were low and indicated no severe impact by compression settlement in the utilized RAP layer. The riding quality of pavement constructed over the RAP-modified base layer was considerably good, with an IRI of 1.1 to 1.22m/km as compared to the maximum value of 1.5m/km. The results demonstrated the potential replacement use of base-course material by RAP material subjected to a heavy traffic load, and suggested further monitoring for the impact on performance in the long run (Saride et al., 2010).

RAP has also been used in runway pavement construction. The utilization was limited to a dosage of 30% and was only allowed on the shoulders rather than on the main runway sections. An asphalt mixture with 17% RAP using the Marshal Design method was used in a taxiway, and performed satisfactorily during the construction and for an observation period of 6 years (Hajj et al., 2010).

The high inclusion of reclaimed asphalt pavement in asphalt construction is not a prevalent practice (Al-Qadi et al., 2007). There is a durability concern with utilizing high rates of RAP in asphalt mixes, which involves the interaction between aged and new virgin binder. If the aged binder is presumed to be inert and the RAP is deemed as a black aggregate, then there is no interaction between the aged and new binder; however, if the aged binder is presumed available for interacting with the new binder, the extent to which this interaction occurs needs to be assessed for implications on the performance of the asphalt mix (Al-Qadi et al., 2007). So far, there has been no approach to correctly measure the degree of interaction between aged and virgin binder in RAP mixtures (Copeland, 2011). Sparse research has been conducted on the usage of RAP at the considerably high rate of > 25%, which could be a breakthrough in utilizing high amounts of RAP in road construction and lead to savings and conservation of natural aggregates (Solanki et al., 2013). The maximum permitted rate of RAP in HMA applications as per MTO specification requirements is 30% (LVM-JEGEL, 2009). It is believed that higher amounts of RAP in HMA application is a reason for premature road failure, which requires examination by routine testing on HMA supplied by contractors. According to Hesp (2019), contractors tend to use higher rates of RAP than permitted, which results in repair costs of millions of dollars, and educating people in the industry is considered a key to the successful implementation of RAP in HMA. There is a concern over the overuse of RAP in field works by contractors in Ontario (Canada). Contractors tend to use as much as they wish in order to save the need to search for new raw aggregates at a higher cost. Thus, strict controls need to be followed in order to ensure that the rate of RAP used is what is in the design formula (LVM-JEGEL, 2009).

A comprehensive survey conducted in 2009 in the USA on behalf of AASHTO revealed that a high rate of RAP has been advocated by most state departments, and yet only limited use has been seen at rates of more than 20%. The average reported rate of RAP used in the US is about 12% (Copeland, 2011). In Canada, the rate of RAP usage varies from one province to another. The province of Ontario suggests a rate of 15% for surface courses, 30% for binder courses, and 50% when testing proves acceptable results. Higher rates are used in Alberta and New Brunswick, where 30% and 40% are respectively recommended, and no limits are set on RAP inclusion in three other provinces, British Columbia, Manitoba, and Saskatchewan (LVM-JEGEL, 2009).

A study executed in Oklahoma in the US explored incorporating high rates of RAP in the upper two layers, i.e., the base and surfacing layers. Field- and plant-mixed samples were prepared for 4 types of aggregate and an asphalt binder PG 64-22 mixed at varying RAP rates of 0%, 10%, 24%, and 40%. The plant-mixed samples were evaluated for their volumetric properties and tested for dynamic shear modulus and resistance to fatigue by using a 4-point beam test. Creep resistance and Hamburg wheel tracking tests were conducted on the field samples. Volumetric analysis showed a rise in VMA (Voids in mineral aggregates) and VFB (voids filled with bitumen) ratio with rises in the RAP rate, and similarly, as the RAP rate increased, the dynamic modulus also increased. Rut-testing showed a reduced rut depth for mixes with higher RAP rates (Solanki et al., 2013). In a study in Pakistan, the deformation behavior of an asphalt mixture containing 30% RAP was measured by means of a wheel-tracking device, and the rut depth exceeded the 10mm limit for loading cycles of 5000, which indicated the vulnerability of the mixes to rutting (Rafi et al., 2014). Moisture-induced damage was significantly reduced as the rate of RAP increased in the mixes, and the fatigue-resistance results were inconclusive. Increased stiffness with increased RAP rates for one type of mixture was observed as well as reduced stiffness for another type (Solanki et al., 2013).

The influence of using rates of 25-40% of RAP on HMA properties at low temperature was examined in a research work in Minnesota, USA. A total of 12 hot mixtures were prepared with PG 58-28 and modified PG58-34 with polymer and tested. Creep resistance was assessed with a bending beam rheometer as per the procedure adopted by Marasteanu et al. (2009). The results indicated that RAP-modified mixes had higher creep resistances than non-RAP mixes. The effects of stiffness, thermal stress, and cracking temperature were more pronounced for

RAP-modified mixes than the non-modified ones. However, the impact on these properties by increasing the RAP dosage in the mixes was not considerably pronounced in comparison with a non-modified binder (Falchetto et al., 2015).

High rates of RAP in base and Sub-base materials were investigated in Oman. Different rates of 0, 20, 60, 80, and 100% RAP were mixed with a crushed stones aggregate for use in a base layer, and the results were encouraging. However, as the rate of RAP increased, there was a decline in strength, and the CBR value dropped to below 70% at RAP rates beyond 20% which demonstrated pronounced noncompliance with the specification requirements for base and Sub-base materials (Taha et al., 2004). Another study conducted an investigation on using high rates of RAP in base material and performed a repeated load tensile modulus test (E, Mr) along with a sensitivity analysis based on the MEPDG of asphalt pavement (Alam et al., 2010). The RAP was used at rates of 30, 50, and 70% and compared with a 100% RAP-base material. The modulus values trend showed improvements with increases in the RAP rate. Sensitivity analysis demonstrated a substantial reduction in fatigue stress as the rate of RAP increased, and resistance to deformation of the Sub-base layer was not discernable (Alam et al., 2010).

Research has explored the field performance of RAP and non-RAP mixtures over a service period ranging from 1 year to three-and-a-half years, which is considered a relatively short period for monitoring. Longer performance periods should be considered in order to assess the trends for RAP performance. While rutting is considerably improved by incorporating RAP in the mixture, fatigue and thermal resistance results are unreliable, as the trend keeps changing (Al-Qadi et al., 2007). There has been a concern in the US about overly stiff mixes when polymer binders are used, which could cause early thermal cracking of the pavement (Copeland, 2011). In a study of highway agencies in the US, 35 out of the 50 agencies targeted constructed reclaimed asphalt pavements RAP on top of all other types of recycled material being used. In this survey, using the RAP as a friction surface course in asphalt pavement was given about 31% of the total received responses (El-Assaly & Ellis, 2001). A supplementary survey was executed in the province of Ontario in Canada by the Ministry of Transportation (MTO) which revealed that even though specifications allow the incorporation of 20 to 50% RAP in base and binder course layers, it is not a very common practice to have an asphalt mixture design with >25% RAP, which is attributed to practical field performance concerns (Copeland, 2011).

3.2.3 Crumbed Rubber (Scrap Tire Rubber)

Crumb rubber from scrap tires is produced through by milling shredded tires to reduce the size of the pieces. This milling process is most often conducted in two approaches, milling at ambient temperature or at ambient temperature moisturized with water, and milling at cryogenic/freezing temperature (Shu & Huang, 2013). The environmental pollution caused by the abundance of scrap tires around cities and the incurred expenditures for remedial actions to overcome the repercussions of disposing of tires in landfills is of great concern. To resolve this concern, one solution is to utilize this waste material in engineering applications including road or pavement construction works. Despite the reported successful incorporation of crumbed rubber in road construction applications, there is still research undergoing to comprehensively investigate this waste and its behavior and impact on pavement performance (Mashaan & Karim, 2014).

There are two distinctive approaches to using rubber tires as a recycled material in asphalt pavement construction: dry and wet methods, which differ in the way that the crumbed rubber is added to HMA. However, they both aim to enhance the performance of the mixture and boost its properties (Huang et al., 2007). In the wet method, asphalt rubber is made by mixing bitumen with the crumbed rubber. This modifies the properties of virgin binder as particles of rubber are swollen into the original asphalt binder. However, crumbed rubber in the dry approach is added as a substitute of 1-3% of fine aggregate in HMA prior to feeding the asphalt binder to the mixture. The mixture produced in the dry approach is commonly dubbed rubberized asphalt, or asphalt rubber (Huang et al., 2007;Shu and Huang, 2013). Crumbed rubber used in the dry process is coarser than that in the wet process, and is called rubber filler (Shu & Huang, 2013). There is also something called waste fiber, which is acquired from the crumbing process of rubber tires as a remnant and has been used in SMA to reduce the drain-down phenomenon that occurs due to a combination of two factors; high rates of bitumen and shortage of fine aggregates proportion in asphalt mixes (Huang et al., 2007).

Whether processed in either the wet or dry approach, the crumbed rubber used in asphalt surfacing applications has several advantages other than improved resistance to skidding. It also provides asphalt mixtures with high shear strength, which is favorable in withstanding imposed traffic loads and preventing the occurrence of rutting in underlying pavement layers (Huang et al., 2007; Mashaan & Karim, 2014). In addition, during freezing and thawing cycles in cold weather conditions, the presence of scrap rubber in underlying layer compositions plays a crucial role in reducing the frost penetration level (Huang et al., 2007). Some other benefits of using crumbed rubber in road applications include enhancing rutting and fatigue resistance, increasing stability, reduced flow values when rubber is used up to a rate of10% , and improving resistance to stripping (Mashaan & Karim, 2014). When added to asphalt binder at up to 30%, significant enhancements in properties such as weathering and stripping resistance have been achieved (Huang et al., 2007).

Significant changes occur in virgin binder because of adding crumbed rubber to it in specific portions, which include alterations in viscosity and temperature susceptibility. Several laboratory tests were conducted to evaluate the influence of rubber size and rate on binder properties and HMA performance wherein three sizes of scrap rubber were added to asphalt cement in accordance in the wet approach at 3 different rates, 5%, 10%, and 20% of the total asphalt binder weight. The tests included the Marshal Stability test, indirect tensile test, compressive strength test, and stripping test. Another set of tests conducted to investigate changes including a penetration test, softening point test, and ductility test. Rubber particle sizes had an effect on the penetration value of asphalt binder where the finer the size, the higher the penetration value became. The findings confirmed that adding crumbed rubber makes the asphalt cement stiffer, which means more viscosity and leads the binder to become less susceptible to temperature changes. Although rubber was found to raise the softening point of bitumen, the increase was not as significant as when plastic was used, and the compressive strength tended to be negatively impaired when the rubber rate exceeded 10%. The indirect tensile strength was also relatively affected by increases in rubber content, and it was concluded that indirect tensile strength is a better indicator of changes in strength with and without the rubber, and was preferred over Marshal Stability for assessing the impact of rubber additives on mixture strength (Tuncan et al., 2003).

The inclusion of rubber with virgin binders and its impact at low and high temperatures has also been examined. Rubber-modified binders possess higher ductility and soften at higher temperatures (Huang et al., 2007). The influence of the rate of rubber used is more pronounced at high temperatures than at lower temperatures. In a study exploring three different base binders used in Alberta, Canada, it was determined that for every 1% increase in crumbed rubber content, increases of 1.2 - 1.5ºC and 0.2ºC in high and low temperature grades respectively were observed. To preserve the viscosity of the asphalt binder at 135ºC, the study suggested maintaining the practice of using 10% crumbed rubber at maximum to keep the viscosity within the limits of 3000 MPa, as per requirements. Also as the rate of crumbed rubber increased, creep resistance improved (Macleod et al., 2007).

The consequences of using rubber on resistance to permanent deformation and fatigue-cracking was explored in Malaysia, which has been trying to cope with the problem of rutting and fatigue-cracking from heavy traffic loads and extremely hot and humid climate conditions. Rates of crumbed rubber used ranged from 0%, 4%, 8%, 12%, 16%, and 20% of the weight of bitumen in order to modify the 80/100 PEN bitumen grade, and were tested throughout a battery of experiments including tests for penetration, softening point, viscosity at 135ºC, and dynamic shear rheometry. The stiffness of the asphalt binder increased as the rate of crumb rubber increased, which demonstrated a high resistance to temperature change. An increase in the stiffness consequently increased rutting resistance, which was a sign of higher shear modulus values. The authors suggested that the size of the crumb particles increases due to the swelling action of the bitumen, which leads to stiffening of the rubberized binder. To investigate the impact on resistance to temperature susceptibility with the same rubber content rate at two different mixing temperatures, 160 and 180ºC, changes in softening temperatures were inconsistent, which was concluded to be due resistance to the change by the chemical properties of the binder and the rubber. A Brookfield Cone test for the viscosity at 135ºC demonstrated that crumb rubber levels should always be kept to less than 20% in order to maintain the viscosity limit of 3000 MPa, according to specification requirements of the asphalt institute. The Dynamic shear rheometry testing was conducted based on the AASHTO modified method D-4 P246 and showed an increase in the shear modulus with the increase in the rate of crumbed rubber. However, the crumb rubber content reduced the phase angle, which reflects the elasticity of the modified mix. The larger the phase angle (δ), the stiffer the material. A purely elastic material has a phase angle (δ) = 0º, whereas a purely viscous material has a phase angle of 90º. In general, a high complex shear moduli is needed to resist rutting, while a lower phase angle is needed to recover from deformation caused by loading (Mashaan & Karim, 2014).

The asphalt binder AC-10 was used with crumb rubber at varying rates in a full-scale field experiment in the field in two sites (Shuler & Estakhri, 2016). In the first project, crumbed rubber was mixed at 22%, 24%, and 26% of total weight of the blend with the AC binder along with dolomite type of aggregates in the lab. At the other site, limestones aggregates were used at 18% and 22% grind rubber. The laboratory mixing temperatures and times varied to better simulate field-produced mixtures. As the mixing time and temperature increased, the rubber molecules disintegrated from the rubber-binder blend. Softening point test results varied with the variation in the rate of rubber in the blend. The authors concluded that the softening tests would not be capable of producing conclusive results on rubber-binder blends (Shuler & Estakhri, 2016).

The wet approach is preferable over the dry approach, as the produced asphalt mixture is more compactable and exhibits better mechanical properties that that of dry mixtures. An assessment study was executed in South Carolina in the US to investigate rubber tires' influence on the mixing properties and performance of constructed pavements. Rubber was added at the rate of 18% of the weight of the binder. The wet approach for rubber tires demonstrated better laboratory results and derived a field performance preferable to the dry approach. An issue with the compacting of dry-mixed rubberized asphalt was discernable. The measured temperature reduced quickly to about 71-77ºC while stagnated for some time, which poses difficulties in compaction and resulted in cracking. Workability was another issue, and hand-working was not practical at some locations. Skid-resistance was also impaired, whereas wet-mixed asphalt demonstrated preferable results and performance. Skidding resistance was improved, and only the tensile strength ratio (TSR) was slightly close to the minimum allowable limit of 85%. The ITS and density of dry-mixed asphalt developed at the beginning of service life, while skid-resistance remained low. According to the authors, the performance was inconclusive and required further in-depth investigation and monitoring (Amirkhanian & Ph, 2001).

The moisture susceptibility of asphalt mixtures modified with crumbed rubber was assessed by evaluating changes in the viscosity and tensile strength ratio (TSR). Several mixes were designed in the lab composed of fine crumb rubber (smaller than 0.425mm) at different rates, RAP from two sources of aggregates of very limited differences in proportions, performance-graded binder of two types, and hydrated lime. As the rate of RAP increased, the viscosity increased, and when 30% RAP was mixed with the stiffer virgin binder PG64-22, the viscosity was higher than that with the softer binder PG52-28. The addition of RAP required a lower OBC, which resulted in higher density. Also, high mixing and compaction temperatures were needed as the RAP rate increased. As the crumb rubber rate increased, the OBC became higher and density was reduced due to the light oil in the rubber, which reduced the coating ability of the binder around the aggregates. Higher rates of RAP resulted in high ITS. The results indicated that the TSR for a mix containing up to 15% crumb rubber was below the 85% required by most specifications. Thus, despite the improvement in stripping resistance considering RAP, when crumb rubber is used, a high rate of RAP is needed or else anti-stripping additives would be the only option left, for which is cost significantly increases (Xiao & Amirkhanian, 2009).

Aside from scrap tire waste being used in asphalt pavement and subgrade applications, it has also reported to have been used in concrete block pavement (CBP). In light of the uncertainty from some laboratory tests on the properties of blocks made with crumb rubber, such as impaired abrasion-resistance and strength, a comprehensive field investigation was executed in Malaysia. Fine crumb rubber of two sizes, 1 to 3mm and 1 to 5mm, was used in a full-scale factory production test to assess the impact on the properties of concrete blocks. The aggregate materials used in the research were in use in Malaysia and had acceptable quality. Rubber was added in increments of 10% to a maximum rate of 30%. The testing included compressive strength, skidding resistance testing with a pendulum tester, and impact testing with the falling weight approach. The rubber-modified concrete blocks were lighter, and as the rate of rubber increased, the weight decreased as well. At rates exceeding 10% rubber, cracks were observed on the surface after 1 day of curing. Also, as the rates of rubber increased, absorption also increased. When the crumb rubber rate went beyond 20%, the adhesion between the rubber and cement mortar was severely affected. Also, as the rate of rubber went up, the calculated bulk, apparent, and saturate surface dry density decreased. Higher crumb rubber rates demonstrated more improvement in compressive strength than for rates of less than 20%. Skid resistance declined and did not manifest any improvement with the addition of rubber in the mixture. High rubber rates provided higher dynamic load resistance in the impact test and better cracking resistance. It was concluded that 10% was the preferable rate at which crumb rubber could be added to concrete block pavement where the strength and toughness were considered satisfactory for traffic pavement applications. Higher rates of 30 and 40% were also suggested for lighter applications such as sidewalks. Crumb rubber in CPB was proved to be effective in absorbing noise, as per the impedance tube method, and improved upon the relative noisy and rough conventional concrete block pavement (Nor et al., 2010).

Despite the advantages reported when utilizing rubber tires in asphalt pavement, there are issues that require further in-depth analysis concerning the potential non-uniformity of scrap tires and a lack of information regarding potential recycling of rubberized asphalt pavement (Amirkhanian & Ph, 2001). The performance of rubberized asphalt at high temperatures is questionable and needs to be exhaustively evaluated (Huang et al., 2007).

3.2.4 Crushed glass

Glass is among several other solid waste by-products, and most recycled glass is derived from building maintenance and reconstruction, glassware, and automobile windows. In most cases, it comes as a mixture of colored glass and various debris such as paper, plastic flakes, food contaminants, and other substances (Arulrajah et al., 2014). Glass does not decompose over a short period of time and can be disposed of by burying. The fragile characteristics of glass make it easy to reduce its size for incorporation into pavement works (Su & Chen, 2002). Glass waste comprises a considerably high proportion of overall waste material in different parts of the globe.

Depending on the processing approach, recycled glass might be produced in three different forms: coarse, medium, and fine-grained. Whilst coarse recycled glass is rarely incorporated in road construction works, medium and fine recycled glass are prevalent in the industry. Fine recycled glass has a less angular shape and a lower flakiness index compared to coarse recycled glass, and it occasionally resembles a natural aggregate in its appearance (Disfani et al., 2011). This waste was found to be effective in improving the mechanical properties of HMA when ground to a fine size before utilizing it as an ingredient in HMA (Su & Chen, 2002). In addition, it is considered a cost-effective substitute in Sub-base materials (Imteaz et al., 2012).

Recycled glass is considered a significant material that can be utilized in road or pavement structures. It possess quality properties that needs to be considered (Huang et al., 2007). Exploiting recycled glass in asphalt pavement construction either in surface layers or underlying layers has successfully been practiced globally (El-Assaly & Ellis, 2001). Improving soft subgrade materials and weak sub-base or base materials is attainable by different approaches. Glass modification is among the effective approaches that has been reportedly successful in underlying pavement structure layers, not to mention also improving the properties of the top surface layer in asphalt pavement (Arulrajah et al., 2014). The literature highlights that the little experience with recycled glass implementation is because of the lack of understanding the behavioral performance of this material when incorporated in various pavement applications (Younus Ali et al., 2011). However, this waste material has given acceptable performance results when used in the surface pavement layer in the amount of 10-15%, providing it does not exceed 4.75mm to preserve the safety of users and vehicles on roads (Huang et al., 2007).

Glass is found to improve safety, as skid resistance develops over the years of pavement service. With the abrasion force induced by traffic load, glass granules become exposed and light-reflection at night develops. The performance of a road constructed with 10% glass after one year of monitoring revealed favorable results, and the surface did not exhibit any deformation (Su & Chen, 2002).

Fine glass is characterized by a flatter Proctor curve resulting from the deficiency in water absorption of fine glass particles (Younus Ali et al., 2011). The results from (Ali & Arulrajah, 2012) demonstrated that as the rate of glass increases in a blend of fine glass and recycled concrete, the absorption rate declines, which is understandable, as recycled concrete would absorb more water and fine glass would absorb less water (Ali & Arulrajah, 2012). Hydraulic conductivity is another important property that needs to be considered when assessing recycled glass as a material in road construction, as it affects the permeability of the material (Ali & Arulrajah, 2012). The results from the Australian study found fine and medium recycled glass to be suitable for a medium-permeable course, and results showed similarities in behavior with natural aggregates (Younus Ali et al., 2011).

Coarse recycled crushed glass is not recommended due to the potential change in gradation during mixing and compaction. The gradation of the recycled glass aggregates could be significantly affected. It has been shown that sand content of coarse recycled glass increased by about 22% after performing modified laboratory proctor test. Incorporating coarse recycled glass would risk the overall aggregate skeleton in the asphalt mixture due to potential segregation (Younus Ali et al., 2011).

An Australian study made an effort to raise awareness of using recycled glass in road construction, as the performance of medium and fine recycled glass was examined through a set of several laboratory tests (Su & Chen, 2002). Consistency tests could not be performed on the fine recycled glass due to its non-plastic behavior. Medium recycled glass exhibited a higher density than fine recycled glass as reflected in the higher strength determined by CBR testing. Direct shear test results indicated a higher friction angle for the medium recycled glass, resulting in higher strength due to better interaction between the particles resulting from the compaction force. Nonetheless, the results for the friction angle were quite like those for natural aggregates. For a better simulation of moving traffic loads on road surfaces, repeated load triaxial resilient modulus tests on fine and medium recycled glass samples collapsed either before testing or after few loading cycles, and thus the testing was called off. The results were encouraging for incorporating recycled glass in various road construction applications, including subgrade base and sub-base materials. The authors recommended further investigation on the inclusion of fine recycled glass in asphalt paving applications (Younus Ali et al., 2011).

Recycled fine glass was added to recycled concrete and waste rock and assessed for base and sub-base applications. Several road sections were constructed in Australia with varying rates of recycled glass. The field performance of the modified sections was compared to the performance of non-modified sections constructed of recycled concrete and waste rock only. The laboratory results reflected favorable enough properties for the recycled material to be used in Sub-bases. Subsequently, two sections were constructed of recycled concrete and waste rock commonly in use in Australia, while other sections were constructed with the addition of 10%, 15%, 20%, and 30% fine recycled glass mixed with recycled concrete in addition to other three sections made of waste rock and 10%, 20%, and 30% fine glass. All the sections were later on paved with a 50mm asphalt layer with 5% fine glass (Arulrajah et al., 2014). Ease of handling and acceptable levels of workability were observed for a waste rock mixture with 15% fine glass. Analyzing the various blends of fine recycled glass and recycled concrete indicated the tendency of blends containing more than 30% fine glass to become coarser and thus harder to compact. This confirms that as the rate of fine recycle glass increases, handling becomes harder. Beyond the specified rate, further compaction by traffic would degrade the aggregates skeleton, and thus it should not be exceeded for sub-base material applications. It was found that 20% fine recycled glass with 80% recycled concrete or waste rock had the optimal performance as derived from the field testing (Arulrajah, Ali, Ph, et al., 2014).

An extensive field evaluation to determine whether a rate beyond 15% could be incorporated was conducted in another study (Ali & Arulrajah, 2012), and the field performance for the base and Sub-base materials was evaluated using a Clegg impact hammer. The CBR results from the Clegg impact hammer test revealed that most base-course constructed sections had insufficient strength. However, the CBR value for the sub-base material was acceptable over most sections except for small areas. Hence, the results suggested that the inclusion of fine recycled glass should be limited to sub-base applications rather than the base material (Arulrajah et al., 2014).

Regarding HMA applications, fine recycled glass was examined in Taiwan by carrying out a Marshall Mix design with two types of binders, 85/100 PEN grade and emulsified asphalt type II. The rates of recycled glass varied from 0%, 5%, 10%, and 15% to replace the minus 4.75mm aggregates (Su & Chen, 2002). The optimum binder content was found to rise as the rate of glass in the mix increased. The performance was evaluated after a year for a section modified with 10% glass by using light reflection, skid resistance, permeability, Benkelman beam, and compaction level tests. Cores made of emulsified bitumen demonstrated higher stability than those with virgin 85/100 binder. The weak bonding strength of the mixed resulted from weak interaction between the particles where bitumen film remained on the surface of the glass aggregates. This means that the hydrated lime intervenes and causes the mixture to become stronger due to better bonding characteristic. Owing to the wearing action by traffic, glass granules seemed to emerge on the surface and a concomitant enhancement in skid resistance occurred, as indicated by the pendulum test measurements (Su & Chen, 2002). Recycled glass, and particularly fine recycled glass, is believed to have an acceptable abrasion resistance comparable to the resistance of natural aggregates in asphalt pavement.

A study in Australia found the LA value for fine and medium recycled glass to be 24% (Younus Ali et al., 2011). Permeability testing following Japanese standards was executed to measure the time required for water to seep through field cores, and the results showed low permeability in light of water being hindered by the characteristics of the glass (Su & Chen, 2002). Stripping was one drawback of using recycled glass in asphalt pavement (Y. Huang et al., 2007). As the rate of lime in a glass-modified asphalt mixture increased, the higher the resistance to rutting of the mix became. Nonetheless, for an economic benefit, an amount of 2% lime is suggested to make the asphalt pavement more durable and less susceptible to moisture-induced damage when fine recycled glass is used (Su & Chen, 2002; Huang et al., 2007). The influence of water damage on composites of recycled glass with the conventional hot mix asphalt, sub-base or base-course has been given little attention. Since recycled glass mixes exhibited low absorption, the impact of moisture-induced damage needs to be investigated. It should be noted that cracks have been observed, and thus further investigation into fatigue resistance should be undertaken (Su & Chen, 2002). Glass-modified asphalt pavement has been reported to be as satisfactory as conventional asphalt pavement when similar machinery and field techniques are used (Huang et al., 2007).

One type of recycled glass used in pavement construction is called foamed glass gravel, which among other applications has been deployed to overcome the failure of soil due to thawing action. It is manufactured purely from recycled glass. Foamed glass waste is characterized by its coarse surface, relatively low density, improved internal angles, and high strength. Foamed glass gravel has been incorporated in asphalt pavement to reduce noise pollution and water damage to the HMA as water seeps through surface voids to the voids between coated aggregates. This recycled material has been successfully used over soft layers in pavement construction works in embankment projects in Europe. Direct shear tests have shown a wider load distribution capability of this method over larger areas. Once the foamed glass gravel is supplied to a site, the construction process involves laying a geotextile layer on top of a soft subgrade layer followed by a foamed glass gravel layer, and then it is encapsulated with the geotextile layer and the process is completed by the laying of the surface course. Additionally, for permafrost soils in locations where the soil undergoes freezing for protracted periods of 2 years or more, as in some parts of Russia and China, the thawing action is disastrous to structures over such soils. With increases in temperature during mild months and the consequent thawing cycles, the properties of the underlying layers deteriorate and the ability of the road structure to carry the traffic load diminishes, resulting in road failure. The utilization of foamed glass gravel as an alternative to reinforce these locations has been efficiently used (Emersleben & Meyer, 2012).

Recycled glass is also considered to be efficient in increasing the strength of soft alkali clay soil. Soft alkali clay soil is available in abundance in Japan and displays unfavorable tendencies when using it in construction works. However, this weak soil has been modified effectively with additives such as hydrated lime and cement.

An investigation study on the impact of adding foamed glass waste on mechanical properties of Alkali clay soil through tests included the CBR test, the unconfined compressive strength (UCS) test, and compressive splitting strength (Taylor et al., 2007). In this study, fine foamed glass of 2.00-4.75mm size was used. The lime rate ranged between 5- 20% while rates of the foamed glass rates ranged from 10 - 25% at an increment of 5% of the total weight of the soft clay for both. The foamed glass- and lime-modified clay soil was compared with the lime-modified clay soil, and the foamed glass waste was found to be effective as a substitute for a certain amount of lime in improving the alkali clay soil. The water content of the alkali clay was high with a plasticity index of 49.9%, which indicated the unsuitability of using the material on its own. It was expected that the foamed glass-modified clay soil would have a lower water content, and a reduction in the range of 10-15% was determined for the mentioned rates of foamed glass and lime used in the experiment. The influence of addition lime on both mixture, alkali and foamed glass-alkali mixture showed similar trend when measuring the unconfined compressive strength. With 10% lime, the strength of the lime-alkali mixture was optimal. High lime content provides improved compressive strength. As the foamed glass rate in the mix increased, the strength increased to the extent that was governed by the percentage of lime in the mix. The CBR results showed an improvement of 68.5% on average for a foamed glass rate of 25%. The authors deduced that foamed glass- and lime-modified alkali soft soil could be utilized in base or Sub-base layers in pavement structures (Taylor et al., 2007).

There are some environmental concerns that acts as a deterrent to using recycled glass in road construction, and this concern could be relieved by investigating the impact of using such glass in road works. An analysis of the short- and long-term toxic impacts and risks associated with such implementation of recycled glass was conducted, and the findings highlighted that the potential substitution of recycled glass in road construction applications would not harm the environment or human health (Disfani et al., 2012). Total concentration (TC) and Toxicity characteristic leaching procedure (TCLP) showed the recycled glass is a hazard-free recycling material. Danger to the environment was not found from using recycled glass, as indicated by the TCLP test. The concentrations of leachate of the investigated glass followed the Australian Leaching Testing Protocol; they were compared with the allowable value for fill material and found to be satisfactory (Disfani et al., 2011). An Australian study compared the level of contaminants in recycled glass with the requirements in terms of the total concentrations and standard leaching procedures, in accord with the Environmental Protection Authority, and the compositions of fine and medium recycled glass were determined to be that of inert solid waste that would not jeopardize the water system even in acid rain circumstances (Disfani et al., 2012).

3.2.5 Steel Slag

Steel slag is produced at steel factories when lime flux mixes with iron ore. The quality of the produced steel has an impact on the quality of the produced steel slag. Steel slag outweighs other waste materials for its ease of collection from steel factories and for the control over its quality. The chemical composition of slag, coupled with the cooling-down process, is what identifies its type (El-Assaly & Ellis, 2001; Bull and Fall 2020a; Xia et al. 2020). Technically, steel slag is most frequently classified into two groups, metallurgic and phosphoric slags. Metallurgical slag is further classified as ferrous or nonferrous, where ferrous slag is characterized by its black color and obtained in two ways, blast furnace and steel smelting (Krayushkina et al., 2012). Steel-smelting slag is a compound mixture of silicates and oxides that hardens and become a strong solid substance once cooling down takes place (Asi et al., 2007).

Steel slag has not only been used in aggregate structures for HMA mixtures, it has also been exploited as an additive in soil stabilization works (El-Assaly & Ellis, 2001). It is believed that employing steel slag in road construction would bring significant savings as compared to conventional natural aggregates (Krayushkina et al., 2012). According to a Ukrainian study, the production cost for natural aggregates for road construction is as much as twice as high as that for steel slag (Krayushkina et al., 2012). In the UK, steel slag has been found to be recyclable, and 98% of this slag is used in aggregates in the construction field (Huang et al., 2007). Between 50 and 70% of the 10-15 million tons of steel slag produced annually in the United States is recycled in the field of road construction (Pasetto & Baldo, 2012). A report issued by the Ministry of natural resources, of Ontario in Canada indicated that there is a concern over the expansion problem of steel slag. The report mentioned that despite all efforts made by steel manufacturers to enhance the quality of such slag, there is still reluctant to pursue utilization of this material in the industry. However, the report stated that city of Hamilton has further laid the asphalt pavement with steel slag on some residential roads and no observations had been noticed on the performance (LVM-JEGEL, 2009). Ontario's Ministry of Transportation is not permitting using slag in Sub-base applications (LVM-JEGEL, 2009).

Field experience with steel slag has revealed substantial advantages from including steel slag in road construction works, including outstanding resistance to moisture-induced damage during freezing and thawing and improved skidding resistance (Krayushkina et al., 2012). The only concern with using such material in road construction is its propensity for expansion. The more that steel slag is used in the road construction field, the less contamination of the environment from the processing and extraction of natural aggregates and from disposing of it in landfills (Krayushkina et al., 2012).

Research by the Saudi University of Petroleum and Minerals and the Strategic Highway Research Program (SHRP) reached the conclusion that using steel slag enhanced the durability of modified hot mix asphalt consisting of polymer modified binder, limestones filling material, fine aggregates (Huang et al., 2007). Another study, conducted by Nottingham Centre for Pavement Engineering in the UK, deduced that an asphalt mixture modified with steel slag would have enhanced durability and significant resistance to deformation (Huang et al., 2007). The most common steel slag produced in the UK is obtained from the well-known Basic Oxygen Steel (BOS), and it was praised by the German Research Association of Steel and Iron to have good performance in asphalt pavement when compared with slag-free asphalt pavement (Huang et al., 2007). Under uniaxial loading on the diameter of the specimens, fatigue resistance testing was conducted at various loading cycles, and simultaneously, the vertical permanent deformation was recorded to evaluate rutting resistance. The results indicate the resistance to both significantly improved with increases in steel slag in the mix. However, at 75% of steel slag in the mix, resistance started to decline due to the high percentage of binder in the mix, which decreased bonding strength (Asi et al., 2007).

Before it can be used in the construction field, slag undergoes size reduction (El-Assaly & Ellis, 2001). It is recommended to use the slag as either fine or coarse aggregates in an asphalt mix in order to control the amount of air voids and reduce potential flushing (Asi et al., 2007). Slag with large grain sizes was examined in China to replace conventional ballast aggregates in a stones mastic asphalt mixture SMA. Rutting resistance was tremendously enhanced (Y. Huang et al., 2007). Coarse-grained steel slag was examined in Turkey with four different Marshall-designed HMAs with two asphalt binders, and the performance of the mixtures was evaluated using indirect tensile strength tests, creep tests, and Marshall Stability and indirect tensile stiffness tests. The results concluded that steel slag improved all mechanical properties of the investigated asphalt mixtures (Ahmedzade & Sengoz, 2009).

It is also important to consider the rate of steel slag added to an asphalt mixture and its ramifications on performance. A high rate is deemed detrimental to the in-service performance of HMA which requires a higher binder content that leads to rutting. Creep tests were conducted on various asphalt mixtures modified with steel slag at different rates ranging from 25-75% and at an increment of 25%. The relationship between the cumulative deformation and number of loading cycles demonstrated that the resistance to deformation is enhanced up to a rate of 75% of added steel slag. It was emphasized that the optimal resistance was with 25% slag, and the impact was neglected at higher rates than that due to the high percentage of bitumen needed to coat the surface area of the angular steel slag aggregates. Carrying out resilient modulus tests on mixtures with various slag rates showed a decline in elasticity at higher rates. In evaluating the addition of steel slag into sub-base materials and owing to the expansion behavior of the slag, there was a slight increase in the resilient modulus at a rate of 10%. However, at a higher percentage of 20%, the modulus value dropped, as cracks were noticed to have developed in the tested specimens, which is ascribed to the behavior of the slag (Ghazireh et al., 2012).

In order to evaluate the impact of including steel slag in asphalt mixtures, a research study was conducted in Jordan to assess the benefits of reducing the adverse effects of steel slag dumped in uncontrolled landfills by incorporating it in asphalt pavements. The performance of local asphalt mixtures was examined by means of laboratory tests at 5 rates of steel slag. Slag was added at 25%, 50%, 75%, and 100%, and the results were compared to the performance of a slag-free mixture. The suite of tests included indirect tensile strength test, resilient modulus test, creep test, moisture sensitivity test, and fatigue and rutting resistance tests. Also, a stripping sensitivity test was executed as per AASHTO T283. It was concluded that bonding strength is improved with the presence of steel slag in the mix. This also was analyzed by the statistical analysis tool ANOVA and complied with the findings (Asi et al., 2007). This has raised the question of whether moisture damage can be evaluated to assess the resistance to various freeze and thaw cycles.

The high durability and rutting resistance benefits from using steel slag outweigh the problem of the heavy specific gravity of mixtures incorporating steel slag, which raises the cost of transporting the mixtures. Steel slag should be considered when used in roads with heavy traffic and when the subgrade layer exhibits weakness in supporting imposed traffic loads (El-Assaly & Ellis, 2001).

3.2.6 Mine Waste

Mining waste by-products are produced during operations at mine sites both before and after processing certain types of materials such as coal, marble, and gold. These processes are considered all over the globe to be the largest source of waste by-products due to the vast volumes produced (Alainachi and Fall 2020; Guo et al. 2020; Ali et al. 2021). The mechanical and chemical properties of mining waste materials depend largely on the type of rocks processed, the type of mining process, and the approach used to handle and store the products at disposal locations (Bull and Fall 2020b). Mine rock waste is a material that is considered suitable as a substitute for use in road construction since it still has not gone through further processing at the mining site. A lack of landfills and the large volumes of waste rock produced at some heavy mining locations make the utilization of such material in road construction potentially attractive.

Mine waste and by-products have been widely used in road construction, in particular in base-courses and in HMA as a frictional additive (El-Assaly & Ellis, 2001). Marble quarry waste is a valuable material that can be used as an aggregate in asphalt mixtures. Due to some drawbacks of the waste's properties, it is often advisable that it be used in the base-course layer or as an ingredient in the aggregate skeleton for the less durable binder course layer (Akbulut & Gürer, 2007). This waste material is the number one solid waste material in Ontario, Canada. Its sources are usually situated in remote areas, causing increased delivery cost. Field trials with mine waste have revealed a serious leaching issue that should be thoroughly analyzed before incorporating this waste in any construction applications (LVM-JEGEL, 2009). The fact that mining locations are stationary makes using mine waste sometimes impractical, and so it can only be considered as a project-based source of material (El-Assaly & Ellis, 2001).

Successful experiences with incorporating mine waste in the road construction field have motivated some countries to investigate the impact of using such material at local scales. For instance, Ghana is a country that produces a very large quantity of waste rock from gold processing, which poses various adverse impacts to inhabitants at mining locations and on the environment. To cope with the abundance of mine waste at certain locations around the country, samples of waste rock materials were collected and evaluated to assess their potential incorporation in road construction. The physical and mechanical properties were examined by unit weight and by water absorption and consistency tests, gradation, flaking, and elongate indexes, aggregate crushing value tests, 10% fine value tests, and aggregate impact value tests (Agyeman & Ampadu, 2016). A gradation analysis after removing plus-20mm particles showed a trend of particle distribution similar to those of base and sub-base gradations. However, further processing of the rock waste was suggested in order to bring the overall gradation within the envelope suggested by the Ghanaian requirements for base and sub-base materials. The tested material was found to be impermeable with only a 0.39% water absorption rate. Other mechanical properties tests demonstrated sound properties that indicated that it could be incorporated into base and sub-base layers, as only the aggregate impact value was below the 25% specification. In the same direction, when chipping was investigated, the waste materials met the durability and strength requirements for surface dressing. This material exhibited no toxicity damage. A conclusion was made that it was possible to substitute natural aggregates with this mining waste, which will bring diverse benefits including the conversation of natural resources, protecting the ecosystem, and reducing the threat to human lives in the vicinity of the mining locations (Agyeman & Ampadu, 2016).

Another waste material produced during mining and rock processing is marble dust and quarry waste. This is a fine by-product from the extraction and cutting of marble at factories and mining locations. Based on the properties of marble ore (marble quarry waste, the lithic waste from the extraction process of the marble at quarries) could represent about 20% (Chandra et al., 2002) to 40% (Akbulut & Gürer, 2007) of the entire marble production. This waste is occasionally utilized as a filling material in HMA applications. Marble quarry dust has been considered as a substitute material for conventional mineral filler in HMA mixtures. Stability, indirect tensile strength, and fatigue resistance have all been proven to be enhanced with the addition of marble dust in aggregate blends (Chandra et al., 2002).

Quarry waste is another mining by-product produced throughout the extraction process of aggregates in quarries before any washing takes place, and it has been effectively utilized in road construction. A 300m road segment was constructed and subdivided into 3 sub-sections. Each section composed of different compositions of an investigated quarry waste. One was a quarry powder and the other was a conventional local crushed stones material. Laboratory compaction levels varied along the three subsections, and the results were compared with the lateritic-only gravel. Plate bearing loading test provides reduced deflection measurement on for the local soil-powder subsection. A two-year observation period demonstrated similar performance of all subsections. The addition of 20% of quarry waste to base-course materials, or 30% to Sub-base s, proved to be efficient and provided acceptable results. This indicates the potential usefulness of incorporating quarry waste in either the Sub-base or base-courses of asphalt pavement (De Rezende et al., 2013).

A research study in the UK focused on determining the influence of various waste materials on a Type 1 sub-base mixture. The wastes used included steel slag, limestones dust, fuel ash, and granulated blast furnace slag. Aside from the control Type 1 mixture, 7 other mixtures, categorized as unbound or bound mixtures, were prepared with different percentages of limestones dust and steel slag. Also, at a fixed rate of dust, the rate of blast furnace slag varied. Limestones dust's effect on the resilient modulus was examined and proved to be positive. There was an increase in the modulus value when dust was added at 10 and 20% owing to the improved interlocking and density (Ghazireh et al., 2012).

3.2.7 Foundry (Moulding) Sand

Foundry sand, or what is also frequently called molding sand, is a fine-grained material consisting of high-quality silica of a uniform size and considerable cleanliness. It is deemed a by-product of the metal casting industry. Sand from foundries has been used in the molding industry for many years owing to its preferable engineering properties (American Foundry Society, n.d.). The literature indicates that foundry sand is re-used at foundries as many times as possible, and then it is disposed of as waste sand. At this stage, it is analyzed and assessed for incorporation in road construction works (Marzouk et al., 2014).

This material has reportedly been used in many aspects of road construction. Aside from utilizing it in sub-base layers, it has also been incorporated in embankments, in backfill material as flowable fills, and as fine aggregates in hot mix asphalt mixtures. As a consequence of the high clay content of foundry sand, it is not viable for use where high-water levels or poor drainage is anticipated. Additionally, foundry sand from nonferrous foundries is not recommended to be used in asphalt mixture applications due to an appreciable amount of heavy metallic substances (Marzouk et al., 2014). In Ontario, Canada, most molding sand is returned to foundries and factories for reuse, and its incorporation was limited to a maximum rate of 15% in asphalt mixtures because of some moisture-induced damage concerns. However, this material is no longer recommended to be used in the base/Sub-base applications, as some technical and environmental ramifications have been reported (LVM-JEGEL, 2009).

Several physical and mechanical tests are performed on foundry sand to examine its potential to be utilized in road construction works. This includes gradation, clay content by the methylene test, proctor test, and compressive strength tests for resilient modulus determination (Marzouk et al., 2014). The performance of molding sand varies, and on most occasions, it is used with the inclusion of other additives in order to enhance its mechanical properties. In France, a comprehensive study was executed to evaluate the use of foundry sand in the sub-base layer of a minor road. The rationale was that there was an abundance of foundry sand in the vicinity of a construction site near the road which would have been used in the proposed work. All material properties tests were executed along with a leaching assessment, which indicated the suitability of using the foundry sand in the sub-base layer. A cement binder was added to the sand to enhance some of the sand’s properties. Two road segments were constructed, i.e., with and without foundry sand, and they were monitored for a period of two years. The investigations revealed that at with 5.5% cement added, the results were promising, and the mechanical performance was considered satisfactory (Marzouk et al., 2014).

Reclaimed/recycled asphalt pavement (RAP), recycled concrete material (RCM), fly ash, and foundry sand were examined regarding their potential utilization in the base and sub-base layers of an asphalt pavement structure. A total of 17 mixtures were examined with a focus on the mechanical and hydraulic characteristics of the recycled material, and therefore attention was given to the characteristics of water retention and hydraulic conductivity, modulus of elasticity, friction angles, and cohesiveness. The use of foundry sand and fly ash was controlled at 15% due to the heavy metal content in these materials. The findings showed that the addition of a low rate of 5-15% of foundry sand did not change either the optimum moisture content or the maximum dry density of the mixtures. However, the resilient modulus results demonstrated that the foundry sand was not suitable for use since the obtained values were much lower than that of the referenced base mixtures. The referenced base mixtures had poor performance and demonstrated a poor shear resistance strength. The study suggested further investigation into the utilization of foundry sand in order to explore its potential for use in road construction base and sub-base layers (Kang et al., 2011).

In Italy, cold in-place recycling (CIPR) has been in use for over 12 years. It is a bound mixture of foam and cement along with reclaimed asphalt. The addition of various recycled waste materials in this process was executed in order to examine whether any performance, economic, or environmental gains could be derived. The investigated mixtures were compositions of various percentages of bottom ash and foundry sand, and physical and mechanical testing was conducted. Also, indirect tensile strength (ITS) tests were carried out to determine the tensile strength ratio (TSR) and the optimum binder content for the foamed mix. It was concluded that foundry sand is of high potential to be a surrogate for natural aggregates in CIPR mixtures, even though less improvement was gained in the strength, stiffness, and rutting resistance of the foamed asphalt mixture with foundry sand than with bottom ash (Pasetto & Baldo, 2012).

3.2.8 Coal Combustion Products and other oil production wastes

During the pulverization of coal at burning facilities or incinerators, a small amount of hardened molten residual substance called bottom ash accumulates at the bottom of the burning device. Its coarse texture is what characterizes this substance (Modarres & Ayar, 2016). It is the fine portion of the residual that is collected during the cooling process of fuel gas before exhausting the gas through the chamber of the processing plant (Athanasopoulou & Kollaros, 2015). The collected ash is either dumped into landfills in the dry approach, or it can be disposed of at certain locations called ponds in the wet process, where the dumped ash is known as pond-ash (Modarres & Ayar, 2016).

In the literature, coal waste is divided into two categories, coarse coal waste and fine coal waste. Coarse coal waste is produced during the mining process and fine coal waste is obtained when coal is washed at a washing plant. The waste resulting from coal-washing is considered a recyclable material. The more waste discarded in this process, the less the amount of ash produced (Modarres & Ayar, 2016). In India, average ash content from the coal-burning process is estimated to be in the range of 30-50% (Saride et al., 2015).

3.2.8.1 Fly-ash

One coal combustion product is called fly ash, which has been defined as a residual non-plastic substance (Athanasopoulou & Kollaros, 2015) produced by burning coal compounds for power generation (Tuncan et al., 2003; Aldhafeeri and Fall 2020). Coal contains a substantial amount of ash which when burned produced a high rate of fly ash which is disposed of as a slurry in landfills. Employing fly ash in road construction not only helps reduce the need for landfill use, but also lessens adverse environmental impacts (Ghazireh et al., 2012). The modest rate of heavy metals in fly ash (Pasetto & Baldo, 2012) and its ultra-tiny particles make using fly ash as a mineral filler in asphalt road construction a desirable practice (Athanasopoulou & Kollaros, 2015). Based on the chemical compounds in the ash, the American Association of State Highway and Transportation Officials (AASHTO) and the American Society for Testing and Materials (ASTM) place fly ash into two classes, i.e., Type C or Type E, where Type C has high calcium content and type E has reduced calcium content (Athanasopoulou & Kollaros, 2015).

The utilization of coal waste by-products and particularly the fly ash from coal combustion in road construction is one approach that has gained popularity based on researches outcomes and from practical perspectives (Modarres & Ayar, 2016). Numerous research studies have confirmed the potential improvements in the properties and quality of HMA by incorporating fly ash (Kim et al., 2012).

Cold in-place recycling (CIPR) has been studied and developed to reduce the economic and environmental burden caused by the inclusion of natural materials in road construction, thereby minimizing construction time and protecting the environment from the emissions from hot asphalt mixtures. Several studies have tried to improve the CIPR technique by investigating some additives and theirs impacts on rutting, fatigue, and strength. Fly ash could be a surrogate for lime and cement in the CIPR process, as fly ash can be used at rates of up to 8-12% as compared to rates of 1-3% of lime and cement in CIPR mixtures. Thus, significant savings are attainable. One research study demonstrated that an economic savings of nearly 25% could be attained when fly ash is used instead of hydrated lime in the CIPR process (Modarres & Ayar, 2016).

Type C fly ash was successfully used with RAP in a base-course layer in the US. The study focused on the developed strength and stiffness of the CIPR mixture by the addition of 10% Type C fly ash. The aged binder and the fines resulted from milling the underneath layer of old pavement that has low strength and low load bearing capacity. The testing involved CBR, resilient modulus tests on lab and field samples, and field observations by means of FWD. The CBR and M_r results of the lab and field samples were inconclusive. The field samples exhibited a CBR value of 66% less than the lab samples, whereas the resilient modulus was 25% higher for the lab samples than for the field samples. The FWD results demonstrated an sever decline in the strength over just one rainy season. Further investigation with regards to the moisture-induced damage of CIPR fly-ashes mixture was concluded (Inputs & Design, 2007).

Type C fly ash was added at 10, 20, 30, and 40% of the total weight of two mixtures of RAP. One mix was additive-free, and a rate of 20% of new aggregates was used with the other. The interaction in the RAP aggregates with new aggregates and fly ash was analyzed by means of X-ray powder diffraction and scanning electron microscope, and the results indicated that the addition of 40% of fly ash to the RAP-modified mixture with 20% new aggregates would be conceivably suitable for Sub-base and base-course materials in light traffic pavements (Saride et al., 2015).

Fly ash was examined in a base-course material. A cement-stabilized base was modified with fly ash at rates 0 - 75% with an increment of 25% of the total weight of the 12% cement, and the changes were assessed by the indirect tensile strength (ITS) test and the compressive strength test. Generally, the compressive strength decreases when the rate of fly ash in the mix increases, and in this study, the results were in compliance with this generic rule. It was concluded that fly ash should not exceed a rate of 25% of the 12% cement used in order to maintain the strength values within specifications (Dimter et al., 2011).

Fly ash has also been successfully used with weak expansive soil, i.e., clay, and proved to meet the requirements for using soil in road construction (Fishman & Member, 1993; Athanasopoulou & Kollaros, 2015). The incorporation rate of fly ash with various types of soft soils ranged from 8-12% so as to enhance the engineering properties of the investigated soil. The compressive strength was pronouncedly improved as the fly ash was added to three clayey soils at rates of 2 to 12% with 2% increment (Athanasopoulou & Kollaros, 2015).

The mitigation impact of fly ash on stripping resistance is an issue that has brought the attention of numerous researchers. A comparison of such impact with the effects of other common additives in asphalt paving such as hydrated lime was executed, and the influence of adding fly ash and conventional hydrated lime to different types of super-paved HMA was assessed. One mix was of high-quality materials with a modified PG 70-28, and the other was a conventional PG 64-22 binder with a low-quality aggregate. Moisture-induced damage was evaluated by using the AASHTO T-283 procedure and asphalt pavement analyzer approaches in addition to the boiling water test and pull-off test by means of the pneumatic adhesion tensile testing instrument PATTI. The results revealed that the influence of fly ash observed the low-quality aggregates. It was concluded that the impact of fly ash on stripping resembled that of hydrated lime; however, further investigation was suggested on a broader scale (Kim et al., 2012). Another study in Argentina on using fly ash and bottom ash with a local sandy-silty non-lateritic soil proposed for base layer. The incorporation of both types of ash brought a significant improvement to the soil properties, but the bottom ash surpassed the fly ash in terms of an instant increased resilient modulus of the material (Lopes et al., 2012).

3.2.8.2 Coal bottom ash

Coal bottom ash, which is sometimes called boiler slag, is another combustion product that results from burning coal. It is a finely graded type of material used as aggregates in road construction works either in the HMA itself or in the base-course material. It has the advantages of high resistance to skidding due to its angular glassy surface (El-Assaly & Ellis, 2001). Bottom ash can be obtained from the bottom of furnaces that burn coal for electric power. Ash obtained from a blast furnace is referred to as furnace bottom ash (Pasetto & Baldo, 2012). The literature states that fly ash is preferred over bottom ash in road construction (Lopes et al., 2012); however, the bottom ash surpassed fly ash in terms of an instant increase in the resilient modulus of the material (Lopes et al., 2012). The incineration of waste material has been reported to produce roughly 25% bottom ash from the total waste material. This ash is rich in organic substances similar to that of virgin aggregate material (Pasetto & Baldo, 2012). About 65% of the total bottom ash produced in Germany in 2003 was reported to have been used, while a much higher rate was reported in France in the same year, which hit 79% (Pasetto & Baldo, 2012). Mississauga and Brampton in Ontario, Canada annually produce 30,000 tons of bottom ash and 3,000 tons of fly ash and Only 10% of that is permitted for use in HMA applications (LVM-JEGEL, 2009).

However, due to the distinctive composition of bottom ash, it is still under investigation for incorporation in road construction, as there are still concerns about leaching and its impact on ground water. One technique proposed to prevent leaching through the inertisation process is the vetrification. That is where the mass of bottom ash is formed by cycles of great cooling and heat which technically known as a vetrification process. Thus, the results vetrified Ash, which is characterized by its glassy texture, is what is to be incorporated in asphalt pavement as aggregates (Bassani et al., 2009).

Owing to the glassy surface textured of bottom ash, it is considered a water-repellent material that absorbs a very low amount of water when submerged for a specific time. Thus, utilizing this material in asphalt mixtures would greatly reduce the amount of asphalt cement needed for desirable performance. From an economic perspective, this is a very sought-after quality providing that further analysis of the mechanical properties of the mixture is conducted. In Italy, investigating the potential substitution of fine aggregates in HMA with various amounts of bottom ash showed promising findings. The results demonstrated a significant improvement in the mechanical properties of HMA, including indirect tensile strength, creep resistance, and water absorption rates. The stiffness modulus increased with increases in ash rates and did not have an impact of the workability of the mixture. Further analysis needs to be undertaken to assess the resistance to stripping of bottom-ash-modified asphalt mixtures (Bassani et al., 2009).

Another study in Argentina on using fly ash and bottom ash with a local sandy-silty non-lateritic soil proposed for base layer. The incorporation of both types of ash brought a significant improvement to the soil properties, but the bottom ash surpassed the fly ash in terms of an instant increased resilient modulus of the material (Lopes et al., 2012).

3.2.8.3 Pond ash

Pond ash has been reported as having great potential to be used in a large amount in asphalt pavement construction. The mixture of bottom and fly ash is carried for disposal of as a slurry mixture of water and ash. This slurry is dumped in lagoons which are termed "ponds". A non-crystalline appearance characterizes pond ash, and it is also considered to be less cementitious. Studies have explored the enhancement of pond ash's properties by using other additives such as fiber and lime dust, which have proved to be useful in terms of strength, durability, and shrinkage and expansion resistance. In one research study (Sarkar & Dawson, 2016), ash was modified with fiber and lime at a rate of 0.2 - 0.5% of fiber and 2, 3, and 5% of lime for use as a sub-base material. Once the pond ash characteristics were identified from maximum dry density and optimum moisture content tests as well as triaxial tests, Plaxis finite element software was used to analyze the stress-strain behavior, and it was revealed that as the rate of fiber and lime increased, vertical compressive strain and horizontal tensile strain imposed on base-course and subgrade layers decreased. Thus, the stiffness of pavement increased with the addition of fiber and lime (Sarkar & Dawson, 2016). Again, it is necessary to further investigate the influence of using pond ash on the CBR values of sub-base materials.

3.2.8.4 Tar Sand

Oil or bituminous sand is a by-product of refining process of crude oil. It is a mixture of sand, clay, and water with a highly viscous asphalt cement in some regions in the world. The world's biggest source of oil sand is located in the western region of Canada, in Alberta (Roshani et al. 2017a,b, 2018). In the US, the size and compositions of the deposits and the availability of water for the extraction process are some obstacles hindering the exploitation of oil sand (Vrtis & Romero, 2015).

Oil sand has been in use in road construction for years. In Canada, back in 1913, oil sand was recorded as an ingredient in road construction works for the first time. Also, it has been reported that using oil sand was thought of by the industry at locations in the vicinity of oil sand deposits in spite of the fact that specification requirements had not been set. Recently, research studies have been trying to shed light on characterizing the properties of oil sands in order for it to be incorporated in asphalt pavement construction. Amongst the properties that have been given attention are the size distribution of the oil sand, binder content, and sensitivity to temperature changes (Vrtis & Romero, 2015).

A study in the US investigated oil sand deposit from Utah which contains about 13% of asphalt cement. This provided an encouraging finding about incorporating the oil sand into asphalt pavement, where there would be significant savings advantage from the ample bitumen content, which in turns helps preserve a substantial amount of virgin bitumen. Sieve analysis results of the Utah oil sand indicated a fine gradation that passed through the 2.36mm screen. Additionally, the influence of temperature was studied by simulating the short-term aging undergone by the binder, and the loss of mass was determined. However, when overheating was observed, a bending beam rheometer test was proposed to examine the changes in properties. The derived relationship between different compaction temperatures and the stiffness modulus of the oil sand indicated a tumble in the modulus when the temperature increased from 104ºC to 127ºC. Therefore, the overheating impacted the mechanical properties of the oil sand (Vrtis & Romero, 2015). Furthermore, a Hamburg wheel tracking device (HWTD) was utilized to check the rutting resistance. Samples of gap graded HMA with the addition of 31.5% to 35.5% of oil sand were prepared and subjected to a simulated wheel load imposed by moving traffic. A deformation of 10mm for 10000 cycles was the measuring criterion for failure, and the tested mixtures did not comply. Further experiments were done, and the results showed poor deformation resistance. Also, the mixing temperature of the oil-sand modified asphalt mixture was lower than that for conventional HMA, and was assessed at 104ºC (Vrtis & Romero, 2015).

It is believed that further comprehensive analysis of incorporating oil sand in asphalt mixture needs to be undertaken, and performance should be observed using available tests. Moisture sensitivity analysis for the modified mixture that also should be done. Whole-tire rubber and tread rubber were used with the oil sand from Utah in the USA, and the results indicated that the viscosity of the sand bitumen increased with the addition of crumb rubber (Holbert et al., 2016).

3.2.8.5 Petroleum-Contaminated Soil

A mixture of sand, clay, silt, and a crude product found in the surrounding area of a refinery as a result of oil spills or as a residue in the bottom of oil tanks is technically known as petroleum-contaminated soil. In the Persian Gulf region and particularly in the Sultanate of Oman, a substantial amount of this material accumulates per annum, and records state that it accounts for about 19,000 tons per year. An investigation was done to assess the potential inclusion of this material in HMA applications, and an asphalt concrete wearing course (ACWC) was prepared with different rates of 0%, 10%, and 20% of this sand. The results indicated that at a rate of 20% sand, a significant reduction would be achieved in the amount of virgin asphalt cement, in spite of the slight out of limit VMA rate. Other than that, the stability and flow values were within specifications but lower than the control mix with 0% sand (Copeland, 2011). It could be worth comprehensively investigating the potential of this waste material on a broader range of HMA types and materials.

3.2.8.6 Oil Shale Ash

The waste material from the process of producing oil shale is called oil shale ash. It is abundant is some regions around the world and has proven to be efficient in some applications. Modifying asphalt binder with oil shale ash is another approach for optimizing the rheological properties of a virgin binding agent, and pioneering research using super-pave testing was performed in Jordan to evaluate the addition of oil shale ash to a virgin asphalt binder. Oil shale ash was added at different rates of 0 - 20% at an increment of 5% and tested by means of dynamic shear rheometry and rotational viscometry. The process was aimed at examining the changes in the properties only at high temperature. Resistance to permanent deformation was expected to improve with an increased rate of ash, and shear modulus testing demonstrated this. However, the elastic behavior of the modified binder was stagnating, as indicated by the phase angle results. Hence, the investigated modifier did not have an impact on the low-temperature behavior of the virgin asphalt cement. The modification of other types of binder by the inclusion of the oil shale ash was suggested for further analysis to evaluate the performance of binders at lower temperatures (Saltan & Findik, 2008).

3.2.9 Pumice Stones (Waste)

Pumice is a volcanic stone produced during the eruption of a volcano that is composed primarily of variable amounts of silica and alumina. In its fine powdered state, pumice is technically called pumicite. It is characterized by a foamy and glassy texture resulting from its crystal structure. The foamy nature of this volcanic stones is a result of the cooling and depressurization that occur simultaneously. Pumice has a high absorption rate due to the high percentage of voids caused by the foamy texture of the stones. Pumice has been used successfully in several fields such as fields construction, chemical, agriculture, dentistry, etc.

Larger dimensions of pumice are typically used in these fields but smaller are often not used as much as larger ones (Saltan and Findik, 2008). However, research on incorporating pumice aggregates in road construction is very limited. Notwithstanding the lack of research, pumice has been reported to be an efficient filling material in HMA (Saltan and Findik, 2008; Saltan, Kavlak and Ertem, 2011).

In Turkey, few studies were conducted to assess the use of locally available pumice in road construction. It was assessed against conventional mineral fillers in HMA, and the results showed an increase in mixture stability and a decrease in asphalt cement when pumice was added. Another study determined that pumice cannot be utilized on its own as an aggregate in a sub-base layer owing to the noncompliance with Sub-base specification requirements. Pumice of 3mm or less in size was found to be non-plastic, and the CBR strength was slightly lower than requirements call for. The authors concluded that pumice can be mixed with another material with high clay content in order to reduce the plasticity and to consider pumice as a stabilizing agent (Saltan & Findik, 2008).

The use of pumice as a stabilizing additive for high-clay-content soil was examined in another study in Turkey. With the very low CBR value and very high plasticity index of the clayey soil, pumice of less than 3mm in size was added at rates of 10%, 20%, 30%, and 40%, and it was revealed that at 40% pumice, the consistency and CBR value were noticeably enhanced, and the mixture satisfied the subgrade requirement for use in road construction. Also, a significant reduction in water content was gained, and consequently the maximum dry density was reduced (Saltan et al., 2011). The resilient modulus assessed from repeated loading tests showed significant improvement as well. It is believed that incorporating pumice into road work would have serious economic advantages if properly utilized in the construction material.

3.2.10 Brake Shoe Fines

The fine dust produced during the manufacturing of automobile brake shoes has also been examined as a filling material in HMA and in the road construction field. An automobile company in China produced a huge amount of this powdered material and supported an attempt to evaluate its potential inclusion in an asphalt mixture. The Marshall Design method was adopted to design a mastic hot mix asphalt with PG 64-22 asphalt binder and different rates of 0%, 15%, 30%, and 45% of brake shoes fines (BSF). The rates used were in proportion to the volume of the original binder. Subsequently, BSF was used at rates of 0%, 2.5%, 5.0%, and

7.5% by total weight of aggregates, and a suite of several laboratory tests was performed on the produced mixtures. Based on an evaluation of the evolutionary properties, dynamic shear rheometry indicated a higher shear modulus at higher rates. In addition, as temperature varied between 5ºC, 20ºC, and 40ºC, the results from a simple performance tester (SPT) demonstrated a rise in the dynamic modulus at the high temperature of 40ºC and a decline at the low temperature of 5ºC. At a rate of only 2.5%, the tensile strength was improved, as indicated by indirect tensile strength (ITS) testing. A conclusion was made on the usage of 2.5% of powdered brake shoe material as a filling material in HMA (Q. Dong & Huang, 2007).

3.2.11 Crushed Clay Masonry and Ceramic Waste Material (Ceramic Whiteware)

Clay masonry waste is collected from demolition and construction activities and could be used to replace the aggregates required for road construction works. Together with recycled concrete aggregates, crushed clay masonry is deemed superior for substituting unbound material in pavement structures. An evaluation of some engineering properties of combined recycled crushed concrete masonry (CCM) and recycled concrete aggregates (RCA) demonstrated a potential utilization of the blend in Sub-base layers for road construction works. (Cameron et al., 2012).

Ceramic waste material (CWM) is obtained from housing fixtures such as those in bathrooms, e.g., toilets and sinks, and it can be reused in new ceramic material. Once this waste is crushed to minimize its size, it can be incorporated in base and sub-base road works. The province of Ontario permits the use of up to 15% of this waste in sub-base granular B materials in combination with glass.

A rate of 15% CWM was used with a Type A granular base material in a small residential road stretch in Toronto. Field trials and observation of a short road section demonstrated satisfying results, and there was compliance with the specification requirements. MTO requirements permit the inclusion of 15% maximum of ceramic waste with granular A type materials in a base-course (LVM-JEGEL, 2009).

A research program in Australia examined two mixtures of clay masonry with recycled concrete aggregates. The first material was a mixture of 20% clay masonry and 80% recycled concrete aggregates, and the other contained 10%, 20%, and 30% clay masonry. Gradation analyses of all the mixtures complied with specification requirements. The abrasion value was within the acceptable limits. Consistency limits also validated as well as the lime limit test. The angles of friction and the cohesion values were determined through unconsolidated compression tests, and results used to correlate with the resistance to permanent deformation. The modulus of elasticity and resilient modulus of the mixtures were obtained by Repeated Load Triaxial Testing. The results indicated that as the clay masonry amount increased in the mixture, there was a decrease in the resilient modulus. This study concluded that a blend of crushed clay masonry and recycled concrete aggregates could possibly be used in a Sub-base layer (Cameron et al., 2012).

3.2.12 Recycled (Reclaimed) Concrete Aggregate (RCA)

Reclaimed concrete aggregate (RCA) is a by-product of crushing concrete rubble in reconstruction and/or renovation activities (Paranavithana & Mohajerani, 2006), and can be acquired through the reconstruction of concrete roads and bridges, for instance. At large, construction and demolition waste constitute a significant portion of concrete masses (Soils et al., 2009). Reclaimed concrete aggregate is composed of aggregates bonded together with a cementitious paste (El-Assaly & Ellis, 2001). However, RCA on most occasions has a high water absorption rate and low density owing to the porosity of the cement paste coating the aggregates (Paranavithana & Mohajerani, 2006). The potential use of RCA would relieve not only the problem of disposing of demolition waste, but also the concern over the depletion of natural materials for road construction works. RCA inclusion varies, from aggregate substitution in HMA applications for base-courses to riprap materials for slope protection (El-Assaly & Ellis, 2001). Using RCA in base and Sub-base applications in road construction is permitted by the MTO in Canada. (LVM-JEGEL, 2009). The implications of using RCA in asphalt mixtures vary considerably. Density and water absorption increase with the deployment of RCA in these mixes, and in HMA applications, the high porosity of recycled concrete aggregates causes high binder absorption and leaving tiny film on the surface of the aggregates. Hence, the mixes become more prone to damage by moisture and are less durable, i.e., they have a lower strength than conventional mixtures composed of fresh aggregates. Volumetric analysis showed that VMA and VFB are high for RCA mixes, unlike the VTM which increased with the utilization of RCA (Paranavithana & Mohajerani, 2006).

A study in Australia compared HMA with new aggregates with another HMA composed of a coarse fraction of RCA mixed with a new fine portion to complete the aggregates skeleton of the mixture. The results were in support of the forgoing influences and suggested further comprehensive investigation (Paranavithana & Mohajerani, 2006).

RCA was used in a comparison study in Pakistan with RAP material. A Marshall Mix design was performed on a control mixture and another two mixtures modified with RCA and RAP at different percentages of 10%, 20%, and 30%. The mechanical tests results revealed that the RCA had a high-water absorption rate compared to the RAP, which in turn had a higher absorption rate than the new aggregate materials. Also, when assessed by ACV and AIV tests, RCA was found to be the least durable material, with RAP coming in second, and the new aggregates were the most durable. The VTM was found to increase with increases in the RCA rate (Rafi et al., 2014).

While the stability and flow characteristics dropped with increases in RCA and RAP, rutting depth measurements were claimed to be a better indication of strength and the resistance reduced as RCA percentage increase owing to the poor quality aggregates reflected in the results of the mechanical tests (Rafi et al., 2014).

Employing RCA in a base material can result in efficient performance when mixed with RAP (reclaimed asphalt pavement). In Spain, newly constructed pavement was assessed for the impacts of incorporating RCA, RAP, and ceramic waste at rates of 75%, 20%, and 5% respectively in a base layer against quarry aggregates in another segment. Proctor, consistency, and abrasion tests were conducted on the material and demonstrated a level of quality comparable to that of the quarry aggregates. Falling weight deflectometry was performed and revealed a significant improvement in strength (Herrador et al., 2012).

A comprehensive 3-year research in Czech Republic was executed to evaluate the impact of employing recycled concrete and reclaimed asphalt pavement on the performance of a bound base layer in asphalt pavement. The laboratory experiments entailed gradation analysis, ITS, compressive strength tests, and moisture-induced damage testing. The modulus of elasticity was evaluated by conducting cyclic trial testing on a full-scale pavement section. Then, a section of asphalt pavement was constructed using the most favorable composition of the material from a 6-month evaluation of a full-scale laboratory model. Then, a comparison was made between the modulus obtained from the FWD results and the modulus obtained in the laboratory (Asi et al., 2007).

Utilizing crushed concrete rubble without additional treatment in Sub-base layers is a common practice in the Netherlands (Hendriks & Janssen, 2001). A comprehensive study was carried out on blends of RCA and RAP contents at different rates of 15%, 30%, and 50%, and the properties were evaluated based on laboratory and field tests against a 100% RCA mixture and a 100% RAP mixture. The water absorption rate was higher for the RAP than for the RCA mixture. Field trials segment with 100% RAP Sub-base failed to meet minimum requirements for density, compaction degree, resilient modulus, and CBR values derived from Clegg Hammer tests (Arulrajah et al., 2014).

RCA has been also favored over natural aggregates in CIPR mixtures. A study in Spain concluded that RCA with a maximum suggested binder content of 4% had a performance comparable to CIPR mixes according to results obtained in the laboratory. The resilient modulus obtained from the indirect tensile stiffness modulus indicated an improvement in the CIPR mixture over the natural aggregate mixture. The porosity of RCA aggregates was revealed in the amount of water lost over a longer curing period (6 to 12 months) compared to that of the natural aggregate mixture (Gómez-Meijideet al., 2016). Recycled concrete aggregates have also been mixed with clay masonry and proved to have a quality superior to that of substitute unbound material in a pavement structure (Cameron et al., 2012).

Airport pavement incorporating recycled concrete aggregates has also reflected good performance. A taxiway at an Italian airport was constructed utilizing recycled concrete slabs, and the formation layer and base layer were modified with recycled aggregates, and promising results were obtained from field and laboratory tests in terms of strength and density (Giustozzi et al., 2012).

In the construction and demolition waste, concrete rubble is frequently mixed with waste bricks, and possesses significant potential for consideration in road constructions. Waste bricks and RCA were considered in a research study in China, and the recycled material was stabilized with a cement-stabilizing agent and tested for UCS. The results revealed the potential use of this waste in sub-base and base layers, as the material was less sensitive to moisture damage. Despite the fact that the bricks were of low strength, evaluating the loss of strength after 7 and 30 days of conditioning revealed an acceptable range of loss as per specification requirements. The authors made the conclusion that the recycled brick and concrete waste would be of great benefit if used in Sub-base and base layer applications (Soils et al., 2009).

According to the Ministry of natural resources, of Ontario, Canada, recycled concrete commonly has some impurities that should be taken into account when considering this option for use in road construction. A proper assessment recycled concrete needs to be conducted in order to fully comprehend its mechanical properties (LVM-JEGEL, 2009).

3.2.13 Coffee Grounds and Cigarette Butts

Recently, rigorous investigation was done at the University of Swinburne in Australia that indicated the potential utilization of coffee grounds from cafés in the pavement subgrade layers, and extensive research is ongoing to explore using such an organic component in road structures. The results of preliminary experiments on coffee grounds demonstrated a promising outcome, and that substantial amounts of this normally discarded material could be incorporated in road construction works in the foreseeable future (Roads & Civil Works Australia, n.d.). Interestingly, research on utilizing cigarette butts with asphalt binders is ongoing, and promising results have been reported (Rahman et al., 2020).

3.2.14 Plastic Materials

Investigation into the inclusion of plastic polymers as an additive in road construction is not new, and has been ongoing for several years (Salem, 2014; Karmakar and Roy, 2016). The literature includes numerous studies conducted on the potential utilization of plastic waste in different forms in road applications, and the conclusions have been encouraging. However, virtually almost all the research studies focused on one type of pavement structure, which is flexible asphalt pavement. Moreover, other than assessing the stabilization of road embankments using recycled plastic pins (RPPs) (Loehr *et al.*, 2000), the attention has been focused on either subgrade soils or hot mix asphalt surfacing (Vasudevan et al., 2012; Sarang et al., 2014; Salem, 2014; Karmakar and Roy, 2016) (Figure 3.2). Limited studies have investigated the alteration of the physical properties of asphalt binders by the addition of certain rates of different types of polymers (Karmakar and Roy, 2016; Maniruzzaman et al., 2016). Some road accessories such as guardrails, noise barriers, and traffic calming tools have been examined with the addition of plastic waste materials (El-Assaly & Ellis, 2001).

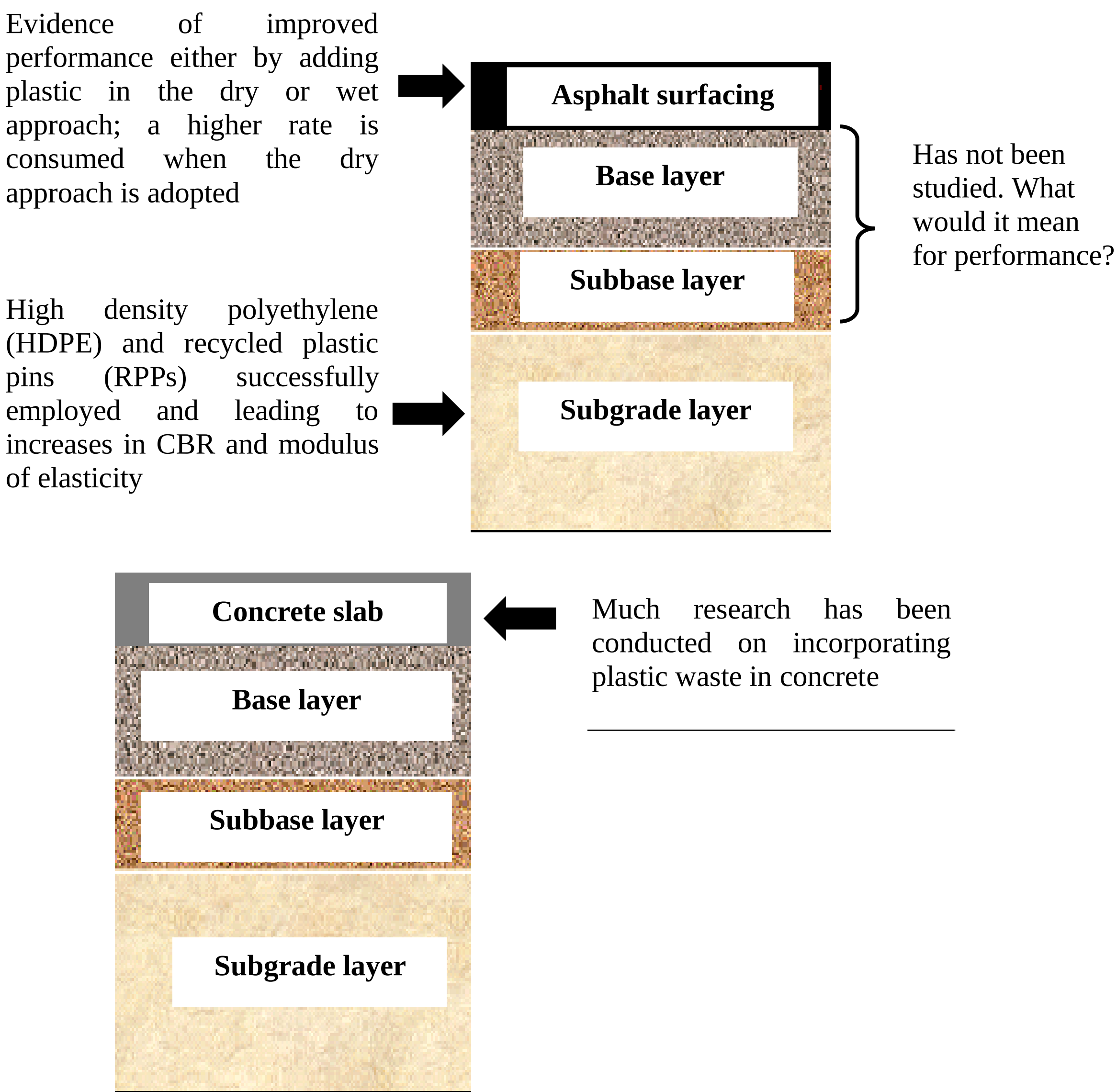

Figure 3.2 Illustration of where plastic has been studies within pavement

3.2.14.1 Applications of Plastic Materials in Road Construction

In road subgrade applications, significant improvements in the elasticity and resistance to deformation of subgrade soil are gained when high density polyethylene (HDPE) (Choudhary et al., 2010) and plastic waste chips (Dhatrak & Konmare, 2016) were incorporated with the soil. The utilization of such resins was considered to enhance overall strength of pavement as a whole structure, as it augments the load-bearing capacity of pavement structures (Vasudevan et al., 2012). Moreover, plastic waste has brought improvements to the mechanical properties and performance of hot mix asphalt mixtures. Marshall stability, voids in total mix and voids in Mineral aggregates improved the density of HMA decreased (Maniruzzaman et al., 2016). Plastics are rich in polymeric substances that play a role in improving the coating and binding properties of asphalt mixtures. Polymers interact with virgin binders, increasing its affinity to the asphalt binder and thus enhancing its rheological properties and minimizing water-induced damage to the mixtures (Salem, 2014; Chavan, 2013). When added to virgin binders, the higher the rate of added plastic to the mixture, the more viscous the asphalt cement becomes, and subsequently the less the penetration value. Thus, the softening point becomes higher and ultimately the temperature susceptibility of bitumen becomes lower (Angelone et al., 2015).

Plastic becomes a molten compound at a low temperature due to its low softening point, and therefore the likelihood of reducing gas emissions becomes significant when used in HMA applications (Vasudevan et al., 2012). It is believed that the promising findings of several research studies on utilizing plastic waste in various asphalt mixture applications would lead to less damage to the environment and reduced greenhouse gas emissions. Similarly, the quality of drainage systems would be improved by reducing the amount of waste buried in the ground (Sojobi et al., 2016). Various factors need to be taken into account when considering plastic as an additive to asphalt cement or to an aggregate's skeleton in HMA, which include environmental effects, cost effects, and impact on the performance of the asphalt mixture (Tuncan et al., 2003).

Polymers have been assessed as additives in HMA applications to reduce the cost of anti-stripping agents. Water is undoubtedly the worst enemy to pavement structures, and despite all precautions, such as good drainage systems and introducing a capping layer in pavement structures aimed at reducing the negative impacts of moisture, numerous surface stresses and deterioration effects occur as a result of water penetration in pavement systems. The literature shows that when plastic is heated to a semi-liquid state and mixed with the aggregates and asphalt binder, the physical composition of the binder is slightly altered, and the bonding characteristic is enhanced. Hot polymers tend to develop a thin coating film over aggregate particles, similar to bitumen film but stronger, and a more moisture-resistant mixture is produced. This behavior promotes plastic waste as a valuable constituent of aggregate blends in asphalt mixtures (Vasudevan et al., 2012).

In another study, plastic pellets were used as an aggregate substitute material for 2.36 - 4.75mm aggregate sizes. A rate of 30% of LDPE was used and the Marshall mix design method was adopted in the study, and improvements in the mixture stability and retained tensile strength were reported (Huang et al., 2007). There has been less attention given to HDPE compared with LDPE in terms of consideration for pavement applications. HDPE was reported to be a better substance and cheaper than using geo-grids in subgrade stabilization applications (Choudhary et al., 2010).

A research study in India examined adding plastic waste strips of different length to width ratios to a locally available sand material. Different mixes with rates of 0%, 0.25%, 0.50%, 1.0%, 2.0%, and 4.0% plastic were tested for CBR and modulus of elasticity (secant modulus), and at a 0.25% plastic waste content, a significant improvement in CBR was recorded, i.e., 70% higher than that of untreated sand which was attributed to the inclusion of the plastic waste. The secant modulus, or modulus of elasticity, for a sample with 4% plastic waste was nearly 3 times higher than that of non-stabilized samples. The significant improvement of the CBR and secant modulus at 4% was ascribed to the interaction between the strips and the sand, which increased the friction (Choudhary et al., 2010). Another investigation was conducted in India on reinforcing fly ash and waste recycled product (WRP) with strips of HDPE plastic at rates of 0%, 0.25%, 0.5%, 1.0%, 2.0%, and 4.0% of the total weight of the ash and WRP respectively. The CBR was calculated for samples with and without plastic strips, and the results showed greater improvements in the CBR and modulus of elasticity for the reinforced fly ash samples than for the reinforced waste recycled product samples, though both indicated enhancements in the CBR and modulus of elasticity values. The highest CBRs and moduli of elasticity were achieved at plastic strip rate of 4% and an aspect ratio of 3.0 (Choudhary et al., 2014).

Another study investigated the potential influence of HDPE on the performance of a Marshall asphalt mixture by mixing bitumen with different percentages of HDPE in the wet approach. The mixing temperatures varied from 145ºC, 155ºC, and 165ºC, and mixing time reduced from 30minutes to 15 minutes to 5minutes. Asphalt cement AC-20 was mixed with varying rates of HDPE of 4, 6, and 8% of the weight of asphalt cement ratio, and the results indicated an improvement in resistance to permanent deformation, a high stability value, and a low flow value for the mixtures. Also, a high resilient modulus and enhanced tensile strength ratio was attained as the modifier positively altered the physical properties of the bitumen, which was expected. An optimum HDPE rate was about 5% of higher acceptable performance and reduced cost of an asphalt mixture was reported (Angelone et al., 2015).

The literature states that the approaches adopted for utilizing plastic waste in pavement resemble those for incorporating crumbed rubber, as plastic can either be shredded or cut into tiny pieces prior to extrusion to small pellets. In the wet approach, plastic resins are added as modifiers to virgin asphalt binder at a rate of ¼ to ½% of the total weight of the HMA mixture (El-Assaly & Ellis, 2001). Alternatively, plastic chips or pellets might be added in the dry approach as an ingredient in the aggregate structure in hot mix asphalt (Huang et al., 2007). Mixing time and temperature in the wet approach are associated with the type of polymer and virgin asphalt cement. Generally, the wet approach outstrips the dry one in terms of ease of mixing and applied shear forces to ensure dispersal of the chips in the bitumen (Kalantar et al., 2012).

In India, a comprehensive study looked at using plastic waste material in a total of six plastic-modified paved roads with various environmental conditions across the country for a service life of 10 years. Plastics were chopped into 2.5-4.36mm sizes and mixed with preheated aggregates at different rates. A rate as high as 25% was used to evaluate the plastics' affinity to asphalt cement, and a rate as low as 1-5% was used to investigate the toughness of aggregates coated with the plastic for hot mix asphalt. The mechanical properties of the coated aggregates were tested using conventional tests such as water absorption and toughness tests. The bitumen used was also assessed by observing the amount of bitumen that was stripped from the coated aggregates after immersion in water at a specified temperature for a specified period. The results indicated that as the percentage of plastics incorporated in the mix was increased, the compressive and bending strengths of the mix increased, and resistance to moisture damage was enhanced to a great extent. Improved binding strength developed between the bitumen and plastic-coated aggregates, as reflected in the high stability and strength values. The long-term performance was observed and assessed between 2007 and 2008 by measuring the incurred deflection, surface irregularities and texture, field density, resistance to skidding, and surface defects. While conventional HMA pavement had an average Benkelman beam value of 1.55mm, the modified-plastic HMA pavement had an average value of 0.85mm, which complies with the required specification range of ½ to 1mm. When the IRI was measured and compared with that of the conventional pavement section, the index was more acceptable, and the field density was proved to decline with time to a certain average value. The measured field density for all 6 monitored pavements showed a lower drop rate than for the non-modified pavement. The modified HMA pavement had good surface conditions, as indicated by the skid resistance value and texture depth measurements (Vasudevan et al., 2012).

Another study, at Integral University in Lucknow, India, was done in order to check on any improvements gained by adding plastic to aggregates structure in asphalt pavement in the dry method. The physical and rheological properties of the bitumen were shown to be improved by the polymer modification, and resistance to stripping and strength were also increased. Characterization of the coated aggregates' properties as water-loving or -hating was conducted by the absorption test. Water absorption declined significantly due to the film formed around the aggregates, and thus the durability increased. The observations made regarding field performance demonstrated that no pavement defects such as permanent deformation, fatigue cracking, potholes, degradation, or raveling were observed, and the roads performed efficiently over the year of service (Salem, 2014).

In Argentina, research was conducted to evaluate the impact of using plastic waste of different types, i.e., flakes or pellets of polyethylene (PE) from silo bags and polypropylene (PP) chips on the properties of Marshal HMA at rates of 2%, 4%, and 6% in comparison with two control HMA mixtures as references. The flaked PE was first added as an additive by total weight, and then the pelleted PE was added as a percentage of the 4.8-2.4mm aggregate size. However, the PP type was to replace the same quantity of aggregate size of 9.5-0.075mm in the mixture. The results showed that the stability of the mixtures increased with the increases in the percentages of the plastic added when compared to that of the references. However, the PP modified mixture demonstrated a decline in stability with increases in the amount of plastic added. Conducting an indirect tensile strength test (ITS) test was useful in determining the fracture energy, which is used as a fatigue resistance and low temperature cracking criterion in Argentina. The FE value was again higher than the control mixture value for both PE types, but it was less than that of the reference mixture for the modified PP type. Moisture damage tests as per the EN-standard revealed encouraging results, and requirements were met by all the mixtures. The dynamic moduli obtained from the master curve results indicated more resistance to low-temperature cracking and more resistance to permanent deformation in the modified mixtures. For all the plastic-modified mixtures, the E* value was low at low temperatures and high at higher ones, which is favorable in resisting temperature-related cracking and rutting. This was reflected by the wheel-tracking tests conducted by a Hamburg machine, where the rutting measured was satisfactory. This research concluded that plastic waste can be incorporated into asphalt mixtures and provides promising rutting and fatigue resistance. For the specific mixtures and materials, 2% of flaked PE was demonstrated to provide the optimum performance (Angelone et al., 2015).

A recent study was executed in Nigeria to evaluate the use of a different type of plastic waste, called polyethylene terephthalate, in asphalt pavement. The dry approach to the inclusion of plastic was adopted with an asphalt cement of 60/70PEN. The AC 60/70 was modified with a conventional polymer to evaluate changes in the asphalt mixture's properties when PET is used. Plastic at different rates was mixed with aggregates to produce plastic-coated aggregates that were later mixed with the 60/70 asphalt cement. Also, various percentages of polymer were added to the AC 60/70 to produce hot mix asphalt. The polymer-modified mixture and the plastic-coat aggregate-modified mixture were assessed by using conventional binder tests and Marshall design analysis. It was concluded that both methods provided increases in stability and air voids with increases in the waste rate; however, the plastic-coated aggregate mixtures were preferable, as they allowed for a better abundant incorporation of PET than the polymer-modified mixture (Sojobi et al., 2016).

Based on the existing conditions and market prices during the time of the study in India, plastic modified pavements could save an exceptional saving amount of money due to the reduced maintenance cost for a seven-year defect-free surface. Also, with a 3.75m lane width, a single ton of bitumen would be saved for each 1 km of road construction, and a 10% savings in the amount of virgin binder would be attained (Vasudevan et al., 2012). In a study on the benefits of using waste plastic instead of virgin binder/polymer in an asphalt mixture, the authors of another study concluded that there was a performance enhancement when utilizing waste polymer to modify the rheological properties of the asphalt binder and produced asphalt mixture. Cost savings benefits were also suggested (Kalantar et al., 2012). Tangible enhancements in the properties when utilizing plastic waste in asphalt pavement brings significant reductions in the use of asphalt cement, hence bringing construction and maintenance cost savings (Sojobi et al., 2016).

3.2.15 Plastic Waste and the Canadian Road Industry

Most recently, the city of Vancouver in Canada has led the Canadian road construction industry in producing what is called plastic-derived wax mixtures in an attempt to develop its own green roads technique. The mixture is produced by refining recyclable plastic wastes into a wax substance. The plastic-derived mix has been categorized as an emission reduction mixture by municipal engineers. In 2012, this new mixture was used on roads after comprehensive testing on small sections in the city in order to assess the quality of the mixture. The advantages included a reduced mixing temperature, which consequently minimized the carbon footprint by an amount of 300 tonnes, less danger to the environment, and reduced gas emissions, which led to a slight increase in cost. Recycling pioneers in the country also doubts that this technology might become a burden to the industry and the current Foamed asphalt mixture, that is thought to be another alternative to the plastic derived warm mix, has been tried and performed satisfactorily with less fuel consumption and less emissions. The concern with the plastic-derived mixture is over its the long-term performance, which remains questionable until a comprehensive evaluation is carried out. The environmental impact of the emissions from the warm mix asphalt is still in questions, and the city of Vancouver is emphasizing that a close assessment is needed before considering the technology (D. P. Ball, 2012).

Aside from road pavement structures, plastic waste has also been examined and practiced in other road construction application such as embankments. Recycled plastic pins (RPPs) derived from plastic waste has been utilized in stabilizing embankments. RPPs are characterized by their favorable modulus of elasticity and high creep resistance as well as by the improved strength of shallow embankment during heavy rains. Dire improvement gained from this utilization in the US. A real-life embankment failure occurred due to shrinking expansion of the soil during wetting cycles which further pose higher and intensified pressure in voids of soil. Several field techniques have gained recognition in this regard, but RPPs are considered the least expensive. A field experimental study showcased the advantages of using RPPS in stabilizing high clay content soil slopes. The underlying layers of montmorillonite, under periodic wetting cycles, caused cracks on the pavement shoulder. Recycled plastic pins were evaluated based on a comparison between a stabilized and non-stabilized section of the slope under investigation, and monitoring was done through inclinometers, topographical investigation, and strain gauges for the treated and the reference sections. The strain gauge measurements indicated stagnated movement at the stabilized section, while movement at the reference section was significant. This was also confirmed by the topographical survey. The authors concluded that using RPPs would surpass other stabilization techniques due to its cost reductions and the advantages of utilizing a potential burden on the environment in embankments.

3.3 Summary of recycled materials in Roads

The following table, Table 3-1, summarizes the main findings of this literature review, underlines the key benefits and disadvantages of using various recycled materials in road construction as well as shows the knowledge gap.

Table 3-1 Summary of key benefits and disadvantages of various recycled materials

Recycled material	**Pavement layer**					**Remarks**
	S.G	**S.B**	**B.C**	**HMA**	**Concrete Surface**	
Glass		X	X	X		• Found effective in improving mechanical properties of HMA when ground to a fine size as ingredient in mix. • Used with other additives, like Lime, to decrease stripping phenomenon • as the rate of fine recycle glass increases, handling becomes harder. • a cost-effective substitute in S.B materials • skid resistance of HMA improved. • Stripping issue in asphalt pavement applications. • efficient in increasing strength of soft alkali clay soil. • In general, there is little experience with its implementation because of the lack of understanding the behavioural performance. • Coarse recycled crushed glass is not recommended due to the potential change in gradation during mixing and compaction.
Crumbed Rubber				X		• Added in two distinguished way, dry and wet approach with dry finer that that in wet approach. • Used with SMA mixes to reduce drain down phenomenon. • Used with HMA to improve skidding resistance • Results in high shear strength of HMA. • Some other benefits include enhancing rutting and fatigue resistance • improving resistance to of HMA to stripping • increase viscosity and makes AC stiffer and of less susceptibility to temperature changes.

						• The influence of the rate of rubber used is more pronounced at high temperatures than at lower temperatures. • As its rate of inclusion increases, creep resistance of HMA improved.
RAP				X		• CIPR low Mr, low CBR, low compressive strength • Hugh rate cause premature failure and durability issue in HMA
RAS				X		• High rate results in low tensile strength and low thermal cracking resistance • Limited researches in Canada • Improve bitumen properties • Reduce % if AC in HMA • HMA properties improved • From performance point of view, max. rate limited to ≤5%, some US states allow up to 7.5% • Ontario allow limited rate due to inconsistency. • With RAP, only 3# is recommended for concerns over aging and brittleness of HMA
RCA		X	X	X		• on most occasions, RCA has a high-water absorption rate and low density owing to the porosity of the cement paste coating the aggregates. • high porosity of RCA causes high AC absorption and leaving tiny film on the surface of the aggregates in HMA application. • Significantly improve strength of B.S materials. • Used in conjunctions with RAP in B.C applications. • Good performance reported in Airport pavement in terms of strength and density.

Mine waste		X	X	X		• widely used in base-courses and in HMA as a frictional additive. • Low absorption rate suggested potential use in S.B and B.C applications • Marble quarry dust considered as a substitute material for conventional filler in HMA mixtures lead to improving Stability, indirect tensile strength, and fatigue resistance
Coal combustion products Coal tar / tar sand				X		• Rich of virgin AC • poor deformation resistance OF HMA • Whole-tire rubber and tread rubber were used with the oil sand in the USA and increased the viscosity of the sand bitumen increased with the addition of crumb rubber
Fly ash	X	X	X	X		• Improve stabilization • Successfully used with RAP in a base-course layer in the US • Used as a mineral filler in HMA as a desirable practice • Incomprehensive behaviour with regards to the moisture-induced damage of CIPR fly-ashes mixture. • Recommended for S.B & B.C materials in light traffic roads • Has been successfully used with weak expansive soil • Improve compressive strength of soil
Pond ash	X	X	X	X		• Improve bonding • large amount in asphalt pavement construction • As rate of inclusion increases, vertical compressive strain and horizontal tensile strain imposed on base-course and subgrade layers decreased. • Require further investigation the influence on CBR values of S.B and B.C. • The incorporation of both types of ash brought a significant improvement to the soil properties

Petroleum-Contaminated Soil				X		• Significant reduction in the amount of virgin AC, in spite of the slight out of limit VMA rate
Oil shale ash				X		• Resistance to permanent deformation improved with an increased rate of ash. • Does not have an impact on the low temperature behavior of the virgin asphalt cement.
Pumice waste				X		• reported to be an efficient filling material in HMA • Increase stability of HMA • Decrease AC content in HMA
Brake shoe fines				X		• The usage of 2.5% of it as a filling material in HMA is recommended to improve resistance to rutting and low temperature cracking.
Crushed Clay Masonry and Ceramic Waste						• Ontario permits the use of up to 15% in sub-base granular B materials in combination with glass. • MTO permit the inclusion of 15% maximum of ceramic waste with granular A type materials in a base course • Blended with RCA, It can be used in the S.B layer
Coffee ground and cigarette butts				X		• Promising results in HMA applications • Conclusive results have not been published yet.
Plastic	X			X	X	• Improve CBR and secant modulus of S.G • Used in soil stabilization • Control drain down of SMA • Dry mixing allows more plastic in HMA • Coating and moisture damage improved • Improved elasticity and resistance to deformation when random HDPE used with S.G

						• Mix stability, VMA & VTM improved and density of HMA decreased • Reduce penetration of AC, increase softening point and lower the temperature susceptibility of vergin AC • less damage to the environment and reduced greenhouse gas emissions.

Where: *S.G is the subgrade layer*
S.B is the subbase layer
B.C is the base course layer
HMA is the hot mix asphalt layer
SMA is the stone mastic asphalt mixture
AC is the asphalt cement binder
CIPR is the cold in place recycling
RCA is the recycled concrete aggregates
VMA is the voids in mineral aggregates in hot mix asphalt

3.4 Research Knowledge Gap

The following conclusions can be drawn from the literature review presented above:

- The plastic waste is one the least investigated solid waste materials regarding their recycling into road constructions materials.

- Almost all the studies on recycling plastic materials into construction materials focus on the use of plastic waste in asphalt surface layer. There is a paucity of studies on the use of plastic waste in concrete surface layer. There is a need to address this research gap.

- A limited number of investigations on the use of plastic aggregates in road subgrade have been conducted. So, the geotechnical performance of road subgrade soils incorporating plastic waste aggregates of different type, shape and content is not understood. There is a need to address this knowledge deficiency.

- No studies on the use of plastic waste aggregates in road sub-base and base layers have been conducted. This lack of knowledge needs to be addressed.

3.5 Conclusions

Numerous alternative materials are being considered for or are already in use in the road-building industry, and there is evidence that some of the afore-mentioned materials are preferable over others in terms of improving the performance of roads. In this chapter, the materials covered included reclaimed asphalt pavement, recycled asphalt shingles, scrap tires, and crushed glass, to name a few. However, plastic waste materials were also chosen for the specific objectives of this study, and the literature on research in these directions was discussed.

From the discussion, it was clear to the author that there is a lack of knowledge in the area of utilizing recycled plastic waste aggregates of different types and shape into the road structure, particularly within the underneath layers i.e., base, sub-base and subgrade layers. Thus, the implication of adding recycled plastic waste to a pavement subgrade is investigated and discussed in the technical paper one followed by the investigation process on the sub-base and base-course in the technical paper number two. It then completed the study with examining the top surfacing concrete layer that was included in article number three.

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4 Chapter 4: Geotechnical characteristics of plastic waste materials in pavement structures applications

4.1 Introduction

4.1.1 Background

In order to achieve the objective of the study, certain road constructions materials were selected. These materials are being used in road construction in building the pavement structure. They include the subgrade soil, the sub-base and base-course materials. Besides, a concrete surfacing material were opted for the study. As for the plastic waste types, the three types of HDPE, PET, and PP were in two forms, flakey and pellets whereas the LDPE type was in ground form. Figure 4.1 illustrate the layout of the proposed study.

Figure 4.1 Structure of the studied pavement layers and incorporated materials of the research study

The following table, Table 4-1, shows the gradation of all three materials as per OPSS requirements. These are then plotted as shown in Figure 4.2 to Figure 4.4. These figures demonstrate the difference in aggregate particles contents between the three materials, the dense grade base-course, the gap graded sub-base and the subgrade soil. They show the different gradation curves for each material with an illustration on the packed aggregates in each skeleton.

Table 4-1. Graduation requirements of granular and subgrade soil according to OPSS requirements

Sieve Size		**Granular A Base-course**	**Granular B type II Sub-base**	**Subgrade Soil**
5.91"	150 mm	N/A	N/A	100
4.24"	106 mm	N/A	100	N/A
1½"	37.5 mm	N/A	N/A	N/A
1.06"	26.5 mm	100	50 – 100	50 – 100
3/4"	19.0 mm	85 – 100	N/A	N/A
0.53"	13.2 mm	65 – 90	N/A	N/A
3/8"	9.5 mm	50 – 73	N/A	N/A
No.4	4.75 mm	35 - 55	20 – 55	20 – 100
No.16	1.18 mm	15 – 40	10 – 40	10 – 100
No.50	300 µm	5 – 22	5 – 22	5 – 95
No.100	150 µm	N/A	N/A	2 – 65
No.200	75 µm	2 – 8	0 – 10	0 – 25

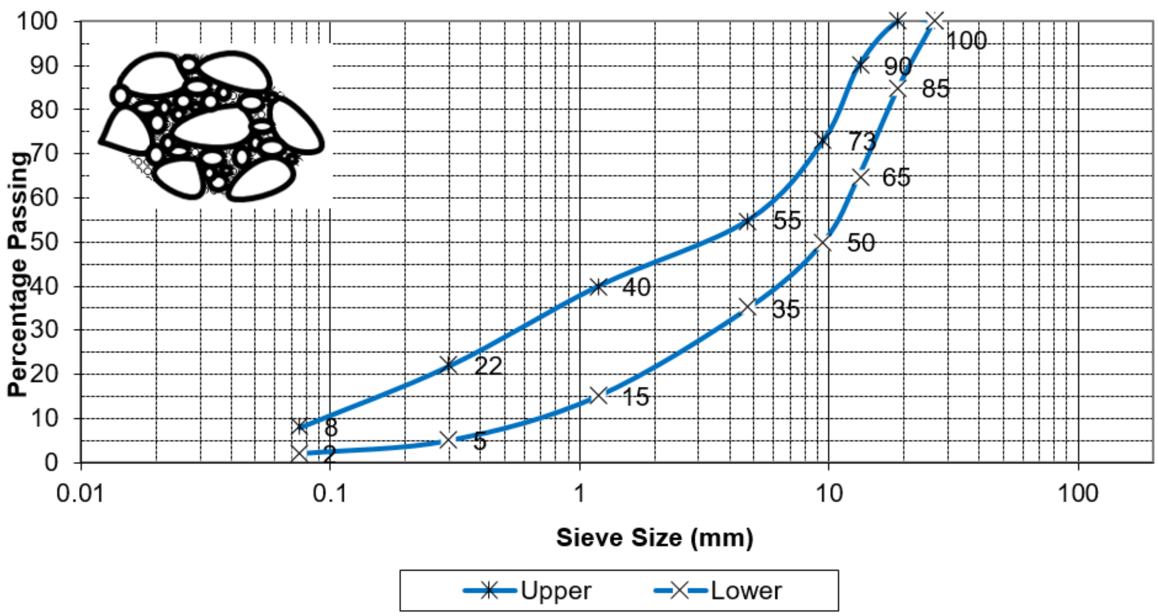

Figure 4.2. Grading limits for the dense graded granular A for base-course

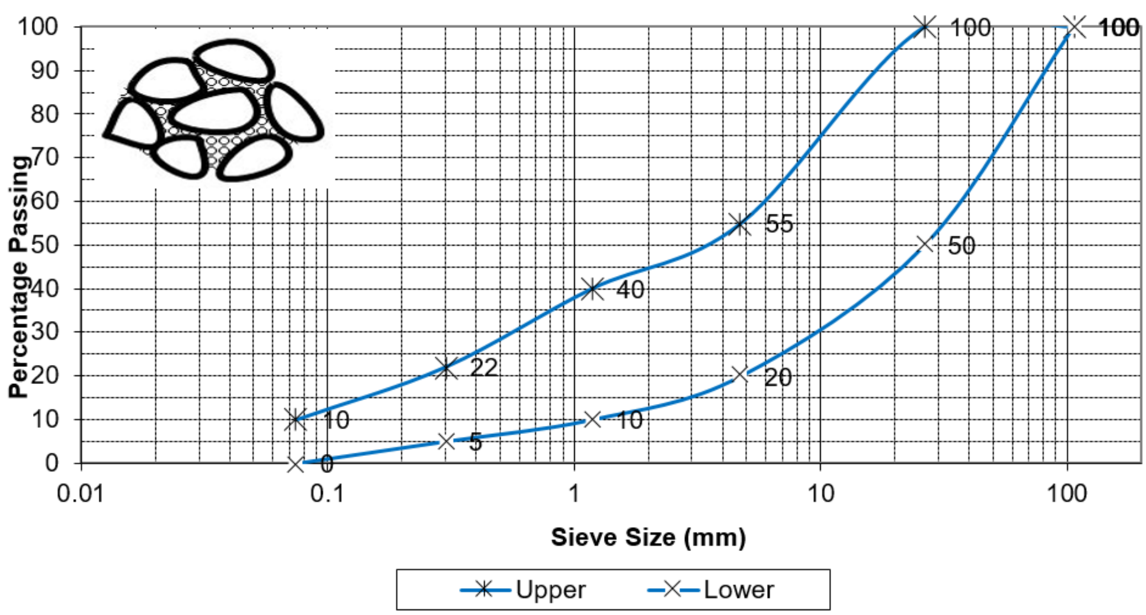

Figure 4.3. Grading limits for the dense graded granular B Type II for sub-base

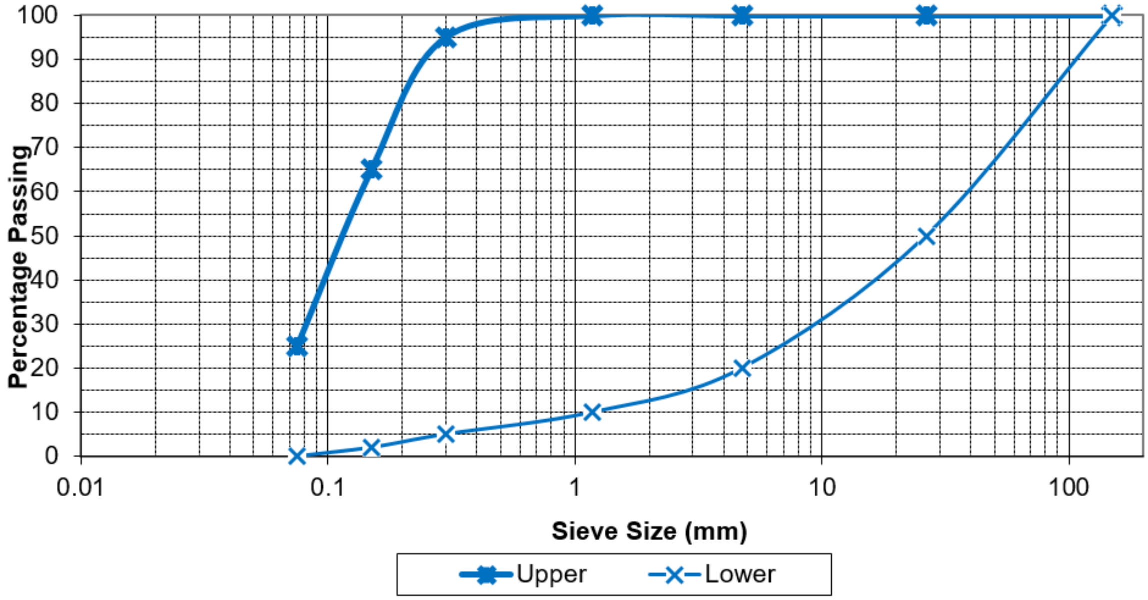

Figure 4.4. Gradation limits for the subgrade soil

4.1.2 Technical Papers structures

This section is divided into three sub-sections. It first focuses on characterizing the geotechnical properties of the subgrade soil with the incorporation of plastic waste. It then drifts to the investigation of impact of adding plastic waste to two different granular materials, the sub-base and base-course. Lastly, the study of the performance of concrete surface layer with the addition of the selected plastic types is added as the wrap up pf the section.

4.2 Geotechnical characteristics of plastic waste materials in pavement subgrade applications

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Journal of Transportation Geotechnics 27(2021) 100472

4.2.1 Abstract

Scarcity of traditional construction materials has motivated researchers to explore alternatives, and besides crushed glass, reclaimed asphalt pavement, and scrap tires, to name a few, plastic waste has also gained attention in recent years. Plastic waste is traditionally re-used or recycled, but it often ends up as trash on curbsides, in landfills, or in our seas and oceans. The substantial amount of plastic waste produced annually worldwide, and its environmental repercussions are the rationale for exploring alternatives in order to recycle plastic waste into construction materials. This exploration can also benefit many industries and would help minimizing adverse environmental impacts associated with dumping tonnes of plastic waste in landfills. Using plastic waste material with soil for soil reinforcement purposes has revealed some improvements in terms of strengths of materials, but nevertheless, this potentiality has not been fully assessed for different types and forms of plastic waste with natural subgrade soil in the road industry. In this paper, ground, flaky, and pelleted shapes of four sorted types of plastic waste from a recycling market were combined with silty or clayey gravel and sand soil of the A-2-7 AASHTO type. These plastic types are low density polyethylene (LDPE), high density polyethylene (HDPE), polyethylene terephthalate (PETE), and polypropylene (PP) resins. Their various geotechnical properties have been assessed thoroughly. The investigation process entails assessing compaction, Californian Bearing Ratio (CBR), strength, resilient modulus, and permeability properties for both natural sub-grade soil and modified sub-grade soil with the aforementioned types of plastic waste. The results obtained show that the addition of plastic wastes decrease the maximum dry densities of the subgrade soils because of the lower relative density of the plastic material compared to the soil particles. It is also found that the addition of plastic wastes can increase or decrease the CBR and M_R values of the subgrade. The nature of change (increase or decrease) and its magnitude are a function of the plastic content, shape and type. Permeability values of many subgrade soil samples increased with the addition of plastic waste, whereas the hydraulic conductivity of some soils modified with plastic remained unchanged. Subgrade soils with plastic had higher friction angle and lower compressive strength than plastic-free soils. The results of this research suggest that partial replacement of subgrade soil material with plastic waste may prove useful in road subgrade applications.

Keywords: Subgrade soil; plastic waste; pavement; road; strength; CBR.

4.2.2 Introduction

Extensive consumption of traditional materials and the increasing scarcity of these materials worldwide have influenced the perspective of several industries, including the construction industry. Thus, the use of various waste materials in other applications has become a necessity. Among these materials, plastic waste is a potential source for the construction industry. Plastic is a durable material that does not rapidly decompose and is inevitably encountered in various aspects of our daily lives in a wide spectrum of applications (Punthutaecha et al., 2007). Plastic is defined as an organic lightweight material (Punthutaecha et al., 2007) with thermal and electrical insulation characteristics (Thompson et al., 2009). Organic compounds that plastic can be made of encompass coal, cellulose from plants and trees, and crude oil (PlasticsEurope, n.d.). Plastic is used in various synthetic or semi-synthetic applications. These applications range from clothing to household items, and includes soft drink bottles, garbage bags, shopping plastic bags, milk pouches, soap dispensers, plastic wraps, straws, and plastic utensils (PlasticsEurope, n.d.).

Plastic recycling businesses have boomed in recent years, and recycling factories have emerged all over the world, including in Canada. Nonetheless, between the years 1975 and 2012, there was an incredible increase in globally produced plastic waste materials, at 280 million tonnes (Jambeck et al., 2015). It is reported that nearly 50% of this plastic is either sent to landfills or recycled for commercial reuse (Rochman et al., 2013). Alleviating the amount of uncontrolled plastic waste through finding better options for consuming it would be helpful for the environment (Jambeck et al., 2015). According to the Plastic Oceans Foundation, 8 million out of the 0.3 billion tonnes of the plastic produced yearly in the UK, USA, Canada, and China goes directly into oceans. This has causes more pollution and harm than human beings could foresee (Plastic Oceans Foundation, 2017). In addition, a report has indicated that 5 to 13 million metric tonnes of the 275 million tonnes of plastic waste produced by 192 coastal countries in the year 2010 wound up in the ocean (Jambeck et al., 2015). In Europe alone, plastic waste was estimated to exceed 8 million tons scattered over various landfills. Without proper waste management, the reported rate of pollution in the seas will become catastrophic by 2025, when it is forecasted to soar to 10 times higher than currently (Jambeck et al., 2015). The Plastic Oceans Foundation is among the organizations endeavoring to raise awareness of the fact that a more useful approach to dealing with this plastic needs to be found (Plastic Oceans Foundation, 2017).

The potential utilization of plastic waste in road work could open the door to more benefits to the industry. Recycled materials encompassed only 7% of an average of 179 million tons of aggregate material used in road construction between 2000 and 2009 in the province of Ontario (Canada). The total consumption of aggregates in the construction industry will rise by 13% over the next 20 years and will hit 186 million tons on average (Altus Group Economic Consulting, 2009), but recycled material was projected to only compose 8% of the aggregate consumption by then (Altus Group Economic Consulting, 2009). Despite the substantial amount of aggregate materials that are consumed in the construction industry, which are predominantly either natural or processed minerals, not much consideration has been given to other alternatives to preserve these resources. Reclaimed asphalt pavement (RAP) is deemed the largest secondary source of material used in road construction; however, there are several other sources that should be investigated to diminish the damage to the environment and the scarcity of natural raw materials. Plastic waste is among the materials which have been rarely considered for use in the road construction field due to a lack of enough evidence on its compatibility and feasibility.

Research studies on the potential use of plastic materials in the road industry have specifically focused on the use of plastic waste in the surfacing layer of flexible pavement (Vasudevan et al., 2012). Pellets of low density polyethylene (LDPE) have been reported to increase stability and tensile strength of asphalt mixtures (Huang et al., 2007). Maniruzzaman et al. (2016) revealed how Marshall stability, voids in total mix (VTM), and voids in Mineral aggregates (VMA) were substantially improved, while the density of the Hot Mix Asphalt (HMA) decreased due to the utilization of plastic (Maniruzzaman, et al., 2016). Chips of comingled plastic brought tremendous enhancement in bonding characteristics and resistance to moisture damage with plastic modified mixtures (Angelone et al., 2015), while flakes and pellets of polypropylene (PP) and polyethylene (PE) led to significant increases in resistance to low temperature cracking and permanent deformation of asphalt mixtures (Angelone et al., 2015). Adding polyethylene terephthalate (PET) chips to asphalt mixtures enhanced the stability and air voids as the rate of inclusion increased (Sojobi et al., 2016), and one plastic mix used in Vancouver, Canada, has shown significant surface quality despite some environmental concerns (D. P. Ball, 2012).

Observations have been made on the changes to the physical properties of bitumen binders when modified with polymers (Karmakar & Roy, 2016; Maniruzzaman et al., 2016).

Polymers actively interact with virgin bitumen, resulting in an increase in the affinity between the two constituents. Thus, they enhance the rheological properties of the asphalt cement and minimize water-induced damage to the asphalt mixture (Chavan, 2013; Maniruzzaman et al., 2016). Periodic surface monitoring has also showed great promise for the performance of plastic-modified mixtures (Salem, 2014). Reduced costs for maintenance and long-lasting defect-free asphalt surface condition are anticipated by using plastic in asphalt mixtures (Kalantar et al., 2012; Vasudevan et al., 2012; Sojobi et al., 2016). However, very few studies have been conducted on stabilizing subgrade soil with plastic waste. Significant increases in elasticity and resistance to deformation have been reported with the addition of strips of high density polyethylene (HDPE) plastic waste to soil (Choudhary et al., 2010). HDPE was reported to reduce the stabilization cost of subgrade soil as opposed to using geo-grids (Choudhary et al., 2010). Loehr et al. (2015) concluded in his research that using recycled plastic pins (RPP) to stabilize embankments would outweigh any other available stabilization approach (Loehr, et al., 2000). Due to the scarcity of studies on the use of plastic wastes in subgrade soils, our understanding of the effect of plastic wastes on the geotechnical properties of subgrade soils is limited.

Thus, the key objective of this paper is to characterize the geotechnical properties of subgrade soils modified with different types (LDPE, HDPE, PET and PP) and forms (flakes, pellets, and ground) of recycled plastic waste in terms of material properties for road applications. A suite of extensive laboratory tests was performed on the modified subgrade soils and their geotechnical properties assessed and compared with that of a virgin pavement subgrade (A-2-7 subgrade) soil.

4.2.3 Experimental Program

4.2.3.1 Materials used

4.2.3.1.1 Subgrade soil

Subgrade soil samples were collected from three different locations in the Ottawa-Gatineau area in Ontario, Canada. The first sample was sourced from the vicinity of the Ottawa Light-Rail transit project (OLRT), while the second sample was supplied by the Lafarge Company in the Gatineau area in Quebec, Canada. Finally, the third soil sample was taken from the proximity of a newly constructed building on the main campus of the University of Ottawa. All samples were initially tested and examined in order to comply with the requirements of subgrade soil in road construction by following the American Standards of Testing and Materials (ASTM), American Association of States Highways and Transports Officials (AASHTO) Standards, and Ontario Provincial Standards Specifications (OPSS).

4.2.3.1.2 Plastic waste

The types of plastic used in the study were obtained from two plastic waste recovery facilities, i.e., Canada Fibers Ltd., situated in Toronto, and Enviroplast Inc., in Montreal. The types of plastic provided included low-density polyethylene (LDPE), high-density polyethylene (HDPE), polyethylene terephthalate (PETE), and polypropylene (PP). These were all post-consumer plastics that had undergone either shredding into flakes, grinding, or extrusion in a high-temperature chamber to be produced in pellets. The three types were used in ground, flaky, and pelleted shapes. Only the LDPE plastic type was ground, while the other types were in flake and pellet forms. Flakes are produced by shredding baled plastic into small chips at recycling facilities, while pellets are produced through extrusion, where the shredded plastic goes into a high-temperature chamber to form round and spherical pellets.

Figure 4.5 Type of plastic used: (a) HDPE flakes, (b) HDPE pellets, (c) PET flakes, and (d) PET pellets

Figure 4.5 and Figure 4.6 show the physical characteristics and appearance of the flakes, pellets, and ground plastic waste used in this study. The pelleted HDPE and PET were cylindrical, with diameters between 3.65 mm and 4.45 mm and thicknesses between 2.70 mm and 1.65 mm respectively. The pellets had a spherical shape and were 3.3 mm in diameter, while the flakes were triangular, between 7 mm x 8.8 mm in diameter and 0.7 mm in thickness.

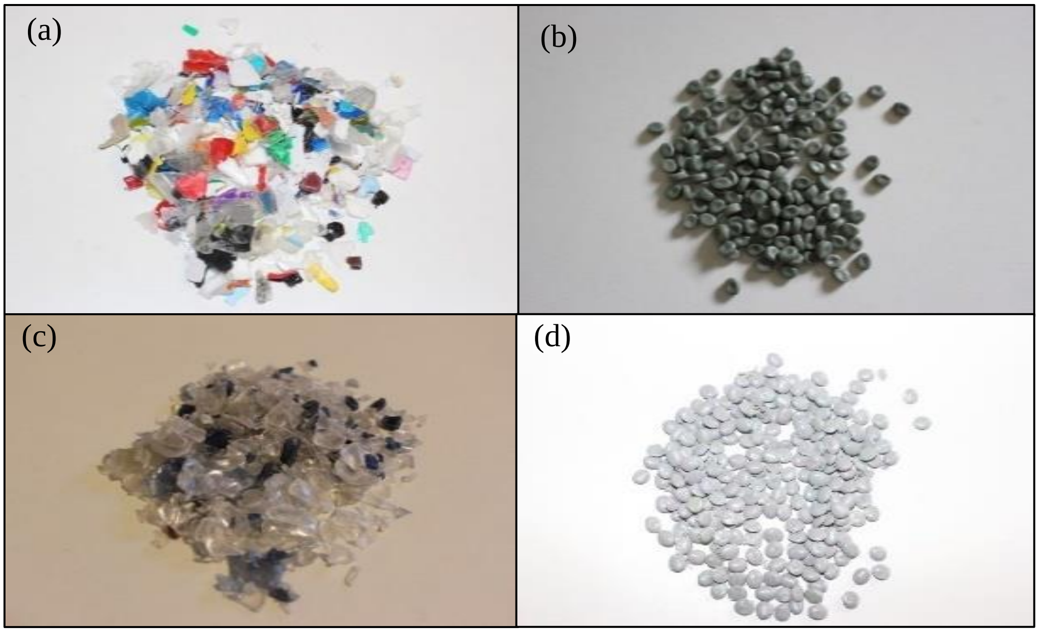

Figure 4.5 Type of plastic used: (a) HDPE flakes, (b) HDPE pellets, (c) PET flakes, and (d) PET pellets

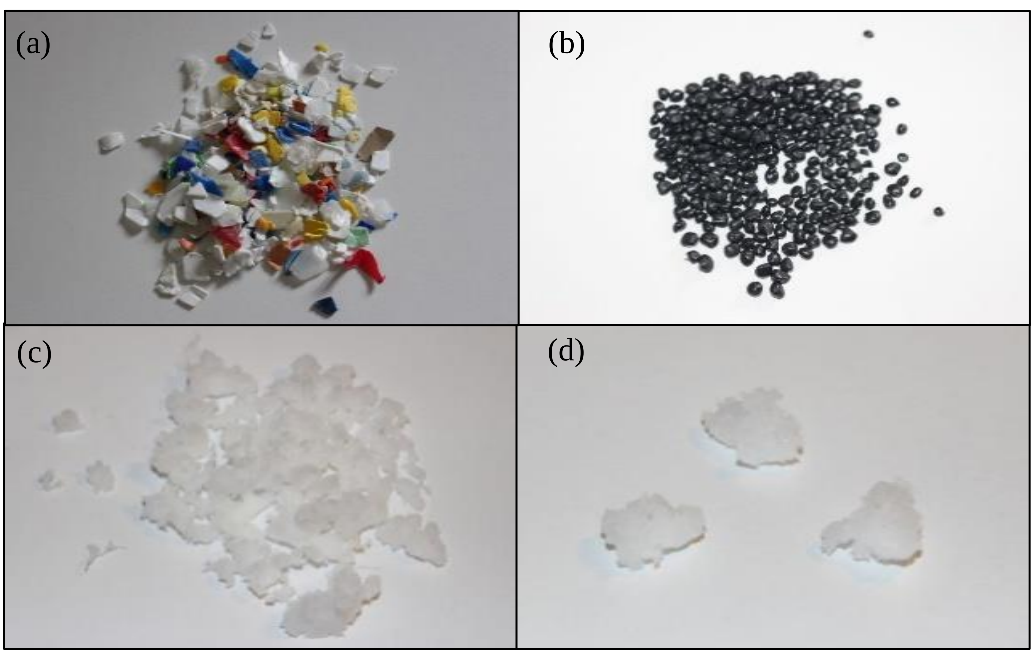

Figure 4.6. Type of plastic used: (a) PP flakes (b) PP pellets (c & d) LDPE ground

For the facilitation of easy handling of the testing results and analysis, samples of tested materials used in the study were then designated by symbols, as shown in Table 4-2 Symbols for the tested samples below.

Table 4-2 Symbols for the tested samples

Identification	Symbol
Natural subgrade soil	S.G
Subgrade soil + ground LDPE + X %	SG-G-LDPE X%
Subgrade soil + flaked HDPE + X%	SG-F-HDPE X%
Subgrade soil + pelleted HDPE + X%	SG-P-HDPE X%
Subgrade soil + flaked PET + X%	SG-F-PET X%
Subgrade soil + pelleted PET + X%	SG-P-PET X%
Subgrade soil + flaked PP + X%	SG-F-PP X%
Subgrade soil + pelleted PP + X%	SG-P-PP X%

(X) Refers to the plastic content in percentage.

4.2.3.1.3 Mixing water

Throughout the investigation process, distilled water was used to prepare the specimens. The water content was ultimately either added as a percentage of the total weight of the soil sample or the total weight of the soil plus the plastic content.

4.2.3.2 Sample preparation

The soil samples were initially dried overnight in an oven at a temperature of approximately 105°C. Subsequently, these samples were mixed thoroughly at the predetermined optimum moisture content (OMC) with the pre-selected plastic contents. The mixing process took place in an automatic mixer, and the mixing time was approximately 7 minutes in order to ensure thorough dispersal of the water in the soil. Visual observation was applied to ensure an enough and thorough mixing process. Samples were then kept in a sealed container (to avoid any evaporation) for a period of 16-24 hours (18 hours on average) before further testing. For each of the following tests, each test was repeated at least twice, and the average value is considered, if the difference between the two obtained values is less than 5%.

4.2.4 Testing Methods

4.2.4.1 Determination of the Moisture Content

Determining moisture content is crucial in paving work, as it helps assess the actual water content in subgrade soils as compared to the required water content for compaction control. The moisture determination was performed as per the D-2216 designation in the ASTM standards (ASTM, 2010). As it was necessary to eliminate the moisture, the soil specimens were left to dry in an oven at a temperature of 100 - 110°C for a period of 18-24 hours before being used for the experiments.

4.2.4.2 Classification of Soils

Classifying soil is an imperative step prior to accepting it for use in road construction, as it has to fall within an acceptable range to be satisfactory for any paving work. Many engineering properties of soil, such as strength, shear resistance, and hydraulic conductivity, are linked with particle distribution (Christopher et al., 2006), and soil compositions that do not fit within standardized requirements are excluded. In executing the grading analysis for the research samples, the samples from the three aforementioned locations were classified by following the methods outlined in the AASHTO classification system. Prior to sampling for classification test, a sample divider was used to reduce the size of the samples. The samples were tested in duplicate to assess the physical properties, as required by AASHTO, and the inputs required for the classification of the soils included gradation and consistency analyses (Atterberg limits). The grain size distribution analysis was conducted in accordance with the ASTM D422 standard (ASTM, 2006a), whereas the Atterberg limit tests were executed in accordance with the ASTM D-4318 standard (ASTM, 2005).

4.2.4.3 Compaction test

A modified proctor compaction test was conducted, since it reflects higher densities for heavier loading conditions, and the tests were conducted in accordance with the ASTM D-1557 standard (ASTM, 2012). In the testing procedure, the specimens were compacted with a 4.5kg hammer in a cylindrical mold with a 100mm internal diameter. The compaction was done with 5 layers and 56 blows. At the end of the compaction of the 5th layer, the mold was removed from the machine, weighed, and dismantled, and the soil specimen was extracted from the mold. A minimum of two samples were taken to dry in the oven at 100-110°C in order to determine the moisture content, in accordance with the ASTM D-2216 standard (ASTM, 2010). The compaction curve was derived for each specimen using a predetermined 4 to 5 points for dry density and moisture content. This obtained the OMC and Maximum Dry Density (MDD) of each sample. To prepare specimens with various plastic content, a pre-specified amount of each type of plastic waste was added before mixing with the water, and the same procedure was followed to obtain the MDD at the OMC for each specimen. The content (in %wt) of ground, flaked, and pelleted plastic for each type is given in Table 4-3 next.

Table 4-3. Content (in weight percentage) and type of plastic waste used with the subgrade soil

Percentage of Plastic Used (%)	LDPE	HDPE	PETE	PP
1 to 5	Ground	Flakes	Flakes	Flakes
10	Ground	Pellets	Pellets	Pellets

4.2.4.4 Strength Index of Soils

The knowledge of the strength index is a pivotal requirement for any soil used in paving work. Soils that exhibit weak strength require thicker pavement structures to protect them and are often prone to premature failure. Thus, the California Bearing Ratio (CBR) of higher value is a good indication of the performance of a pavement structure. The literature indicates that although this test is an indirect reflection of the strength, unlike the resilient modulus, the fact that it is more affordable but still correlated with the resilient modulus has made is very common in pavement engineering practices (Christopher et al., 2006). The CBR was considered as useful for assessing the strength of the soils in this research. Once the OMC and MDD values were obtained from the compaction testing, the CBR specimens were prepared by subjecting them to modified compaction. The CBR tests were performed according to the ASTM D-1883 standard (ASTM, 1999). Each specimen was compacted in a 6-inch cylindrical mold in five layers, with 56 blows in total for each layer. The testing was conducted on un-soaked specimens using an in-laboratory-fabricated piston, simulating the actual CBR penetration test. The force for the penetration is plotted against the penetration value. The force was recorded for penetration values of 0.5mm, 1.0mm, 2.0mm, 4.00mm, 5.mm, 7.5mm, 10mm, and 12.5mm. The CBR strength was then obtained by comparing the ratio of the forces for penetration values of either 2.5mm or 5.00 mm.

4.2.4.5 Hydraulic Conductivity Test

Hydraulic conductivity is one of the main material inputs to assess the transient moisture profiles in unbound materials and estimate their drainage characteristics. In this research, the falling head approach was adopted for determining the saturated hydraulic conductivity of subgrade soils. The test procedure follows the ASTM standard D5084 (ASTM Standard D5084, 2017). For each sample, a sufficient quantity of dry soil was sampled and subsequently saturated overnight at the optimum water content in a sealed, airtight bag. Prior to testing, a porous stone was placed at the bottom of the permeameter along with a filter paper situated on the stones. The soil was then poured into the permeameter and compacted gently with a free-falling tamper. At approximately 20 mm from the upper edge of the permeameter, another filter paper was placed to cover the top of the specimen prior to placing another porous stone on top. The cap for the cell was then attached, and all hoses were connected to conduct the test. The initial measurements before performing the test included recording the height and diameter of the specimen to estimate the volume of the soil. The procedure for preparing the modified subgrade soil sample was similar to the procedure for preparing a natural subgrade soil sample. The samples were then ready for testing in the falling head apparatus. The time required for the water levels to change in the standpipes along with the initial and final water level measurements were recorded to compute the coefficients of permeability of all the specimens.

4.2.4.6 Unconfined Compression Strength Test

Unconfined compressive strength (UCS) testing was conducted to determine the compressive strength of subgrade soils, and was executed in accordance with the ASTM standard D-2166/D-2166M immediately after compaction (ASTM, 2013). The UCS testing was performed on specimens of both untreated and plastic-treated subgrade soils, and a total of 56 specimens (28 in duplicate) were prepared for testing. The maximum compressive strengths of all specimens were then tabulated and plotted against percentage of plastic to observe any changes in compression of the soil with changes in proportions of the plastic utilized.

4.2.4.7 Static Triaxial Shear Test

The shear properties of the investigated subgrade soils were assessed while the soils were at a consolidated undrained state. The consolidated undrained (CU) triaxial test is presumed as enough for the short-term strength analysis process, where water does not have time to dissipate. The triaxial CU testing was performed according to the ASTM D4767 standard

(ASTM, 1995). In this procedure, consolidation is performed while the drainage valve of the triaxial cell is open, and the valve is then closed when applying the three levels of confining pressure in the shear stage. A total of 56 specimens were prepared in the same way as for the UCS test. The OMC was added to the soil, which was then compacted in a 100mm-high 50mm-diameter split mold by using the in-laboratory loading frame. During the saturation, a constant 35 kPa effective stress was maintained on the specimens in order to get to the maximum saturation state. This procedure required monitoring the increase in the back pressure when increasing the cell pressure in order to keep within the 35kPa limit. Subsequently, the consolidation phase began by closing the drainage valve. The back pressure reached during saturation was maintained while increasing the cell pressure until the pore pressure decreases of each combination. Pressures of 70 kPa, 150 kPa, and 200 kPa were consequently applied until failure of the three samples. Eventually, the shear stage commenced under the undrained conditions, and the maximum applied stresses at failure were recorded.

4.2.4.8 Estimation of the Repeated (Cyclic) Resilient Modulus Test

The modulus of resilience (M_R) of a subgrade soil is an important factor in paving design, as it estimates the resistance of repeatedly applied loads and induced stresses. When paving material is prone to cyclic loading, it is defined as the ratio of deviator stress to the recoverable strain (Li & Selig, 1994). The M_R for soil and granular materials is either measured experimentally or predicted by means of empirical, analytical or numerical models. Experimental results are considered as a more direct measurement of stiffness in paving design, and follow AASHTO T-307 (AASHTO, 2017). The strength of soil, measured by means of the CBR test, and stiffness are the two properties of soil that are well correlated in multiple models used globally (Christopher et al., 2006).

In the present work, the modulus of resilience is estimated using well-established prediction models, as is common practice (Drumm et al., 1990). Various models have been developed for estimating a modulus of resilience (Iyengar et al., 2012), including for the M_R of soil materials as well as granular materials in paving applications. Some of these prediction models are contingent on the materials' properties, including CBR, unconfined compressive strength, and plasticity. Iynegar et al. (2012) reported three models developed by different researchers, and these models are summarized in Table 4-4 (Iyengar et al., 2012).

Table 4-4. Prediction Models for MR

UCS (psi)	Model M_R (psi)	Developer
< 200	= 124 * UCS + 9,980	Thomson,1986 (as cited in (Iyengar et al., 2012))
200 > and < 500	= 1,000 * UCS	Little, 2010 (as cited in (Iyengar et al., 2012))
> 500	= 1,200 * UCS	Barenberg, 1977 (as cited in (Iyengar et al., 2012))

The Mechanistic-Empirical Pavement Design Guide (MEPDG) has also suggested the following model to predict the resiliency of stabilized soil material for Level 2 designs (Research Program Transportation Research Board National, 2004) :

M_R (psi) = 2555 ^{x} $CBR^{0.64}$, where CBR is a percentage (1)

According to Abu El-Maaty Bhairy, equations to estimate the resiliency of materials when considering CBR values have evolved over the years. The following was proposed by the United States Army Corps of Engineers in 1975 (Abu El-Maaty Behiry, 2014):

M_R (psi) = 5409 ^{x} $CBR^{0.71}$, where CBR is a percentage (2)

The correlation equation for CBR and M_R which was incorporated in the AASHTO 1993 design guide was first proposed by Keukelom and Klomp. It is, however, limited to a maximum of 10% or less soaked CBR for a soil and is given as follows (Abu El-Maaty Behiry, 2014):

M_R (psi) = 1500 ^{x} CBR, where CBR is a percentage (3)

Sukumaran noted a slight different correlation model which was developed by the Council for Scientific and Industrial Research (CSIR) of South Africa (as cited in (Abu El-Maaty Behiry, 2014)). The model was based on assumptions similar to those in the US Army Corps of Engineers' approach:

M_R (psi) = 3000 ^{x} $CBR^{0.65}$, where CBR is a percentage (4)

Another estimation approach categorized material properties into two groups in terms of CBR rate values when predicting a modulus of resilience. The first is for CBR below 5%, and the second is for materials over 5%.

M_R (psi) = 2350 ^{x} $CBR^{0.70}$, where CBR is a percentage (5)

M_R (psi) = 3250 $\times$ $CBR^{0.50}$, where CBR is a percentage (6)

Nevertheless, the debate over the reliance on CBR values to estimate the modulus of resilience is still ongoing. Drumm et al. (1997) questioned this dependency and claimed that the dependence on CBR can be misleading. Li and Selig (1994) relied essentially on the correlation developed previously by Thomson and LaGrow for estimating the value of the resilience of soil. This relationship is contingent upon the percentage of material particles passing through a 2-micrometre sieve, as well as the plasticity of the particular soil. The derived formula is given as follows (Li & Selig, 1994):

$$M_R = 30800 + 677 * \% \text{ Clay} + 821 \text{ (PI)} \quad (7)$$

where PI is the plasticity index of the material.

Another model was established by Çeleri in 2007 that considers other aspects of the materials properties. Other than the CBR ratio and the consistency limits, the optimum moisture content is also included in the model, as follows (Abu El-Maaty Behiry, 2014):

$$M_R = 228376.7946 - (1479.8978 \times LL) - (12381.4217 * OMC) + (689.5 \times CBR) + (152.9164 \times LL \times OMC) \quad (8)$$

However, another model in the literature only estimates the modulus of resiliency for CBR rates ranging from 2 to 21 percent. This is shown in the following equations (Çöleri, 2007):

$$M_R = k_i \, \sigma_d^{\,k2} \quad (9)$$

$$K_i = 10^{(1.0016 + 0.043\,CBR)} \quad (10)$$

$$k2 = -\frac{1.9557}{CBR} + 0.1705 \quad (11)$$

In this research, the correlation formula suggested by the Mechanistic-Empirical Pavement Design Guide (MEPDG), i.e., Equation (1), and the correlation equation based on compaction and CBR properties, i.e., Equation (8), are adopted to estimate the modulus of resilience of the soil.

4.2.5 Results and Discussion

4.2.5.1 Classification of Subgrade Soils

Overall, the results categorized all tested soil samples in the A-2-7 class according to the AASHTO classification system, which is a commonly used in paving applications. It is deemed to be a good to excellent subgrade soil for paving applications. The classification results for all tested soil samples are summarized in Table 4-5 below.

Table 4-5. Gradation analysis results of soil samples

Site	Specimen No.	Gravel (%)	Sand (%)	Silt & Clay (%)	Water Content (%)	AASHTO Classification
I	1	1.10	97.30	1.60	24.3	A-2-7
	2	1.10	96.30	2.60	23.8	
II	1	1.70	95.80	2.50	20.5	
	2	1.50	96.70	1.80	22.6	
III	1	0.40	99.20	3.30	8.2	
	2	0.70	95.60	3.70	9.2	

The results from the grain size analysis are plotted in a semi-log graph Figure 4.7 to show the gradations of the tested soils, and it is shown that all soil samples had a similar gradations, as per table 2 of OPSS 1010 : gradation analysis of subgrade soil (Ontario Provincial Standard Specification, 2003), with significantly low clay and silt content. This was essential to ensure similar soil has been chosen for further investigation in this research.

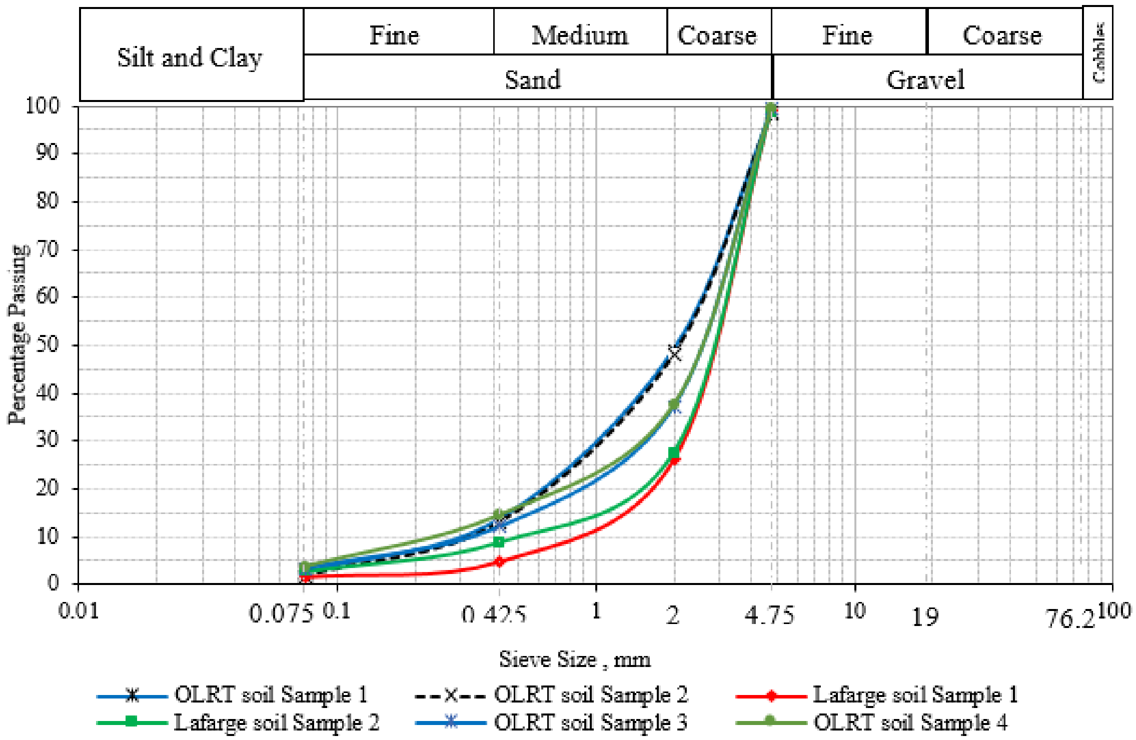

Figure 4.7 Grain size distribution of soil samples

4.2.5.2 Optimum Moisture Content and Maximum Dry Density

The values of the OMC and MDD for all combinations are presented in Table 4-6. Figure 4.8 to Figure 4.11 feature the changes in moisture content with dry densities for the subgrade soils with each type of plastic. From these figures, it can be inferred that as the proportion of plastic increased, the moisture content and corresponding dry density decreased accordingly. Because plastic material (LDPE, HDPE, PET, PP) has a relative density that is much lower than that of soil mineral particles, it is obvious that when the plastic content increased, the MDD will also decrease. Dhatrak and Konmare (Ali et al., 2011) observed similar trends, and concluded that this behavior was attributed to the low density of the plastic used (Dhatrak & Konmare, 2016). The results presented in Table 4-6 and Figure 4.8 to Figure 4.11 indicate that the type and shapes of the plastic used don’t affect the dry density in a significant manner.

Table 4-6. Summary of optimum moisture content and maximum dry density of different compositions of subgrade soils

	Optimum Moisture Content (OMC), %						Maximum Dry Density (MDD), g/cm³					
Subgrade Soil (SG)	20.5						1.625					
Percentage of Plastic, %	1	2	3	4	5	10	1	2	3	4	5	10
SG-G-LDPE	19.5	18	20	18	19	19.5	1.563	1.588	1.525	1.570	1.510	1.574
SG-F-HDPE	19.5	17	20.5	19.5	19	19.5	1.575	1.588	1.560	1.570	1.545	1.510
SG-P-HDPE	17	18	19	18	20	17	1.650	1.625	1.538	1.590	1.590	1.525
SG-F-PET	17	18.5	17.5	18.5	17	17	1.645	1.560	1.575	1.625	1.645	1.520
SG-P-PET	17	18	21	19.5	17	17	1.588	1.580	1.513	1.610	1.570	1.430
SG-F-PP	19	18	19	17	19	19	1.600	1.585	1.550	1.530	1.525	1.575
SG-P-PP	19	19	21.5	19	17	19	1.580	1.575	1.530	1.563	1.560	1.420

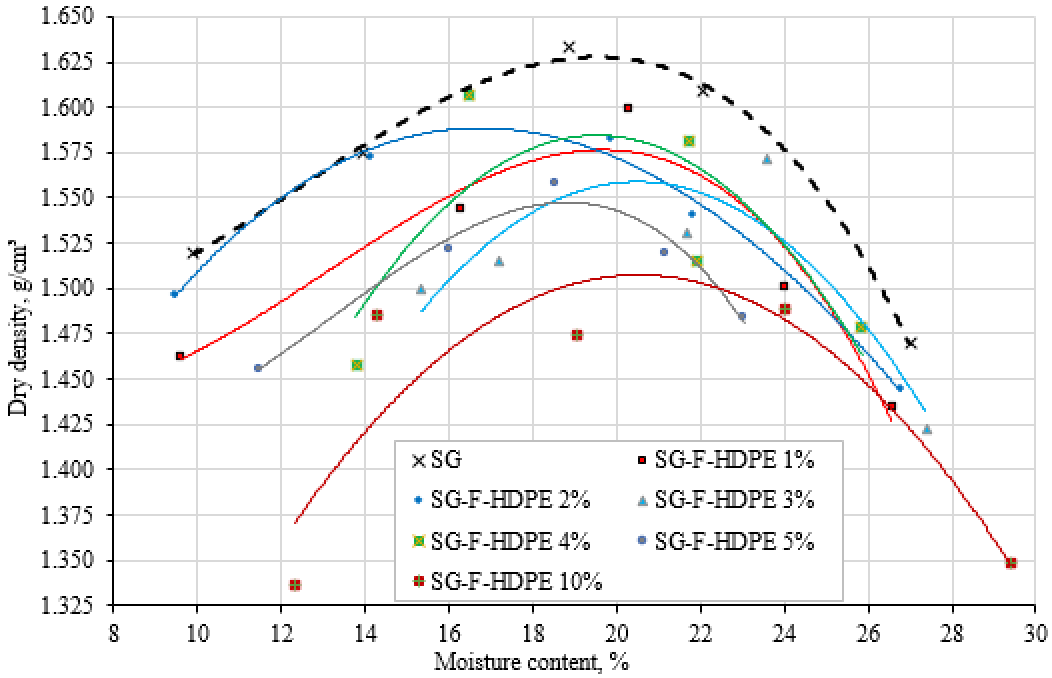

Figure 4.8 Compaction curves of the natural and flake-HDPE modified soil

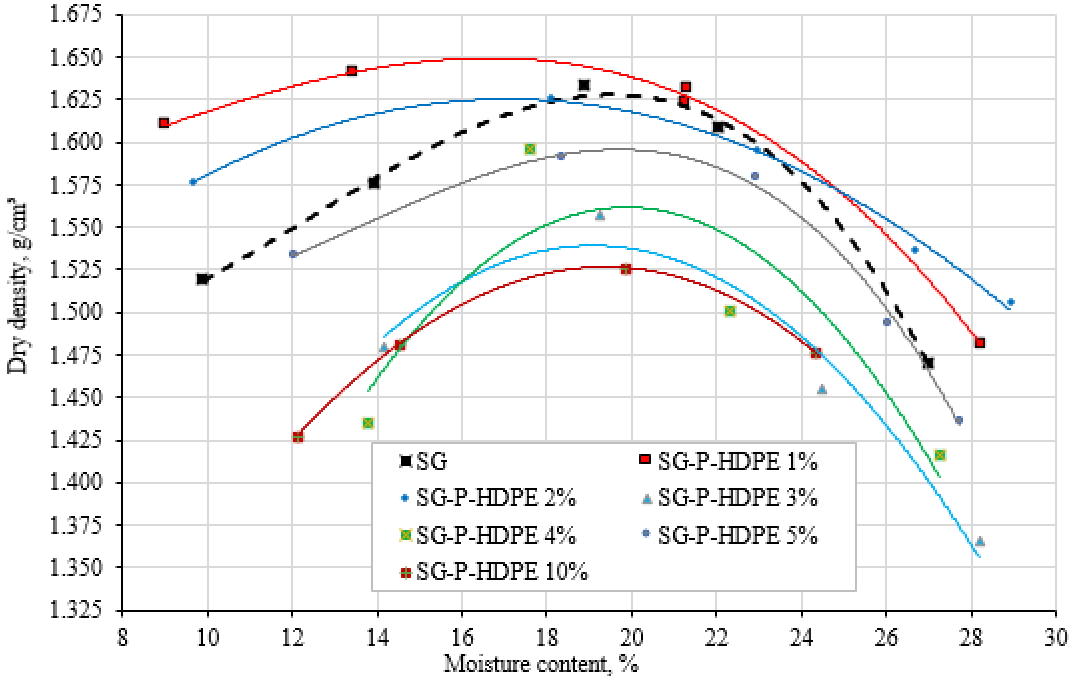

Figure 4.9 Compaction curves of the natural and pellet-HDPE modified soil

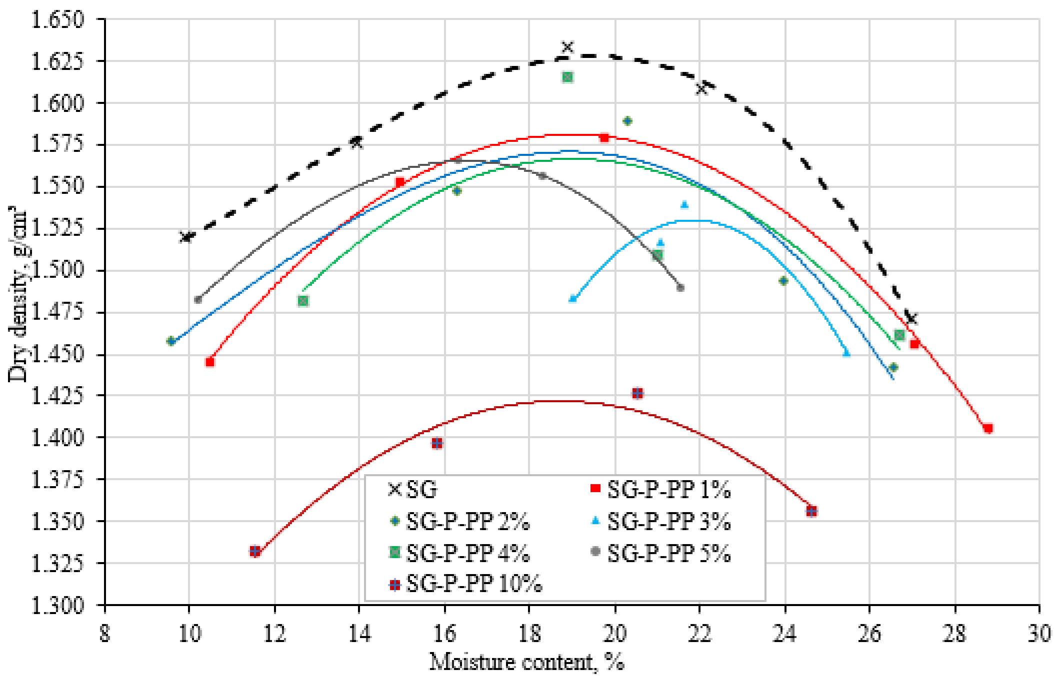

Figure 4.10 Compaction curves of the natural and pellet-PP soil

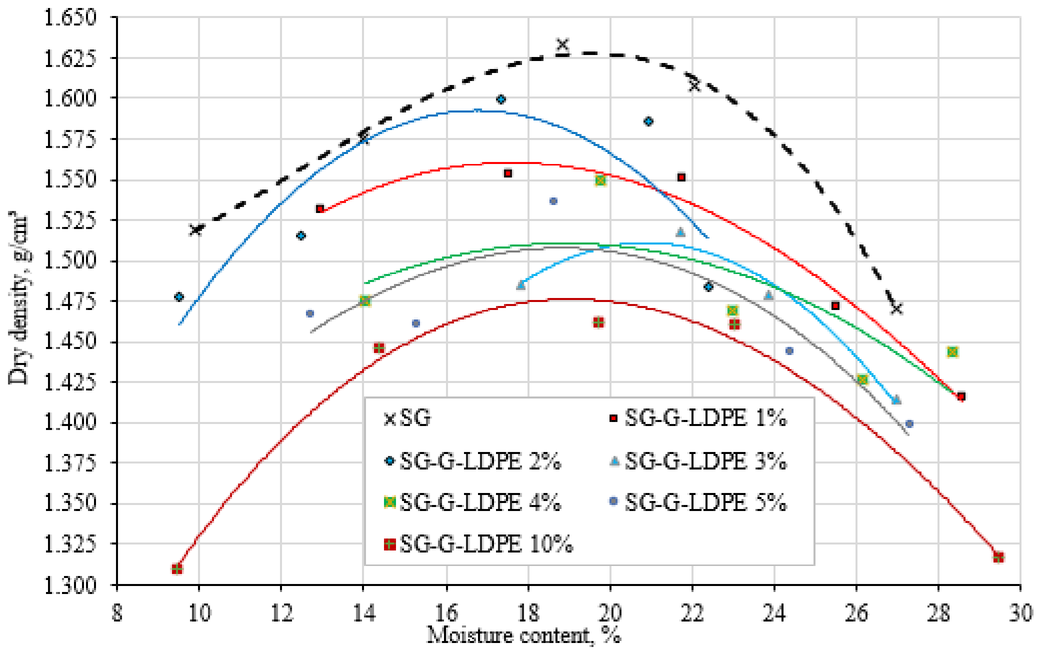

Figure 4.11 Compaction curves of the natural and ground-LDPE modified soil

4.2.5.3 CBR Results

Table 4-7 summarizes the CBR results for natural soil and soils with various combinations of plastic waste content. The load-penetration values are plotted to obtain the CBR values from the penetration testing, and thus the effects of the proportions of the various plastic shapes on the soil samples are easily seen. The CBR curves for different material combinations are illustrated in Figure 4.12. From Table 4-7 and Figure 4.12 it is obvious that, depending on the plastic content, the strength of the soil samples can be improved with the addition of plastic of different shapes. The CBR values ranged from 25% to 41% for the soil samples with SG-F-HDPE, SG-P-HDPE, PET, and PP; however, the strengths only ranged from 21% to 29% for the SG-G-LDPE samples. One can note that the behaviour of the soil with ground LDPE was the most changeable, and this might be related to the irregular shape of the ground LDPE causing fluctuations in the CBR values, unlike the other types of plastic (Dhatrak & Konmare, 2016).

Table 4-7 Summary of CBR values of different rates and combinations of subgrade soil with plastic waste

Subgrade Soil (SG)	California bearing ratio, CBR (%)					
	29.2					
Percentage of Plastic, %	1	2	3	4	5	10
SG-G-LDPE	29.2	29.2	30.7	27.7	20.8	22.6
SG-F-HDPE	24.8	24.8	24.8	26.3	25.5	26.3
SG-P-HDPE	40.9	39.4	36.5	32.8	30.7	20.4
SG-F-PET	27.4	27.4	29.2	31.0	31.0	34.7
SG-P-PET	32.1	27.4	27.4	27.4	25.5	27.7
SG-F-PP	28.1	28.1	28.5	29.2	30.7	38.0
SG-P-PP	32.1	30.7	30.7	27.7	30.7	29.2

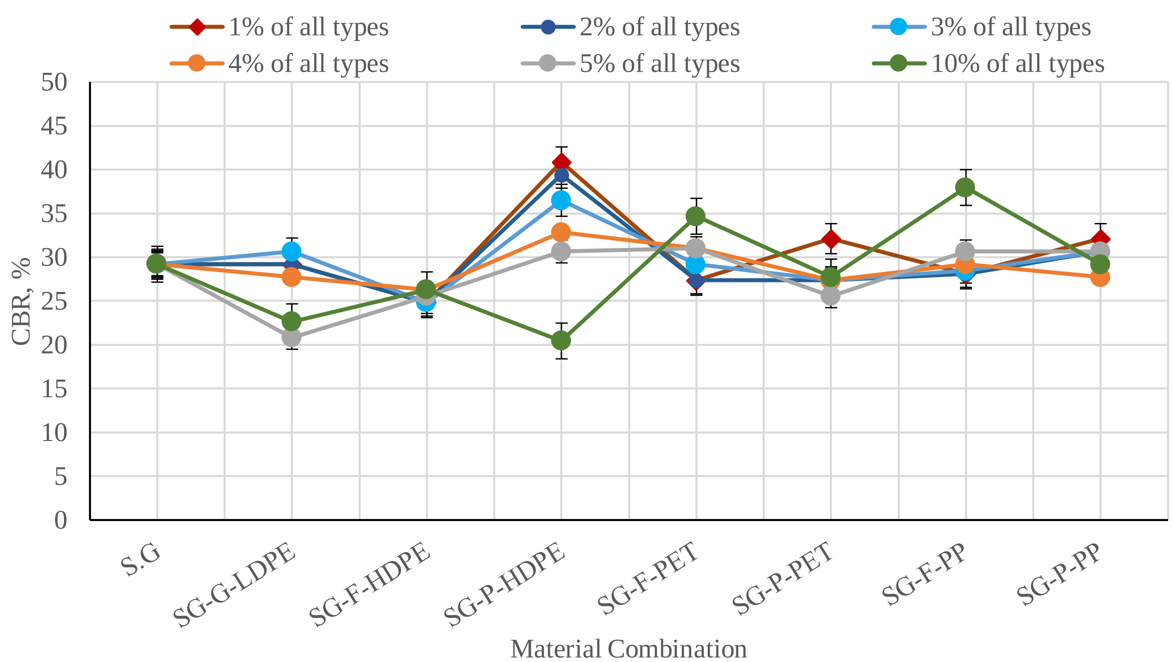

Figure 4.12. Changes in CBR values with changes in plastic types and rates

The changes in the CBR of the soils with plastic varied depending on the plastic type and content (1%, 2%, 3%, 4%, 5%, and 10%), type and shape, as shown in Table 4-7 and Figure 2.8. For all types and shapes of plastic, except those with SG-G-LDPE, there is a plastic content that significantly increased the CBR compared to CBR of the unmodified subgrade soil (S.G.).

However, the results on the impact of plastic content on the CBR also indicate that, for a given type and shape of plastic, the CBR value and plastic content is not a simple linear relationship. Indeed, there is a critical value beyond which the CBR decreases as the plastic content is increased. The optimal plastic content is related to the reinforcing ability of the plastic. When a small amount of plastic is added, the plastic particles are easily dispersed throughout the subgrade soil, which poses a potential for increased CBR values (Dhatrak & Konmare, 2016). The literature indicates that the elongation effect on the frictional force at the interface between the soil particles and plastic material plays a role in improving soil strength (Choudhary et al., 2014). At too high plastic content, the plastic might tend to overlap, causing weaker interaction between the soil particles, resulting in a lower strength (Disfani et al., 2012). This observed plastic waste-induced improvement of the CBR values may suggest that the addition of plastic waste to subgrade soils could result in the reduction of the thickness of the paved structure, which, in turn, will lead to a decrease of the cost of the road or pavement structure. Indeed, the CBR value is a predominant factor that influences any paved structure, and according to AASHTO, the design method and approaches indicate that the sturdier the material is, the higher the CBR value (Christopher et al., 2006), which is reflected by requiring less paving material on top of the subgrade soil. Thus, the overall cost of the paved structure will decrease, as illustrated in Figure 4.13 (Huang, 2004). This figure shows that as the strength of subgrade soil increases, the required thickness of the base-course layer on top decreases. Subsequently, the percentage of reliance of the strength of the sub-base materials shifted as the CBR of subgrade soil improved. Greater CBR values govern the sustainability at which subgrade soil can handle the load transferred to from the above layers. Many specifications require a subgrade to have a CBR value higher than 10 % to be considered structurally sufficient for pavement applications (Ali & Arulrajah, 2012); (Emersleben, 2012);(Arulrajah et al., 2014). Literature also indicated that a subgrade soil of a CBR percentage below 6% is considered unstable and is either improved or replaced prior to any further road construction works (Bandara et al., 2015). Since the pavement structural acts as a unity in transferring imposed loading to the underneath subgrade layer, the thickness of the top laid Sub-base or base materials can be efficiently reduced as the strength of the bottom layer improved (Neopaney et al., 2012).

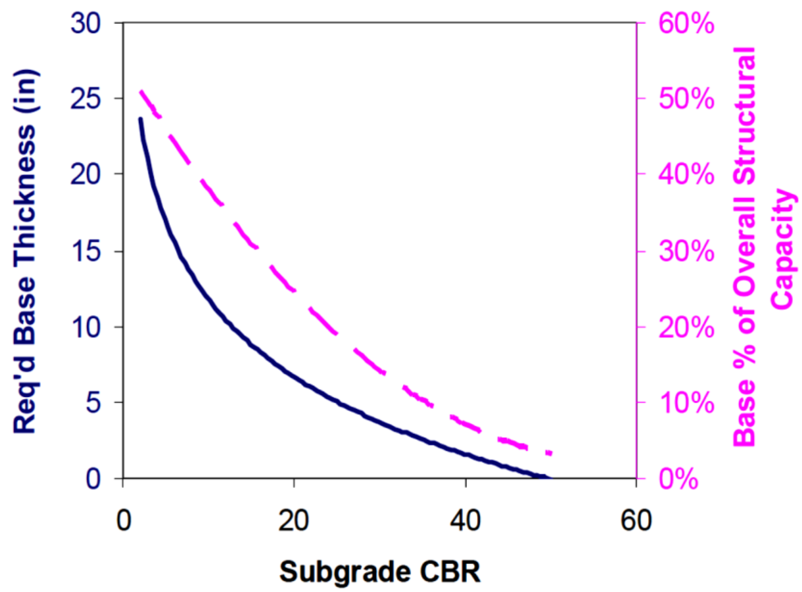

Figure 4.13. Effect of CBR value on paved structure thickness (Christopher et al., 2006)

4.2.5.4 Hydraulic Conductivity

The saturated hydraulic conductivity of the natural subgrade soil was found to be 4.6 x 10^{-6} cm/s (Figure 4.14). The changes in the coefficients of hydraulic conductivity for all combinations of materials are presented in Table 4-8, and illustrated in Figure 4.14. The results show differences in the hydraulic conductivity of the soil samples modified with plastic. The results indicate that, in general, the subgrade soil modified with plastic has a hydraulic conductivity value similar to or higher than that of the virgin subgrade. The magnitude of the plastic-induced changes in the hydraulic conductivity is a function of the plastic type and content. The pellet-modified soil has higher permeability in comparison to the flake-modified soil, which could be associated with the additional space created between soil particles with the addition of pellets. Thicker pellets, in particular, provides more room for water to pass through, as shown in Figure 4.15. From the results presented in Table 4-8 and Figure 4.14, it can be concluded that the permeability of the studied subgrade soils can be increased if the suitable amount and type of plastic is used. This would have positive impact on the pavement performance and maintenance. In pavement application, it is more preferable to have higher coefficient of permeability of unbound materials (Huang, 2004; Brockenbrough, 2012). This will enhance the drainage of the water infiltrated in the soil as well as restrict the accumulation of water within the soil medium. Water from surface infiltration and other sources increases pavement maintenance costs and shortens pavement service life (Shuler & Estakhri, 2016).

Table 4-8. Coefficients of permeability for different rates of incorporated plastic

	k, (cm/s)					
Subgrade Soil (SG)	$4.6*10^{-6}$					
Percentage of Plastic, %	1	2	3	4	5	10
SG-F-LDPE	$8.8*10^{-6}$	$3.5*10^{-5}$	$3.5*10^{-5}$	$1.4*10^{-4}$	$6.0*10^{-6}$	$2.8*10^{-6}$
SG-F-HDPE	$5.0*10^{-6}$	$4.1*10^{-6}$	$1.3*10^{-5}$	$8.0*10^{-6}$	$4.7*10^{-6}$	$5.6*10^{-5}$
SG-P-HDPE	$4.6*10^{-6}$	$5.8*10^{-5}$	$1.0*10^{-5}$	$6.9*10^{-5}$	$5.6*10^{-6}$	$5.6*10^{-5}$
SG-F-PET	$8.4*10^{-6}$	$4.4*10^{-6}$	$5.3*10^{-6}$	$7.1*10^{-6}$	$5.2*10^{-6}$	$1.3*10^{-5}$
SG-P-PET	$4.6*10^{-6}$	$6.5*10^{-6}$	$4.3*10^{-6}$	$4.7*10^{-6}$	$7.0*10^{-7}$	$3.6*10^{-6}$
SG-F-PP	$1.9*10^{-6}$	$4.5*10^{-5}$	$1.7*10^{-5}$	$8.5*10^{-6}$	$6.8*10^{-5}$	$1.3*10^{-4}$
SG-P-PP	$2.2*10^{-6}$	$3.5*10^{-6}$	$1.0*10^{-5}$	$5.7*10^{-5}$	$3.4*10^{-6}$	$4.5*10^{-6}$

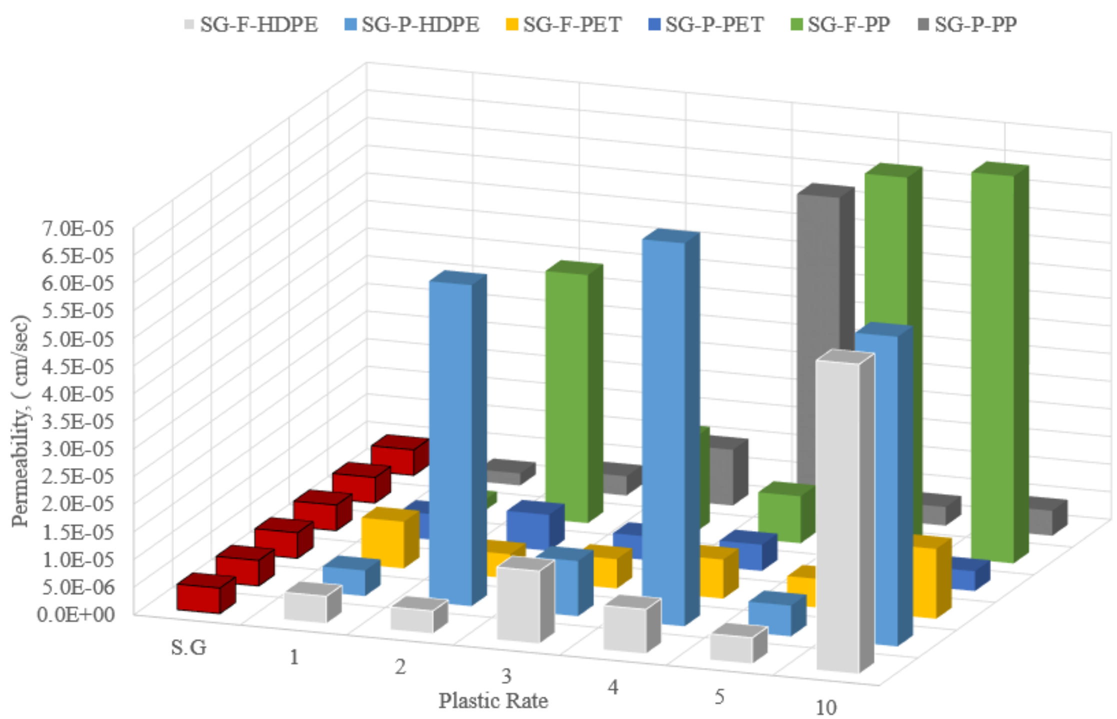

Figure 4.14. Coefficients of permeability of the investigated materials

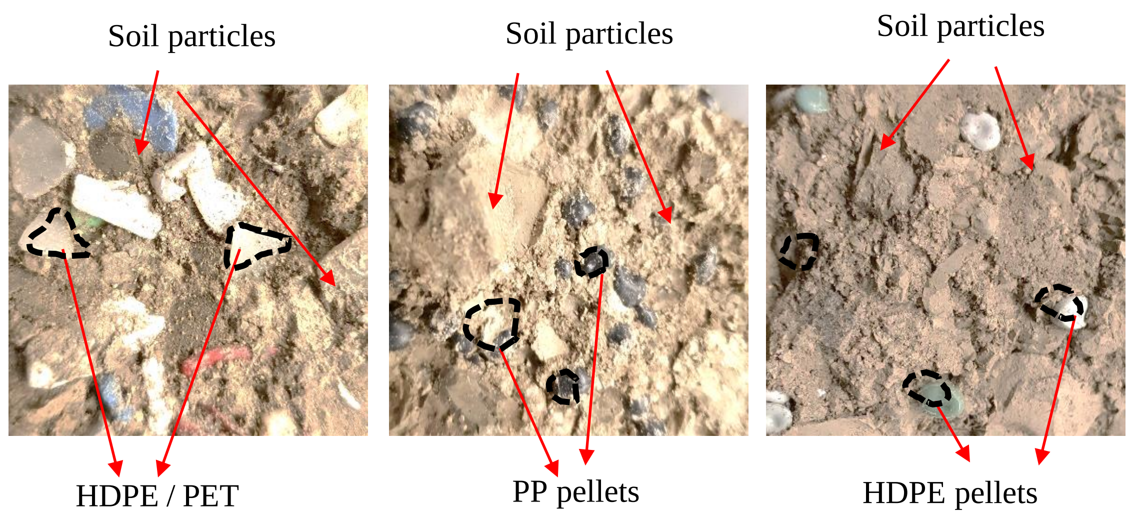

Figure 4.15. Photographic images show dispersion of plastic flakes/pellets within soil particles

4.2.5.5 Unified Compressive Strength (UCS)

Table 4-9 summarizes the results from the UCS tests. There were variations in the UCS values of the subgrade soils modified with plastic waste. Again, the shapes of the added flakes, pellets, and ground particles are reflected in the compressive strength of the soil, as also observed by Neopaney et al., (Neopaney et al., 2012). The UCS values declined with the addition of plastics of various shapes; however, it can be noted that the UCS values when using LDPE particles were, in general, considerably lower compared with using the other materials. This UCS decrease due to the introduction of plastic pieces to subgrade soil can be attributed to the fact that plastic particles that replace the soil particles result in a softer surface, leading to less internal friction and hence, lower compressive strength and bearing capacity (Jha et al., 2014). Moreover, there is no simple linear relationship between plastic content and the UCS of the subgrade. The SG-P-HDPE sample at a low content (1%) shown a relatively similar strength (1149 kPa) to the natural soil (1210 kPa). The UCS values declined at 3% plastic content for almost all types, and then showed an increase with higher plastic content before dropping at a plastic percentage of 10%. The mechanism responsible for this plastic-induced decreased in strength was already discussed above.

Table 4-9. UCS values for various plastic content and combinations with subgrade soil

	Unconfined Compressive Strength, UCS (kPa)			
Subgrade Soil, (SG)	1210.5			
Percentage of Plastic, %	1	3	5	10
SG-F-LDPE	513	471	616	625
SG-F-HDPE	961	577	875	557
SG-P-HDPE	1150	1025	963	729
SG-F-PET	676	535	292	479
SG-P-PET	696	713	738	894
SG-F-PP	920	786	550	403
SG-P-PP	954	896	839	317

4.2.5.6 Triaxial Shear Strength

The triaxial test results are presented in Table 4-10. Shear properties of natural and plastic-modified soils based on total normal (Mohr–Coulomb criterion is used to describe the shear failure envelopes). The internal friction angles varied from 29° to 48°, whereas the cohesion values ranged from 39 kPa to 69 kPa. In general, the modified subgrade soils show higher internal friction angle values than the unmodified subgrade. This behavior can be attributed to the plastic-induced reinforcement of the soil, which is due to the physical interaction between plastic and soil particles. Indeed, in the presence of plastic, there is friction at the interface between plastic and soil particles, which increases the internal friction angle of the modified subgrade (De Rezende et al., 2005). Table 4-10 indicates that most of the modified subgrade soils have lower cohesion values compared to the virgin subgrade soil. The SG-F type, out of all of the plastics used, provided slight improvements in cohesion, in contrast with LDPE, which showed otherwise. The soils modified with ground LDPE exhibited lower cohesion as opposed to other material combinations (Consoli et al., 2002). The flaked HDPE and PP samples exhibited similar results in terms of cohesiveness (59 to 69 kPa and 63 to 65.5 kPa, respectively) and were virtually identical to the cohesion of the natural soils (65.5 kPa). The lower cohesion values are attributed to the weak adhesion between the plastic and soil particles (Ali & Arulrajah, 2012). It should be underlined that, in the practice, the strength parameters (i.e., cohesion, friction angle) are not usually required or determined for unbound pavement materials. Indeed, the stresses that develop in any well-designed pavement are well below the failure strength of the unbound materials (Bin-Shafique et al., 2004). Thus, this plastic-induced decrease in the shear strength parameters (i.e., cohesion, friction angle) would not have significant practical implications with respect to the use of recycled plastic wastes in subgrade soils.

Table 4-10. Shear properties of natural and plastic-modified soils based on total normal stresses

	C, Cohesion, kPa				θ Internal friction angle, °			
Subgrade Soil (SG)	6.80				32.0°			
Percentage of Plastic, %	1	3	5	10	1	3	5	10
SG-G-LDPE	9.6	10.6	10.3	9.30	33.0°	40.0°	39.0°	36.0°
SG-F-HDPE	13.4	13.6	11.9	13.8	41.0°	40.0°	38.0°	46.0°
SG-P-HDPE	8.2	8.0	8.1	7.8	29.0°	30.0°	31.0°	29.0°
SG-F-PET	10.4	9.2	10.6	12.2	39.0°	37.0°	43.0°	38.5°
SG-P-PET	10.5	9.6	9.8	8.5	37.0°	41.0°	42.0°	38.5°
SG-F-PP	12.6	12.6	13.0	13.1	38.0°	39.0°	43.0°	48.0°
SG-P-PP	7.2	9.4	11.4	7.9	34.0°	37.0°	34.5°	31.0°

4.2.5.7 Modulus of Resilience of the Soil

The results of the correlation analyses based on equations (1) (Model 1) and (8) (Model 2) are summarized in **Error! Not a valid bookmark self-reference.** and Table 4-12. The CBR model (Model 1) revealed slightly higher moduli of resilience as compared to the physical properties model (Model 2). Results obtained from Model 2 show that the plastic does not have a significant impact on the resilient modulus of the soils. While results based on the Model 1 indicate that the plastic waste can increase or decrease the resilient modulus depending on the plastic content, shape and type, as also illustrated in Figures 12. This figure shows the moduli of resilience based on the Model 1 according to the percentages, type and shape of plastic used. It can be observed that, with the selection of a suitable type and amount of plastic, an increase in the M_R value (compared to M_R value of the virgin subgrade soil) can be achieved. Among the types of plastic, the flake form tended to provide higher M_R than the pellet form, and the ground form of LDPE seemed the weakest. This plastic shape-induced change in M_R value is likely due to the fact that the flake, pellet and ground forms have different aspect ratios (length/width). Previous studies on sand reinforced with strips of reclaimed HDPE have concluded that when the aspect ratio is small, the plastic strips are too short to develop sufficient friction at the ends of the strips. Consequently, little tensile force is generated in the strip (Sarang et al., 2014). Additional physical and microstructural tests would be necessary to better understand the fundamental mechanisms responsible for the decrease or increased in M_R values due to the addition of plastic waste to the subgrade soil.

Table 4-11**.** Moduli of resilience (M_R) for the investigated soil based on Model 1

	M_R (kPa), Model 1					
Subgrade Soil (SG)	152.7					
Percentage of Plastic, %	1	2	3	4	5	10
SG-G-LDPE	152.7	152.7	157.5	147.7	122.9	129.7
SG-F-HDPE	137.6	137.6	137.6	142.7	140.2	142.7
SG-P-HDPE	189.3	185.0	176.1	164.6	157.5	121.5
SG-F-PET	146.5	146.5	152.7	158.7	158.7	170.4
SG-P-PET	162.3	146.5	146.5	146.5	140.2	147.7
SG-F-PP	149.0	149.0	150.2	157.7	157.5	180.6
SG-P-PP	162.3	157.5	157.5	147.7	157.5	152.7

Table 4-12. Moduli of resilience (M_R) for the investigated soil based on Model 2

	M_R (kPa), Model 2					
Subgrade Soil (SG)	154.4					
Percentage of Plastic, %	1	2	3	4	5	10
SG-G-LDPE	154.53	154.60	154.52	154.59	154.50	154.48
SG-F-HDPE	154.50	154.62	154.45	154.51	154.53	154.51
SG-P-HDPE	154.73	154.67	154.60	154.63	154.52	154.59
SG-F-PET	154.64	154.57	154.63	154.59	154.66	154.69
SG-P-PET	154.67	154.59	154.44	154.52	154.63	154.64
SG-F-PP	154.55	154.59	154.55	154.65	154.56	154.61
SG-P-PP	154.47	154.56	154.44	154.54	154.66	154.55

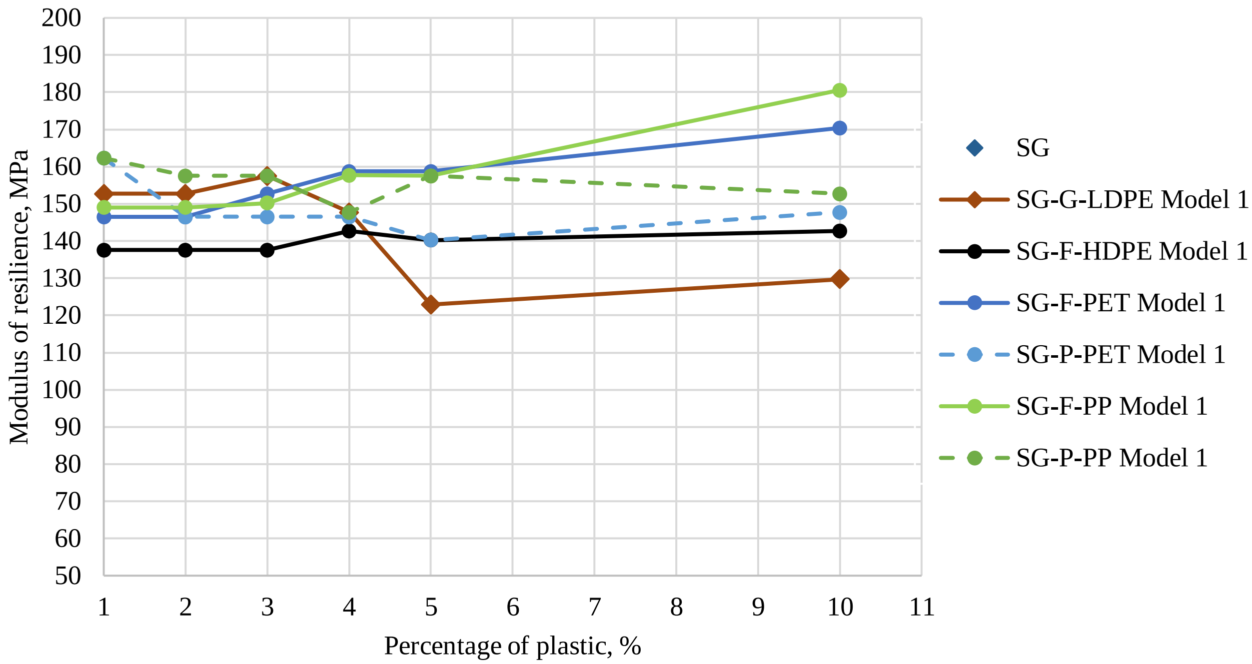

Figure 4.16. M_R for all material combinations (Model 1) (M_R of SG = 152.7)

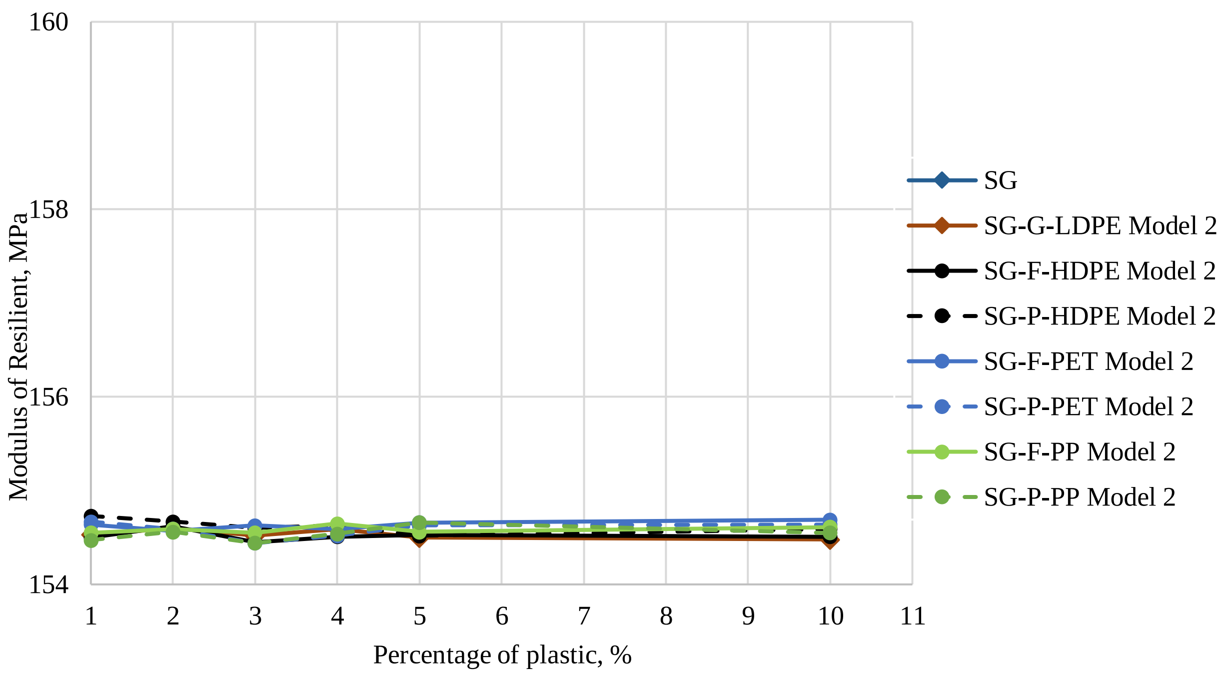

Figure 4.17. M_R for all material combinations (Correlation Model 2) (M_R of SG = 154.4)

4.2.6 Summary and Conclusions

In this paper, the results of research on the potential use of plastic wastes to partially replace soil particles in subgrade are presented and discussed. Several types (LDPE, HDPE, PET, and PP), contents (1 to 10 wt%) and shapes (ground, pellet, flake) of plastic wastes, and an A-2-7 AASHTO-subgrade soil were considered in this study. Key geotechnical properties (relevant for subgrade) of subgrade soils with different type, shape and content of plastic wastes were investigated. Based on achieved results, the following conclusions can be made:

- Compaction curves indicated that the addition of plastic wastes decreased the maximum dry densities of the subgrade due to the lower relative density of the plastic material compared to the soil particles. Moreover, the maximum dry densities decreased as the content of plastic increased, regardless of the plastic type and shape.
- Addition of plastic wastes to the studied subgrade soil changed its CBR. The nature of change (increase or decrease) and its magnitude are a function of the plastic content, shape and type. Ground, flaked, and pelleted plastics affect the strength of soils in different ways, with pellets being more influential than the other shapes. For all types and shapes of plastic, except those with SG-G-LDPE, there is a plastic content that significantly increased the CBR compared to CBR of the unmodified subgrade soil. This plastic waste-induced enhancement of the CBR values could indicate that the addition of plastic waste to subgrade soils could result in the reduction of the thickness of the paved structure, which, in turn, will result in a decrease of the cost of the road or pavement structure. The better the CBR, the less the required thickness of any pavement structure.
- Permeability values of many subgrade soil samples increased with the addition of plastic waste, whereas the permeability of some soils modified with plastic remained unchanged. The nature and magnitude of the plastic-induced changes in the hydraulic conductivity depend on the plastic type and content. The pellet-modified soil has higher permeability in comparison to the flake-modified soil. Thus, the permeability of the studied subgrade soils can be increased if the suitable amount and type of plastic waste is used. This would have positive effect on the pavement performance and maintenance.
- UCS tests showed that the addition of plastic wastes decreased the strength of the subgrade, irrespective of the plastic type, content and shape. Moreover, there is no simple linear relationship between plastic content and the UCS of the subgrade.

- Subgrade soils with plastic had higher friction angle than plastic-free soils due to the plastic-induced reinforcement of the soil. However, most of the modified subgrade soils have shown lower cohesion values compared to the virgin subgrade soil.
- In terms of resilient behaviour, plastic waste can increase or decrease the resilient modulus depending on the plastic content, shape and type. With the selection of a suitable type and amount of plastic, a MR value higher than that of the virgin subgrade soil can be obtained. Among the types of plastic, the flake form showed higher MR than the pellet form, and the ground form of LDPE appeared the weakest. This shape-induced change in MR value is likely due difference in aspect ratio.

The results of this study suggest that partial replacement of soil material by plastic waste may prove useful in road subgrade applications. However, despite the encouraging results obtained, further studies are necessary: (i) to provide a better understanding of the performance and behavior of the proposed plastic-modified subgrade materials by conducting geotechnical and microstructural tests, and at different scales; (ii) optimize the content, shape and type of plastic; (iii) assess the durability of the plastic-modified subgrade soils; (iv) determine the economic benefits that be gained through their use. Research in this area has not been accomplished for the project in this manuscript, but presently being implemented in a new project.

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4.3 Geotechnical Properties of Pavement Base and Sub-base Materials Containing Plastic Waste (Technical Paper II)

Mukhtar Abukhettala, Mamadou Fall

4.3.1 Abstract

Degradation of natural materials ensuing the production of granular materials have drawn attentions of many researchers. Several non-conventional materials have been investigated, assessed and examined in terms of quality and performance within pavement structures. Among these non-conventional constituents is plastic waste. This manuscript presents the results of an experimental study conducted to investigate the potentiality of finding a way to incorporate plastic waste within the sub-base and base-course layers of pavement structures. Four different types of recycled plastic, namely low-density polyethylene (LDPE), high-density polyethylene (HDPE), polypropylene (PP), and polyethylene terephthalate (PET), were deployed. These four types of recycled plastic were in three forms (ground, flakes, and pellets) and added at varying rates of 1%, 5%, and 10%. The physical properties of the granular type A, and granular B type II were examined before proceeding with a series of geotechnical tests after incorporating the plastic. The results revealed a decline in the optimum moisture content and its corresponding maximum dry density for all types of plastic. The results also showed a noticeable improvement in strength of some base and sub-base materials modified with plastic despite variation with the shape of utilized plastic. The permeability was reduced, which is favorable to protect pavement subject to moisture. All in all, the pellets brought higher improvements compared to the ground and flake forms of plastic.

Keywords: Pavement; base-course; sub-base; strength; road; plastic waste

4.3.2 Introduction

A pavement structure is essentially a multi-layered system in which the top layer is subjected to the most intensive loads from the anticipated traffic. The applied load is transmitted through the multi-layered structure to the underlying formation layer. The goal is to absorb the load intensity and reduce it and to an acceptable magnitude that the underlying layers can safely endure. This multi-layered system is designed such that the concentrated load on the top surface gradually transfers downwards to the base, thus allowing for using lower-quality materials in the lower layers (Huang, 2004).

Structurally, the pavement is fundamentally classified into two main types: flexible structures and rigid structures. Flexible pavement is commonly favored over rigid pavement for several reasons, including flexibility, cost and material availability. Traditionally, a typical pavement structure encompasses several layers stacked over one another. These layers are the sub-grade, sub-base, base-course, and surfacing layers. The sub-grade layer is formed of a natural or imported soil of an adequate quality material. However, sub-base and base materials are most often a composition of different mineral particles packed together when compacted to form a strong skeleton within any pavement structure. The load-carrying function is heavily dependent on the upper base and sub-base layers. The performance of any road structure relies heavily on the mechanical properties of the ingredients of all layers within the structure. Thus, it is crucial to furnish the structure with quality materials that can sustain the applied loading.

A substantial amount of construction materials from various sources, including natural resources, are consumed in the construction of roads as well as in rehabilitating deteriorated road segments every year. In recent years, endeavors have been put forward to explore alternatives to conventional materials without compromising the quality, the performance, and ultimately the anticipated level of endurance over the design period of any road pavement (Christopher et al., 2006).

Sourcing for alternatives to traditional materials in the road construction industry has been driven by many factors. This includes shortages in the supply of traditional materials and the environmental damages inflicted on habitats from exploring and processing these raw materials. In recent years, there has been a shift towards exploiting various secondary materials and by-products through substitution of natural materials with potentially viable secondary or tertiary materials. Rezende et al. (2015) defined secondary materials as materials which possess quality-related properties as per the performance requirements for pavement materials in road structure applications (Rezende et al., 2016).

A plethora of potential materials have been investigated in recent decades for use in road pavement. Such potential materials include but are not limited to reclaimed asphalt pavement (RAP), crushed glass (CG), foundry sand (FS), mine waste (MW), pumice waste (PW), and reclaimed concrete aggregates (RCA) (Al-Qadi et al., 2007; Saltan & Selcan Findik, 2008; Reddy et al., 2014; Arulrajah et al., 2014; Yazoghli-Marzouk et al., 2014; Xuan et al., 2015; Agyeman & Ampadu, 2016). Mine rock waste was examined in Ghana and considered of great potential for base or sub-base applications (Agyeman & Ampadu, 2016). In addition, fine recycled glass was used with recycled concrete aggregates in Australia, and improvements in base-course properties were been reported (Arulrajah et al., 2014). Interestingly, cigarette butts have also recently been considered as a modifier to asphalt bitumen (Rahman et al., 2020) .

However, literature on exploiting plastic waste within road structures is sparse and limited. Moreover, most of previous studies on incorporating plastic waste into pavement layers, focused on exploiting the plastic as bonding agents in the surfacing layers, either to modify the physical properties of asphalt binders or replacing the binders altogether (e.g., Vasudevan et al., 2012; Sarang et al., 2014; Sojobi et al., 2016). Due to the paucity of studies on the use of plastic wastes in sub-base or base materials, an understanding of the effect of plastic wastes on the performance, particularly on the geotechnical properties, of sub-base and base materials is limited. There is a need to address this knowledge and research gap. Exploitation of plastic waste within road sub-base and base systems would be of great benefit, not only environmentally, but also economically.

Indeed, tons of plastic debris are thrown away each year, everywhere, polluting lands, rivers, coasts, beaches, and oceans. Every year, 8 million metric tons of plastic end up in our oceans, and in 2025, the annual input is estimated to be about twice greater according to UC Santa Barbara’s National Center for Ecological Analysis and Synthesis (NCEAS) (Plastic Oceans Foundation, 2017). Moreover, according to The World Economic Forum, plastic production has significantly increased over the last half-century, growing from 16.5 million tons in 1964 to 343 million tons in 2014; it is projected to double by 2036 (Word Economic Forum, 2016). It has been reported that nearly 50% of this plastic is sent to landfills (Rochman et al., 2013). In Canada, nearly 90% of plastics end up incinerated, or in our landfills, lakes, parks and oceans (Environmental Defense Canada, 2020). Thus, alleviating the amount of uncontrolled plastic waste through more viable options, such as incorporation of plastic waste into road base or sub-base materials, would help lessen the repercussions to the environment as well as reduce the material cost in road construction (Jambeck et al., 2015).

Considering the facts mentioned above, a research program has been carried out at the University of Ottawa to investigate the feasibility of incorporating plastic waste within the sub-base and base-course layers of pavement structures. The key objective of this paper is to present and discuss the impact of various percentages and types of recycled plastic of different shapes on the geotechnical properties of base and sub-base granular materials. The granular materials tested include the granular B type II for sub-base plastic application and the granular A plastic for base-course applications as per Ontario Provincial Standard Specifications (Ontario Provincial Standard Specification, 2003).

4.3.3 Materials and Methods

4.3.4 Materials

4.3.4.1 Plastic waste materials

Four types of plastic materials were used: the ground low-density polyethylene (LDPE) type, the high-density polyethylene (HDPE), the polyethylene terephthalate (PETE), and the polypropylene (PP). They are in ground, flakes and pellets forms, as supplied by two plastic waste recycling centers: Canada Fibers Ltd., located in Toronto Ontario, Canada, and Enviroplast Inc., in Montreal Quebec, Canada. These recycled materials are recycled plastics that have undergone cleaning, sorting, and eventually cut into chips/flakes by shredding or made into pellets by the process of extrusion at the facility.

Physically, pelleted HDPE and PET types are cylindrical in shape and are differentiated by thickness. While PP pellets are spherical, the flakes of all types are triangular with a 7.0mm base, 8.8mm height, and 0.7mm thickness. However, ground LDPE plastic has irregular rounded shape of 3-5mm. The physical characteristics of the utilized types are illustrated in Figure 4.18.

a) Pelleted PP b) Pelleted HDPE c) Flakey HDPE, PETE

Figure 4.18. Types and physical characteristics of used flakes and pellets

4.3.4.2 Granular sub-base and base materials

Two types of granular material were employed in the current study. The first was granular B type II and the second was granular type A, as per OPSS (Ontario Provincial Standard Specification, 2003). The granular B type is selected for the sub-base applications and granular A type is used in the base-course applications in the province of Ontario, Canada. Both granular materials are originated from granite and are produced by crushing at the quarry and supplied by Lafarge company in Gatineau, Quebec (Canada).

The physical properties of both materials are relatively similar except for the fact that the base-course material is sturdier. Any material that is intended for base-course applications needs to have higher load-sustaining properties because the load intensity transferred from the surface layer is always more intense, as opposed to the lower sub-base layer (Huang, 2004).

Figure 4.19. (a) Production of the granular material at the quarry, and (b) the as-received granular material used for the investigation

4.3.4.2.1 Sub-base material

The sub-base material, granular B type II, is an open grading that lacks few particles sizes and it is coarser than granular base-course type A material. The percentage passing through sieve size 26.4mm ranges from 50 to 100% for the Sub-base and 100% passing for the base-course material (Table 4-14, Figure 4.21).

4.3.4.2.2 Base-course material

The type A granular is dense-graded and is finer than the sub-base material. The percentage of material passing sieve size 26.4 mm was 100%, and the fine material passing the number 200 (0.075mm) sieve ranged between 2 and 8% inclusively (Table 4-14, Figure 4.22).

The following schematic diagram presented in Figure 4.20 shows the structures of both materials. The granular A type is more structurally homogenous. It is a well graded material with higher contact points between the particles, which makes it bond more firmly together as a unit.

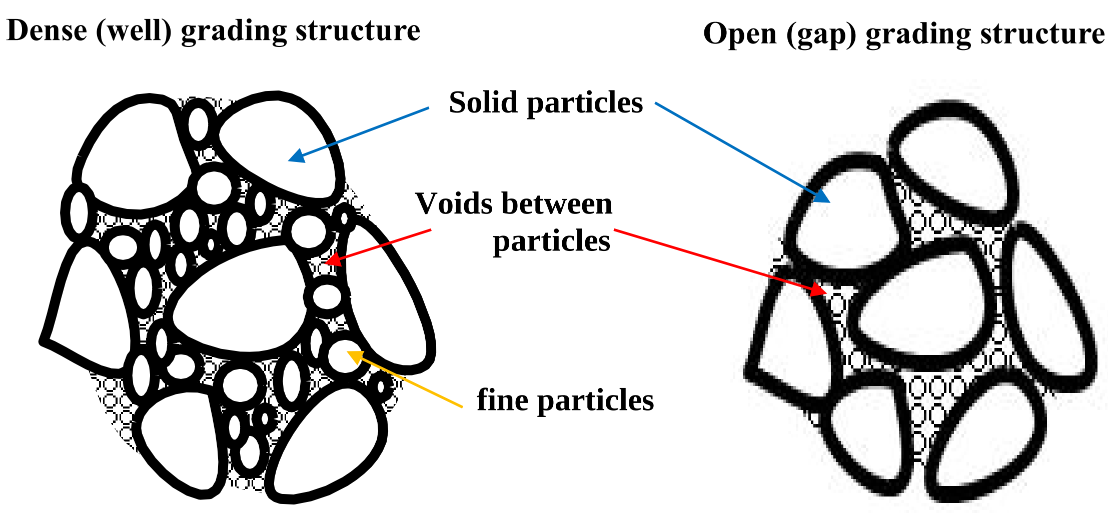


Figure 4.20. Schematic diagram of dense and open grading aggregates

4.3.5 Sampling and characteristics of the granular materials

4.3.5.1 Sampling

In order to obtain representative samples of the materials with a minimal distortion, a proper sampling is required. The quartering approach was adopted and conducted in accordance with the ASTM C702 specification standard (ASTM, 2011). In this procedure, the material is spread over a flat surface area and divided into four quarters, with each two opposed quarters subsequently mixed together and then compiled and split again. The process continues until the required weight for each sample is obtained.

4.3.5.2 Physical properties of used granular materials

Both granular materials were assessed for compliance prior to their utilization in the investigation process. Preliminary tests were conducted to assess physical properties. These tests were done either in accordance with OPSS or in their equivalents from the American Society for Testing and Materials (ASTM), or the American Association of State Highways and Transport Officials (AASHTO) standards.

Furthermore, consistency limits were determined through Atterberg testing. This was conducted as per the ASTM D-4318 specification standard (ASTM, 2005). Both the sub-base and base-course materials had non-plastic characteristics as per the test requirements. The water content of the received materials, as per ASTMD-2216 (ASTM, 2010), was 12.9% and 15.2 % for the sub-base and base-course, respectively, and the maximum dry densities, as per ASTM D-2216, for the sub-base and base materials were 2.880 g/cm^3 and 2.870 g/cm^3 at 7.0% and 7.3%, respectively. The obtained results are illustrated in Table 4-13.

Table 4-13. Physical properties of the sub-base and base-course granular materials

Properties	Granular B Type II, Sub-base	Granular A, Base
Liquid Limit, L.L %	23.2	19.5
Plastic Limit, P.L%	0	0
Plastic Index, P.I%	N.P*	N.P
Passing #40, %	18.2	17.8
Gravel content (Retaining #50), %	83.1	86
Sand content (Passing #50 - Retaining #200), %	12.6	8.2
Fine content (Passing #200), %	4.3	5.8
Water Content, W.C%	12.9	15.9
Nominal Maximum aggregate size, mm	106	26.4

**N. P refers to nonplastic term*

4.3.5.3 Gradation analysis

Particle size distribution analysis was conducted to verify the gradations of the granular materials under investigation. A total of three samples from the sampling quartering were obtained, tested, and then analyzed. The average gradation was computed and graphically plotted and are shown in Figure 4.21 and Figure 4.22.

The amount of fine material passing the 0.075mm sieve size, at 4.3%, was relatively less than for the granular A samples, which was 5.8%. However, both materials fell within the grading envelopes of the OPSS requirements, and subsequent tests proceeded.

Table 4-14. Grading analysis of average samples of granular B type II and granular A material

Sieve Size		Percentage Passing, %	
Inch	mm	Granular B Type II Sub-base	Granular A Base
5.91	150	-	-
4.24	106	100	-
1½	37.5	-	-
1.0	26.4	82.8	100
¾	19	-	92.6
½	13.2	-	84.7
⅜	9.5	-	69.3
No.4	4.75	34.0	41.0
No.16	1.18	23.1	23.8
No.40	0.425	18.2	17.8
No.50	0.3	16.9	14.0
No.100	0.15	-	-
No.200	0.075	4.3	5.8

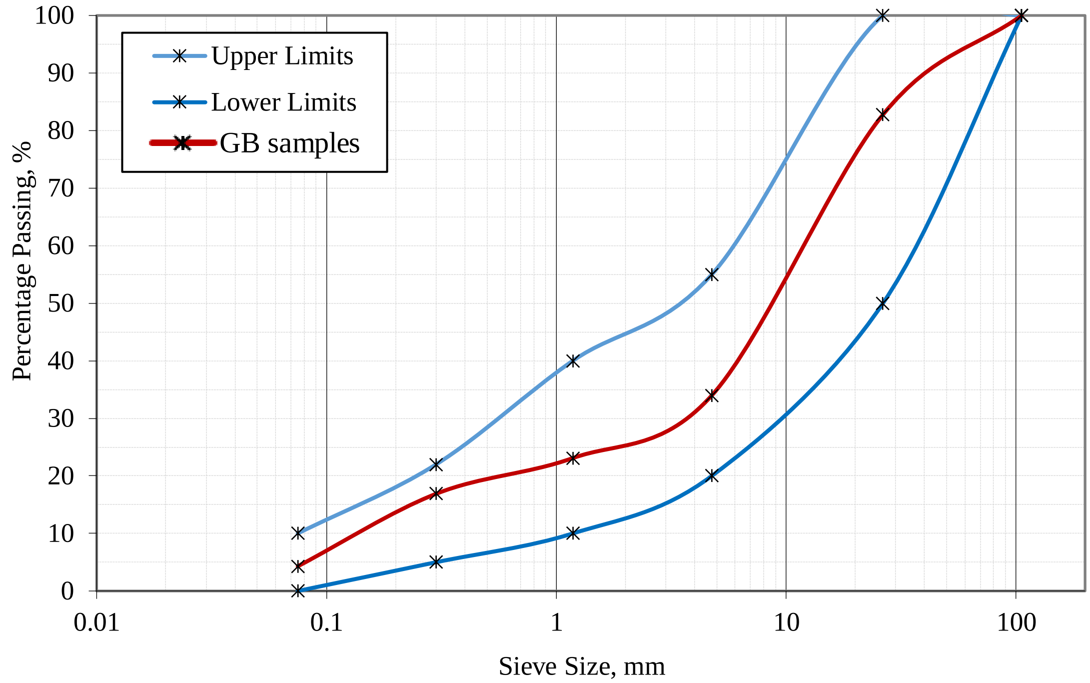


Figure 4.21. Gradation curve of the investigated granular B type II material

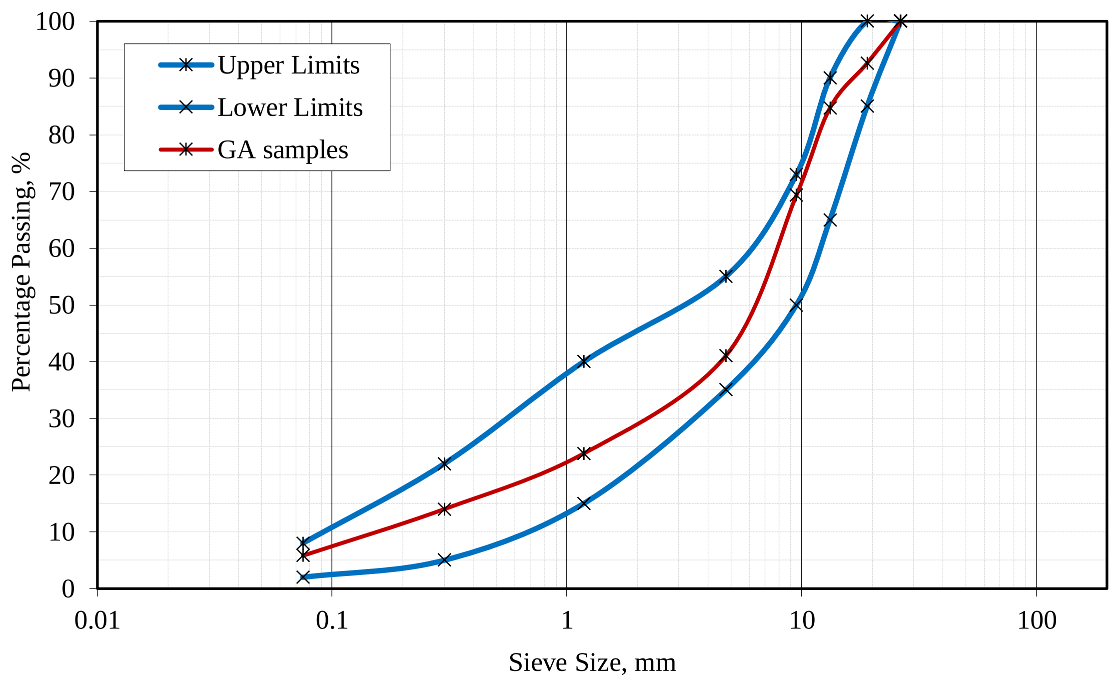


Figure 4.22. Gradation curve of the investigated granular A material

4.3.6 Sample preparation

The rates at which the plastic material was incorporated were 1% (weight proportion), 5%, and 10% for each combination during the compaction tests. Mixing the granular material together with the plastic was accomplished manually, as well as adding the needed moisture content for testing. This process of mixing continued until thorough mixing was visually observed, which was in an average of 2 minutes for most combinations. The specimens were then left to stand for an average period of 1 hour in a sealed pail before proceeding with the experimental work to preserve the incorporated moisture contents.

4.3.7 Testing and estimation methods

The work conducted includes several experiments, such as the determination of compaction properties, strength and hydraulic conductivity as well as estimating the resilient modulus. For each of the following testing procedures, each test was repeated at least twice, and the average value is considered and reported, if the difference between the two obtained values is less than 5%. All testing experiments are discussed below.

4.3.7.1 Compaction test

The testing was conducted either in accordance with the standard or modified proctor compaction (ASTM, 2007; ASTM, 2012). The standard method has evolved over time with the demand from heavier traffic volume and anticipated imposed stresses on pavement structures. Thus, modified proctor compaction was adopted for the sake of this research work. In the testing approach, a 6-inch steel mould was used instead of the 4-inch mould for the sub-grade soil. The material was compacted in five layers, with a total number of 56 blows per each layer. Once the compaction was done, moisture content samples were taken to evaluate the actual water content during the compaction process. The relations of varied water content against the achieved dry densities are then plotted to acquire the maximum dry density for the granular material and its combination with the plastic waste. The specimens were identified with labels for the ease of handling during the testing procedure and for analyzing the results. These labels are summarized below in Table 4-15.

Table 4-15. Labels for tested materials and material combinations

		Combination	Granular B type II Sub-base layer	Granular A Base layer
Granular materials		**GB for "granular B type II" Sub-base material, GA for "granular A" base-course material**		
Plastic Type	**Plastic Shape**	**Composition**	**Combination with GB**	**Combination with GA**
LDPE	Ground	with 1% ground LDPE	GB-G-LDPE-1%	GA-G-LDPE-1%
		with 5% ground LDPE	GB-G-LDPE-5%	GA-G-LDPE-5%
		with 10% ground LDPE	GB-G-LDPE-10%	GA-G-LDPE-10%
HDPE	Flakes	with 1% Flakey HDPE	GB-F-HDPE-1%	GA-F-HDPE-1%
		with 5% Flakey HDPE	GB-F-HDPE-5%	GA-F-HDPE-5%
		with 10% Flakey HDPE	GB-F-HDPE-10%	GA-F-HDPE-10%
	Pellets	with 1% pelleted HDPE	GB-P-HDPE-1%	GA-P-HDPE-1%
		with 5% pelleted HDPE	GB-P-HDPE-5%	GA-P-HDPE-5%
		with 10% pelleted HDPE	GB-P-HDPE-10%	GA-P-HDPE-10%
PET	Flakes	with 1% Flakey PET	GB-F- PET-1%	GA-F- PET-1%
		with 5% Flakey PET	GB-F- PET-5%	GA-F- PET-5%
		with 10% Flakey PET	GB-F- PET-10%	GA-F- PET-10%
	Pellets	with 1% pelleted PET	GB-P- PET-1%	GA-P- PET-1%
		with 5% pelleted PET	GB-P- PET-5%	GA-P- PET-5%
		with 10% pelleted PET	GB-P- PET-10%	GA-P- PET-10%
PP	Flakes	with 1% Flakey PP	GB-F- PP-1%	GA-F- PP-1%
		with 5% Flakey PP	GB-F- PP-5%	GA-F- PP-5%
		with 10% Flakey PP	GB-F- PP-10%	GA-F- PP-10%
	Pellets	with 1% pelleted PP	GB-P- PP-1%	GA-P- PP-1%
		with 5% pelleted PP	GB-P- PP-5%	GA-P- PP-5%
		with 10% pelleted PP	GB-P- PP-10%	GA-P- PP-10%

4.3.7.2 Determination of the California bearing ratio (CBR)

The ability of a material to sustain deformation or resist breaking due to applied mechanical forces is referred to the strength in mechanics of materials. In the sub-base and base-course materials in pavement applications, the most commonly utilized approach to assess the strength is by experimentally evaluating its California bearing (CBR) ratio (Huang, 2004). The CBR percentage represents the resistance of a compacted material to the penetration of a plunger of 50mm diameter at certain penetration values. The CBR test is conducted according to the ASTM D-1883 specification standard (ASTM, 1999). The specimens were compacted in the 6-inch cylindrical mould in five layers, with 56 blows in total for each layer. The penetration measurements were recorded at 0.5, 1, 1.5, 2, 4, 5, 7.5, 10, and 12.5mm, and corresponding load readings were taken. Plotting the penetration and load readings together resulted in obtaining the CBR values at penetration depths of 2.5mm or 5.0mm.

4.3.7.3 Hydraulic conductivity

In order to assess the saturated hydraulic conductivity of the investigated materials, permeability testing was conducted by employing the constant head method. The testing procedure complied with the ASTM D-2434 specification standard for granular materials (ASTM, 2006b). In this approach, the discharge of a specific quantity of water is measured over time, and the heads' heights are simultaneously recorded. The discharge elapse time is noted, and the procedure is repeated a minimum of three times. The average measurement for the three recorded values is then reported. The testing water temperature was always close to 23°C.

4.3.7.4 Determination of resilient modulus

The capability of any pavement structure material to recover from deformation caused by traffic loading is attributed to the elastic property of the material. In pavement work applications, it is prudent to evaluate this property for all incorporated materials for which a depression in the surface of the pavement might affect its performance. Deformation is a major defect primarily in flexible pavements. It either occurs in the underneath base or sub-grade layer or in the top surfacing asphalt layer (Huang, 2004).

Resistance to deformation is assessed by determining the modulus of resilience of the materials. The modulus of resilience is key in the design approaches for pavement structures and is defined as the ratio of deviator stress to the recoverable strain after a serious of cyclic loading (Li and Selig, 1994; Kang et al., 2011). Laboratory testing of a modulus of resilience is time-consuming and expensive. Therefore, numerous empirical models have been developed and employed to predict the moduli of resilience of soil and granular materials alike. Many models are widely accepted, recognized, and proven to be reliable (Drumm et al., 1990). Thus, implementing some of the afore-mentioned models was a viable choice for determining the moduli of resilience in this research.

Three CBR-dependent empirical models have been selected for this research to estimate the M_R values of the investigated materials, and the three models are given in Equations 1 to 3 below. While the first model (Model 1) represents a numerical equation proposed by the Mechanistic-Empirical Pavement Design Guide (MEPDG) (Research Program Transportation Research Board National, 2004), the second (Model 2) is an outcome of the research work conducted by the United States Army Corps of Engineers in 1975 (Abu El-Maaty Behiry, 2014), and the last one (Model 3) is a prediction equation developed by the Industrial Research Council of South Africa (Abu El-Maaty Behiry, 2014).

$$M_R \text{ (psi)} = 2555 \times CBR^{0.64} \quad (1)$$

where the CBR is substituted in percentage,

$$M_R \text{ (psi)} = 5409 \times CBR^{0.71} \quad (2)$$

where the CBR is substituted in percentage,

$$M_R \text{ (psi)} = 3000 \times CBR^{0.65} \quad (3)$$

where the CBR is substituted in percentage.

4.3.8 Results and Discussions

4.3.8.1 Compaction test results

The results of the compaction tests are summarized in Table 4-16 and presented consequently in Figure 4.23 and Figure 4.24. Table 4-16 provides a summary of the obtained maximum densities and the corresponding optimum moisture contents for various material combinations. From this table, it can be noticed that the computed values of the maximum dry density (MDD) and optimum moisture content (OMC) had a relatively close pattern over the set of rate and type of plastic materials mixed with the sub-base and base materials (Table 4-16). However, regardless of the type of granular materials (A or B) and characteristics (type, shape, content) of plastic considered, the MDD values of the plain base and sub-base materials are higher than those of the granular materials with plastic material. While the MDD of the plain sub-base was recorded at 2.880 gm/cm^3 at 7.0 OMD, the plain base-course materials gave a value of 2.870 gm/cm^3 at a slightly higher OMC percentage of 7.3%. This decrease of the MDD value induced by the addition of plastic waste is attributed to the fact that plastic material has a relative density (~1.0-1.5) that is much lower than that (~2.6-2.8) of the main mineral particles (quartz, feldspar) the granular materials, A, B, are made of. Thus, the low density of plastic granules was likely responsible for the behavior (Chowdhury et al., 2013). Indeed, the plastic granules, flakes, and pellets have replaced some soil mineral particles in the compacted granular material modified with plastic waste. Consequently, the plastic granules partially occupy spaces (voids) between the aggregate particles, as illustrated in Figure 4.25, which in turn decreases the density of the compacted material. This figure schematically illustrates the interactions between soil aggregate particles and the added plastic of two different types, flakes and pellets (note, the illustrations depict the well graded granular material A only).

Table 4-16. Summary of obtained maximum densities (MDD) and the corresponding optimum moisture content (OMC) for various material combinations

Property	Optimum Moisture Content (OMC), %			Maximum Dry Density (MDD), g/cm^3		
Combination	**1%**	**5%**	**10%**	**1%**	**5%**	**10%**
GB		7.0			2.880	
GA		7.3			2.870	
GB-G-LDPE-	9.0	9.0	8.0	1.990	1.985	1.688
GA-G-LDPE-	7.6	7.8	7.3	1.887	1.925	1.910
GB-F-HDPE-	3.5	3.5	3.3	1.980	1.963	1.913
GA-F-HDPE-	6.3	6.7	6.9	1.996	1.892	1.884
GB-P-HDPE-	3.8	3.8	3.9	1.945	1.930	1.915
GA-P-HDPE-	6.2	6.2	7.1	1.878	1.910	1.890
GB-F-PET-	8.5	8.5	7.7	2.080	2.075	1.885
GA-F-PET-	7.6	7.7	7.3	1.996	2.015	1.982
GB-P-PET-	5.9	5.9	6.4	2.040	2.025	1.950
GA-P-PET-	6.9	6.8	7.2	2.005	2.100	2.018
GB-F-PP-	8.3	8.3	6.3	2.030	2.035	1.860
GA-F-PP-	7.6	7.7	7.4	2.020	2.014	2.000
GB-P-PP-	8.0	8.0	6.0	2.065	2.055	1.890
GA-P-PP-	7.7	7.8	7.2	2.012	2.022	2.102

It is also clear from the results of the compaction tests that the plastic content influences the compaction characteristics (MDD, OMC) of the sub-base and base materials with plastic wastes. Figure 4.23 and 24 show typical examples of the impact of the shape, content and type of plastic on the compaction curves and characteristics of the granular B and A materials, respectively. The comparative analysis of these figures indicates that the compaction behaviours of the base (granular A) and sub-base (granular B type II) materials show relatively similar pattern. Moreover, in general, the higher the rate of plastic incorporated in the combined mixture, the lower the obtained maximum dry density became (Figure 4.26 and Figure 4.27), and similarly, the lower the optimum moisture content (except for samples, HDEP flakes and pellets, and PET pellets) (Figure 4.28 and Figure 4.29) developed. In practice, lower maximum dry density would mean less compaction energy is required to control the in-situ degree of compaction for pavement applications (Christopher et al., 2006; Delatte, 2008; Opus International Consultants Limited, 2016). This decrease of the optimum moisture content with higher rate of plastic can be attributed to the following factors: (i) plastic material does not absorb water unlike natural aggregates that constitute the sub-base and base materials (Nsaif, 2013); (ii) plastic pieces take the place of some voids in the compacted granular materials.

However, the achieved results show that, for a given type of granular material and plastic content, the type and shapes of the plastic used have only minor effect on the dry density of the compacted sub-base and base materials.

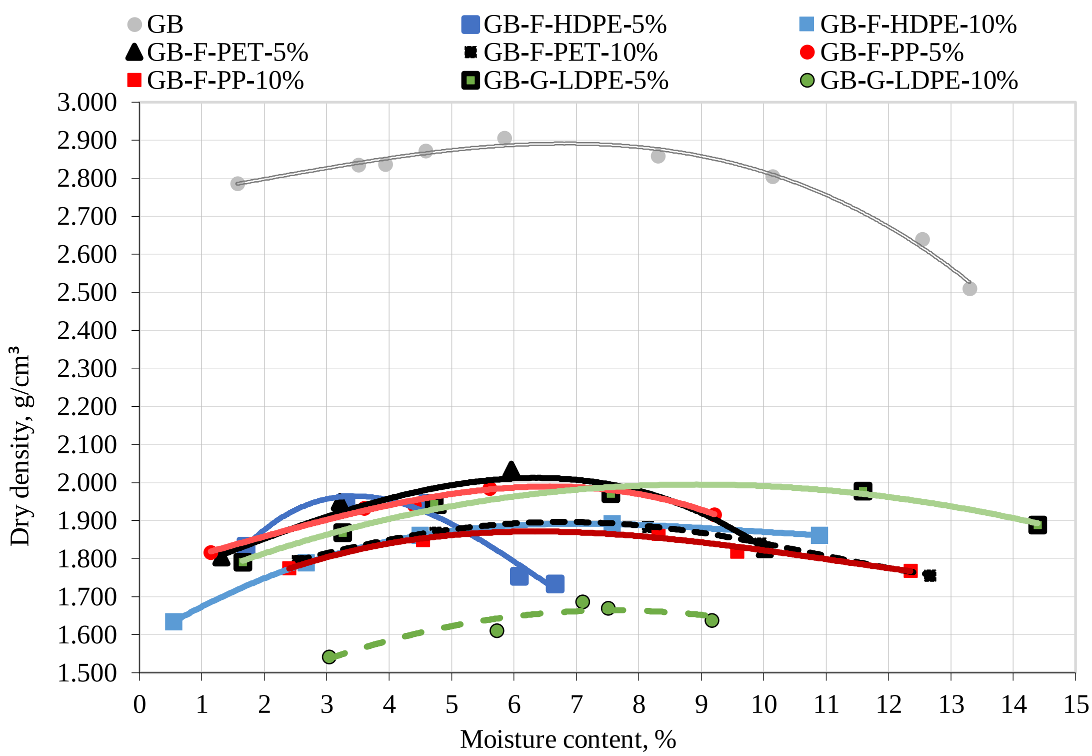


Figure 4.23. Changes in compaction properties for flaky types with granular B material

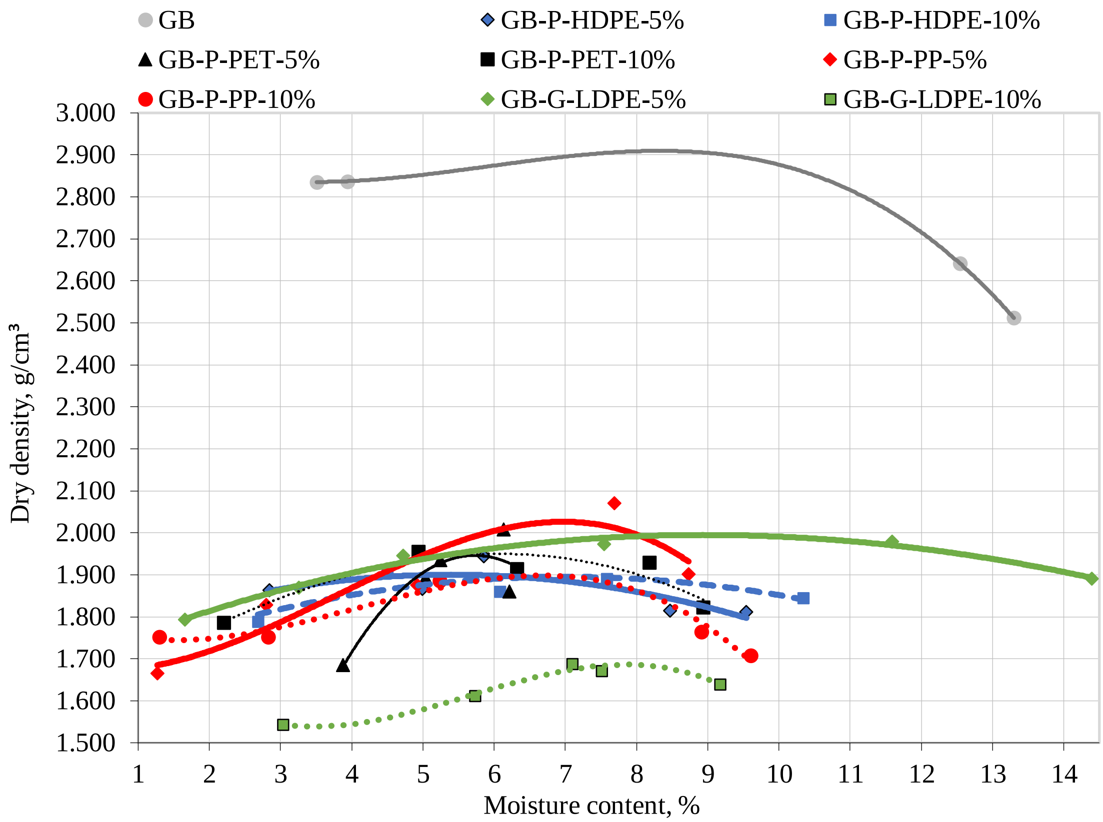


Figure 4.24. Changes in compaction properties for pellet types with granular B material

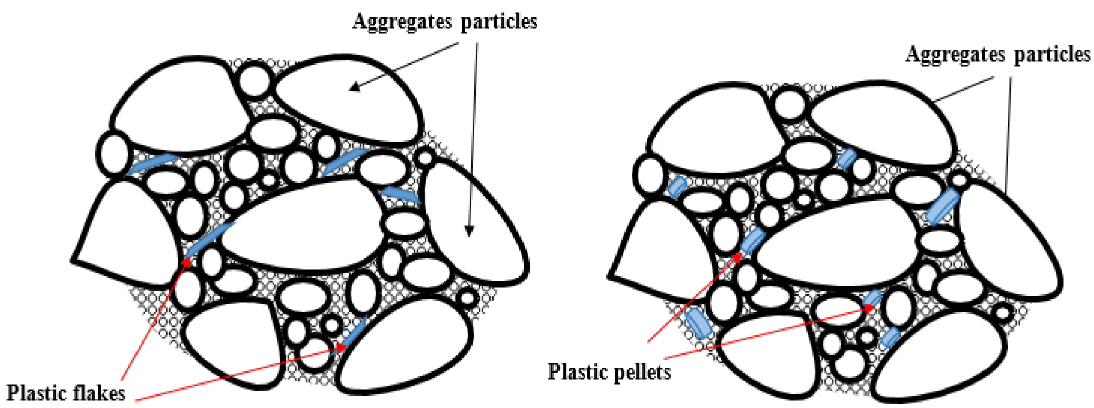


Figure 4.25. Spaces occupied by plastic chips and pellets in the aggregate structure

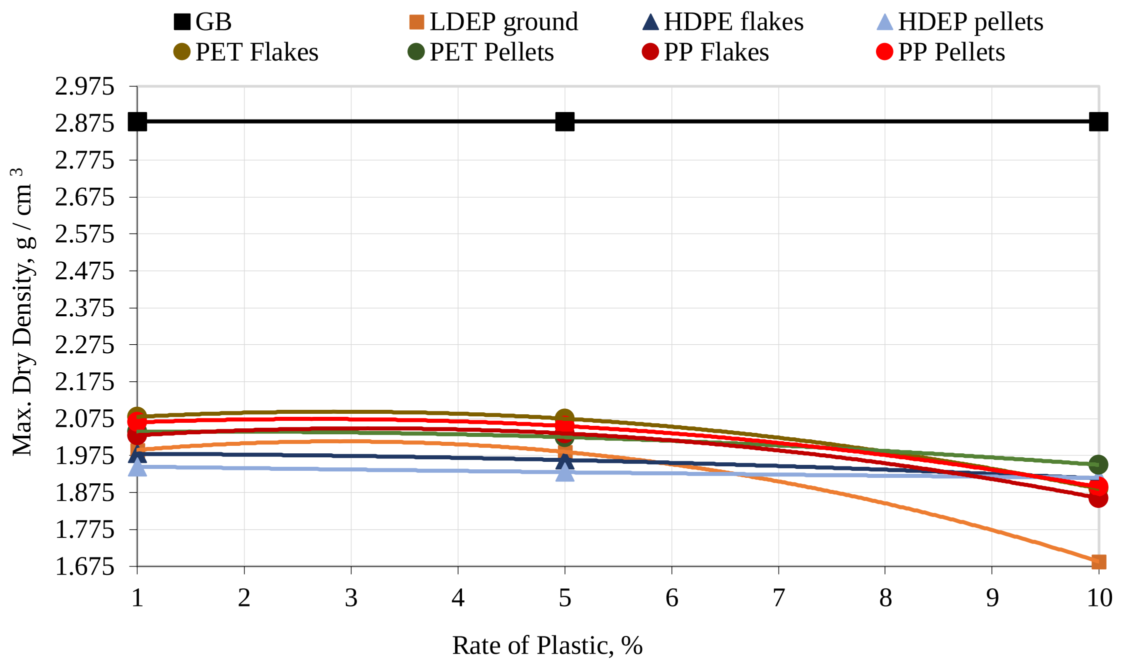


Figure 4.26. Change in maximum dry densities with change in rate of plastic used with granular B type II material

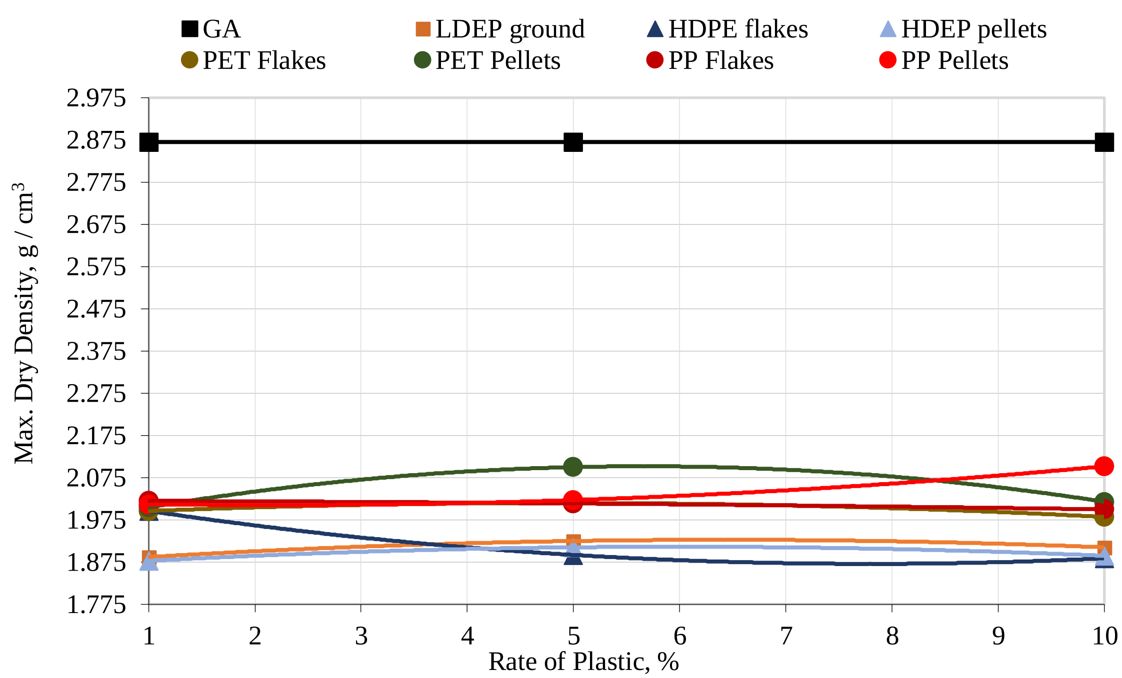


Figure 4.27. Change in maximum dry densities with change in rate of plastic used with granular A type material

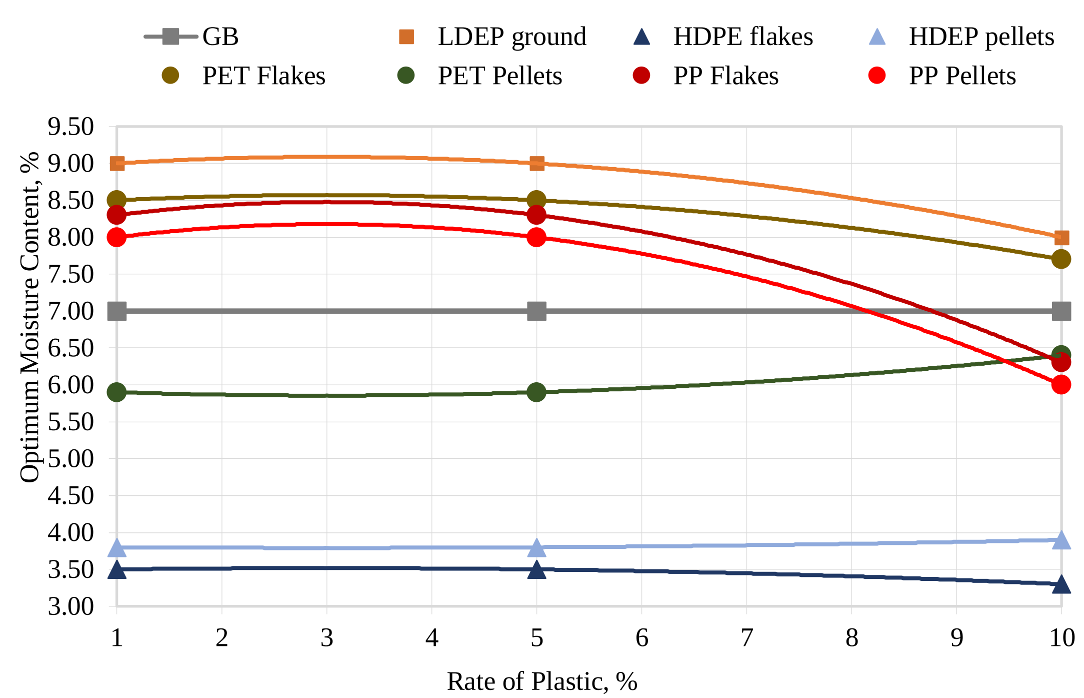


Figure 4.28. Change in OMC with change in rate of plastic used with granular B type II material.

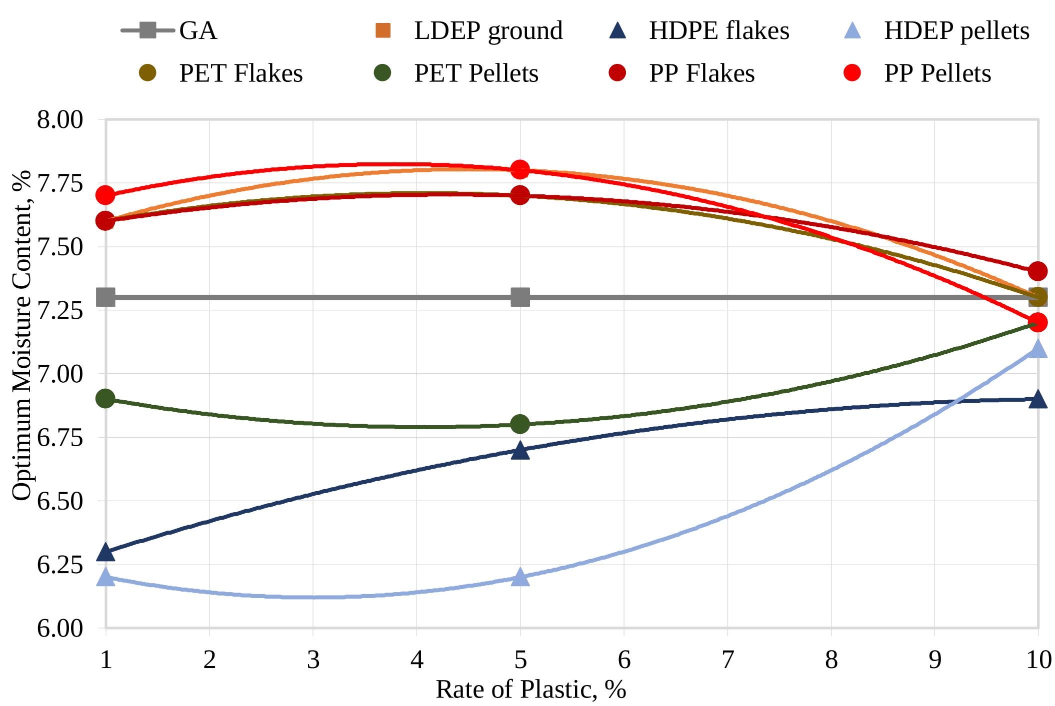


Figure 4.29. Change in OMC with change in rate of plastic used with granular A material

4.3.8.2 CBR strength

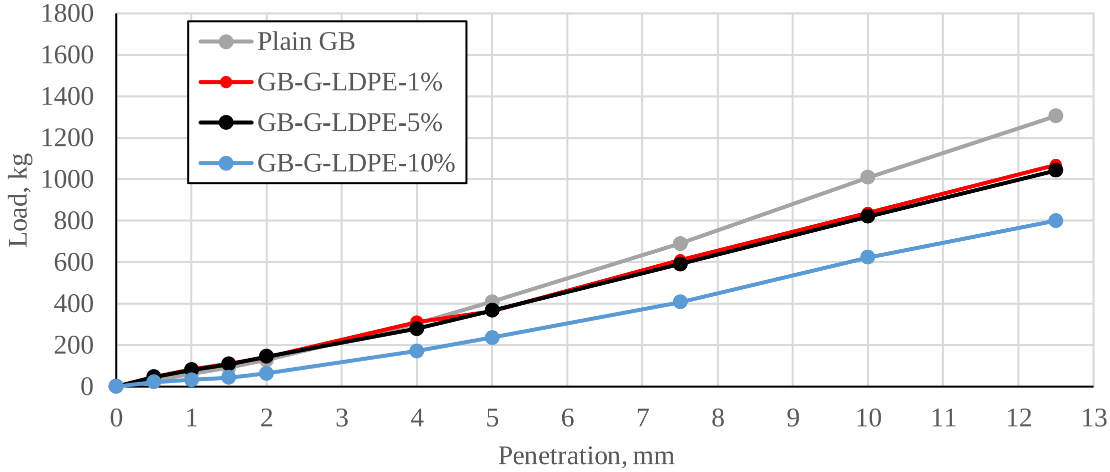
The results of the conducted CBR tests are summarized in Table 4-17. CBR values of 23.4% and 41.4% for the plain sub-base and base-course materials, respectively were obtained. The CBR values of the sub-base and base materials that incorporate plastic waste varied between 9.7 to 48.7 and 38.9 to 56.0, respectively. Table 4-17 also indicates that, for a given plastic type and shape, the addition of plastic to the granular material can result in an increase of the CBR of the sub-base or base material. For all types and shapes of plastic (except for GB-G-LDPE) there is a plastic content that moderately or significantly increased the CBR compared to CBR of the unmodified sub-base or base materials. However, the CBR value and plastic content is not a simple linear relationship. The CBR increase due to the presence of a specific (optimal) proportion of plastic in the granular material is related to the reinforcing ability of the plastic. When an optimal amount of plastic is added, the plastic particles are easily dispersed throughout the sub-base or base materials, which creates a potential for increased CBR values (Dhatrak and Konmare, 2016). Previous studies concluded that the elongation effect on the frictional force at the interface between the particles of the granular material and plastic material plays a role in enhancing soil strength (Choudhary et al. 2014).

Table 4-17. CBR values for all combinations

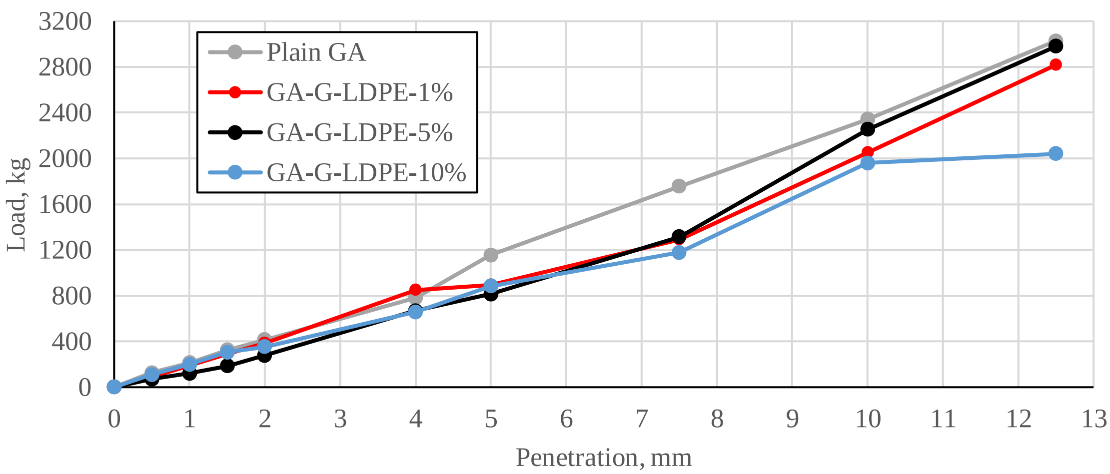
CBR, %							
	Granular B Sub-base				**Granular A Base**		
GB samples	**23.4**			**GA samples**	**41.4**		
% Plastic	**1%**	**5%**	**10%**	**% Plastic**	**1%**	**5%**	**10%**
GB-G-LDPE-	17.5	19.0	10.7	GA-G-LDPE-	42.8	39.9	41.4
GB-F-HDPE-	21.9	10.7	34.1	GA-F-HDPE-	43.8	40.9	43.8
GB-P-HDPE-	29.2	30.2	21.9	GA-P-HDPE-	41.4	58.4	38.9
GB-F-PET-	23.8	33.1	25.3	GA-F-PET-	38.9	56.0	40.9
GB-P-PET-	23.4	47.7	48.7	GA-P-PET-	41.8	42.8	39.4
GB-F-PP-	28.2	31.6	24.3	GA-F-PP-	39.4	41.4	43.8
GB-P-PP-	23.4	11.7	9.7	GA-P-PP-	39.4	44.8	39.9

However, the results also show that, for a given type of granular material (sub-base or base material), the change (increase or decrease) in the CBR of the sub-base and base material is not only a function of the plastic content, but also of the type and shape of the plastic (Table

4-17, Figure 4.30 -4.32). For a given type of granular material and shape of plastic, the type of plastic that induces the highest increase in CBR value depends on the plastic rate. Similarly, the shape of plastic that provides the sub-base or base material with the highest CBR value for a given plastic type also depends on the plastic rate.

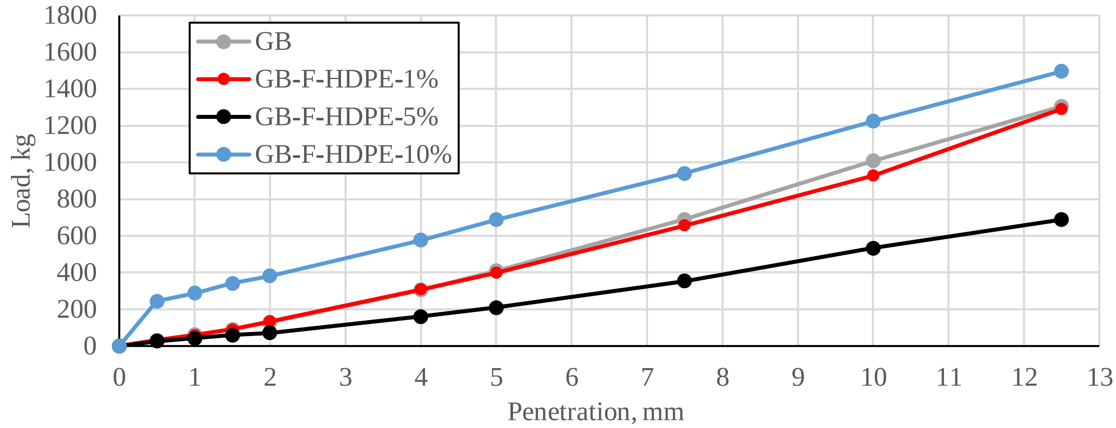


a) LDPE type with granular B sub-base material

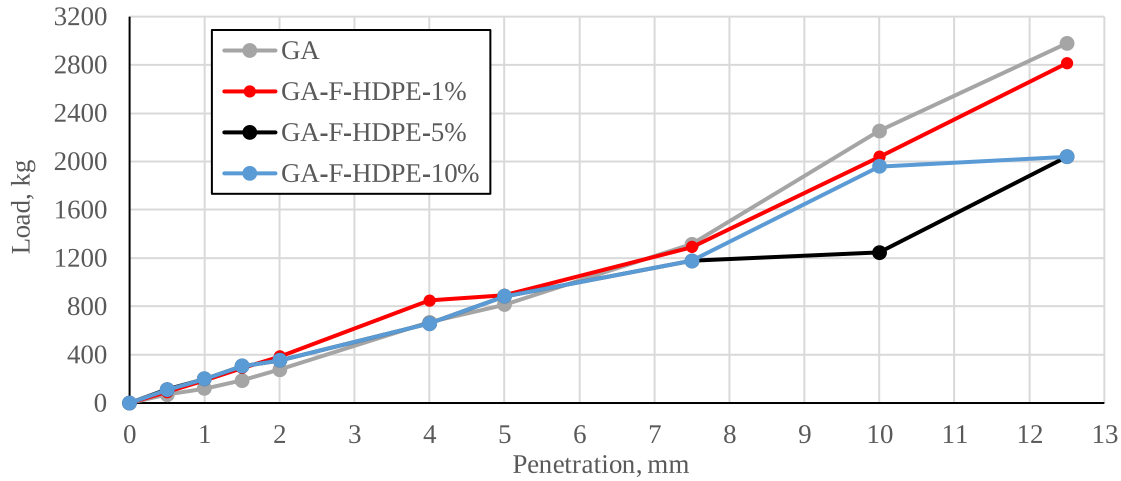


b) LDPE type with granular A base-course material

Figure 4.30. CBR results for LDPE type with (a) granular B sub-base material and (b) granular A base-course material

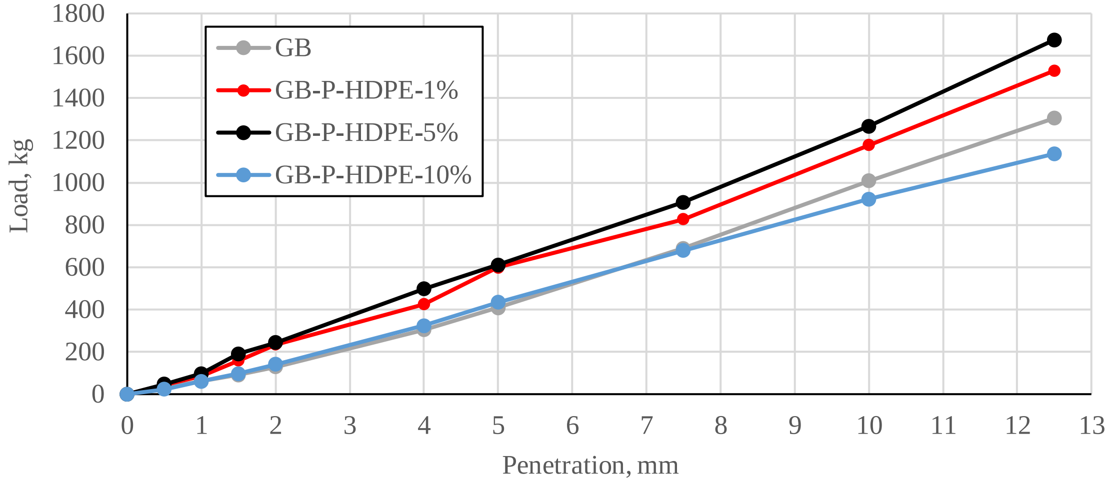


a) Flake HDPE type with granular B sub-base material

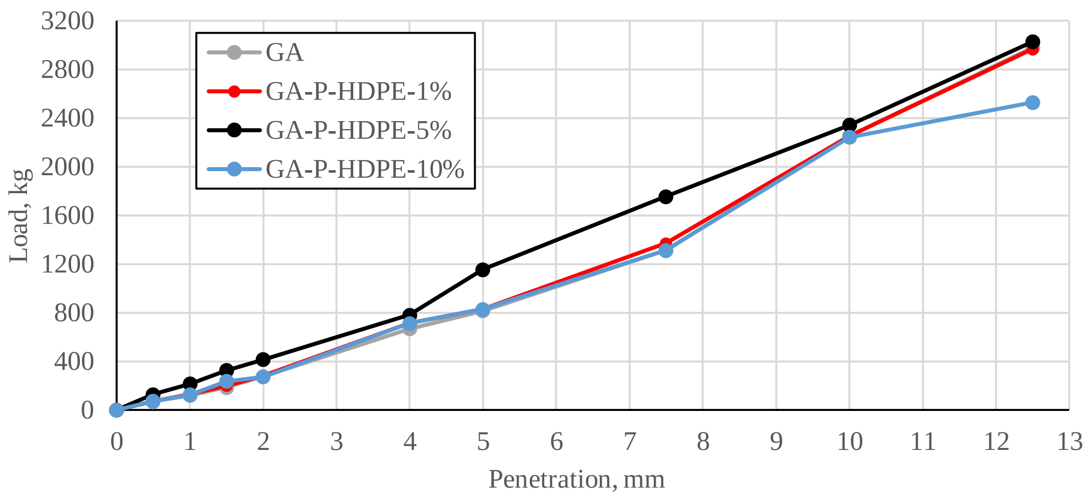


b) Flake HDPE type with granular A base-course material

Figure 4.31. CBR results for flake HDPE type with (a) granular B sub-base material and (b) granular A base-course material



a) Pelleted HDPE type with granular B material



b) Pelleted HDPE type with granular A material

Figure 4.32. CBR results for pelleted HDPE type with (a) granular B sub-base material and (b) granular A base-course material

It can be deduced from the graphs, table and results presented above that the type, rate, and shape of plastic incorporated with the granular material can significantly increase or decrease the strength property (CBR) of the sub-base or base materials. All these three factors related to the plastic (type, shape, rate) need to be considered to achieve the highest increase or enhancement of the CBR value of the sub-base or base materials. An improvement in material strength, as reflected in the CBR values, has a significant impact on the overall required thickness of pavement structure over the sub-grade to protect it from traffic loading, and thus on the cost of the pavement or road. The stronger the granular material over the sub-grade layer, the less the thickness of pavement required (Huang, 2004; Christopher et al., 2006) and the lower the construction cost of the pavement or road. Figure 4.33 illustrates the importance of having a high resistance base within any pavement structure. It depicts how the required thickness of the asphalt layer declines as the CBR of the base-course increases and subsequently increases the structural dependence on the base layer.

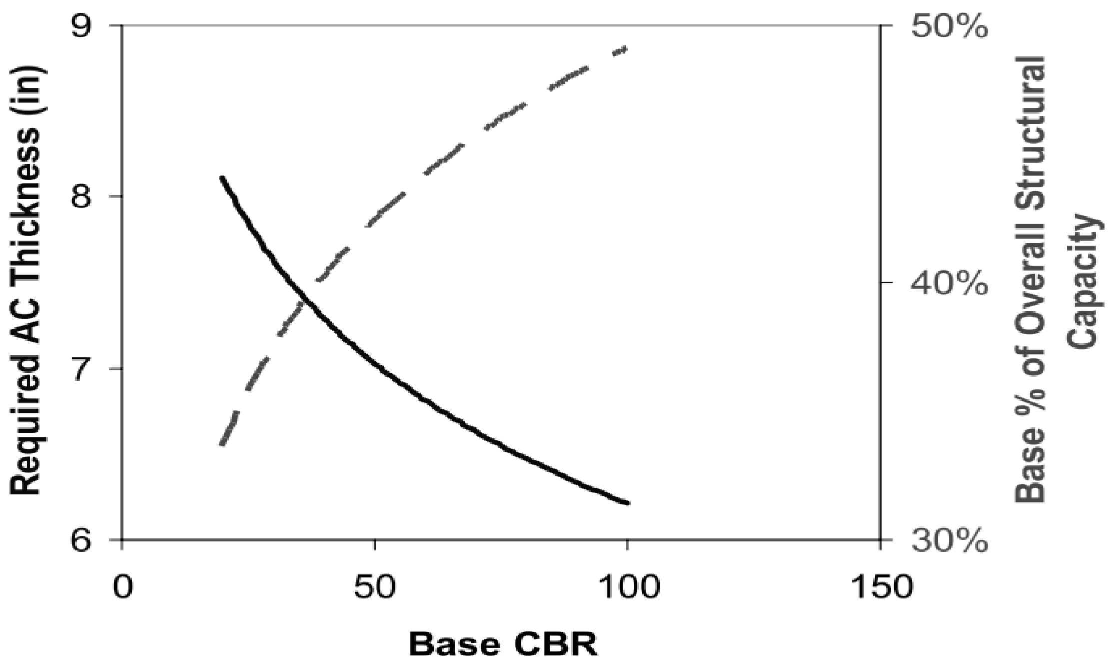


Figure 4.33. Influence of CBR of base material on required pavement thickness (Christopher et al., 2006)

4.3.8.3 Estimation of modulus of resilience

Each modulus of resilience (M_R) was predicted numerically by means of the three models (Model 1, Model 2, Model 3) described previously. The results are presented in Table 4-18 and Table 4-19.

Table 4-18. M_R values based on the three predicted moduli for granular B

	Modulus of Resilience (M_R), MPa								
	Model 1			**Model 2**			**Model 3**		
GB	132.5			349.8			160.6		
Plastic rate, %	**1%**	**5%**	**10%**	**1%**	**5%**	**10%**	**1%**	**5%**	**10%**
GB-G-LDPE	110	116	80	285	302	201	133	140	97
GB-F-HDPE	127	80	169	334	201	457	154	97	205
GB-P-HDPE	153	156	127	409	419	334	185	190	154
GB-F-PET	134	165	139	354	447	370	162	201	169
GB-P-PET	132	209	212	350	580	589	161	255	259
GB-F-PP	149	161	136	399	433	359	181	195	165
GB-P-PP	132	85	75	350	214	187	161	102	91

Table 4-19. M_R values based on the three predicted moduli for granular A

	Modulus of Resilience (M_R), MPa								
	Model 1			**Model 2**			**Model 3**		
GA	190.9			524.5			232.6		
Plastic rate, %	**1%**	**5%**	**10%**	**1%**	**5%**	**10%**	**1%**	**5%**	**10%**
GA-G-LDPE	195	186	191	537	511	524	238	227	233
GA-F-HDPE	198	189	198	546	520	546	241	231	241
GA-P-HDPE	191	238	183	524	670	502	233	291	223
GA-F-PET	183	232	189	502	650	520	223	283	231
GA-P-PET	192	195	185	528	537	506	234	238	225
GA-F-PP	185	191	198	506	524	546	225	233	241
GA-P-PP	185	201	186	506	555	511	225	245	227

The US Army Corps of Engineers' predictive model (Model 1) presented the highest values as compared to the other models. The fact that these are CBR-based equations manifested on the results in terms of having a pattern similar to that encountered when observing the CBR of the material. Essentially, however, the sub-base material exhibited a lower modulus as compared to the base-course materials for the three utilized models.

The modulus of resilience rose at 5% before declining again at 10%, as shown in the graph in Figure 4.34 and Figure 4.35. Nonetheless, this pattern was reversed in the case of pelleted HDPE, where it dropped at 5% before increasing again at the 10% rate of inclusion. Overall, the resilience of the material declined as the rate of plastic increased after 5%, which indicates that the 5% rate can be considered as the optimum rate for this investigation.

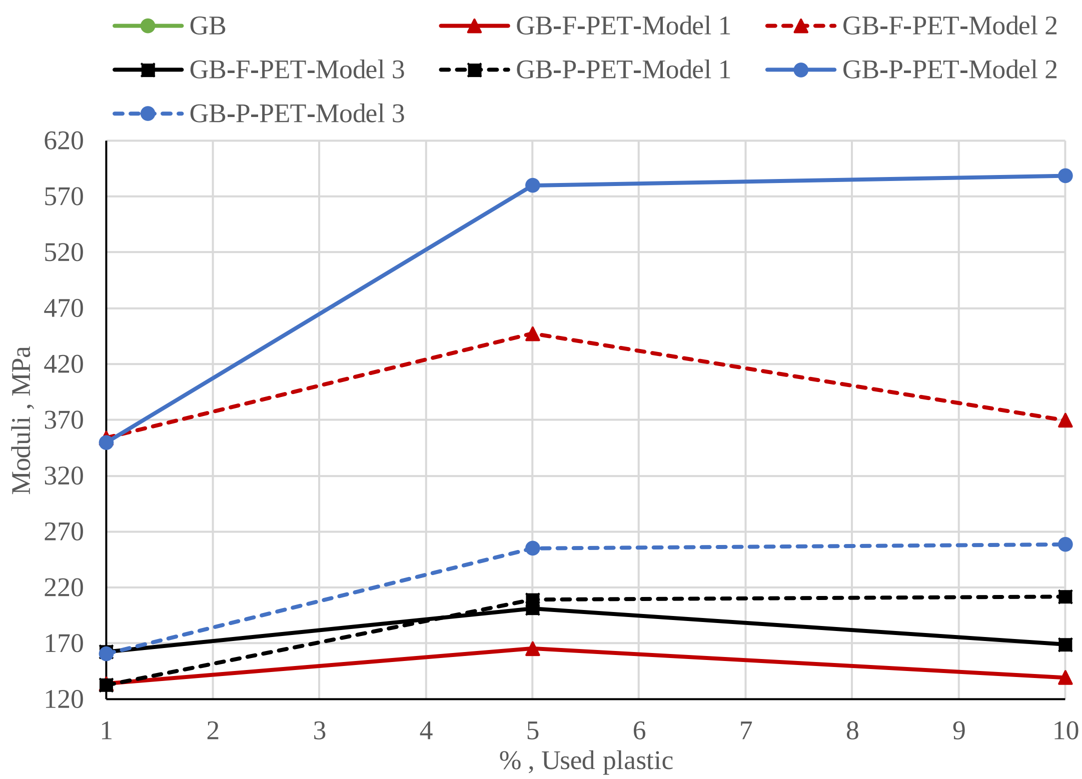


Figure 4.34. M_R of the three models for the combinations of sub-base material with PET

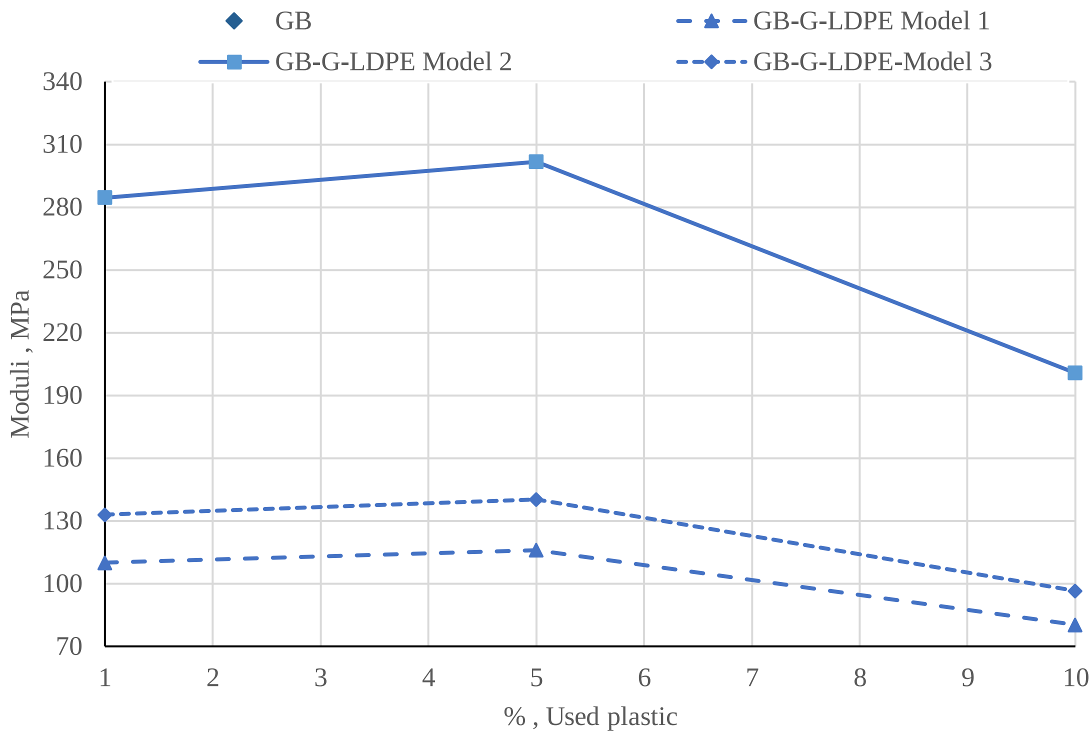


Figure 4.35. Obtained modulus of resilience for granular B type II material with LDPE combinations

Model 1 and 2 displayed higher moduli of resilience for the granular B type II material with ground LDPE. The US Army Corps of Engineers' model had the most conservative outcome amongst the three prediction models used with ground LDPE (Chowdhury et al., 2013). These results from the models would be best validated with an experimental work in order to compare the results. However, this is enough to present the trend the granular materials show with the addition of recycled plastic.

The AASHTO design attaches clear importance to thoroughly factoring in this parameter when determining the required structural thickness of any pavement and to mitigate the detrimental effects of unrecoverable strains. Typically, this value for the base materials is higher than that for the sub-base and the sub-grade owing to the materials properties and the natural on the imposed stresses on the upper layers in pavement (Huang, 2004; Christopher et al., 2006; Schaefer et al., 2008).

4.3.8.4 Hydraulic conductivity

The assessment of hydraulic conductivity is essential for sub-base and base granular materials in order to determine the drainage properties, such as the required period the materials need to be drained of water. It is also an indicative factor for whether lowering the water table during a particular construction project is required to circumvent the detrimental effects of moisture within the pavement structure (Christopher et al., 2006).

The changes in the coefficients of permeability of saturated hydraulic conductivity for all combinations of materials and plastic type, shapes and rates are given in Table 4-20. Interestingly, the coefficient of permeability values for both granular materials were predominantly increased by the addition of ground LDPE, while they generally decreased when the other type of plastic (HDPE, PET, PP) with different shapes (flaked and pelleted types of plastic) were added. This is probably due to the non-uniform shape of the ground LDPE particles, which created significant room for water to flow through the spaces between the solid granular material particles. The results also revealed a lower permeability of the base materials as compared to the sub-base, which is most likely due to the relatively higher proportion of fine materials in the base-course material, as shown in Figure 4.36 and Figure 4.37.

Table 4-20. Permeability of both granular materials with different combinations

Permeability, k (cm/sec.)							
	Granular B type II				**Granular A**		
GB		**1.021**		**GA**		**0.6413**	
Plastic rate, %	**1%**	**5%**	**10%**	**Plastic rate, %**	**1%**	**5%**	**10%**
GB-G-LDPE-	1.4925	2.5210	2.2520	GA-G-LDPE-	1.4776	2.1680	2.2250
GB-F-HDPE-	0.2717	0.4018	0.5765	GA-F-HDPE-	0.3396	0.5224	1.4500
GB-P-HDPE-	0.3633	0.1316	0.2322	GA-P-HDPE-	0.3996	0.1605	0.2740
GB-F-PET-	0.2706	0.4018	0.5765	GA-F-PET-	0.3518	0.5425	0.8359
GB-P-PET-	0.4282	0.1118	0.2716	GA-P-PET-	0.4710	0.1363	0.3205
GB-F-PP-	0.3211	0.4018	0.5765	GA-F-PP-	0.4496	0.6229	0.9224
GB-P-PP-	0.4134	0.1118	0.2716	GA-P-PP-	0.4547	0.1386	0.3246

The obtained results are also illustrated graphically in Figure 4.36 and Figure 4.37 for better understanding of the impact of addition of plastic on the coefficient of permeability of the sub-base (Figure 4.36) and base (Figure 4.37) materials. These figures generally indicate a decrease in the permeability coefficient of the sub-base and base materials with the incorporation of plastic (except for LDPE). The granular B type II sub-base material exhibited higher values of conductivity with the inclusion of ground LDPE, as shown in Figure 4.36. The sub-base specimens exhibited lower hydraulic conductivity by an average of 33% when different plastic types and shapes were incorporated with the GB material as opposed to an average 75% decrease with the GA material. However, ground LDPE demonstrated coefficients 204% and 305% higher than the GB and GA with no plastic, respectively. The aforementioned decrease of the coefficient of permeability due to the addition of the plastic, HDPE, PET, PP, can be attributed to the fact the plastic particles occupy the spaces between the large mineral aggregates and thus, reduce the porosity of the granular material, which in turn decreases the permeability.

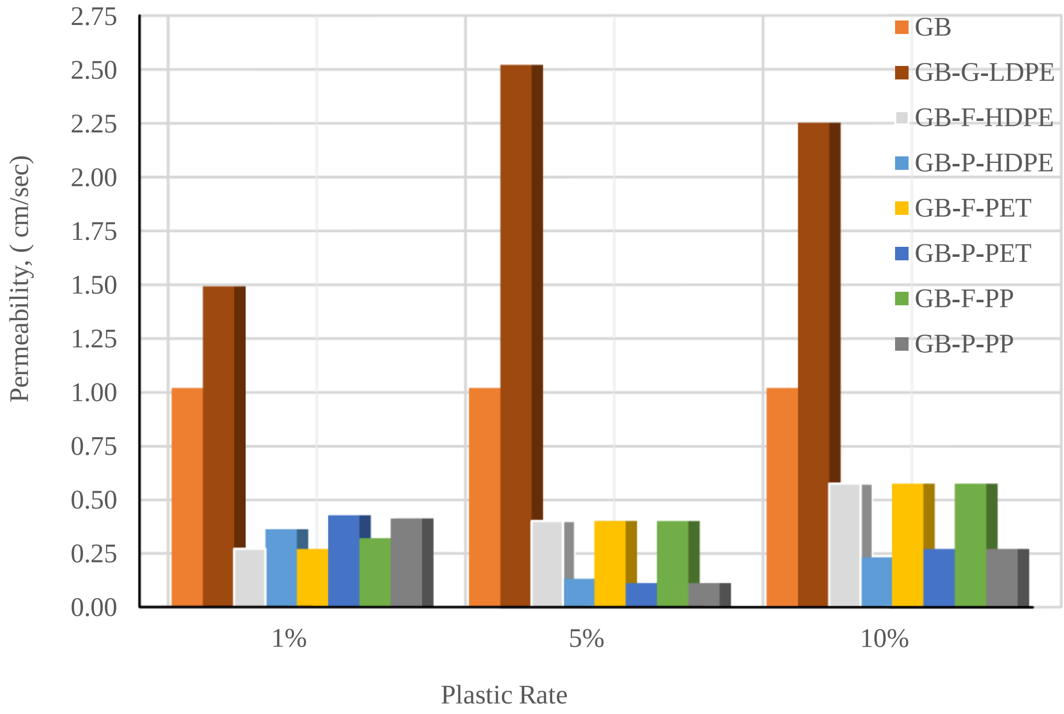


Figure 4.36. Hydraulic conductivity of granular B type combinations

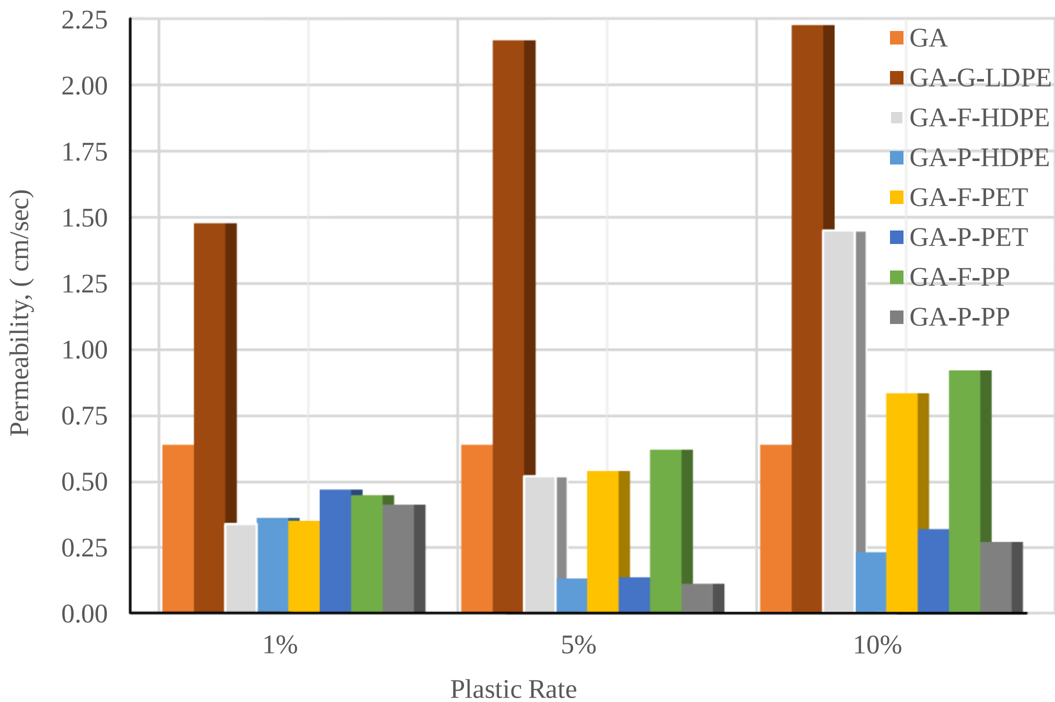


Figure 4.37. Hydraulic conductivity of granular A combinations

4.3.9 Summary and Conclusions

This investigative study was carried out to examine the geotechnical properties of two types of granular materials intended for the sub-base and base-course applications with the incorporation of recycled plastic. These granular materials were of two different gradation types; granular B type II for the sub-base and granular material Type A for the base-course applications and they were combined with several types (LDPE, HDPE, PP, and PET) of plastic in different forms (ground, flakes, and pellets) and at different rates (1%, 5%, and 10%). The changes of the geotechnical properties of the two investigated materials have been observed, assessed and analyzed. The assessment was conducted experimentally through many standardized tests and the following conclusions can be made:

- The granular B type II and A type materials exhibited lower maximum dry densities at a relatively lower water content once mixed with plastic. This decrease of the maximum dry densities induced by the addition of plastic waste is related to the low relative density of the plastic material compared to that of the main mineral (quartz, feldspar) particles that make up the studied granular materials.
- In general, the higher the plastic content, the lower the maximum dry density became, and similarly, the lower the optimum moisture content (except for samples, HDEP flakes and pellets, and PET pellets) of the compacted sub-base or base materials with plastic. However, for a given type of granular material and plastic content, the type and shape of the plastic have insignificant effect on the dry density of the compacted sub-base and base materials.
- It is also found that the type, rate, and shape of plastic incorporated with the granular material can significantly increase or decrease the CBR of the sub-base or base materials. The increase or decrease of the CBR value is a function of the type, shape and content of plastic waste. This CBR increase induced by the presence of a specific (optimal) amount of plastic in the granular material is due to the reinforcing ability of the plastic. All the aforementioned factors related to the plastic (type, shape, rate) need to be considered to achieve the highest increase or enhancement of the CBR value of the sub-base or base materials with recycled plastic. This observed plastic waste-induced enhancement of the CBR values may suggest that the addition of plastic waste to the studied sub-base and base

materials could result in the reduction of the thickness of the paved structure, which, in turn, will result in a decrease of the construction cost of the road or pavement structure.

- The resilient modulus of the base and sub-base materials improved with certain types and shapes of incorporated plastic. Moreover, the resilient modulus of the material generally increased as the plastic rate increased up to 5%, whereas it declined as the rate of plastic increased after 5%, which indicates that the 5% rate could be considered as the optimum rate for this investigation.
- The addition of ground LDPE generally resulted in the increase of the hydraulic conductivity of both granular materials, while the coefficient of permeability of the latter generally decreased when the other type of plastic (HDPE, PET, PP) with different shapes (flaked and pelleted types of plastic) were added. This decrease of the coefficient of permeability due to the addition of the plastic, HDPE, PET, PP, can is related to the fact the plastic particles occupy the spaces between the large mineral aggregates and thus, diminish the porosity of the granular material, which in turn decreases the permeability. On the other hand, the non-uniform shape of the ground LDPE particles resulted in the generation of significant space in the pore structure of the granular material for water to flow through the areas between the solid particles. The obtained results suggest that partial replacement of the studied sub-base and base granular materials with plastic waste may prove useful in road applications and could help to reduce the amount of plastic waste to be managed and the material cost in road construction. However, further research is required to (i) optimize the plastic characteristics (content, type and shape) with respect to the performance of the base and sub-base materials, (ii) study the resilient modulus of the granular materials with optimal plastic characteristics, and (iii) conduct large scale or field scale testing of the performance of sub-base and base materials with the optimal plastic characteristics.

Acknowledgments:

The Authors would like to thank Canada Fibres Ltd. and Enviroplast Inc. for the provision of the required recycled plastic, and to Lafarge for providing the researcher with the needed granular materials.

4.3.10 References

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4.4 Effect of plastic waste on the fresh and hardened properties of concrete pavement material

Mukhtar Abukhettala, Mamadou Fall

4.4.1 Abstract:

Alternative materials in the construction field has been there for years. Materials that have been potentially evaluated include plastic waste materials which pose high burden on societies and the ecosystem. In this paper, an attempt was made to investigate the influence of incorporating recycled plastic aggregates of different forms and shapes on the fresh (density, workability, air content) and hardened (compressive, split and flexural strength, water absorption and sorptivity, permeability) properties of a concrete pavement material. The investigation was carried out experimentally on fresh and hardened concrete mixtures. Performed tests include slump, density, compressive strength, splitting tensile strength, bending strength, water sorptivity, permeability, and electrical conductivity tests. Some of the tests were conducted at different curing ages to evaluate the influence of time on the investigated properties. The concrete mixture was mixed with three resins, namely Low-density polyethylene (LDPE), High-density polyethylene (HDPE) and Polyethylene terephthalate (PET) in three different forms; flakes, pellets, and ground, and in three percentages;1%, 5%, and 10% by weight of the mixture. The addition of plastic aggregates significantly decreased the slump values and density of the fresh concrete, while it increased its air content. Compressive, splitting and flexural strength decreased with the increase of added plastic aggregates. The measured bending strength values reflected a more ductile concrete with added recycled plastic. The dry densities of the hardened concrete materials decreased as the content of plastic waste increased, regardless of the type and shape of plastic. Furthermore, concrete with plastic aggregates showed higher water absorption capacity and coefficient of sorptivity and permeability. Moreover, mixing of concrete with plastic aggregates resulted in more permeable materials. The study provides significant information regarding the suitability of such supplement materials for incorporation with concrete mixtures and for pavement or road applications.

Keywords: Pavement; Road; Surface-course; Concrete; Plastic waste; Recycling

4.4.2 Introduction

Concrete pavement has made quite a lot of our roads in recent years. Particularly in paving runways and aprons which fundamentally are subjected to heavy loads. Applied loads on concrete slabs are distributed largely over abroad area unlike the mechanism of load distribution in flexible pavement. The typical cross section of concrete pavement is illustrated in Figure 4.38. Causes of concrete slab failures are commonly associated with the variation in temperature and the persistence bending due to loading which induce high tensile and flexural stresses that ultimately lead to fatigue stress failure (Mathew, 2006). The required thickness of the slab is highly dependable on these fatigue stress induced on the pavement structure. However, reinforcement is only introduced to control the cracks induced by the contract and expansion of the slab.

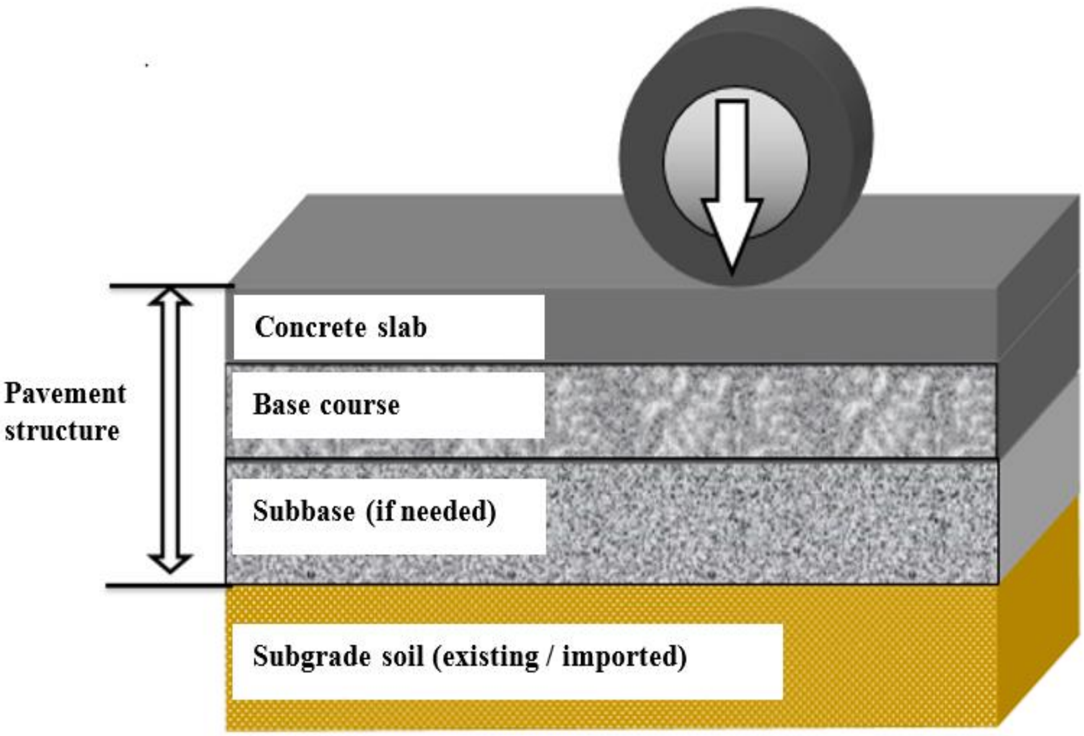


Figure 4.38. Typical lay out of a concrete pavement structure

The endurance of a concrete slab is then of an essence to sustain imposed stresses. Durability of the concrete plays a substantial role in prolonging the service life of the structure and in governing routine and periodical maintenance. Durability is affected by concerted factors when acting concurrently destroy the integrity of the slab rapidly. Factors include rate of water absorption, compressive and tensile strength, influence of capillary action, density and abrasion of aggregates (Leung et al., 2016) .

With the evolved construction industry and the emerging construction materials, there has been a huge need to revisit the possibility of incorporating various so-called waste materials in more meaningful industries including construction. This would not benefit only the environments, but assist in preserving natural resources and coping with existing shortage of spaces for landfills (Siddique et al., 2008). Materials considered in the past included scrap tires, crushed glasses, foundry sands, as well as plastic waste which are mainly collected from demolition works (Siddique et al., 2008) along with recycled plastic fibers (Bhogayata & Arora, 2018). With the huge consumption of plastic materials in everyday lives of people around the world, it jumped from a consumption amount of 5 Million tonnes to over a 100 Million tons in 50 years' time worldwide. Only 7% of plastic waste is recycled in the US where an estimate of 80% of plastic generated waste is diverted to landfills in the US (Siddique et al., 2008). This has led to revolve the stance on dwelling on a waste material to a more useful material in the academic research and industry to reduce the burden on societies (Siddique et al., 2008). Some characteristics of plastic which makes is usable and preferable in applications of daily lives include their low density, low cost, and high durability (Gu & Ozbakkaloglu, 2016). Some adverse properties entail the very slow biodegradability of most types of plastic which largely impacts the environment (Sharma & Bansal, 2016)

Low-density polyethylene (LDPE), High-density polyethylene (HDPE) and Polyethylene terephthalate (PET) are the highest produced types of plastic waste among other plastic types in the US in 2000 (Siddique et al., 2008) and represent the highest recycled resins among other types (Gu & Ozbakkaloglu, 2016). In Europe, almost half of the consumed plastic in 2012 was reported to be diverted to waste landfills. That was about 25 million tons out of the 45 million tons. In the same year, the US citizens consumed nearly 32 million tons of plastic in which merely 8.8% was recycled (Sharma & Bansal, 2016). In India, 12.3% of plastic waste, in which PET accounts for the largest portion, is generated annually out of 40 million tons of solid waste. Flakes of plastic generated by shredding the baled plastic in recycling factories is a common type of plastic waste that undergoes re-processing to other industries (Siddique et al., 2008).

With the advance in material characterization and testing, the use of ordinary concrete in paving works has evolved. Many additives and by-products have been explored and tested in this direction to measure benefits on concrete while maintaining a durable and long-lasting product. Using additives and other admixture to improve the quality of concrete has been very common in recent years, this includes fly ash, slag, recycled concrete aggregate as well as plastic waste. The aim is to reduce the environmental impact associated with producing concrete and its ingredient while cultivating the durability of the concrete (Delatte, 2008).

The environmental and economic risk associated with coping with increasingly huge plastic waste diverted efforts towards benefiting rather than disposing this material along with other quasi-waste materials (Sharma & Bansal, 2016). Therefore, considering the increased awareness of environmental concerns and associated pollution and material consumption, there has been a substantial leap in the field of construction when it comes to the raw materials. Numerous studies have been performed on the engineering properties (e.g., strength) of concrete incorporating plastic wastes (e.g., Sharma & Bansal, 2016; Siddique et al., 2008; Bhogayata & Arora, 2018; Choi et al., 2005). Despite the tremendous contributions made by these studies to better understand the impact of plastic waste on the engineering properties on concrete, the effect of the shape and different types of plastic waste on the fresh and hardened properties was largely ignored. Moreover, most of these previous studies focused on the hardened properties, particularly, strength. There is a paucity of information about the effect of plastic waste on the fresh and other hardened (e.g., permeability, capillary water absorption) properties of concrete pavement. There is a need to address this knowledge and technology gap.

The main objectives of the study are to examine the influence of adding recycled plastic of different types and shapes on the properties of concrete mixture for pavement (surface-course) applications. The compatibility of the incorporated plastic on the properties of fresh and hardened concrete are investigated.

4.4.3 Experimental Program

The experimental program is presented in Figure 4.39 . It is basically composed of two phases which focus on different aspects. The first phase concentrates on properties of freshly patched concrete. The second stage focuses on properties of hardened concrete that cured for sometimes after patching.

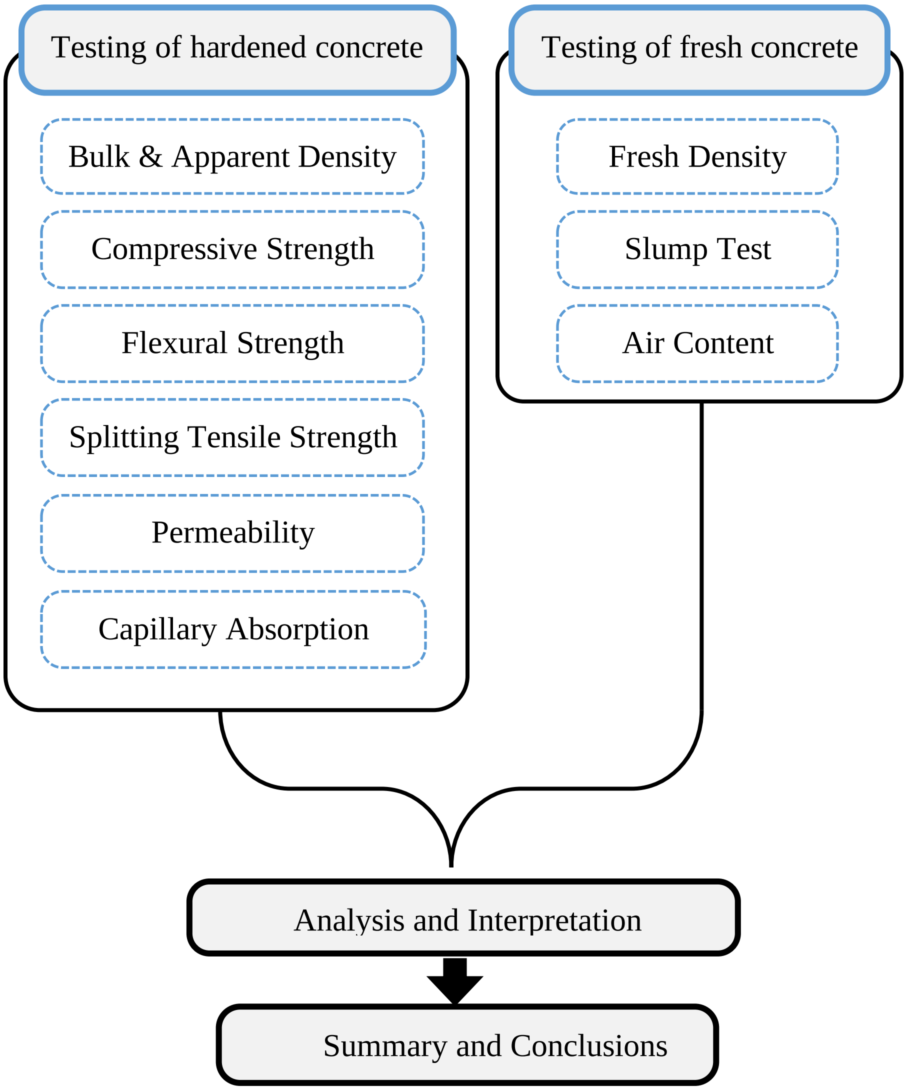


Figure 4.39. Experimental program and methodology

4.4.4 Materials and Methods

4.4.4.1 Concrete material components

A crushed gravel of a maximum aggregate size of 10mm was used alongside with crushing sand to produce the concrete mixture. The combination reflects well graded materials with a specific gravity of 2.710 gm/cm^3 for the gravel and 2.690 gm/cm^3 for the sand. Additionally, these components had an absorption rate of 1.20% and 1.25% for the gravel and sand, respectively. The fine sand had a finesse modulus (FM) of 0.80 as produced by the accumulated percentage of materials retained on sieves 9.5 mm to 0.150 mm inclusively. Literature pointed out that a material with FM that falls out of the 2.3 to 3.1 range is deems ultra-fine material (Delatte, 2008). The Portland cement type I was used for producing the concrete mixture. Constituents were blended together to produce a 30MPa concrete mixture which would be used as a reference in this investigation. The grading analysis of the utilized materials for the concrete mixture is given Table 4-21 next.

Table 4-21. Grading of materials incorporated in the concrete mixture

Sieve size,		**Percentage of materials passing, %**
mm	**Inch**	
9.5	⅜ in.	96.1
4.75	No.4	74.6
2.36	No.8	57.2
1.18	No.16	38.7
0.600	No.30	28.6
0.300	No.50	22.3
0.150	No.100	17.3
0.075	No.200	12.4

4.4.4.2 Concrete mixture design

A concrete mixture of 30 MPa strength was selected for the study as a control. The range of 20.7 MPa to 35.9 MPa is applied in most Canadian jurisdiction. The strength of a concrete mixture proposed by the structure design of pavements depends on the traffic loading and situational factors pertaining to prevailing conditions of the designed pavement. Any strength value may satisfy the design according to the input factor for the design. In this study, the mix design proportions along with the other components including plastic rates and types are presented in Table 4-23. The highest rate of incorporation of all types of plastic was 10%. Figure 4.40 shows the sieve analysis results of the selected materials of the concrete mixture.

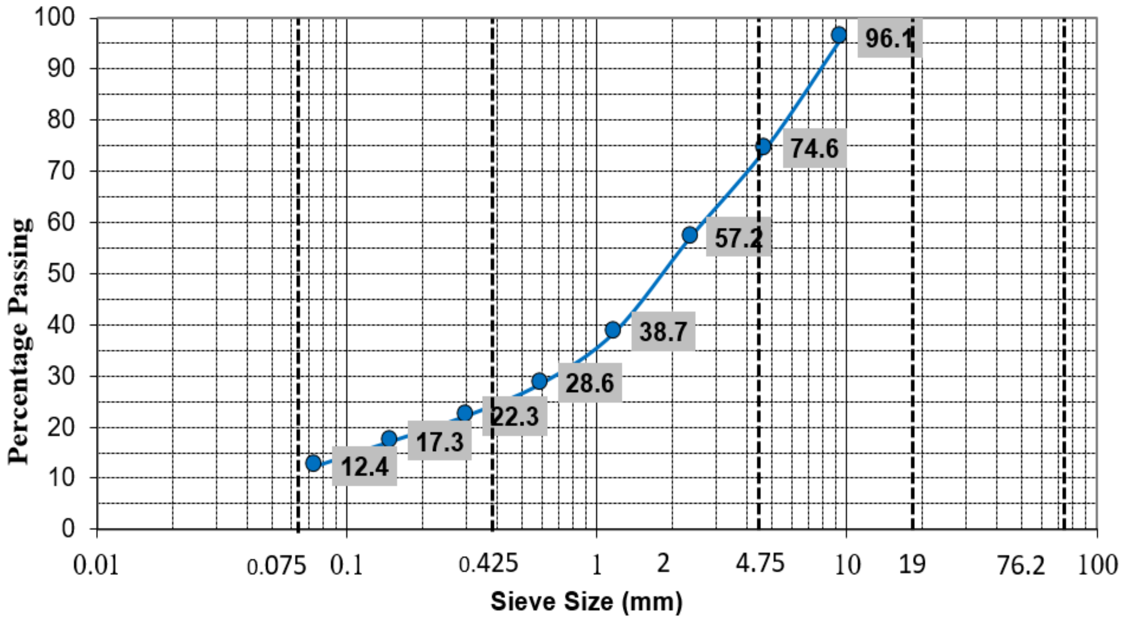


Figure 4.40. Grading Analysis for the aggregate materials used in the mixture

4.4.4.3 Plastic Waste Materials

Three different types of recycled plastic were incorporated in this study. They are as follow; low density polyethylene (LDPE), high density polyethylene (HDPE), and polyethylene terephthalate (PET) polymers. These types were employed in three different forms; ground, flakes, and pellets. The physical characteristics of these types are shown in

Table 4-22 and then presented in Figure 4.41.

Table 4-22. Plastic materials properties incorporated for the study

Type of Plastic	**LDPE**	**HDPE**		**PET**	
Form/Shape	**Ground**	**Flakes**	**Pellets**	**Flakes**	**Pellets**
Characteristic	**Irregular**	**Rectangular**	**Cylindrical**	**Rectangular**	**Rectangular**
Length (L), mm	-	8.80	-	8.80	
Width (W), mm	-	7.00	-	7.00	
Diameter (D), mm	-	-	3.65	-	4.45
Thickness (T), mm	-	0.7	2.70	0.7	1.65
Aspect Ratio (AR) = L/d		1.25	1.35	1.25	2.70

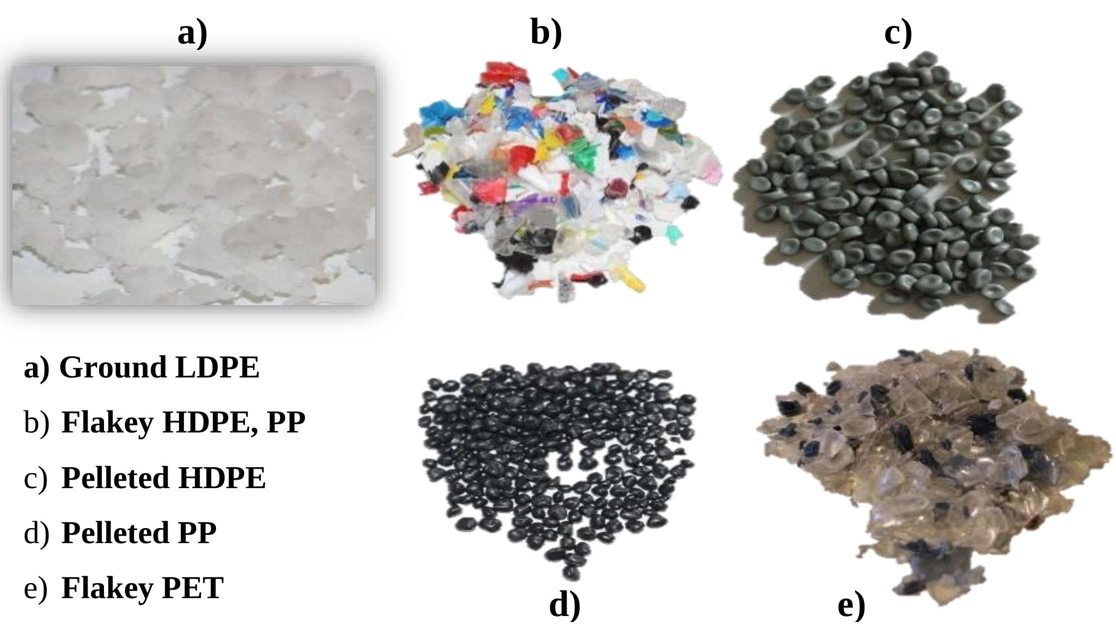


Figure 4.41. Types and shapes of recycled plastic used in the study

The mix design proportions, and amount of plastic used for each combinations are all presented in Table 4-23 below. It is worth noticing that the incorporation rate was based on previous studies (Gu & Ozbakkaloglu, 2016; Sharma & Bansal, 2016) and in accordance with the observed workability of trail mixtures in the laboratory prior to proceeding with actual experimental works.

Table 4-23. Concrete mixture proportions

Series	**Plastic, %**	**Gravel, kg/m^3**	**Sand, kg/m^3**	**Water/Cement Ratio, w/c**	**Cement, kg/m^3**	**Water, kg/m^3**
LDPE ground	1	838.92	921.0	0.45	379.30	170.70
	5	838.92	921.0	0.45	379.30	170.70
	10	838.92	921.0	0.45	379.30	170.70
HDPE flakes	1	838.92	921.0	0.45	379.30	170.70
	5	838.92	921.0	0.45	379.30	170.70
	10	838.92	921.0	0.45	379.30	170.70
HDPE pellets	1	838.92	921.0	0.45	379.30	170.70
	5	838.92	921.0	0.45	379.30	170.70
	10	838.92	921.0	0.45	379.30	170.70
PET flakes	1	838.92	921.0	0.45	379.30	170.70
	5	838.92	921.0	0.45	379.30	170.70
	10	838.92	921.0	0.45	379.30	170.70
PET pellets	1	838.92	921.0	0.45	379.30	170.70
	5	838.92	921.0	0.45	379.30	170.70
	10	838.92	921.0	0.45	379.30	170.70

4.4.5 Sample preparation

Prior mixing of all ingredients, batches of all predetermined weighs of all components, as per Table 4-23, were prepared and put aside in separate pans. The distilled water was weighed aside on graduate cylinders to be added later in the process. An automated mixer was utilized for mixing the concrete mixture. During the blending process, constituents of the concrete mixture were poured into the electric mixer with half the amount of water added and allowed mixing for an average period of 5 minutes prior to adding the remaining quantity of water. Mixing was then continued for an additional 5 - 8 minutes so as to achieve a through blend of the whole materials with the water. Once homogenous mixture was observed, the predetermined amount of the set percentage of plastic was also poured into the mix and allowed to exhaustively blend into the mixer for another 5 minutes. The mixture was then considered to be ready for further experimental works. Mixing tool and process are presented in

Figure 4.42.

a) Concrete mixer **b) Plastic added to the mixer**

Figure 4.42. Concrete mixer and mixing process of concrete with plastic

Subsequently, preparation of concrete samples begun. Concrete samples were prepared in accordance with ASTM C-192 specification standards (ASTM Standard C192/192M, 2007). Cylinders were casted in 100mm diameter and 200 mm in height casting mould. Beams were however 400mm in length and 100 mm in width and depth. Replicates samples were prepared, and results were averaged later in the testing. After a 24-hour of patching, cylinders and beams were cured in water in large baskets (tanks) at an average temperature of 20°C. Curing of samples is shown in Figure 4.43. Testing process was phased into two stages; fresh concrete testing and hardened concrete mixing, as indicated in the experimental scheme program (Figure 4.39).

Figure 4.43. Illustration of samples curing in water prior to testing

4.4.6 Testing of fresh concrete

Shortly after mixing, testing of fresh samples was conducted. The impact of the addition of plastic materials of various types on the concrete mixture was assessed in terms of the density, slump flow (workability), and air voids contents as illustrated more in depth in the following sections. For each of the following testing procedures, each test was repeated at least twice, and the average value is considered and reported, if the difference between the two obtained values is less than 5%.

4.4.6.1 Workability testing

The term workability refers to the ease of handling of the mixture without excessive segregation occurs. It is the measure of how manageable to work with the concrete during construction. Traditionally, workability of any concrete mixture is examined through the slump flow. Workability is measured by running the slump test as described in ASTM C-143 standard testing procedure (ASTM Standard C143/C143M, 2015). The procedure allows compaction of the mixture in four layers in a frustum mould with a 100mm freely fallen steel rod. Once compaction is completed, the frustum in inverted and the dropping distance of the mix from the top end of the frustum is measured.

4.4.6.2 Density of fresh concrete

For the purpose of this experimental test, specimen was prepared in duplicates to determine the value of the density of freshly mixed concrete. Testing was conducted in a 90mm diameter mold of a19.7mm height. Each specimen was compacted in two layers with a 25 blow of a 100mm tampering rod. The mold was tapped on the outer side with a mallet after compacting each layer inside the mold. Testing was conducted as per ASTM C567 standard (ASTM Standard C567, 2004). The mold was weighed empty and then upon filling with the mix. The weight of the compacted concrete was then calculated and converted using the volume to the utilized mold into a density of the freshly mixed concrete.

4.4.6.3 Percentage of air entraining

Measuring air entraining in fresh concrete is an acceptable practice. It is crucial to estimate the percentage of air entrainment in any concrete mixture (Delatte, 2008). Evaluation of air content in a concrete mixture is necessary in computing the yielding of cement in the mixture and maintaining the average value within an acceptable limits pertaining to the mixture design criteria (Minnesota Department of Transportation, 2003). Concrete may undergo corrosion due to the intrusion of water and air within the concrete matrix (Leung et al., 2016). Means to control air invasion is by controlling the air voids contents through this test. The rate of air entrained in the concrete mixture plays a role in lessening the adverse impact of freezing and thawing in the mixture as well as empowering the workable mixture through having an entrained air ranges from 4 – 7.5% in general (Delatte, 2008). Moreover, concrete pavements in Canada and many other regions in the world are exposed to severe winter conditions, including low temperatures, snowfall, the application of large quantities of deicing agents, and several freeze-thaw cycles. Water increases in volume upon freezing, applying stresses that can surpass the tensile strength of concrete pavement. Repeated cycles of freezing and thawing can result in cumulative damage in the form of cracking and scaling. The air voids offer empty spaces within the concrete pavement material that function as reservoirs for the freezing water, releasing pressure and preventing damage to the concrete pavement (Portland Cement Association, 1998).

This test was undertaking according to ASTM C231 standard procedure (ASTM Standard C231/C231M, 2010) using an air meter. A sample of freshly mixed concrete is poured in the experimental bowl of 207mm in diameter in three layers. Each layer is compacted with a steel rod 25 times. Air is pumped into the meter until water starts to come out from the petcock that is unused to fill in water into the bowl. Air pressure is then realized, and the measurement of air entrained in indicated in the air voids measurement gauge.

4.4.7 Testing of hardened concrete

4.4.7.1 Bulk density of concrete cylinders

In order to assess the impact of incorporating the aforementioned types of plastic at the prescribed percentages on the density of hardened concrete after a curing ages of 7 and 28 days respectively, the specification standard ASTM C642 (ASTM Standard C642, 2013) approach was implemented. In this testing procedure, samples of a weight of not less than 800gm were cut from larger cylinders and subjected to cycles of drying out in an oven following by period of immersion in water and boiling water for various period of time. Each test was performed twice to ensure the repeatability of the results.

4.4.7.2 Compressive strength

It is an indication of developing the strength of the mixture over time to the desirable compressive strength. It also gives a better comprehension of the changing of strength with time in construction activities. Literature has shown that compressive test could be the single most reliable test for concrete mixtures exposed to less harsh ambient conditions (Concrete Institute of Australia, n.d.). This test was executed in accordance to ASTM C-39 testing standard (ASTM Standard C39/39M, 2015). For comparison purpose, samples were dissected at ages of 14 and 28 days. Each test was performed carried out twice to ensure the repeatability of the results.

4.4.7.3 Splitting tensile strength

Specification standard ASTM 496 (ASTM Standard C496, 2014) was conducted to measure the tensile strength of the concrete mixtures. For this testing, curing age was maintained at a 28-period of age. During testing, samples were subjected to diagonal loading to failure. The concept of testing is illustrated in Figure 4.44.

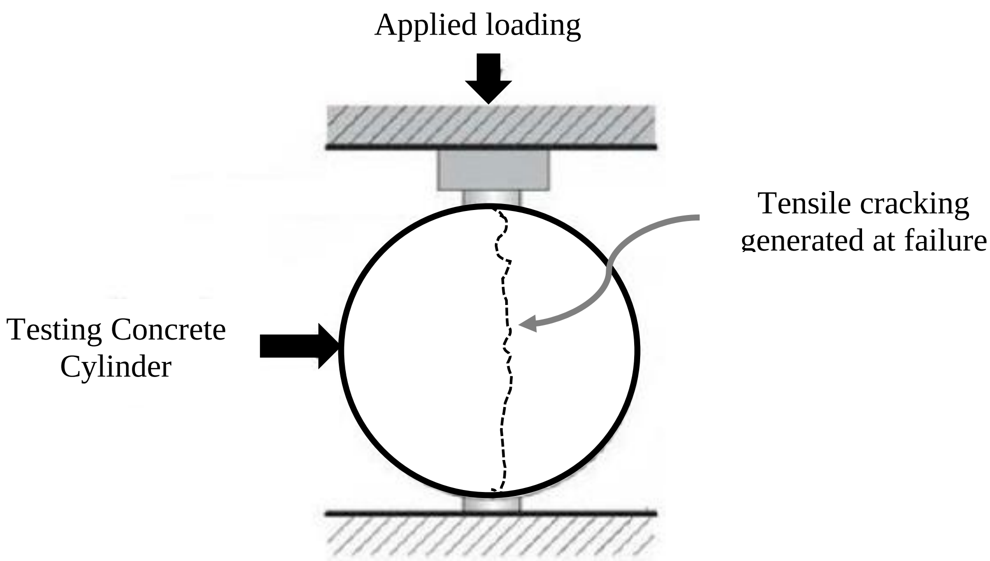
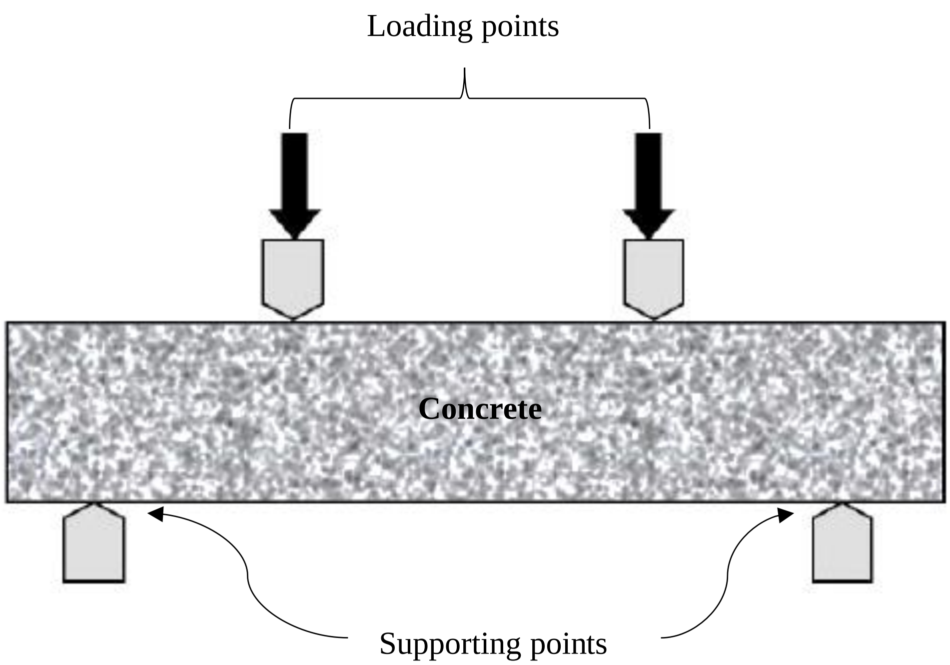


Figure 4.44. Splitting tensile strength concept

The National Ready Mix Association (NRMCA), however, indicated in a technical report that there is limited experience with reliance on splitting strength of concrete to assess the tensile strength of the concrete mixture (National Ready Mixed Concrete Association, 2000). Each test was performed twice to ensure the repeatability of the results.

4.4.7.4 Flexural strength using simple beam with third-point loading

Flexural strength is also interchangeably called fracture or bending strength and occasionally called modulus of rupture (MOR) of concrete slabs. Literature shows that this parameter is indirectly reflect on the tensile strength of concrete subjecting to tension during bending. Pavement concrete slabs are repeatedly subjected to tension under traffic loading at the bottom interface with the underneath layer (Huang, 2004) and assessing this parameter is paramount in determining the strength of pavement, particularly, in runways paving works of airports (National Ready Mixed Concrete Association, 2000). As a matter of fact, measurement of this property is essential in many occasions. Particularly, in thin pavement layers where the pavement endures substantial load stresses results in cycles of tensile and compressive fatigues (Huang, 2004). Experimental testing is conducted as per ASTM C-78 procedure using simple beam with third point loading (ASTM Standard C78/C78M, 2010). The schematic description of the test is shown in Figure 4.45



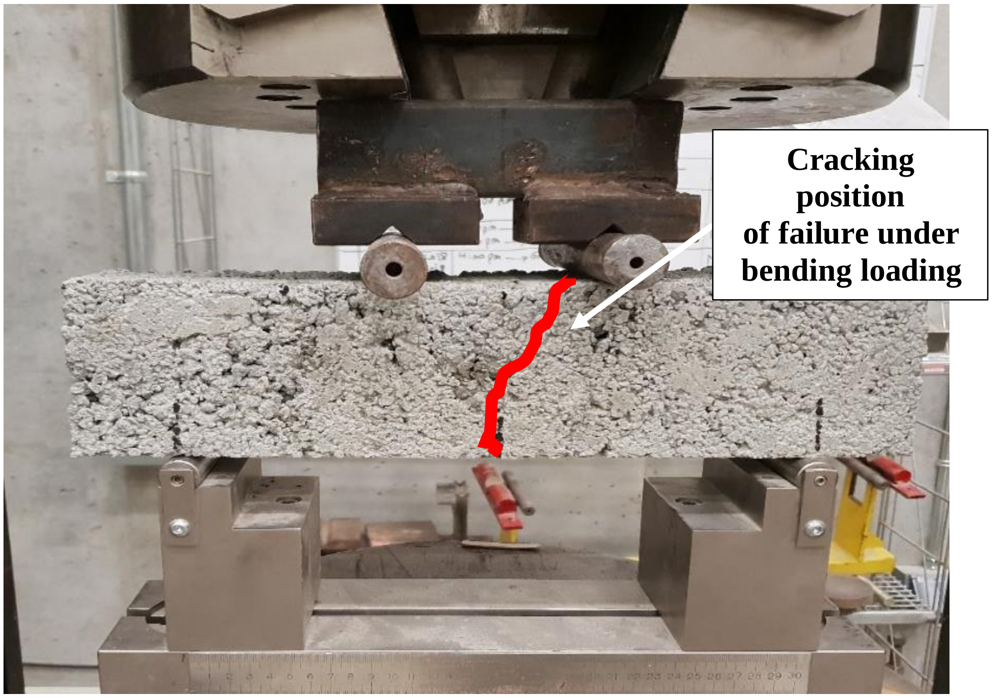


Figure 4.45. Flexural testing

4.4.7.5 Permeability

The importance of fluid transport on the durability of concrete pavements is well-acknowledged. A key fluid transport property of concrete pavement material is permeability. The rate of which water is transported through concrete matrix under gravity (pressure) is substantially affected by the interconnection of the pore volume and structure. This is collectively known as a measure of the permeability of concrete. The measurement is controlled by Darcy's law and numerous approaches are adopted for measuring it. For this study, to examine the influence of incorporating the prescribed types and rates of plastic with concrete on the penetration of water flow through the concrete matrix, the flexible wall permeameter method to determine permeability was selected. The testing was done in according with ASTM specifications standard ASTM D5084 (ASTM Standard D5084, 2017) on water-saturated concrete samples. The constant head permeability test using flexible wall cell was conducted on concrete samples with or without plastic.

4.4.7.6 Capillary absorption tests

Capillary absorption is another key fluid transport property of concrete pavement which affects its durability and long-term performance. Porosity, which is also an indicative to pore volume and structure, is reflected by the amount and speed of water suction by means of capillary action (Concrete Institute of Australia, n.d.). An increase in porosity of concrete material is associated with a greater vulnerability to aggressive agents. The amount and rate at which voids are filled is commonly measure by the capillary test. This test is also called a sorptivity test of hardened concrete. The standard procedure to carry out this test is stated in ASTM designation C1585. It measures the rate of attacking the concrete specimen by the surrounding water. The test is simple in principle and does not require much time to be performed. The test is important because amount of water absorbed due to the capillary action leads to detrimental impacts on the durability of the concrete.

In the testing procedure, as shown in the testing diagram presented in Figure 4.46, samples are first dried at a temperature of 50°C in an oven for a period of 7 consecutive days, prior to cutting them into thinner specimens of 40 – 50 mm in thickness. They are then sealed on perimeter by means of an adhesive (an epoxy, water-proof material, or a tape). In our testing, samples were cured at two ages of 7 and 28 days, respectively, and sealed with a tape of which covered a thin layer of plastic sheet wrapped around the thin specimen. The sealant allows a unidirectional flow of water from the surface subjected to submerging. Specimens were then placed on a wide tray over short glass rods lifting the specimens few inches from the base of the tray. This permits free flow of water from the underneath. Water level is raised above the base of the specimens by a submerging distance of approximately 5mm. Advance measurements of initial weights and cross-sectional areas of all specimens were done prior to commencing the test. The layout of this testing procedure is presented in Figure 4.46.

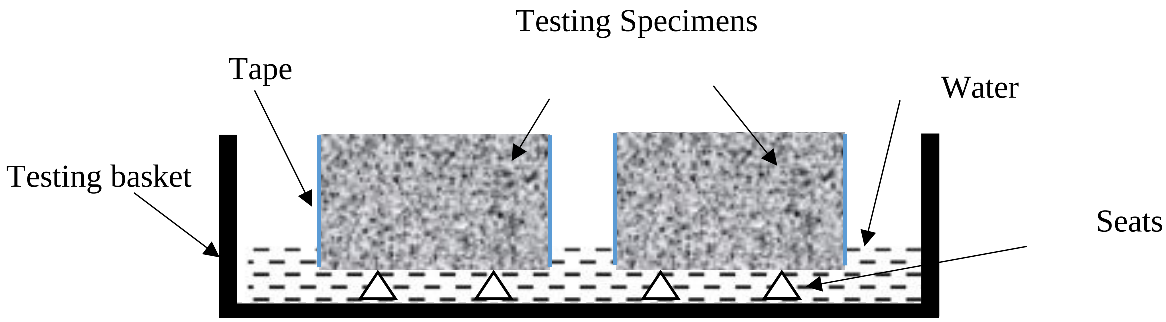


Figure 4.46. Testing scheme of capillary test

During testing of capillary suction, mass of each sample is recorded immediately at periods of 5, 10, 30, 60, and 120 minutes and tray was each time refilled to maintain the water level at the prescribed level from the base of the specimens. The amount of absorbed water due to capillary action is recorded. Thus, this amount of water per the cross-sectional areas of the specimen is then plotted against the square root of time. The slope of the best fitted line between obtained points is finally reported as the sorptivity of the examined specimen. In our tests, all combinations were tested in duplicates and then averaged. Sorptivity is often reported in $mm/min^{½}$ even though the volume of absorbed water is measured per unit of surface area of the sample in the square root of time, i.e. $mm^3/mm^2/min^{½}$ (Concrete Institute of Australia, n.d.).

The equation followed is explained as follows:

$$i = S * t^{½} \quad \text{(equation 1)}$$

where (i) denotes for the amount of water absorbed per the cross-sectional area of the specimen and is measured in gm/mm^2, (S) stands for the sorptivity and measured in ($g/mm^2/min^{½}$) and (t) is the time interval in minutes.

4.4.7.7 Electrical conductivity

Electrical conductivity is the inverse of resistivity and reflects the ability to conduct electrical current within the material. Once concrete is poured, it undergoes hydration process over time until reaching full strength. It is often called the bulk electrical conductivity due to the fact that the flow of the current passing through all ingredients of the concrete mixture. Electrical conductivity is a vital parameter that can reveal the level of ion migration, which is associated with cement reactions and water content (Li and Fall, 2016; Jiang et al. 2017). So, the monitoring of the electrical conductivity is an effective means of assessing the progress of cement hydration and tracking the microstructural changes that happen within the hydrating concrete materials. Thus, the variation over this conductivity throughout the process of hardening was assessed with the addition of plastic to the concrete mixture. The testing set up covered an early age of concrete after patching. This period was initially examined and found to be at three hours long on average before it becomes hard enough to remove the testing sensor out of the cylinder. Measurements were made an interval of 1 minutes for the whole period. The electrical conductivity (EC) of the samples was monitored by using a 5TE EC sensor (Decagon Devices, Inc.). This sensor measures EC in the range of 0-23 dS/m with the accuracy of ±0.1. Figure 4.47 illuminates a schematic diagram of the procedure of testing.

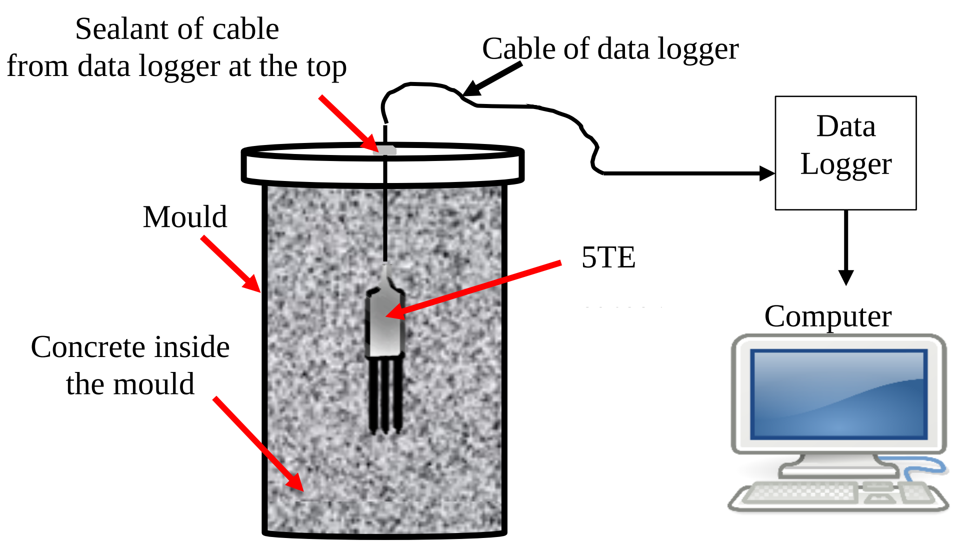


Figure 4.47. Schematic diagram of running electrical conductivity testing

4.4.8 Results and Discussions

4.4.8.1 Properties of the fresh materials

The obtained values of density, slump, as well as the air content are presented in Table 4-24. For a better visualization of the changes in the concrete mixture behavior with the addition of plastic, graphical relationships between plastic contents and the desired property of fresh concrete are shown in Figure 4.48 to Figure 4.50.

Table 4-24 and Figure 4.48 show changes in the measured densities at various combinations of plastic waste. Concrete mixture with plastic exhibited lower densities at all combinations, apparently as plastic content increased, densities decreased. However, PET plastic combinations exhibited slightly different behavior. The density value decreased at 1% of pelleted PET prior to a slight increase at a percentage of 5% and then followed the trend of all other types and started to drop again at a percentage of 10%. Reduced density of concrete specimens can be interpreted as an effect of adding low densities particles of plastic (Siddique et al., 2008). Indeed, the density of plastic is lower than that of sand by about 70% (Ismaila and Al-Hashmi, 2008). Incorporation of recycled plastic with the tested concrete reduced bulk density of plastic-contained concrete by about 40% at a content of 5% and about 60% at 10% of plastic as opposed to that of the plain concrete which provided an average density of 2.317 gm/cm³.

Table 4-24. Properties of tested fresh concrete specimens

Series	**Plastic Rate**	**Density, gm/cm^3**	**Air Content, %**	**Slump Value, mm**
Plain Concrete	0 %	2.317	5.5	110
Concrete + HDPE Flakes	1 %	2.110	7.0	0
	5 %	2.070	9.0	100
	10 %	1.699	5.9	0
	1 %	2.242	6.5	0

Concrete + HDPE Pellets	5 %	2.234	7.0	0
	10 %	1.970	5.8	0
Concrete + PET Flakes	1 %	2.121	6.5	0
	5 %	2.076	8.5	0
	10 %	1.728	6.2	0
Concrete + PET Pellets	1 %	2.119	7.0	0
	5 %	2.214	7.8	0
	10 %	2.030	6.8	0
Concrete + LDPE ground	1 %	2.310	7.0	0
	5 %	2.106	4.5	0
	10 %	1.899	6.0	0

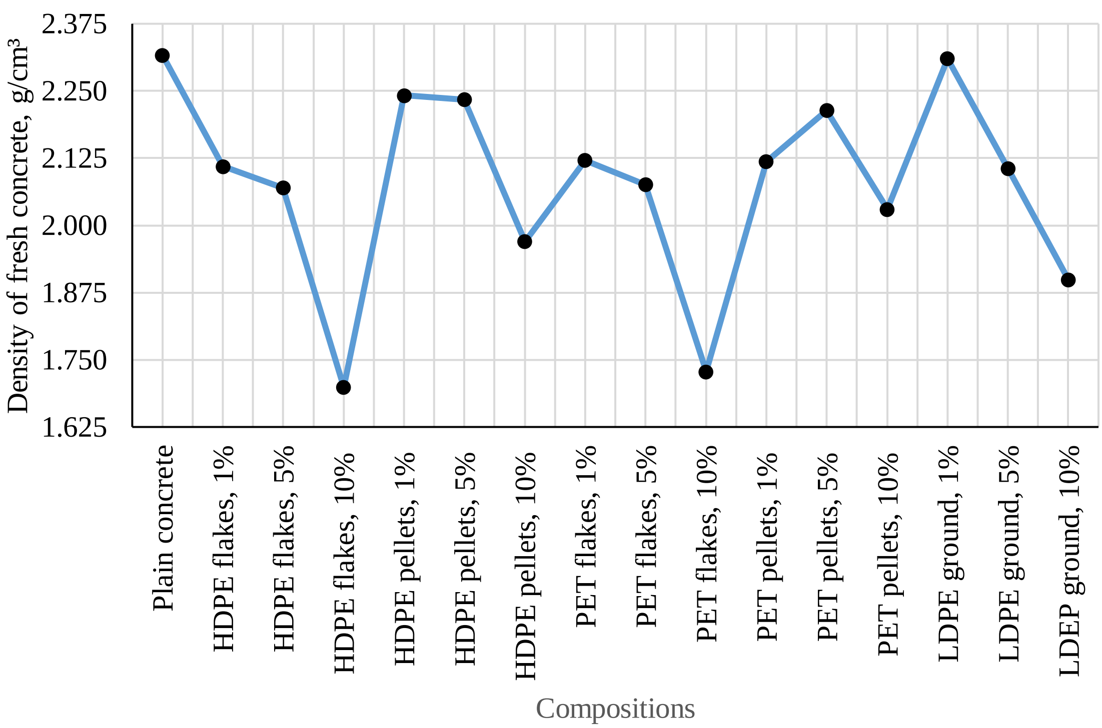


Figure 4.48. Density of fresh concrete samples

As for the workability, which is measured by means of the slump test, Figure 4.49 and Table 4-24 show the changes occurred after the incorporation of plastic. Slump dropped dramatically from a value of 110mm (concrete without plastic waste) to a complete stagnant state where measurement indicated a zero-slump value. This slump decrease can be explained by the fact that some particles are angular, and others have non-uniform shapes leading to less fluidity (Ismaila and Al-Hashmi, 2008). Batayneh et al. (2007) concluded that a decrease in the slump value is a factor of the sharper shape of added plastic particles to a concrete mixture (Batayneh et al., 2007). Plastic fibres were reported to impact the fluidity of the concrete mixture and reduce the slump value of the tested mixture (Bhogayata & Arora, 2018). Foti (2013) reported the impact of the circular shape of PET bottles on the workability and concluded that the higher the rate of incorporation the less practical the concept is due to reduced workability (Foti, 2013). Batayneh et al. (2006) reported a slump value ranged from 55mm to 80mm with the addition of up to 20% of ground unsorted types of plastic waste with concrete mixture (Batayneh et al., 2007). In our results, slump of LDPE modified concrete shrunk by 100% when 1%, 5%, and 10% of ground LDPE plastic where added to the mixture. Despite the drop in slump due to the addition of plastic, the concrete mixtures with plastic waste have easy workability and are suitable for use in pavement or road applications. Indeed, workability has a wide range from very low (at slump = 0–25 mm) applied for vibrated concrete in roads or other big sections, to high workability (at slump = 100–180 mm) applied for sections with congested reinforcement (Koehler and Fowler, 2003; Ismail and Al-Hashmi, 2008).

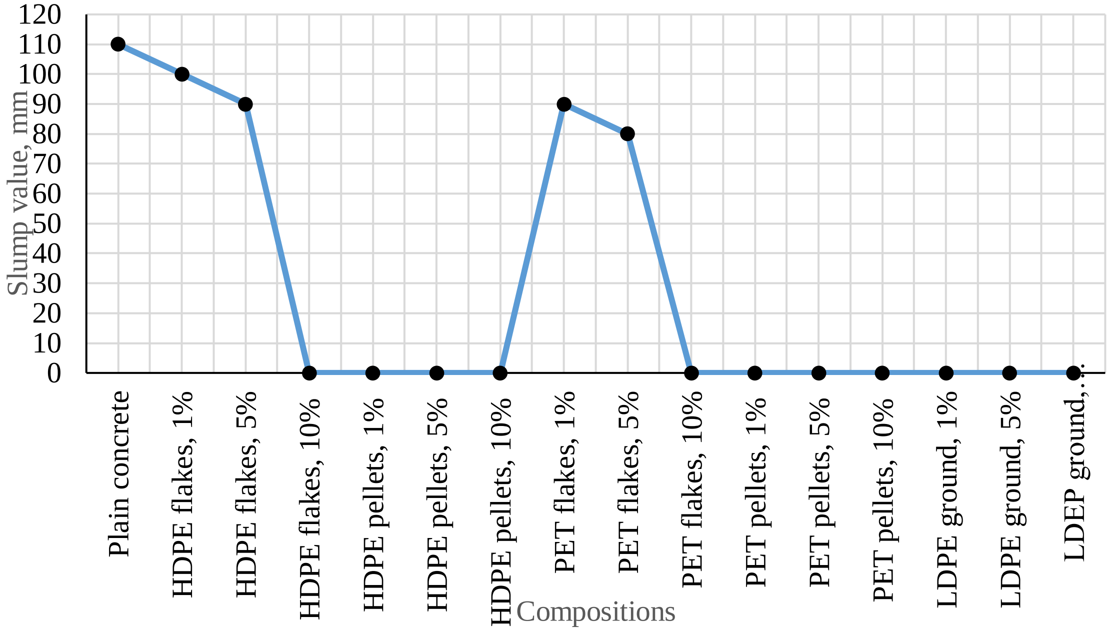


Figure 4.49. Changes in workability as measured by slump values

Results showed more workable concrete mixtures that contain flakes over that contain pellets. Flakes of plastic are known to reduce the slump of the concrete mixture where the sharp edges of these chips may have contributed larger than pellets on the slump value. This was claimed when an additional 20% of plastic particles were used to replace fine particles in a concrete mixture and slump value reported to be less (Batayneh et al., 2007). Siddique et. Al (2008) noted an opposite observation when plastic aggregates were added to concrete mixes at different rates (Siddique et al., 2008). They concluded that the increase is associated with the non-absorbed water content around the aggregates (Siddique et al., 2008). An example of the very low slump value of the concrete mixture is illustrated in the Figure 4.50.

Figure 4.50. Zero slump concrete mixtures

The entraining air content values obtained during testing are summarized in Table 4-24 above and shown in Figure 4.51. The acceptable percentage of air content is usually maintained at a percentage of 6 to 7.5% ±1.5% for concrete pavement exposed to severe conditions (exposed to wet-freeze-thaw conditions, deicers, or other aggressive agents) (Portland Cement Association, 1998). The achieved results in this study show air content values that are ranging between 4.5% and 9% of all combinations. In other words, most of the studied pavement concrete materials meet the aforementioned criterion, and thus would perform adequately when exposed to the conditions mentioned above. From Table 4-24 and Figure 4.51, it can be also noticed that the addition of plastic aggregate to the concrete increases the air content of the concrete pavement material (except for LDPEP, 5%). Similar findings were also made by Chen et al. (2015). Their test results showed that, compared to the control mix (without the incorporation of any plastic particles), around 110%, 167%, and 387% higher air content were found in concrete when fine aggregate was substituted by 30%, 50%, and 100% plastic aggregate, respectively. The authors stated that the increase in air content of concrete because of the addition of plastic particles is related to the irregular shape, immiscibility of plastic and natural sand, and hydrophobic nature of plastic.

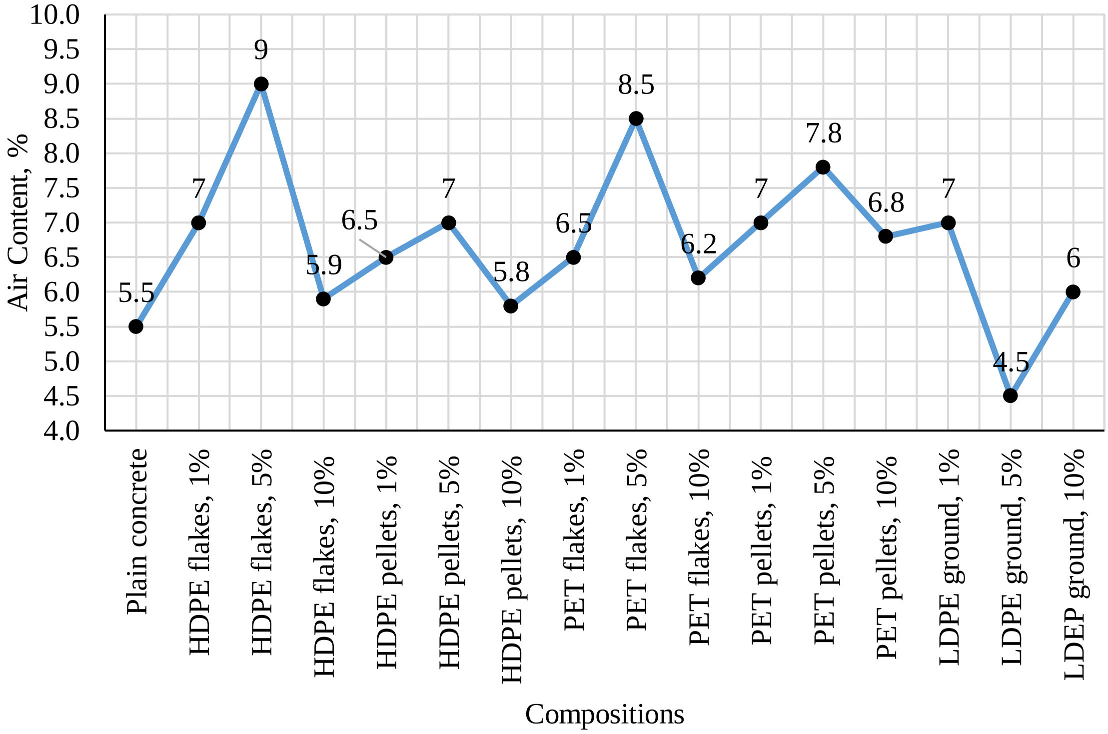


Figure 4.51. Measured air contents values for all different combinations of tested concrete pavement materials

4.4.8.2 Properties of the hardened materials

4.4.8.2.1 Bulk and apparent densities

The measured bulk and apparent densities of tested concrete mixtures, along with the percentage of water absorption (will be discussed below), prior and after mixing with the predetermined percentage of plastic are summarized in Table 4-25 and shown in Figure 4.52.

Table 4-25. Measured densities after 7 and 28 days of curing

Series	**Plastic Rate**	**Bulk Dry Density, gm/cm^3**		**Apparent Density, gm/cm^3**		**Water absorption, %**	
		7 days curing	28 days curing	7 days curing	28 days curin g	7 days curing	28 days curing
Plain Concrete	0 %	1.913	2.097	2.073	2.120	1.69	1.04
Concrete + HDPE Flakes	1 %	2.058	1.985	1.996	2.108	3.25	2.85
	5 %	2.210	1.849	1.852	1.879	2.92	3.23
	10 %	2.677	1.620	1.669	1.657	6.28	5.80
Concrete + HDPE Pellets	1 %	1.986	1.976	1.994	2.107	1.54	1.18
	5 %	2.091	1.919	1.941	1.947	1.82	1.67
	10 %	2.443	1.712	1.726	1.732	5.33	3.15
Concrete + PET Flakes	1 %	1.968	1.899	1.925	2.008	2.10	2.85
	5 %	2.190	1.750	1.848	1.875	2.87	3.20
	10 %	2.520	1.680	1.665	1.660	6.15	5.20
Concrete + PET Pellets	1 %	2.410	1.868	2.055	1.984	1.85	2.95
	5 %	2.412	1.736	1.770	1.754	2.90	3.53
	10 %	2.185	1.853	1.890	1.875	2.22	1.92
Concrete + LDPE ground	1 %	1.992	1.893	1.844	1.966	2.16	1.82
	5 %	2.119	1.896	1.770	1.919	3.32	1.44
	10 %	2.513	1.683	1.539	1.708	7.43	3.99

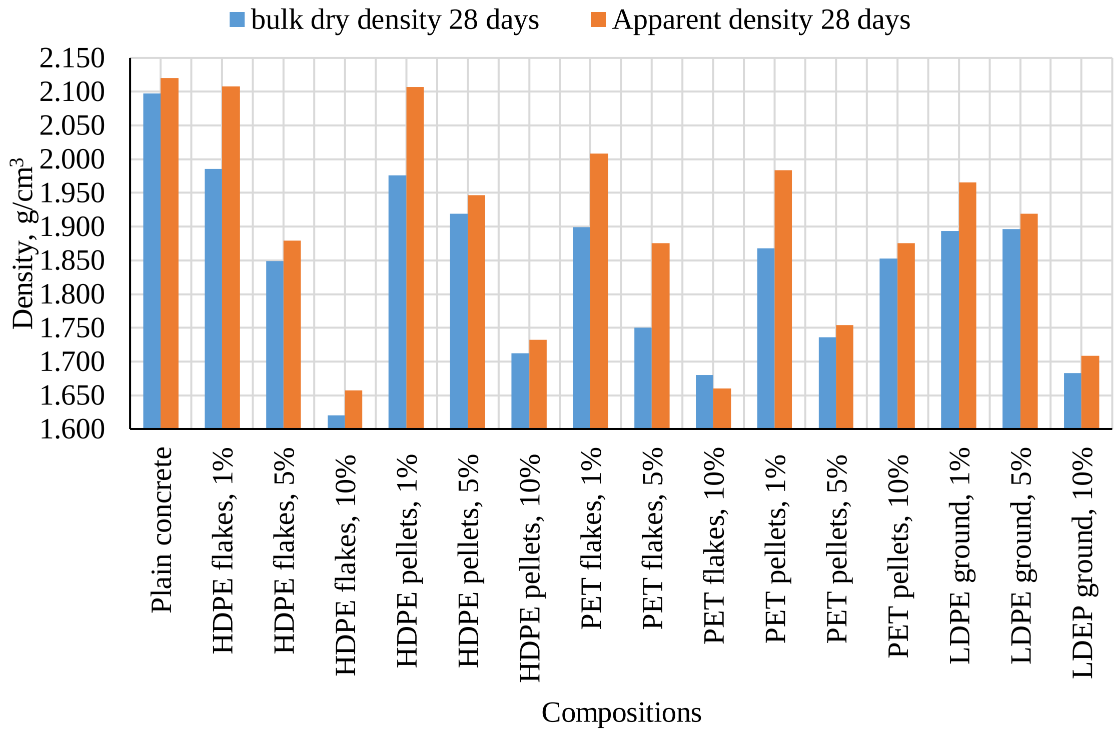


Figure 4.52. Variation of densities of hardened concrete modified with plastic

The graph, Figure 4.52, shows that the addition of plastic aggregates reduces the measured densities. Moreover, the measured densities decrease as the content of plastic waste increases, regardless of the type and shape of plastic. At 1% incorporation of plastic, an average reduction of 6-8% was observed and 20-25 % reduction at a percentage of 10% of added plastic. For samples with or more than 5% of plastic aggregates content, the dry density was lower than 2.0g/cm^3, which is the minimum dry density needed for structural lightweight concrete in accordance to the RILEM LC2 classification (Hannawi et al. 2010). Apparent and bulk densities followed closely similar trends over the same age of curing as shown in the graph. Similar observation of declined bulk density was also reported by Marzouk et al. (2005) when proportion of sand materials in the concrete mixture was substituted by percentages of up to 100% of shredded PET and shredded HDPE (Marzouk et al., 2015). Al-Manaseer & Dalal (1997) attributed this reduction to the low specific gravity of the added plastic pieces (Al-Manaseer & Dalal, 1997). Indeed, the density of concrete material is a function of the specific gravity of the mix components and compactness of concrete. Since the specific gravity of waste plastic material is commonly lower than that of natural aggregates or minerals (density of plastic is lower than that of sand by about 70%, Ismail and Al-Hashmi, 2008), it is obvious that the densities of concrete with plastic waste will decrease in proportion to the substitution level (Babafemi et al. 2018).

4.4.8.2.2 Water absorption, capillary suction and sorptivity

The water absorption or sorptivity of concrete material can be considered as a quantification of the capillary forces as exerted by the pore structure inducing the liquids to be drawn into the concrete (Hall, 1989). Water absorption test by immersion measures the ratio of water absorbed during the process of submerging samples in water at temperature of 21°C followed by a drying cycle in an oven at a temperature of 110°C. Samples of concrete submerged in water will acquire additional weight through which voids are filled with water (Concrete Institute of Australia, n.d.). This is an indication of percentage of water absorption. The change is water absorption is presented in Figure 4.53 and Figure 4.42. A glimpse at the trend of the two age periods of curing indicates a similar observation of absorption behavior. There has been a rise in absorption with all combinations of plastic waste. This would suggest that the addition of the plastic waste resulted in the coarsening of the pore structure of the concrete since water absorption indicates the degree of porosity of materials (Babafemi et al. 2018). Furthermore, it can be seen from Figure 4.53 that the water absorption generally increases as the plastic content increases, which indicates a coarsening of the pore structure of the concrete material.

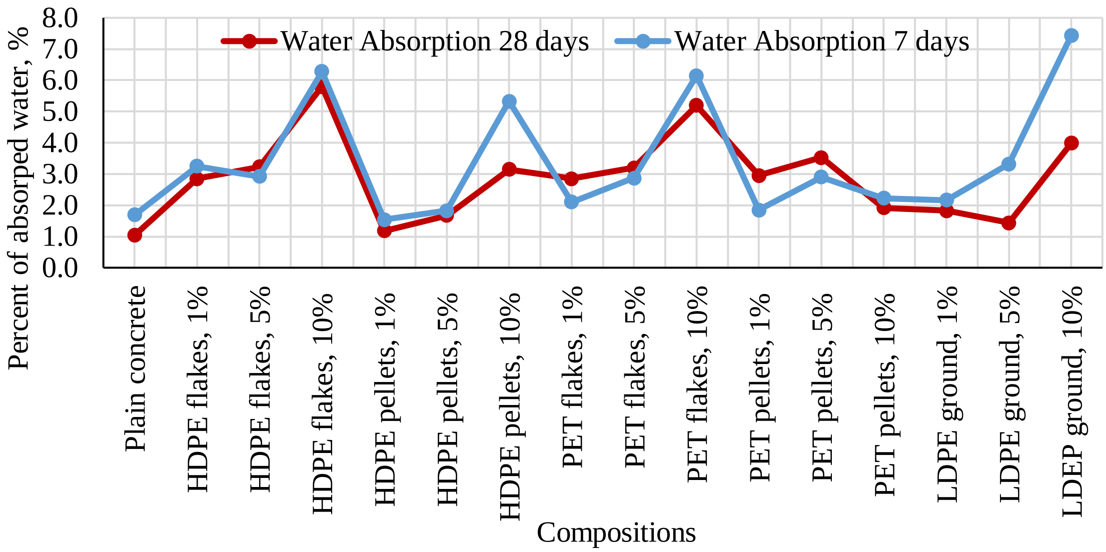


Figure 4.53. Variation of absorption of water of hardened concrete modified with plastic

As mentioned previously, sorptivity test quantifies the rate of ingress of water into the pores of the concrete by capillary suction. To compute the sorptivity coefficient (S) according to equation 1, the cumulative amount of water absorbed per cross sectional area (i) was plotted against the square root of time, then a best fit line was achieved by regression analysis and then S was calculated from the gradient of the best fit straight line. The obtained results of capillary suction and sorptivity coefficients are presented in Table 4-28. Capillary suction and sorptvity coefficients results are presented against the percentage and types of plastic, observation time, and curing ages which varied between 7 days and 28 days. This tables shows that the concrete without plastic gives the lowest capillary suction and absolute value of sorptivity coefficient, regardless of the curing time. In other words, the addition of plastic result in higher sorptivity for the concrete material. The increase in sorptivity can be more than 100%. This increase due to the incorporation of plastic waste is due to the plastic aggregates-induced coarsening of the pore structure of the concrete material. This explanation with the findings of Ruiz-Herrero et al. (2016), who observed about 200% and 140% higher porosity in concrete at 28 days with 20% inclusion of polyethylene and PVC plastic aggregates.

Table 4-26 Capillary suction results and obtained sorptivity values

Series	Plastic Rate	mm³ of water / mm² of area 7 days 5 minutes	60 minutes	Sorptivity	mm³ of water / mm² of area 28 days 5 minutes	60 minutes	Sorptivity
Plain Concrete	0 %	0.0004	0.00010	- 0.000007	0.0002	0.00040	0.00002
Concrete + HDPE Flakes	5 %	0.0009	- 0.00020	- 0.00020	0.0010	0.00010	- 0.00020
	10 %	0.0025	0.00010	- 0.0004	0.0021	0.00010	- 0.00030
Concrete + HDPE Pellets	5 %	0.0009	- 0.00020	- 0.0002	0.0007	0.00002	- 0.00010
	10 %	0.0009	0.00060	- 0.00004	0.0005	0.00060	- 0.000008
Concrete + PET Flakes	5 %	0.0008	- 0.00018	- 0.00020	0.0012	0.00010	- 0.00030
	10 %	0.0022	0.00012	- 0.00038	0.0018	0.00010	- 0.00030
Concrete + PET Pellets	5 %	0.0005	0.00030	- 0.000002	0.0010	0.00020	- 0.00010
	10 %	0.0008	0.00050	- 0.00004	0.0007	0.00050	- 0.00005
Concrete + LDPE ground	5 %	0.0017	0.00003	- 0.00030	0.0006	0.00003	- 0.00020
	10 %	0.0025	0.00030	- 0.0003	0.0014	0.00002	- 0.00020

4.4.8.2.3 Compressive, splitting tensile and flexural strengths

In concrete pavement materials, mechanical properties, such as compressive, splitting and flexural strengths, are counted among the most important parameters defining the suitability of the material for application in pavement engineering practice. Compressive strength and splitting tensile strength tests were conducted after a curing age of 28 days. However, for the importance of bending strength (flexural strength) in pavement applications as a structure endures cyclic of imposed flexural loads over the service life, MOR was examined at two curing ages; at 28-day and at 56-day curing period. This was an attempt of investigate the influence of time on the concrete mixture bending strength with the addition of recycled plastic. Obtained results are summarized in Table 4-27 as well as presented in Figure 4.55 and Figure 4.57

The variation in both compression and split tensile strengths are presented in Figure 4.55. Compressive strength of the concrete presented the highest value when plastic was not introduced (Table 4-27, Figure 4.55). In other words, the addition of plastic waste decreased the compressive strength of the concrete pavement material. Obtained compressive strengths ranged as low as 4.9MPa and spunk the high of 15.1MPa for plastic contained concrete mixture. This is virtually 23% to 72% of that obtained for the plain concrete which is a reduction of about 30% to 77%. However, at a rate of 5% of all incorporated types, the general observation was that the strength would dwindle at percentages of 1 and 5% of plastic and increased again at the highest rate of incorporation of 10%. Quite similar observation was noticeably recorded with the split tensile strength. The splitting tensile strength was found to drop by an average percentage of 48% at a rate of plastic 1%, by 30% at a percentage of 5% and then 45% at a high rate of 10% of all types. Similar observations were made by Siddique et al., (2008), who observed that splitting tensile strength reduced when plastic aggregates incorporated with concrete mixture unlike the ductility which indicated a raise for better long life performance against cracks in concrete (Siddique et al., 2008). Batayneh et al. (2007) concluded that the low strength of ground plastic used up to a 20% in the concrete mixture led to a reduction of up to nearly 72% in compressive strength (Batayneh et al., 2007). Batayneh et al. (2007) reported a slight less effect of the ground plastic on splitting tensile strength as compared to that on the compressive strength for the reduced strength of the plastic particles (Batayneh et al., 2007). Foti (2011) found that adding different forms of PET plastic from shredded bottle at rates of 0.5% and 0.75% to regular concrete mixtures also led to reducing the compressive strength of concrete (Foti, 2011). Compressive strength of concrete incorporated recycled PET was found to continue to develop with the additional curing time and drop with the change is curing temperature (Rebeiz & Craft, 1995). However, Bhogayata & Arora (2018) indicated that the use of styrene butadiene rubber (SBR) latex blended with plastic fibers at various rates (0 – 20%) was found to have significant impact on the concrete, including of improving not only the workability (by 3 to 13%) of concrete, but also on compressive strengths (Bhogayata & Arora, 2018). Moreover, Chowdhury et al. (2013) highlighted the increased in splitting tensile strength up 2% of PET fibres in the concrete mixture (Chowdhury et al., 2013). Hossainet al. (2016) computed splitting strengths of 8 to 11% of the compressive strengths for concrete mixtures modified with 5,10, and 20% of PET aggregates (Hossain et al., 2016).

This reduction in (compressive, split tensile) strength due to the addition of plastic waste aggregates can be attributed to the: (i) reduction in adhesive strength between the surface of the plastic and the cement hydration products (e.g., C-S-H, CH); (ii) coarsening of the pore structure of the concrete due to the addition of the plastic, as evidenced by the results of water absorption and sorptivity tests, which were already discussed above; (iii) hydrophobic surface of the plastic. Plastic is regarded as a hydrophobic material; accordingly, this characteristic may restrict the water necessary for cement hydration from entering through the structure of the concrete specimens during the curing period (Ismail and Al-Hashmi, 2008). This may slow down the cement hydration in hardening concrete with plastic waste, thereby producing a concrete with lower strength. This later statement is experimentally supported by the results from the electric conductivity (EC) monitoring of the concrete samples with various types and percentages (0%, 5%, 10%) of plastic as presented in Figure 4.54. This figure shows the effect of the percentage and types of plastic on the EC-time curves for the studied concrete samples. All of the curves display a common behaviour, with a rise up to the peak value of EC, then followed by a progressive decline. From this figure, it can be observed that the EC of concrete without plastic reaches the highest peak value of (5.7 mS/cm) after only three minutes. After the peak EC is reached, the EC gradually decreases with curing time. According to the working principle of EC sensors, there is a positive correlation between EC and ion movement, which indicates the rate of the chemical reaction in the concrete pavement materials (Li and Fall, 2016). These results show that, for a given type of plastic, as the plastic waste content of the concrete is increased, a longer time is needed to reach the peak maximum of the EC, which indicates the retardation of the cement hydration process. Moreover, lower initial and peak values of samples with a plastic content of 1, 5 and 10% for a given type of plastic indicate a reduced cement reaction intensity. Both of these mechanisms negatively affect the strength of concrete pavement material.

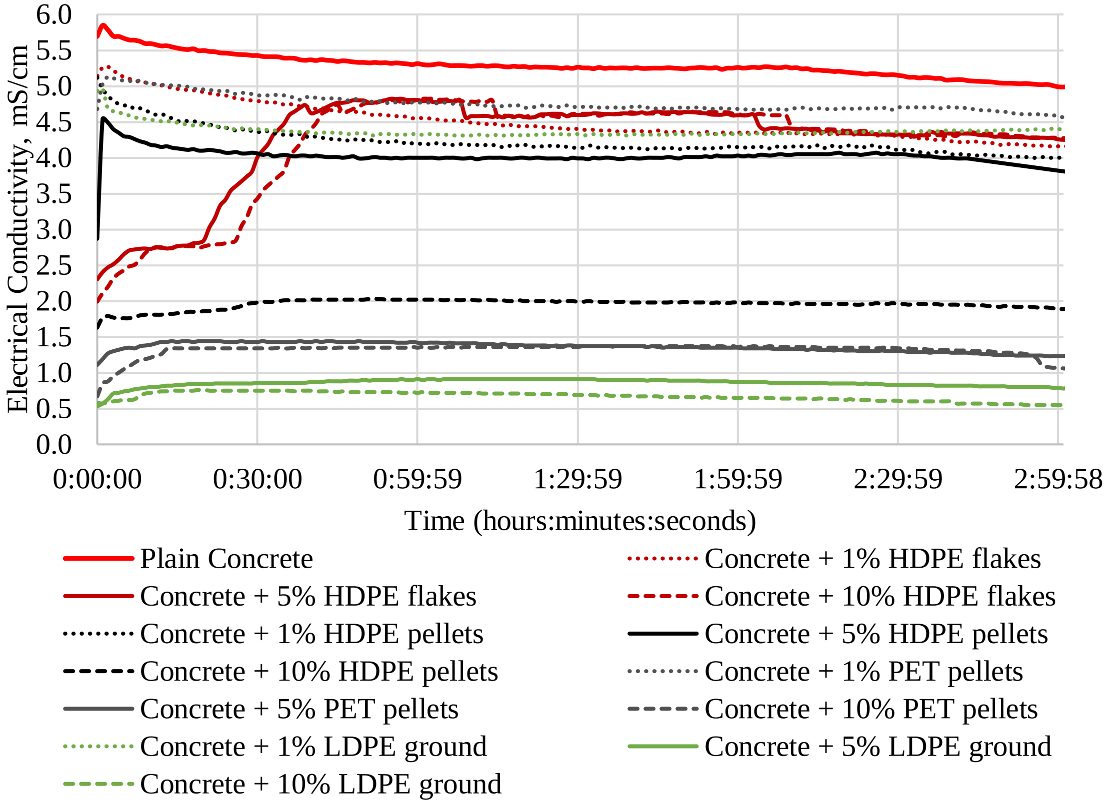


Figure 4.54 Change is electrical conductivity over the course of three hours of recording

Table 4-27 Measured strength of concrete with the addition of plastic

Series	Plastic Rate	Compressive Strength, MPa	Tensile Strength, MPa	Modulus of Rupture (MOR), MPa	
				28 days	56 days
Plain Concrete	0 %	21.2	16.8	4.0	3.2
Concrete + HDPE Flakes	1 %	13	8.2	1.0	2.6
	5 %	11	5.2	1.8	1.7
	10 %	12	7.7	1.0	0.6
Concrete + HDPE Pellets	1 %	15.6	9.1	2.7	2.4
	5 %	7.0	4.9	2.7	2.3
	10 %	11.6	8.3	1.5	1.6

Concrete + PET Flakes	1 %	12	7.9	1.0	2.3
	5 %	10	5.4	1.8	1.5
	10 %	9.8	8.2	1.0	0.9
Concrete + PET Pellets	1 %	10.2	7.6	1.9	1.8
	5 %	3.5	3.5	1.9	1.6
	10 %	6.0	4.9	3.1	2.5
Concrete + LDPE ground	1 %	11.1	8.4	1.7	1.1
	5 %	7.7	6.3	1.7	0.4
	10 %	5.8	4.5	1.4	1.6

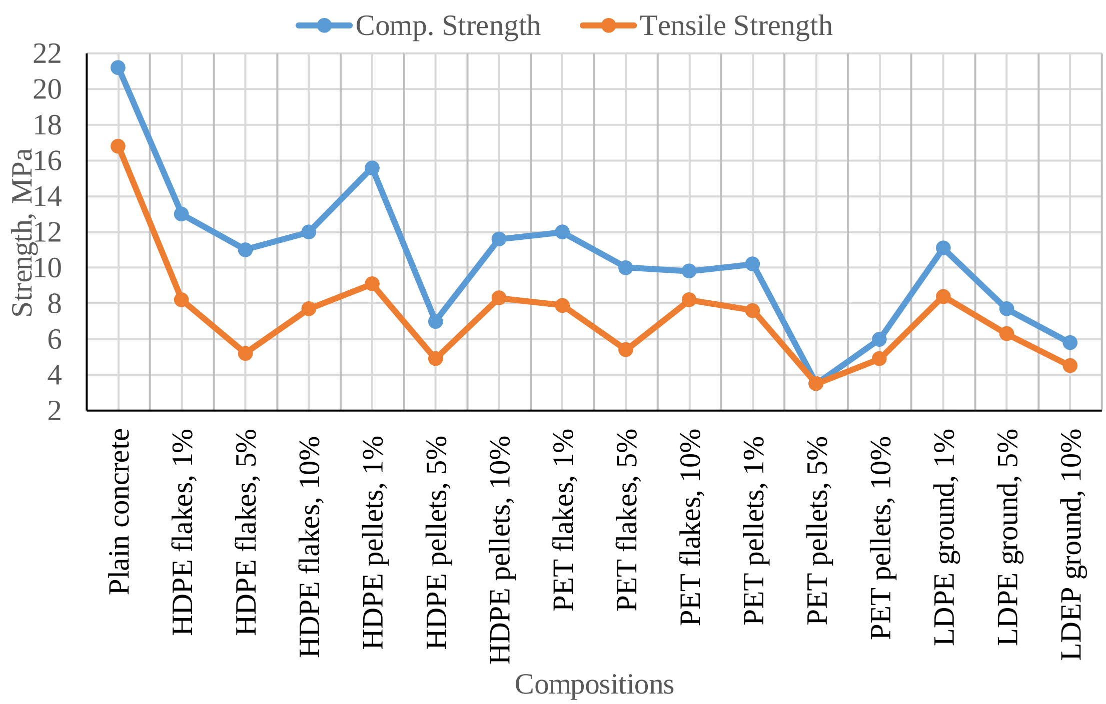


Figure 4.55. Compressive and tensile strength results of all material compositions

The influence of the type, shape and percentage of plastic on the flexural strength of the concrete samples cured at 7 and 28 days is illustrated Figure 4.56. Figure 4.56 shows that the addition of plastic waste reduces the flexural strength of the concrete, regardless of the type, shape and rate of plastic waste. The flexural strength of the plain concrete is about 4.0 MPa, whereas that of the concrete with plastic wastes varies between 3.1 MPa and 0.4 MPa, depending on the type, shape and rate of plastic waste as well as on the curing time. These findings are consistent with the conclusions of previous studies. For example, Hossain et al. (2016) concluded that the bending strength of concrete mixtures modified with various percentage of PET waste reduced as compared to plain concrete mixtures (Hossain et al., 2016). Marzouk et al. (2005) reported that flexural strength would decline by 88% when substitution of sand portion in the tested concrete mixture reached up to a percentage of 50% of both PET and HDPE waste (Marzouk et al., 2005). Foti (2011) determined that concrete mixture of an average modulus of rupture of 4.45 MPa lose some strength (18%) when a percentage of 0.5% of shredded PET was incorporated with the mixture (Foti, 2011). The mechanisms responsible for this observed drop in flexural strength induced by addition of plastic waste are same as those responsible for the plastic-induced decreased in compressive and split tensile strength.

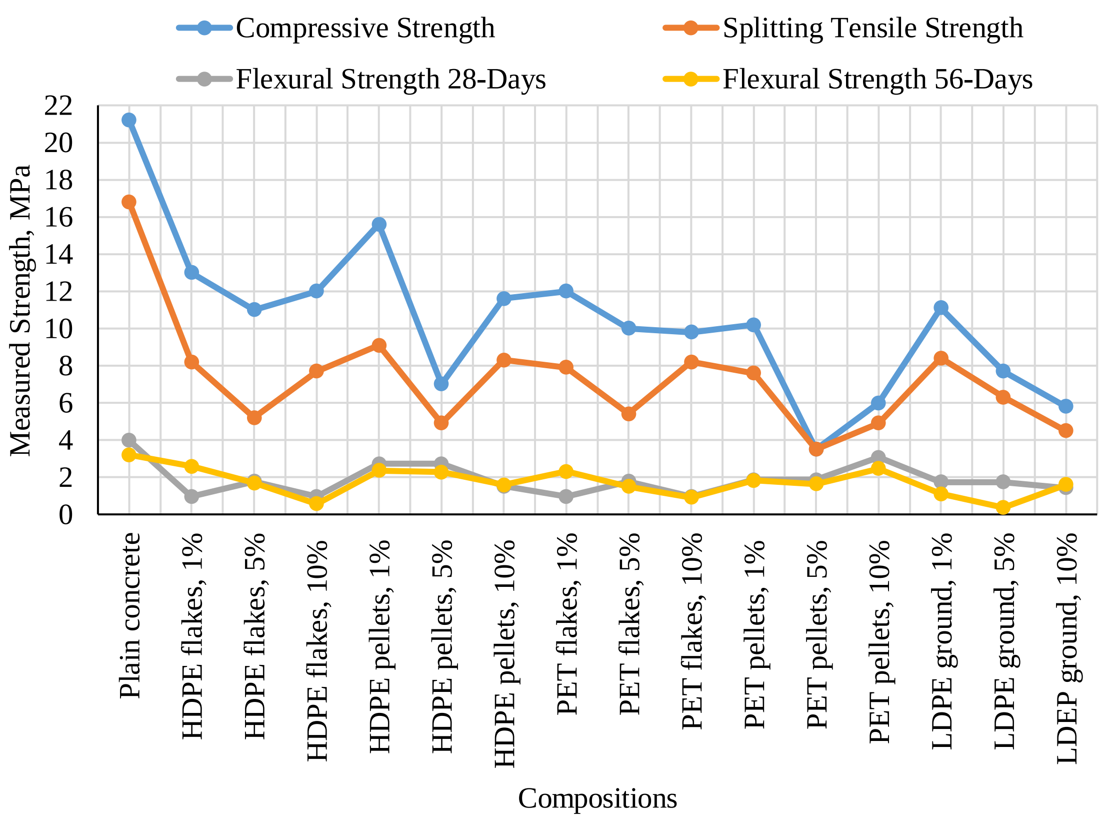


Figure 4.56. Flexural strength as opposed to compression and tensile strengths

4.4.8.2.4 Permeability

The results of the conducted water permeability tests are summarized in Table 4-28. Results indicate a slight increase of the coefficient of permeability of the concrete when 5% or 10% the natural aggregates are replaced plastic aggregates. Hannawi et al. (2010) made similar observations and concluded that such impact resulted from the weak bonding of cement and plastic. Another additional mechanism, i.e., the coarsening of the pore structure and increase of the porosity of the concrete pavement material because of addition of plastic aggregate, contributes to the increase in permeability of the concrete samples. This argument is consistent with the conclusions of water absorption and sorptivity tests presented and discussed earlier.

Table 4-28. Coefficient of permeability for samples of plain concrete and concrete mixed with plastic

Series	Plastic Rate,	Permeability Coefficient, (cm/sec)
Plain Concrete	0 %	**2.73 * 10-4**
Concrete + HDPE Flakes	1 %	$2.62 * 10^{-4}$
	5 %	$3.62 * 10^{-4}$
	10 %	$9.09 * 10^{-4}$
Concrete + HDPE Pellets	1 %	$1.04 * 10^{-4}$
	5 %	$1.20 * 10^{-4}$
	10 %	$8.56 * 10^{-4}$
Concrete + PET Flakes	1 %	$4.38 * 10^{-4}$
	5 %	$5.71 * 10^{-4}$
	10 %	$7.83 * 10^{-4}$
Concrete + PET Pellets	1 %	$4.82 * 10^{-4}$
	5 %	$6.21 * 10^{-4}$
	10 %	$5.43 * 10^{-4}$
Concrete + LDPE ground	1 %	$6.74 * 10^{-4}$
	5 %	$8.75 * 10^{-4}$
	10 %	$4.12 * 10^{-4}$

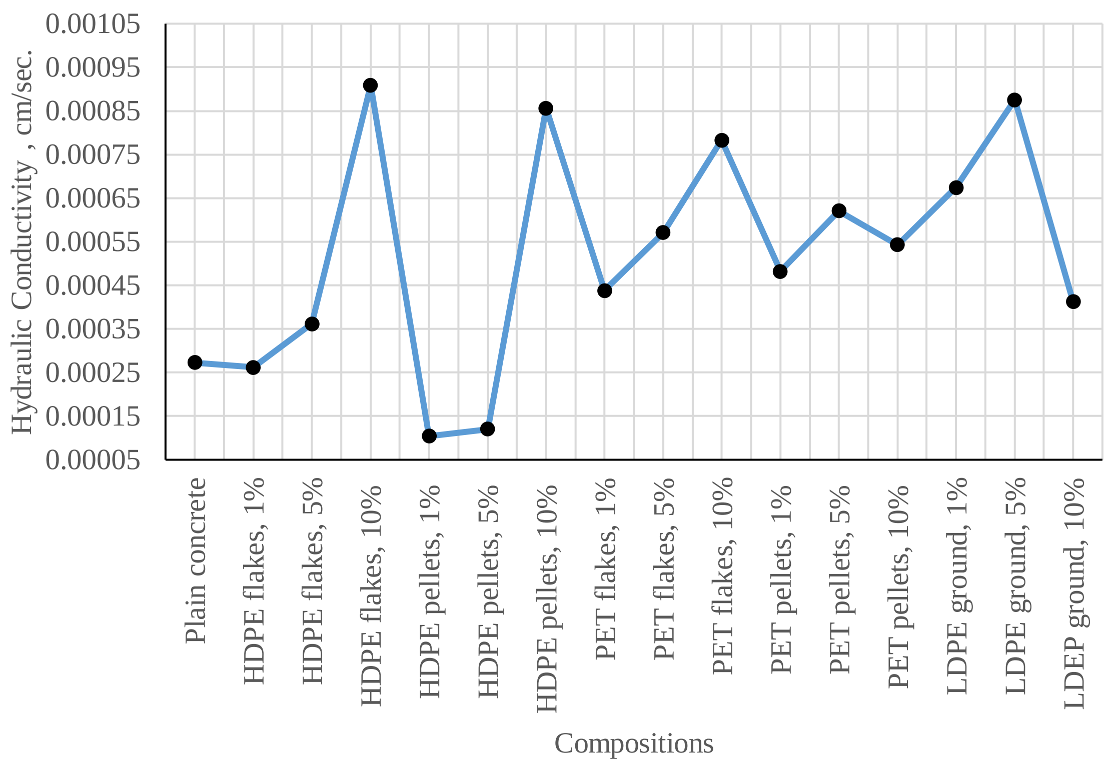


Figure 4.57. Hydraulic conductivity results

4.4.9 Summary and Conclusions

In this paper, the experimental results of a study which aims to assess and understand the effect of addition of plastic waste aggregates of different types, shape and content on the fresh (density, workability, air content) and hardened (strength, water absorption and sorptivity, permeability) properties of concrete pavement materials are presented. The following conclusions are made based on findings:

- The fresh properties of the studied concrete pavement materials are significantly affected by the addition of plastic aggregates. The density of the fresh content decreases as the plastic content increases, regardless of the type and shape of plastic, because the density of plastic is about 70% lower than that of sand. The addition of plastic aggregates to the fresh concrete mixtures significantly decreases their slump values (down to 0) regardless of the shape, type and content of plastic. Results showed more workable concrete mixtures that contain flakes over that contain pellets. However, despite this significant decrease in the slump of these fresh concrete mixtures, these mixtures with plastic waste have easy workability and are suitable for use in pavement or road applications. The addition of plastic aggregates to the concrete mixtures increases their air content because of to the irregular shape, immiscibility of plastic and natural sand, and hydrophobic nature of plastic. The air content values of the concrete with plastic are ranges between 4.5% and 9% of all combinations. This plastic aggregates-induced increase in air content is beneficial for the freeze-thaw resistance of the concrete pavement materials.

- The addition of plastic waste aggregates decreased the compressive, splitting and flexural strength of the concrete pavement samples, irrespective of the type, shape and content of plastic. The mechanisms responsible for plastic-induced strength-decrease are (1) decrease in adhesive strength between the surface of the plastic and the cement hydration products (e.g., C-S-H, CH); (2) plastic aggregates-induced coarsening of the pore structure of the concrete; and (3) hydrophobic surface of the plastic.
- The dry densities of the hardened concrete materials decrease as the content of plastic waste increases, regardless of the type and shape of plastic, because the density of plastic material is lower than that of sand by about 70%. For pavement concrete samples with or more than 5% of plastic content, the dry density is lower than 2.0 g/cm^3, which is the minimum dry density needed for structural lightweight concrete in accordance to the RILEM LC2 classification.
- The addition of plastic aggregates deteriorates the durability properties of the concrete, such as water absorption and sorptivity, and permeability. Concrete without plastic aggregates has the lowest capillary suction and sorptivity coefficient as well as coefficient of permeability. This is because the addition of the plastic waste aggregates leads to the coarsening of the pore structure of the pavement concrete material.

- Findings of the study could assist in determining the feasibility of using recycled plastic in producing light weight concrete mixture for various pavement application.

Acknowledgment

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5 Chapter 5: Synthesis of the Results and Implications for Application in Road Engineering

5.1 Synthesis and Integration of the Results

This chapter synthesizes and integrates the achieved outcomes of the three technical articles included in chapter four. It sheds the light on the importance of the outcomes and their effects on roads pavements structures. Each of the three papers focuses on a particular pavement or road structure layer; while the first and the second articles concentrate on the "geotechnical properties" of a pre-selected subgrade soil, a Sub-base and a base-course granular material currently in use in pavement applications in Ontario, Canada, the third research article turns the attention on "assessing the performance a surface concrete material modified with plastic waste". Overall, this chapter summarizes the key findings and put an emphasize on the conclusions from the three articles and what changes that brings to pavement structures. Summary of conducted work on each of pavement layers is illustrated in the following diagram in Figure 5.1.

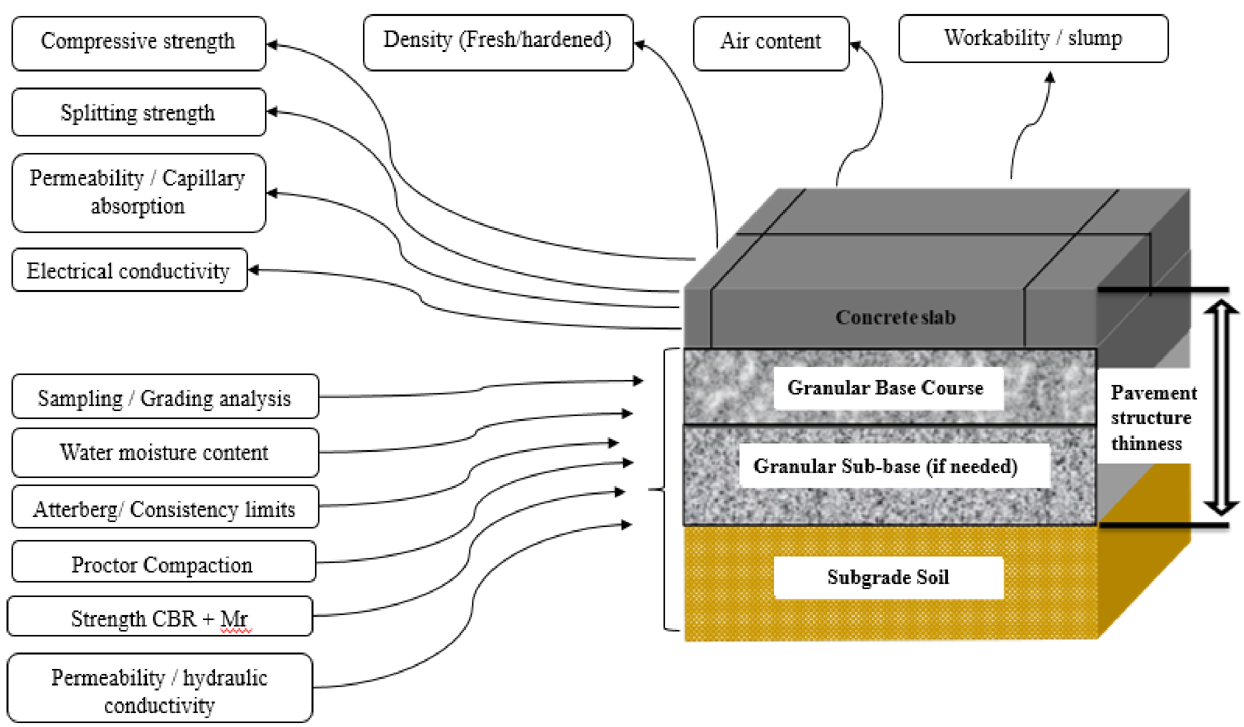


Figure 5.1. Summary of conducted experimental work

5.1.1 Pavement subgrade soil:

The four selected types of the assorted recycled plastic waste (LDPE, HDPE, PET, and PP) of three shapes (ground, pellet, and flake) used at rates varying from 1 to 10 % by weight, were employed with a pre-selected type of subgrade soil classified as an A-2-7 AASHTO. Performing multiple pavement-related experimental works revealed important results.

Proctor compaction tests showed a declined amount of water contents corresponding to lower maximum dry densities as well (lighter weights of plastic chips were associated to this declination). As one of the most common indication of the strength of soil and granular material, CBR for most tested plastics showed improved strength value which leads to a reduction of the thickness of the paved structure, and in turns, it will result in a decrease of the cost of the road or pavement structure where higher CBR materials require less thickness of any pavement structure. The resilient modulus indicated an improved value with certain types and shapes of added plastic with higher value for flakes plastic than other shapes. Shear properties tested by the UCS test showed a reduction in the strength despite the shape and type of used plastic. The nature and magnitude of the plastic-induced changes in the hydraulic conductivity depend on the plastic type and content. The pellet-modified soil has higher permeability values as opposed to the flake-modified soil. Thus, the permeability of the studied subgrade soils can be increased if the suitable amount and type of plastic waste is used. This would have positive effect on prolonging pavement performance and associated required maintenance.

5.1.2 Suggestions for adding plastic to subgrade soil

The results of this study suggest that partial replacement of soil material by plastic waste may prove useful in road subgrade applications. However, despite the encouraging results obtained, further studies are necessary:

- to provide a better understanding of the performance and behavior of the proposed plastic-modified subgrade materials by conducting geotechnical and microstructural tests, and at different scales;
- optimize the content, shape and type of plastic;
- assess the durability of the plastic-modified subgrade soils;

- determine the economic benefits that be gained through their use. Research in this area has not been accomplished for the project in this manuscript, but presently being implemented in a new project.

5.1.3 Pavement sub-base and base-course application

The geotechnical properties of two types of granular materials were assessed; granular B type II for the sub-base and granular material Type A for the base-course applications were combined with four types (LDPE, HDPE, PP, and PET) of plastic waste recycled in multiple forms (ground, flakes, and pellets) and at 1%, 5%, and 10% rates. The following findings were observed:

The mineral composition of granular as opposed to plastic particles in terms of the unit weight was responsible for the resulted lower maximum dry densities at a relatively lower water content when plastic was added at the predetermined rates. It was also concluded that the type and shape of added plastic is irrelevant in the resulted dry density of both granular materials. On the other hand, the added rate, shape, and type of plastic can predominantly affect the achieved CBR value. Improved CBR can significantly impact the required thickness of pavement structure as also indicated in the pavement design manual and school of practices around the world. In the same direction of investigation, the resiliency results by the resilient modulus demonstrated relatively similar trend as the CBR did. However, at a rate of 5% of using plastic, the highest M_R was reached despite the type, and shape of used plastic. The fact that plastic particles occupy the spaces between the large mineral aggregates led to diminishing the porosity of the granular material, is the reason for the decreased coefficient of permeability when HDPE, PET, and PP plastics of different form were added. Thus, the permeability increased with the addition of ground LDPE to both granular materials due the non-uniform shape of the used LDPE particles that generated significant space in the pore structure of the granular materials for water to flow through the areas between the solid particles.

5.1.4 Suggestions for adding recycled plastic to granular materials

The obtained results of examining the incorporation of recycled plastic to two different graded granular materials suggest the followings:

- Partial replacement of granular B type II sub-base material and granular A base granular materials with plastic waste may prove useful in road applications
- This replacement could help to reduce the amount of plastic waste to be managed.
- It also may help bring the cost of road construction materials down.
- Results suggest that research is required to optimize the plastic characteristics (content, type and shape) with respect to the performance of the base and sub-base materials.
- Also, future study is necessary to assess the resilient modulus of the granular materials with optimal plastic characteristics, and
- It is to utmost need to conduct large scale or field scale testing of the performance of sub-base and base materials with the optimal plastic characteristics.
- Results reflect the performance of tested materials in regular weather conditions. Testing the performance of materials modified with recycled plastic would be beneficial for a more conclusive result.

5.1.5 Pavement surfacing layer (finding and suggestions)

In the final research article, the impact of adding plastic waste of different types, shape and content on fresh and hardened concrete was investigated. The following conclusions are made based on findings:

- The fresh properties of the studied concrete pavement materials are significantly affected by the addition of plastic aggregates.
- The density of the fresh concrete mixture decreases as the plastic content increases, regardless of the type and shape of plastic, because the density of plastic is about 70% lower than that of sand.
- The addition of plastic aggregates to the fresh concrete mixtures significantly decreases their slump values regardless of the shape, type and content of plastic. Despite the significant decrease in the slump of freshly mixed concrete, these mixtures with plastic waste have easy workability and are suitable for use in pavement or road applications.

- The addition of plastic aggregates to the concrete mixtures increases their air content, which ranged from 4.5% to 9% of all combinations, because of to the irregular shape, immiscibility of plastic and natural sand, and hydrophobic nature of plastic. This plastic aggregates-induced increase in air content is beneficial for the freeze-thaw resistance of the concrete pavement materials.
- The addition of plastic waste aggregates decreased the compressive, splitting and flexural strength of the concrete pavement samples, irrespective of the type, shape and content of plastic.
- The dry densities of the hardened concrete materials decrease as the content of plastic waste increases.
- The addition of plastic aggregates deteriorates the durability properties of the concrete, such as water absorption and sorptivity, and permeability. Concrete without plastic aggregates has the lowest capillary suction and sorptivity coefficient as well as coefficient of permeability. This is because the addition of the plastic waste aggregates leads to the coarsening of the pore structure of the pavement concrete material.
- Findings of the study could assist in determining the feasibility of using recycled plastic in producing light weight concrete mixture for various pavement application.

5.2 Advantages and Disadvantages of Recycled Plastic Waste in the Tested Materials of Pavement Structure

Obtained results indicate that incorporating recycled plastic materials in pavement or road layers has some advantages, but also disadvantages. This section points out advantages and disadvantages of incorporating recycled plastic materials into road layers or materials so as to facilitate the selection of the suitable recycled plastic materials and further development of the technology of recycling plastic waste materials into road construction materials. The main advantages or benefits and disadvantages are discussed below.

- The reduced OMC at lower MDD due to the addition of plastic waste materials means a reduced compaction effort on road construction sites, which, in turn, will lead to a reduction of the cost related to the compaction as required soil moisture content and compaction energy are both reduced.

- Obtained results indicate that addition of plastic wastes to the granular road layers can increase or decrease their CBR values depending on the types, shape and the adding rate of plastic. An increased CBR value will not only enhance the mechanical performance of the road structure, but also is linked with a thinner road or pavement structure. The latter is obviously associated with a lower construction comstruction cost of the road. In opposite, it should be anticipated that plastic waste-modified pavement layers with reduced CBR values will show poorer mechanical performance. Moreover, a thicker pavement structure is required in this case. The results discussed above suggest that the selection of the suitable type, shape and content of plastic waste is critical for obtaining higher CBR values, and thus for the cost-effective design of road that incorporates layers with recycled plastic wastes. Since the MR was estimated based on CBR-dependent models, the aforementioned conclusion applies to the MR of the granular materials too.
- The incorporation of plastic waste led to a raise in the permeability of the subgrade materials. This is beneficial to the mechanical performance of the road structure since higher hydraulic conductivity value is associated with enhanced drainage ability of the water infiltrated in the subgrade layer. Good drainage can increase as well as maintain the strength of the subgrade. However, the sub-base and base materials with plastic waste have lower values of saturated conductivity than those without plastic materials. In other words, the addition of pastic waste to the sub- and base materials decreases their water drainable ability, which is not benefical to the mechanical performance of the pavement structure.
- The observed decrease in compressive, splitting and flexural strength of the concrete due to the addition of plastic waste aggregates indicates that this plastic modified concrete material might not suitable pavement surfacing layer materials for heavy traffic roads, but might be utilized in light weight concrete application (e.g., roads with light traffic, parking areas, pedestrian walkways, and greenhouses).
- This plastic aggregates-induced increase in air content enhances the freeze-thaw resistance of the concrete pavement materials, which is an advantageous characteristic in cold regions. However, the addition of plastic aggregates deteriorates the other durability properties of the concrete, such as water absorption and sorptivity, and permeability. These two opposite effects on durability of pavement materials with recycled plastic

materials need to be considered in the practical application/implementation of this developed road or pavement material.

5.3 Comparison of obtained Results with Previous Results

As discussed earlier, past investigations on the potential use of recycled plastic materials in the road industry have basically focused on the use of plastic waste in the surfacing layer of flexible pavement. This use of the recycled plastic material has enabled to enhance the properties of asphalt mixtures, such as stability, tensile strength, and resistance to low temperature cracking. However, to the best knowledge of the PhD candidate, only one study on plastic aggregates into road/pavement layers has been conducted (Dhatrak & Konmare, 2016). Dhatrak & Konmare (2016) assessed the impact of adding non-sorted shredded plastic (at a rate of 0 to 2.5%) to a subgrade soil on the compaction characteristics and CBR of the subgrade soil modified with these plastic aggregates.

In this section, the compaction and CBR results obtained in the study by Dhatrak & Konmare (2016) are compared with those obtained in this thesis.

Figures 5.1 and 5.2 feature the compaction curves of the subgrade soils modified with plastic aggregates obtained by Dhatrak & Konmare (2016) and in the present study, respectively. From these figures, it can be observed that both studies have obtained similar results with respect to the impact of adding plastic aggregates on maximum dry density of the subgrade soil. From these figures, it can be seen that as the proportion of plastic increased, the corresponding maximum dry density decreased accordingly. In other words, the modified road subgrade soil becomes lighter. Moreover, the optimum water content generally decreased as the plastic content increased.

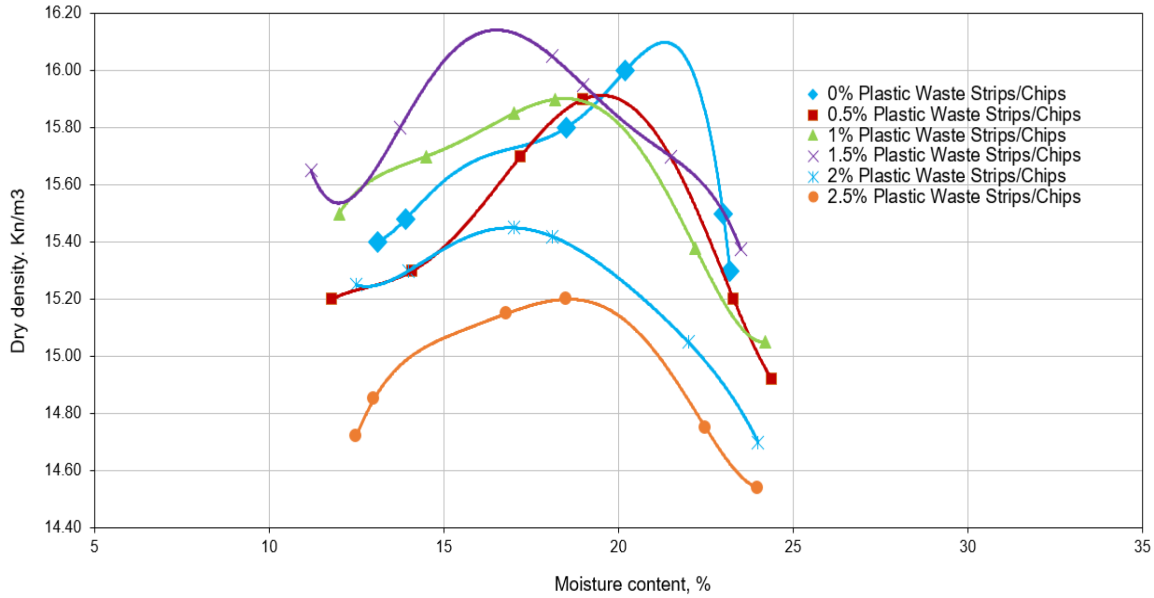


Figure 5.2 Compaction curves of a soil mixed with shredded plastic of various contents and types (Dhatrak & Konmare, 2016)

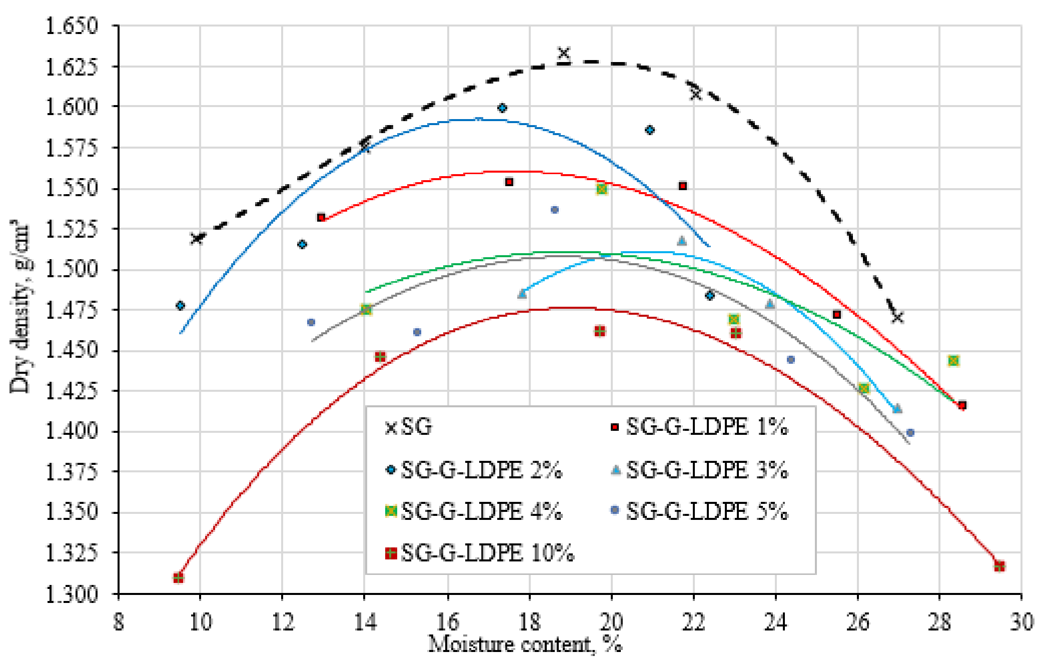


Figure 5.2 Compaction curves of the natural and ground-LDPE modified soil (obtained in this study)

The CBR curves of the modified subgrade soil for different proportion of plastic aggregates obtained by Dhatrak & Konmare (2016) are shown in Figure 5.3, whereas the CBR curves for different material combinations of plastic type, shape and content acquired in the presebt study are illustrated in Figure 5.4. The comparison of these two figures indicate that the CBR results obtained in the present study are also in agreement with those obtained by by Dhatrak & Konmare (2016). Indeed, Figure 3 shows that depending on the plastic content, the strength of the subgrade soil samples can be enhanced with the addition of plastic aggregates. Similar observations or conclusions were made in the Technical Paper I of the present thesis manuscript.

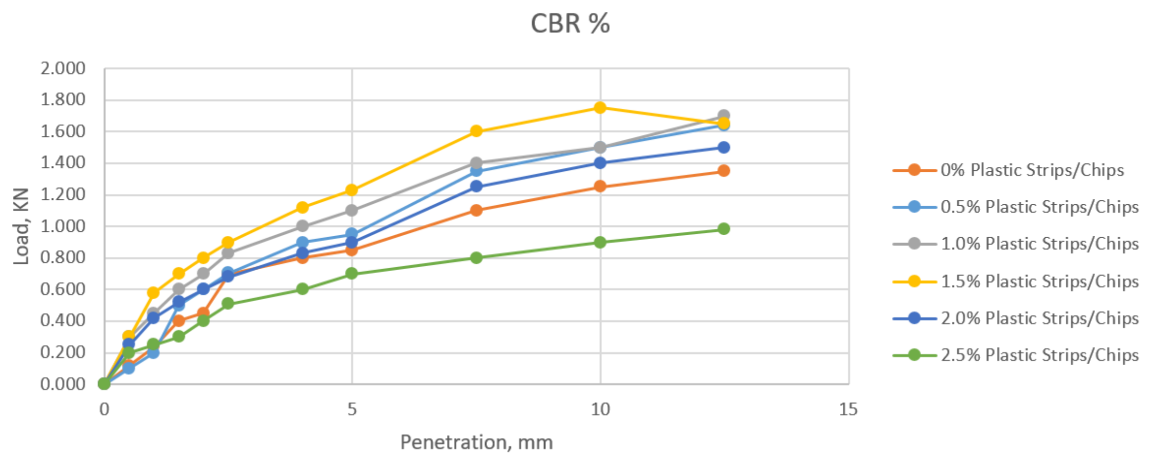


Figure 5.3 CBR results of tested non-soaked soil with various percentage of plastic chips (Dhatrak & Konmare, 2016)

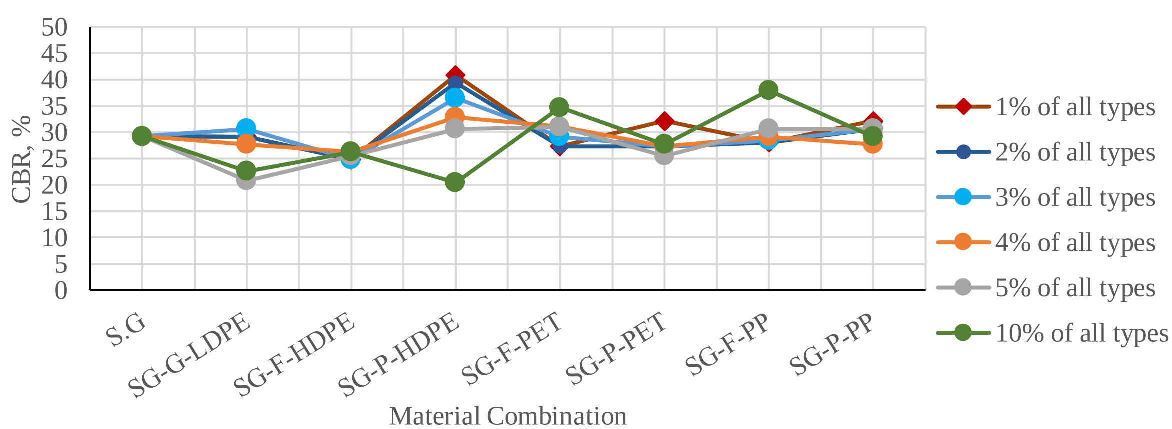


Figure 5.4. Changes in CBR values with changes in plastic types and rates

5.4 Summary and Conclusions

The thesis is structured in papers format and the following paragraphs highlight the main the drown conclusions from each technical paper.

In sub-chapter 4.2, ground, flakey and pelleted LDPE, HDPE, PET, and PP types of recycled plastic waste were incorporated at rates ranged from 1 - 10wt% with an A-2-7 AASHTO type of subgrade soil. Geotechnical tests were carried out and results collected. The results indicated reduced maximum dry densities of modified samples as opposed to the density of the plain soil which minimize the required field compaction efforts of the soil in practice events. Additionally, the strength of the soils as measured by the CBR test was fluctuated. This fluctuation was associated with the types and content of plastic used with the soil. Designers aimed at reducing the required pavement thickness that is attributed mostly to the better CBR value of the soil. Pellets in this research showed favorable attained CBR results. Similarly, pellets-induced changes to the modified soil is an indicative that better pavement performance might be attainable with the studied recyclable plastic waste.

Furthermore, in sub-chapter 4.3, a gap graded Sub-base material along with a dense graded base granular material was examined geotechnically with the addition of various types of recycled plastic in many forms. Both materials shown lower water contents at reduces densities when incorporated plastic as opposed to when did not. The CBR also was enhanced for some types of plastic over others. That was attributed to the shape of added plastic. Thin Pellets presented better CBR results as compared to longer and thicker counterparts. The resilient modulus improved with certain types and shapes of incorporated plastic. Reduced permeability serves as a good indicative parameter from a design perspective, as the less the time it takes for the material to drain water, the preferable is the pavement structure.

In the final sub-chapter, chapter 4.4, which covers the final technical article in the theses, a concrete surfacing material of 30MPa was selected. The selection of concrete over asphalt mixture was opted for aspects included environmental reason as less menace may pose by concrete cement over asphaltic bituminous mixture. Prominently, ductile concrete mixtures resulted when plastic was added with less absorption of water predominantly manifested with flakey and pelleted plastic over ground despite the raised permeability as measured by the hydraulic conductivity of the mixture. We concluded that several light concrete applications can be utilized with the proposed plastic materials.

It is then summarized that the occurred changes to the properties of all tested materials may have an impact of reducing the overall pavement thickness required to an applied traffic load. Pavements for light traffic could be a potential tentative consumer for materials incorporated plastic waste recycled of multiple shapes and of various types. Aside from that, having to provide potential utilization of the over burden waste by the potentiality of consuming plastic in the road construction industry account for a leap in the field. Further materials ought to be examined with wider range of recycled plastic. It is however an early stage to come to a decisive conclusion as further examination is necessary.

5.1 References

Dhatrak, A.I., Konmare, D. S. (2016). Laboratory Performance of Randomly Oriented Plastic Waste in Subgrade of Flexible Pavement. *International Journal of Innovative Research in Science, Engineering and Technology*, 5(3), 3969–3976.

6 Chapter 6: Conclusions and Recommendations for Future Studies

6.1 Conclusions

The work presented in this thesis is a combination of two stages. The first one focuses on the geotechnical impact of recycled plastic waste on the subgrade soil, sub-base, and base-course material in pavement structures applications. The second phase is assessing the performance of a concrete surfacing material with the addition of recycled plastic waste. This chapter addresses the key conclusions and provides recommendations as well for future research studies.

The results of this study suggest that partial replacement of soil material by plastic waste may prove useful in road subgrade applications. However, despite the encouraging results obtained, further studies are necessary: (i) to provide a better understanding of the performance and behavior of the proposed plastic-modified subgrade materials by conducting geotechnical and microstructural tests, and at different scales; (ii) optimize the content, shape and type of plastic; (iii) assess the durability of the plastic-modified subgrade soils; (iv) determine the economic benefits that be gained through their use. Research in this area has not been accomplished for the project in this manuscript, but presently being implemented in a new project

The two materials have slightly different compositions in terms of aggregates sizes. This may have impacted the proctor densities parameters. The granular B type II and A type materials exhibited lower maximum dry densities at a relatively lower water content once mixed with plastic.

The obtained CBR results indicate enhancements in strength which changed with changes in shapes of the incorporated plastic. Pellets of thinner diameter presented better CBR results as compared to longer ones of thicker diameter. Resilient modulus is best determined experimentally and compared with available models. Thus, the results obtained reflect theoretical calculations as it was mainly based on the tested CBR values in the laboratory.

The following conclusions are made based on findings of testing using concrete with various types and shapes of recycled plastic materials. Recycled plastic in concrete application can be used in variety of applications including utility components, transportation-related components and some marine construction materials as well as some minor repair work. This explain the potential incorporation of concrete contains plastic waste in non-structural application which in turns reduce the incurred cost. This aim may in turn help in promoting the tested concrete as a lightweight material.

6.2 Recommendations for future studies

- Other soil types are suggested to further better comprehend the behaviours of the soils in subgrade applications. This could be achieved through conducting microstructural tests alongside with the geotechnical tests. The examination could also include the optimization of plastic types and rates with various soil types. Assessing the durability of the plastic-modified subgrade soils though the investigation of impact of freezing and thawing of various cycles is also crucial to understand the behaviours of the plastic-modified soils, particularly in area susceptible to frost-heave actions.
- It would be more conclusive to investigate the impact of adding additional types of plastic with other types of granular materials. It could be also beneficial to examine the addition of other alternative materials such as reclaimed asphalt pavement (RAP), or crushed glass.
- Testing the modulus of resilience experimentally would help correlate model results with lab results and come up with a new model for estimating the modulus of resilient of subgrade soil, as well as the granular Sub-base and base materials.
- The impact of freezing and thawing is paramount to study in cold regions. It is essential to further examine the influence of freeze-thaw cycles on the investigated materials.
- Providing availability of testing equipment, compliance with safety regulations, testing asphalt concrete with the utilized recycled plastic would pave the way towards further research paths.

7 Appendixes:

7.1 Materials and Testing equipment

Figure 7.1 Subgrade soil bags supplied from site for testing

Figure 7.2. Granular materials used for the study

Table 7-1 Graduation requirements for granular B type II sub-base and type A base-course material

Sieve size		Granular B type II, Sub-base		Granular A, Base-course	
Inch	**mm**	**Lower Limit, %**	**Upper Limit, %**	**Lower Limit, %**	**Upper Limit, %**
4.24"	106	100	100	-	-
1.06"	26.4	50	100	100	100
3/4"	19.0	-	-	85	100
0.53"	13.2	-	-	65	90
3/8"	9.5	-	-	50	73
No.4	4.75	20	55	35	55
No.16	1.18	10	40	15	40
No.40	0.425	-	-	-	-
No.50	0.30	5	22	5	22
No.200	0.075	0	10	2	8

Figure 7.3. Types of plastic used in the research

Figure 7.4. Sample splitter

Figure 7.5. Sieve shaker used for grading analysis

Figure 7.6. material automatic mixer

Figure 7.7. mixer for preparing Sub-base and base batches for later testing

Figure 7.8. Automated proctor compactor

Figure 7.9. Sample inside proctor mold after compaction

Figure 7.10. CBR machine

Figure 7.11. Permeability mold

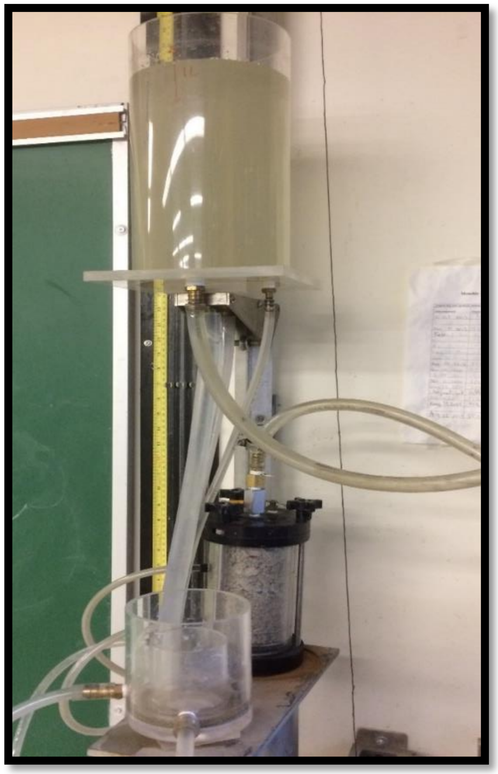


Figure 7.12. Device used for Permeability test

Figure 7.13. Soil sample with pellets type of plastic for moisture testing

Figure 7.14 . Soil sample with flakey type of plastic for moisture testing

Figure 7.15. Soil sample with pellets type of plastic for moisture testing

Figure 7.16. Concrete mixer and mixing process of concrete with plastic

Figure 7.17. Illustration of samples curing in water prior to testing

Figure 7.18. Slump Testing

Figure 7.19. measuring density of fresh concrete

Figure 7.20. Aerometer for measuring air content of fresh mixture

Figure 7.21 Shear samples being weighed

Figure 7.22. Samples in oven being prepared for density testing

Figure 7.23. Zero slump concrete mixtures

Figure 7.24 Concrete beam ready for flexural testing

Cracking position of failure under bending loading

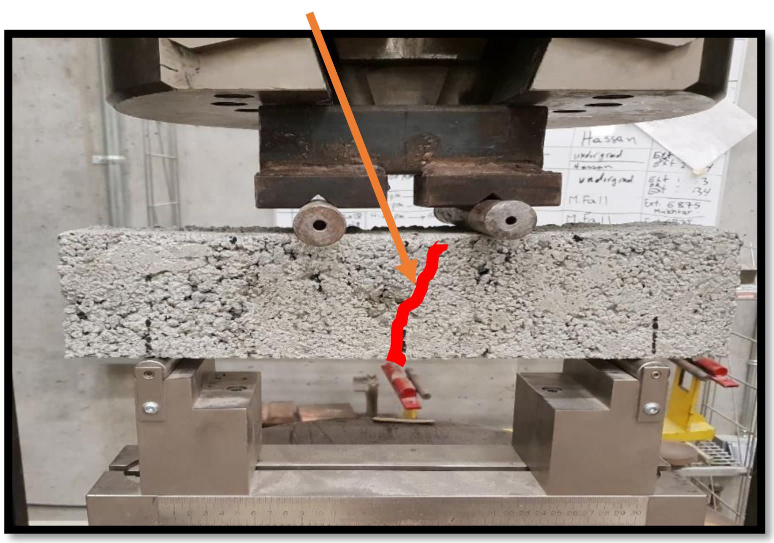


Figure 7.25. Flexural testing

Figure 7.26 Cylinder samples for capillary testing

Figure 7.27 Samples casted in cylinders and connected to data acquisition for conductivity testing

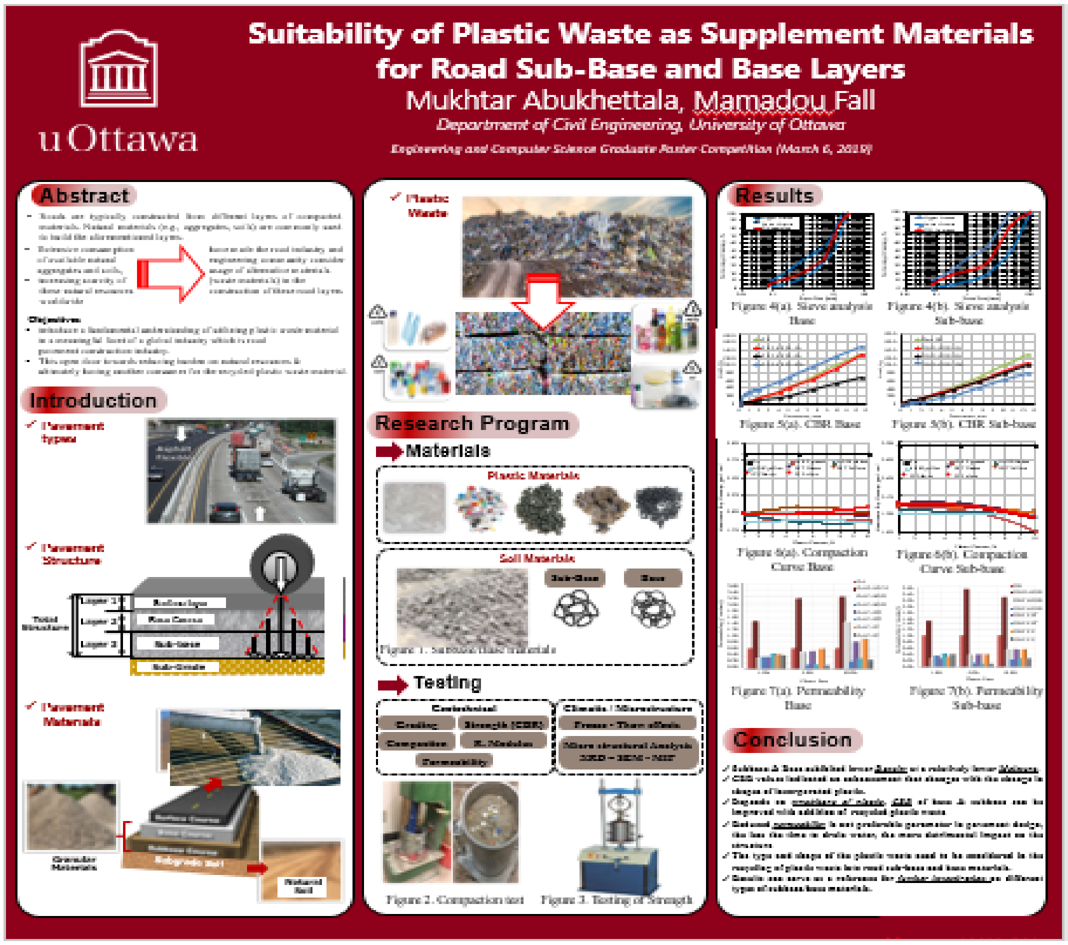


Figure 7.28. Poster during the graduate engineering poster competition

7.2 Compaction proctor test

Proctor compaction procedure has two main types which differ in the compaction effort applied to the material. The standard test applies the 2.5kg compaction hammer that was then evolved to tailor the increase in loading over the year to become a 5kg hammer. Also, while the standard test compacts the sample in three consecutive layers in the same manner, the modified test applies compaction to five similar layers instead of the three. The objective is to determine the highest density at the optimum moisture where voids are at their lowest. The modified proctor is more common than the standard test. The higher the compaction force, the higher the obtained density and the lower the moisture content is. The two components that are essential for identification is the MDD at the OMC.

These standard tests are executed in the laboratory and are conducted according to ASTM D698 for the standard test and ASTM D1557 for the modified compaction test. Overall, the variation in the OMC from the modified test is about 2-5% higher than that of the standard test, and the variation of the MDD is about 85-97% higher in the modified test than in the standard. Illustration of the proctor curve in showed in figure (ASTM, 2012; Ameratunga et al., 2016)

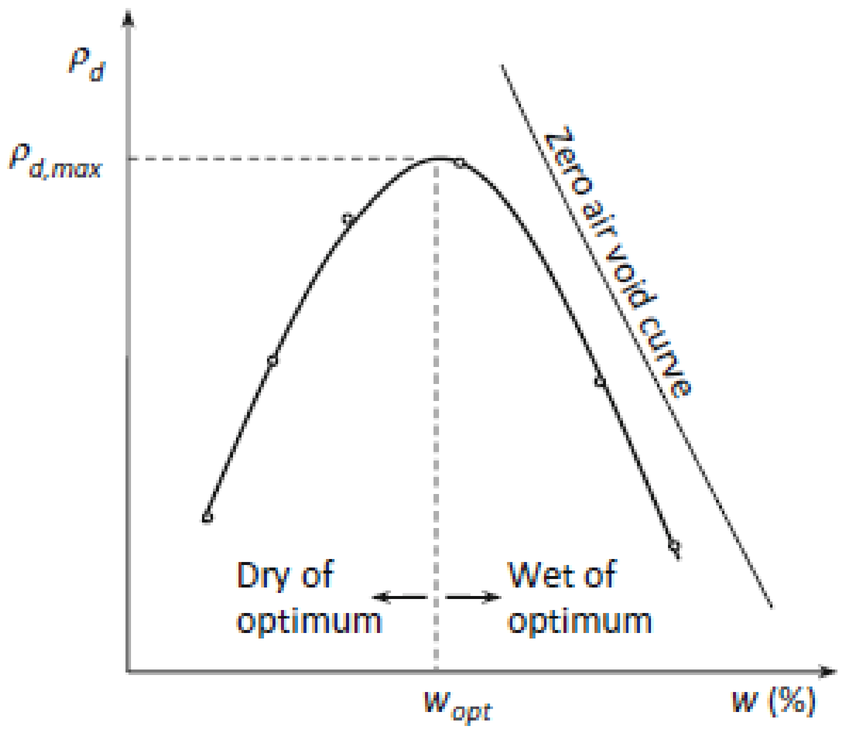


Figure 7.29. Obtaining the proctor curve of the compaction test (Ameratunga et al., 2016)

7.3 Samples of test results

PROCTOR TEST		Reference (AASHTO T190, ASTM D1557)	
Material	SUBGRADE SOIL		
Plastic waste type	NA	Sampling date	24-May-2017
Rate of usage	NA	testing date	31-May-2017

Volume of mould 1 (V1) 983.0 cm3 Volume of mould 2 (V2) 940.0 cm3

Test No.			1		2		3		4		5	
Mass of mould + base + compacted specimen,	M1	gm	5780.0		5985.0		6035.0		6055.0		6055.0	
Mass of mould + base,	M2	gm	4210.0		4220.0		4210.0		4210.0		4220.0	
Mass of compacted specimen,	M = M1 - M2	gm	1570		1765		1825		1845		1835	
Bulk Density,	r = M / V	g/cm^3	1.670		1.796		1.941		1.963		1.867	
Container No.			A	B	H	F	C	D	A	B	C	
Mass of container,	m1	gm	2.14	2.14	2.15	2.14	2.14	2.14	2.09	2.07	2.07	
Wet soil + container,	m2	gm	65.24	70.39	47.69	48.76	68.06	74.27	83.18	76.95	106.86	
Dry soil + container,	m3	gm	59.36	64.45	42.06	43.10	57.17	63.30	68.33	63.59	84.59	
Mass of moisture,	m4 = m2 - m3	gm	5.88	5.94	5.63	5.66	10.89	10.97	14.85	13.36	22.27	
Mass of dry soil,	m5 = m3 - m1	gm	57.22	62.31	39.91	40.96	55.03	61.16	66.24	61.52	82.52	
Moisture Content ,	W = m4 / m5 x 100	%	10.3	9.5	14.1	13.8	19.8	17.9	22.4	21.7	27.0	
Average Moisture Content ,	W	%	9.9		14.0		18.9		22.1		27.0	
Dry Density,	rd = 100r / (100 + w)	g/cm^3	1.520		1.576		1.633		1.608		1.470	
Max Dry Density		Mg/m^3	**1.620**									
Optimum Moisture Content		**%**	**20.0**									

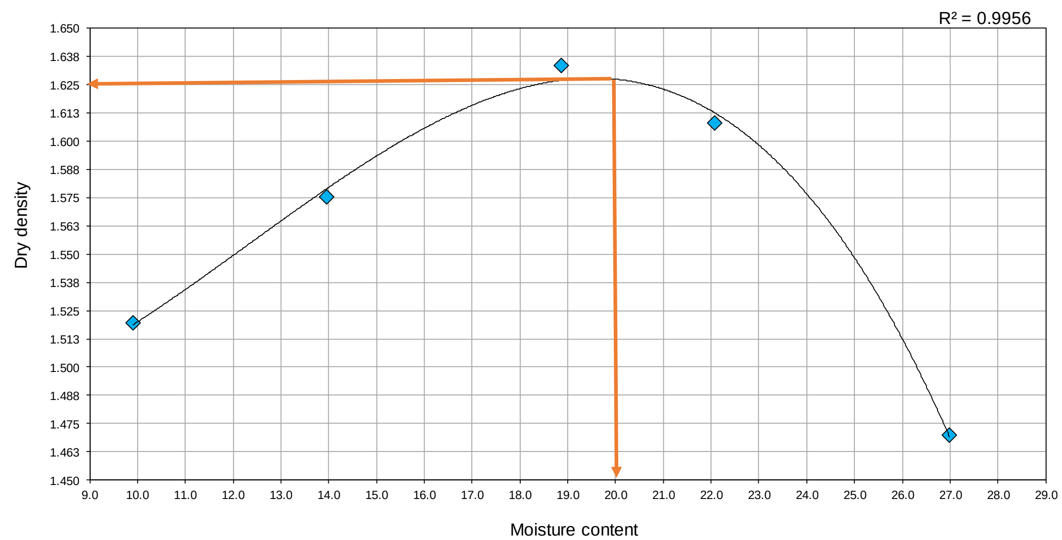


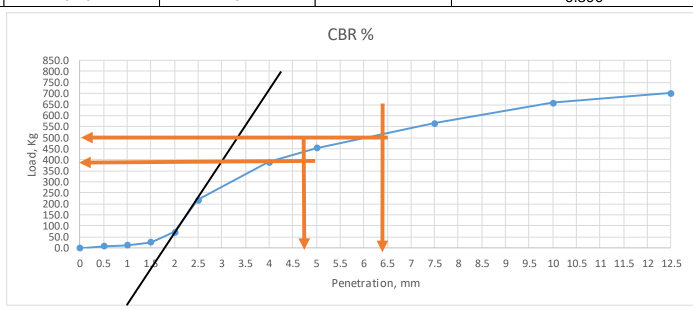
Figure 7.30. Proctor test of plain subgrade sample

California Bearing Ratio (CBR)							
Material Source	Subgrad soil			Sources		A-2-7	
Type of Material	**1% ground LDPE**			Testing date		15-Aug-2017	
Weight of compacted Sample + Mold		gm	**7870**	**Swelling Rate**			
Weight of CBR Mold only		gm	**4195**	Initial height of sample		mm	-
Weight of compacted Sample		gm	**3675**	Initial reading of the guage		mm	-
Height of Sample (**h**)		mm	**114.9**	Final reading of the guage		mm	-
Diameter of soil sample (**D**)		mm	**152.2**	Difference in guage readings		mm	-
Volume of soil sample (**V**)		cm³	**2089.4**	Swelling Ratio = (Difference in reading / Initial height of sample) * 100			
Wet density of Soil		gm/cm³	**1.759**	Swelling Ratio		%	-

Water Content Ratio, %				
				after soaking
Mass of container,	m1	gm		-
Wet soil + container,	m2	gm		-
Dry soil + container,	m3	gm		-
Mass of moisture,	m4 = m2 - m3	gm		-
Mass of dry soil,	m5 = m3 - m1	gm		-
Moisture Content ,	W = m4 / m5 x 100	%		-
Average Moisture Content ,	W	%		

Penetration Date

Penetration, mm		Load dial reading	Conversion Factor	Total load,	Corrected Load,	Corrected Load,
mm	inch	mm		KN	KN	Kg
0	0	0.000	0	0.000	0.000	0.0
0.5	0.02	0.002	2	0.089	0.089	9.1
1	0.04	0.004	4	0.134	0.134	13.6
1.5	0.06	0.010	10	0.260	0.260	26.5
2	0.08	0.038	38	0.711	0.711	72.5
2.5	0.1	0.124	124	2.134	2.134	217.5
4	0.15	0.226	226	3.823	3.823	389.7
5	0.2	0.260	260	4.445	4.445	453.1
7.5	0.3	0.324	324	5.556	5.556	566.4
10	0.4	0.378	378	6.445	6.445	657.0
12.5	0.5	0.407	407	6.890	6.890	702.3



CBR @ 2.5mm = $\frac{400}{1370}$ * 100 = 29.2

CBR @ 5.0mm = $\frac{525}{2055}$ * 101 = 25.5

% CBR = **29.2** %

Figure 7.31. Sample of CBR results of the subgrade with 1% of LDPE

Soil Classification Test		(AASHTO T-27, BS 1377 Part 2)	
Research			
Material Source	Lafarge	Sampling date	13/05/2017
Type of Material	Subgrade Soil	Testing date	15/05/2017

Sieve Analysis Test (SAMPLE 1)

Sieve Size		Weight Retained	%	
inch	(mm)	(gm)	Retained	Passing
No.4	4.750	9.30	1.1	98.9
No.10	2	589.0	72.8	26.0
No.40	0.425	172.0	21.3	4.8
No.200	0.075	25.3	3.1	1.6
Pan	-	13.3	1.6	0.0
Total		808.90	-	-
L.L		45.00	P.I	18.1
Group Index G		0.01(F200 - 15)(PI - 10)		0.0

General classification	Granular materials (35% or less of total sample passing No. 200)						
	A-1			*A-2*			
Group classification	*A-1-a*	*A-1-b*	*A-3*	*A-2-4*	*A-2-5*	*A-2-6*	*A-2-7*
Sieve analysis (percent passing)							
No. 10	50 max.						
No. 40	30 max.	50 max.	51 min.				
No. 200	15 max.	25 max.	10 max.	35 max.	35 max.	35 max.	35 max.
Characteristics of fraction passing No. 40							
Liquid limit				40 max.	41 min.	40 max.	41 min.
Plasticity index	6 max.		NP	10 max.	10 max.	11 min.	11 min.
Usual types of significant constituent materials	Stone fragments, gravel, and sand		Fine sand	Silty or clayey gravel and sand			
General subgrade rating	Excellent to good						

Figure 7.32. Sample of sieve analysis results of a subgrade sample

Atterberg Limits	(ASTM D4318, BS 1377, AASHTO T89 & 90)			
Research	0			
Material Source	Lafarge		Sampling date	13/05/2017
Type of Material	Subgrade Soil		Testig date	15/05/2017

PLASTIC LIMIT

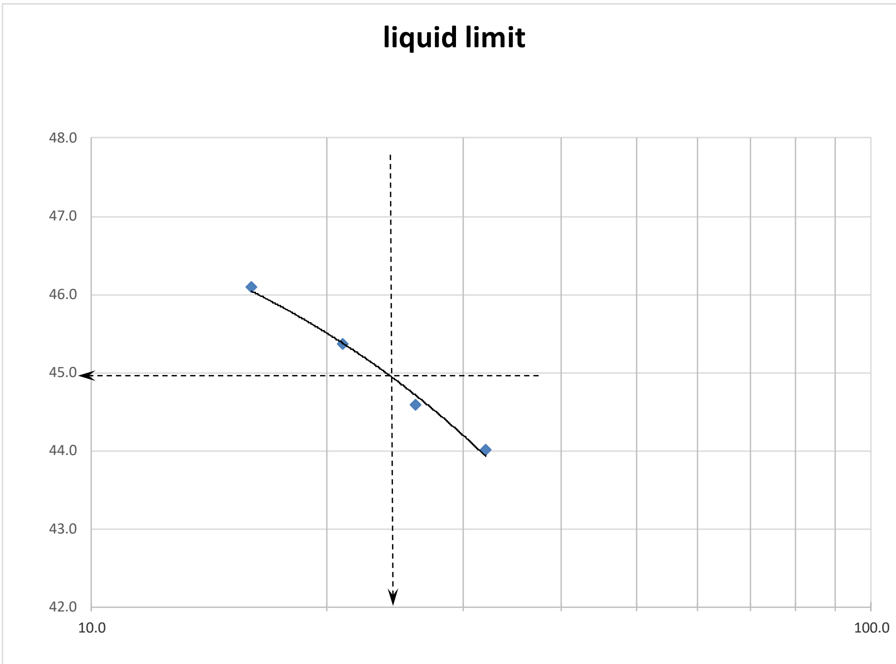
Container No		1	2	3	4	5
Mass of container, M1	g	2.13	2.12	2.10	-	-
Mass of wet soil + container, M2	g	5.00	5.50	5.40	-	-
Mass of dry soil + container, M3	g	4.36	4.78	4.74	-	-
Mass of moisture, M4 = M2 - M3	g	0.6	0.7	0.7	-	-
Mass of dry soil, M5 = M3 - M1	g	2.2	2.7	2.6	-	-
Moisture Content, W = M4 / M5 x 100	%	28.7	27.1	25.0	-	-
Average Moisture Content, W	%	**26.9**				

LIQUID LIMIT

Container No		1	2	3	4	5
Mass of container, M1	g	2.19	2.13	2.12	2.14	-
Mass of wet soil + container, M2	g	13.60	10.30	11.20	13.20	-
Mass of dry soil + container, M3	g	10.00	7.75	8.40	9.82	-
Mass of moisture, M4 = M2 - M3	g	3.6	2.6	2.8	3.4	-
Mass of dry soil, M5 = M3 - M1	g	7.8	5.6	6.3	7.7	-
Moisture Content, W = M4 / M5 x 100	%	**46.1**	**45.4**	**44.6**	**44.0**	**-**
Number of Blows		**16.0**	**21.0**	**26.0**	**32.0**	**-**

Figure 7.33. Sample of Atterberg limit results of a sample of subgrade soil

Atterberg Limits	(ASTM D4318, BS 1377, AASHTO T89 & 90)		
Research	0		
Material Source	Lafarge	Sampling date	13/05/2017
Type of Material	Subgrade Soil	Testig date	15/05/2017



From the Graph :

Liquid Limit	=	45
Plastic Limit	=	26.9
Plasticity Index	=	18.1

Remarks

* Sample 1 is classified as A-2-7 Silty or Clayey gravel and Sand

* Sample is rated as ecellent to good as subgrade soil

Figure 7.34. Sample of determining subgrade soil type

Hydraulic Conductivity Test				**ASTM D-5084**	
Material Source	Subgrad soil			Sources	A-2-7

Type of Material	LDPE ground		**1.00%**	Testing date	9-Jul-2017
Inner dia. of Permeamter (**D**)	cm	**7.52**			
Area of Permeamter (**A**)	cm^2	**44.39**			
Length of soil permeameter (**L**)	cm	**15.10**			
Diameter of Standpipe (**D1**)	cm	**0.50**			
Area of Standpipe (**a**)	cm	**0.20**			
Weight of Dry soil (**m**)	gm	**616.37**			
Temparature	°C	**24.0**			
Volume of soil sample (**V**)	cm^3	**670.32**			
Dry density of soil	gm/cm^3	**0.920**			

Trial No.	**Total initial Head h0 (cm)**	**Total Final Head H1 (cm)**	**Time Interval for head to drop t, (sec.)**	$K_T = \frac{a*L}{A*t} * \ln\frac{h0}{h1}$ **(cm/sec.)**		$K_{20} = K_T * \frac{\eta_T}{\eta_{20}}$ **(cm/sec.)**
1	80	79	81.00	0.0000104		0.0000094
2	79	78	86.00	0.0000099		0.0000090
3	78	77	100.00	0.0000086		0.0000078
				Average		**0.0000088**

Type of Material	LDPE ground		**2.00%**	Testing date	9-Jul-2017
Inner dia. of Permeamter (**D**)	cm	**7.52**			
Area of Permeamter (**A**)	cm^2	**44.39**			
Length of soil permeameter (**L**)	cm	**15.10**			
Diameter of Standpipe (**D1**)	cm	**0.50**			
Area of Standpipe (**a**)	cm	**0.20**			
Weight of Dry soil (**m**)	gm	**613.75**			
Temparature	°C	**24.0**			
Volume of soil sample (**V**)	cm^3	**670.32**			
Dry density of soil	gm/cm^3	**0.916**			

Trial No.	**Total initial Head h0 (cm)**	**Total Final Head H1 (cm)**	**Time Interval for head to drop t, (sec.)**	$K_T = \frac{a*L}{A*t} * \ln\frac{h0}{h1}$ **(cm/sec.)**		$K_{20} = K_T * \frac{\eta_T}{\eta_{20}}$ **(cm/sec.)**
1	81.5	75.5	110.00	0.0000464		0.0000422
2	75.5	74.5	26.00	0.0000342		0.0000312
3	74.5	73.5	27.00	0.0000334		0.0000304
				Average		**0.0000346**

Figure 7.35. Sample of permeability test results

Unconfined Compressive Strength				ASTM D-2850			
Material Source		1% LDPE		Sampling date		8-Jul-2017	
Source		SG soil		Testing date		7-Sep-2017	
Sample Dia, mm		50.00		Initial area, mm²		1963.5	
Sample Height (**h**), mm		100.00					
Initial dial reading (**S0**)		NA					
Strain %	**Compressive Stress, (q) Kpa**	**Strain %**	**Compressive Stress, (q) Kpa**	**Strain %**	**Compressive Stress, (q) Kpa**	**Strain %**	**Compressive Stress, (q) Kpa**
0.17	47.3	0.75	156.8	1.22	296.1	1.81	452.7
0.19	49.6	0.76	160.0	1.24	299.6	1.83	455.8
0.21	51.8	0.78	163.4	1.25	303.8	1.84	459.4
0.22	54.3	0.79	166.5	1.26	307.3	1.86	463.8
0.24	56.8	0.80	169.3	1.27	311.3	1.87	466.3
0.25	59.4	0.81	172.9	1.29	314.5	1.88	468.9
0.27	61.9	0.83	176.4	1.30	319.0	1.89	472.7
0.28	64.3	0.84	179.1	1.32	322.5	1.91	475.9
0.30	66.6	0.85	182.8	1.33	326.5	1.92	478.6
0.31	69.4	0.87	186.3	1.35	330.5	1.93	481.1
0.33	71.7	0.88	189.3	1.36	334.5	1.94	484.7
0.34	73.9	0.89	192.8	1.38	338.3	1.96	487.7
0.36	76.6	0.90	196.3	1.39	341.7	1.97	489.5
0.37	79.1	0.92	199.8	1.41	346.0	1.98	492.7
0.39	82.0	0.93	202.5	1.42	349.4	1.99	495.2
0.40	84.1	0.94	206.3	1.44	353.3	2.00	497.3
0.42	87.2	0.95	209.5	1.46	357.3	2.02	499.3
0.43	89.6	0.97	214.1	1.47	361.2	2.03	501.5
0.45	92.1	0.98	216.7	1.49	364.9	2.04	503.6
0.46	94.5	0.99	220.3	1.50	369.0	2.05	505.1
0.48	97.4	1.00	223.8	1.52	372.4	2.06	506.7
0.49	99.7	1.02	227.5	1.53	377.1	2.07	508.3
0.50	102.5	1.03	230.5	1.55	381.0	2.08	509.4
0.52	105.1	1.04	234.1	1.56	385.1	2.09	510.5
0.53	108.1	1.05	238.0	1.58	390.6	2.11	510.6
0.55	111.2	1.06	241.5	1.59	395.5	2.12	512.1
0.56	113.8	1.07	244.9	1.61	399.0	2.13	512.2
0.57	117.1	1.09	248.5	1.62	403.5	2.15	512.3
0.59	119.8	1.10	252.1	1.64	407.8	**2.16**	**512.6**
0.60	122.7	1.11	255.4	1.65	411.6	2.17	512.4
0.61	125.3	1.12	258.7	1.67	414.7	2.19	511.7
0.63	128.4	1.13	262.4	1.68	417.9	2.20	512.0
0.64	131.6	1.14	266.1	1.70	422.5	2.21	511.9
0.65	134.5	1.15	269.5	1.71	426.4	2.23	511.0
0.67	137.7	1.16	273.2	1.73	430.0	2.24	510.4
0.68	140.6	1.17	277.6	1.74	434.1	2.25	509.4
0.69	143.5	1.18	281.1	1.76	437.6	2.27	509.6
0.71	146.6	1.19	284.5	1.77	441.7	2.28	508.3
0.72	150.0	1.20	288.4	1.79	445.1	2.30	506.8
0.74	153.3	1.21	292.1	1.80	449.0	2.31	505.4

Figure 7.36. Sample of UCS testing reading

7.4 Subgrade compaction properties

Soil samples were obtained from three different resources in the Ottawa-Gatineau region. Each of the received samples was tested for compliance with AASHTO classification system, prior to proceeding further with the investigation. According to the AASHTO system, and commonly practiced approaches in pavement design, soil samples have to possess certain requirements and adhere to metrics and fall within the prescribed categories in the AASHTO classification prior to be deemed suitable for any road works. Once the three tested soils in the research exhibited similar values, they were considered similar. The compaction curves (with all data points in the graphs) and characteristics of subgrade soils without plastic are shown in Figures 7.37, while those of the soils with plastic are shown in Figures 3.38 to 3.41.

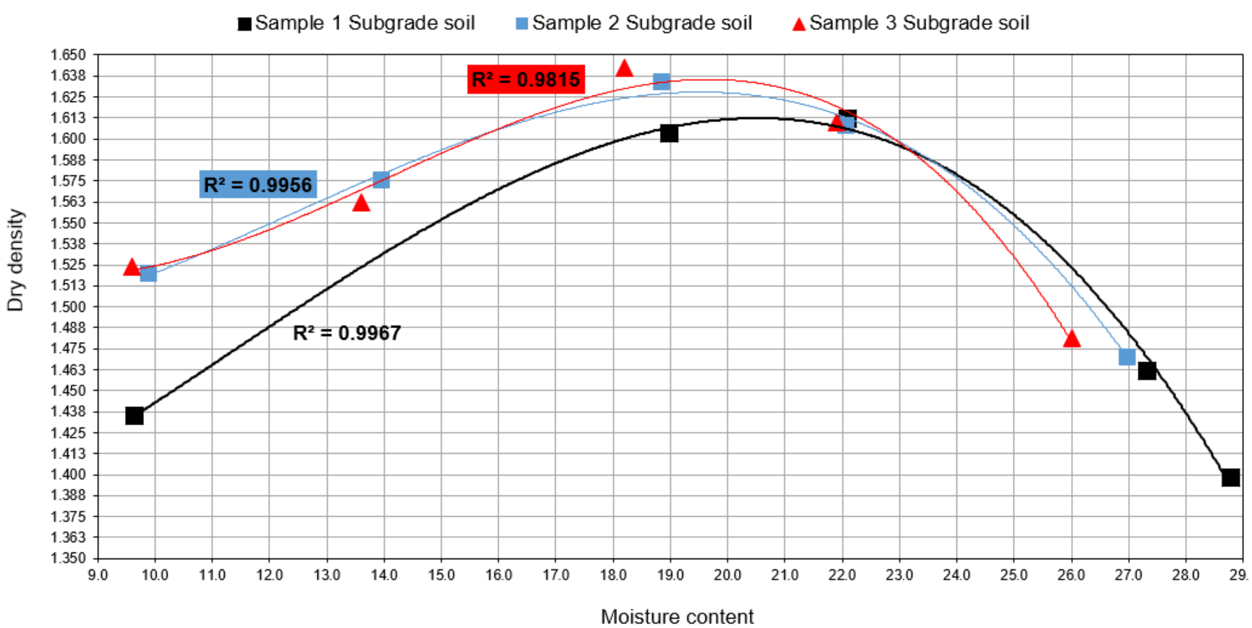


Figure 7.37. Compaction curves of the three soil samples without plastic waste

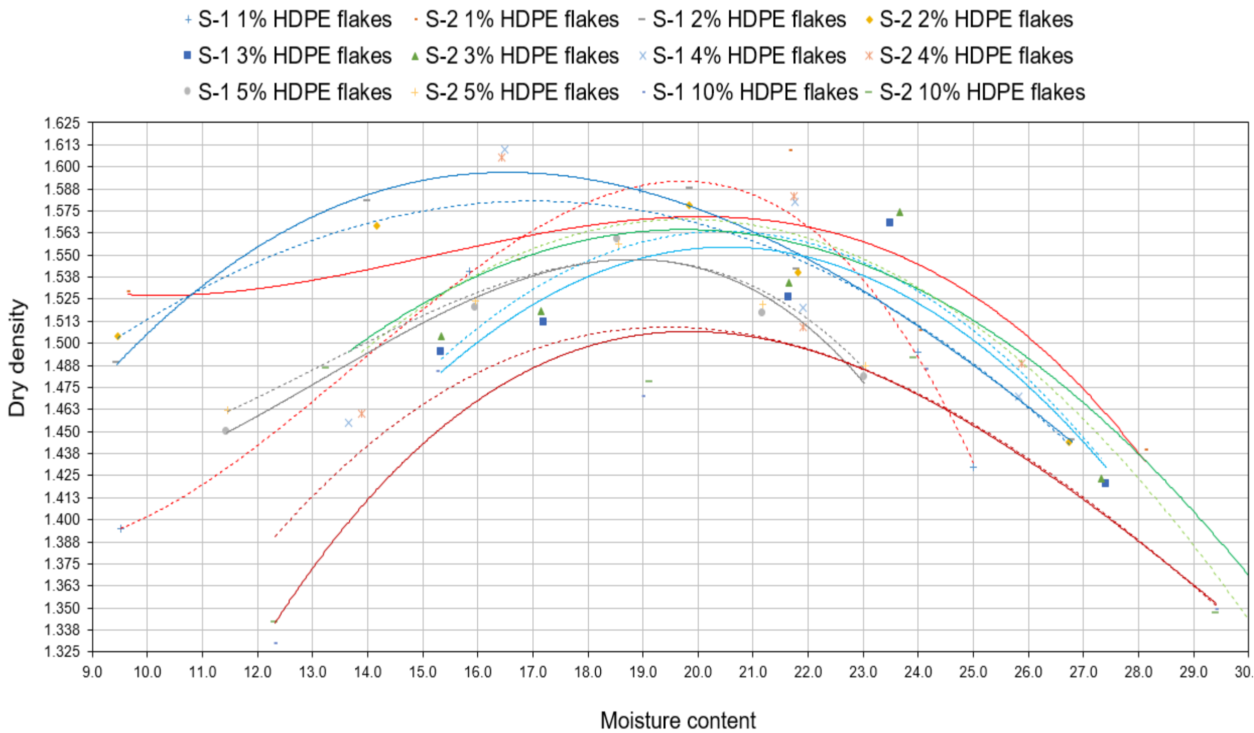


Figure 7.38 Compaction curves of the natural and flake-HDPE modified soil

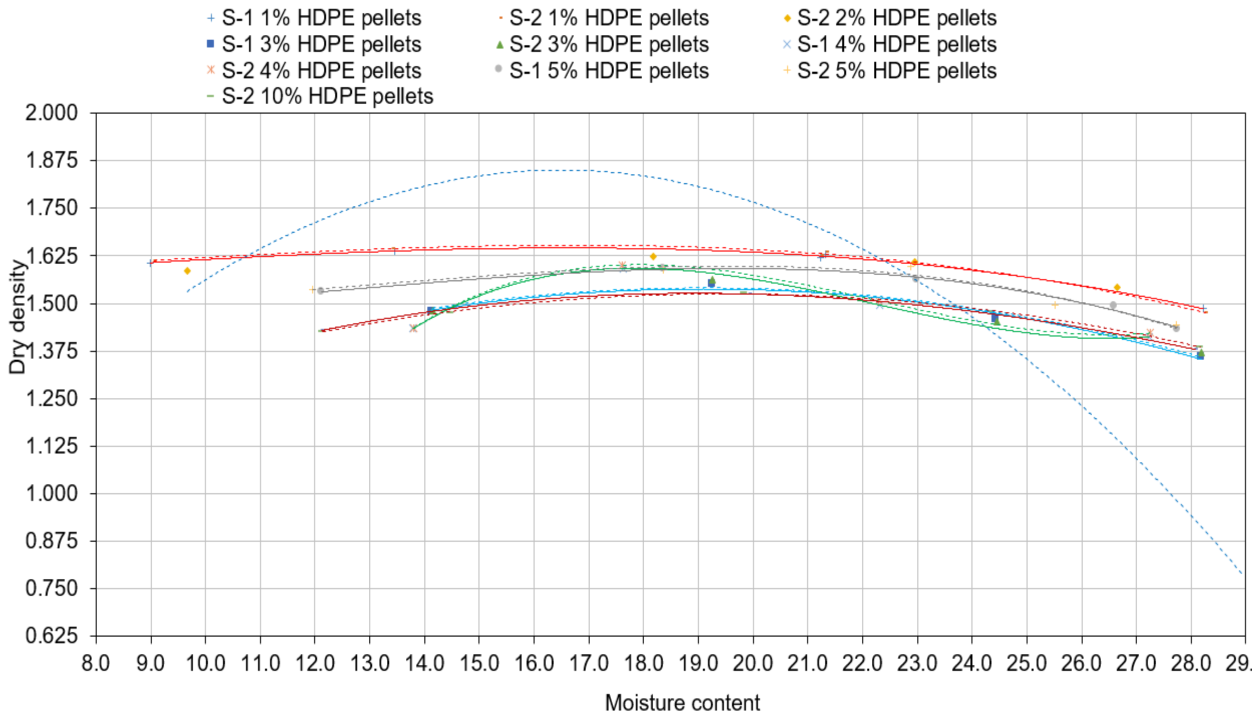


Figure 7.379 Compaction curves of the natural and pellet-HDPE modified soil

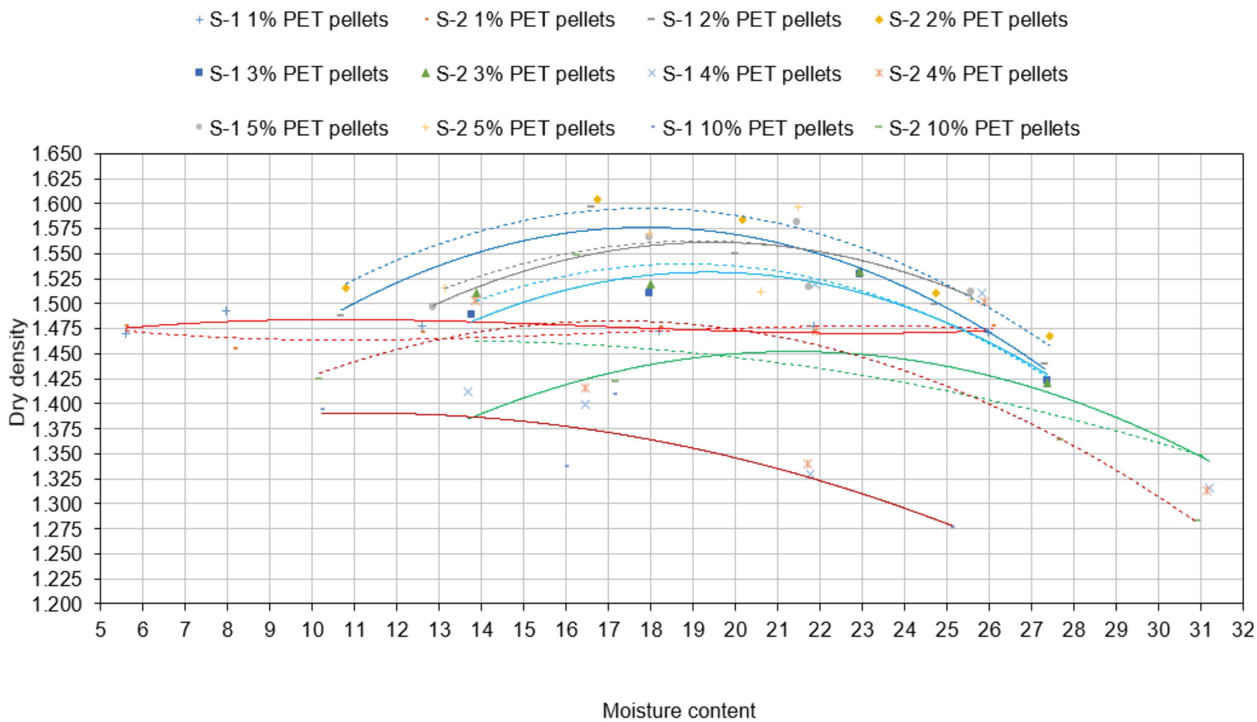


Figure 7.380 Compaction curves of the natural and pellet-PP soil

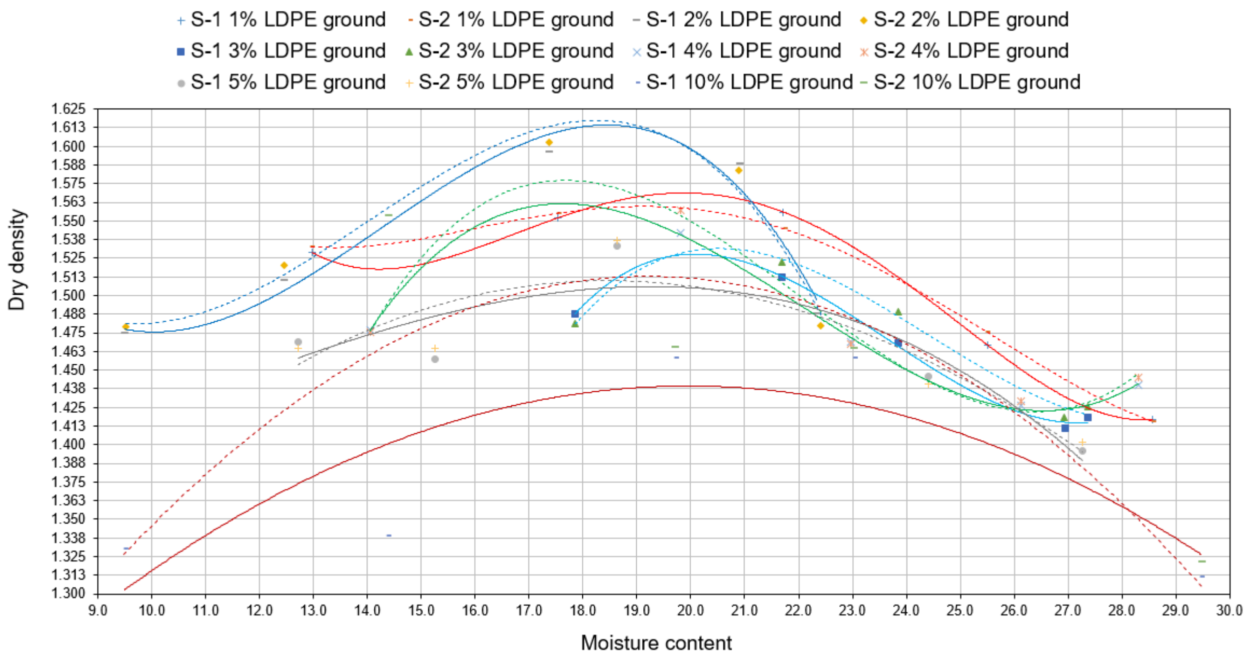


Figure 7.41 Compaction curves of the natural and ground-LDPE modified soil

7.5 Compaction characteristics and curves of the granular B materials

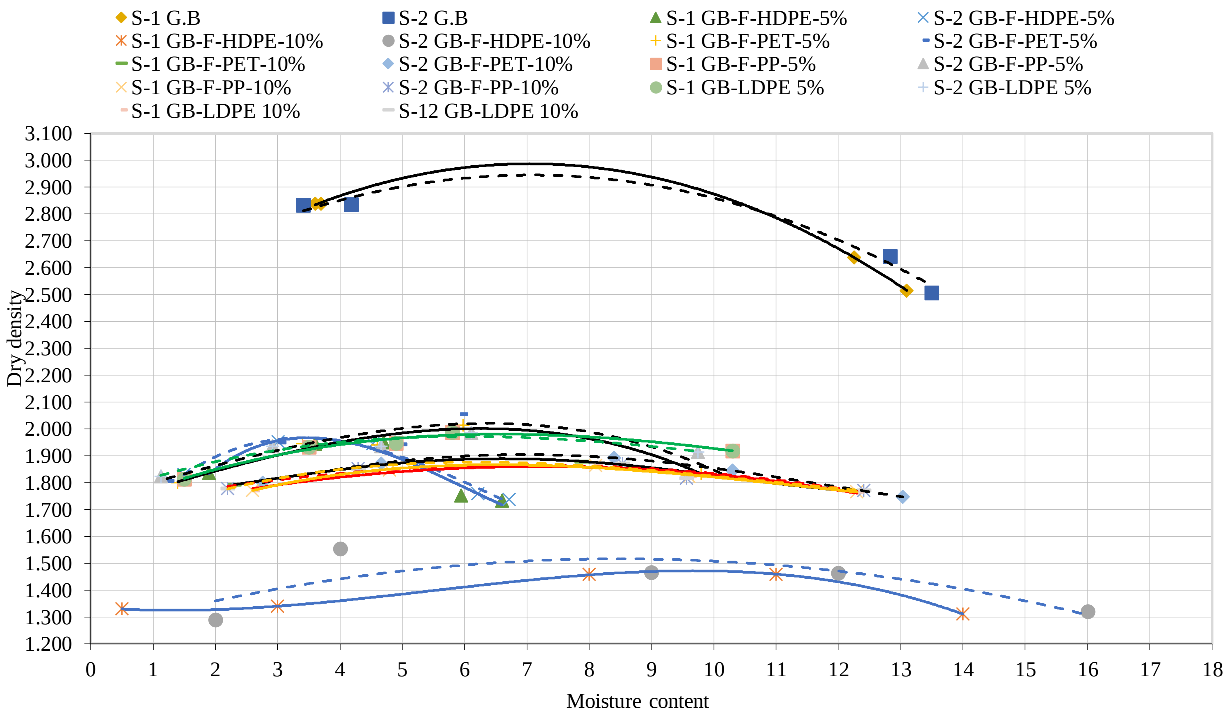


Figure 7.42. Changes in compaction properties for flaky types with granular B material

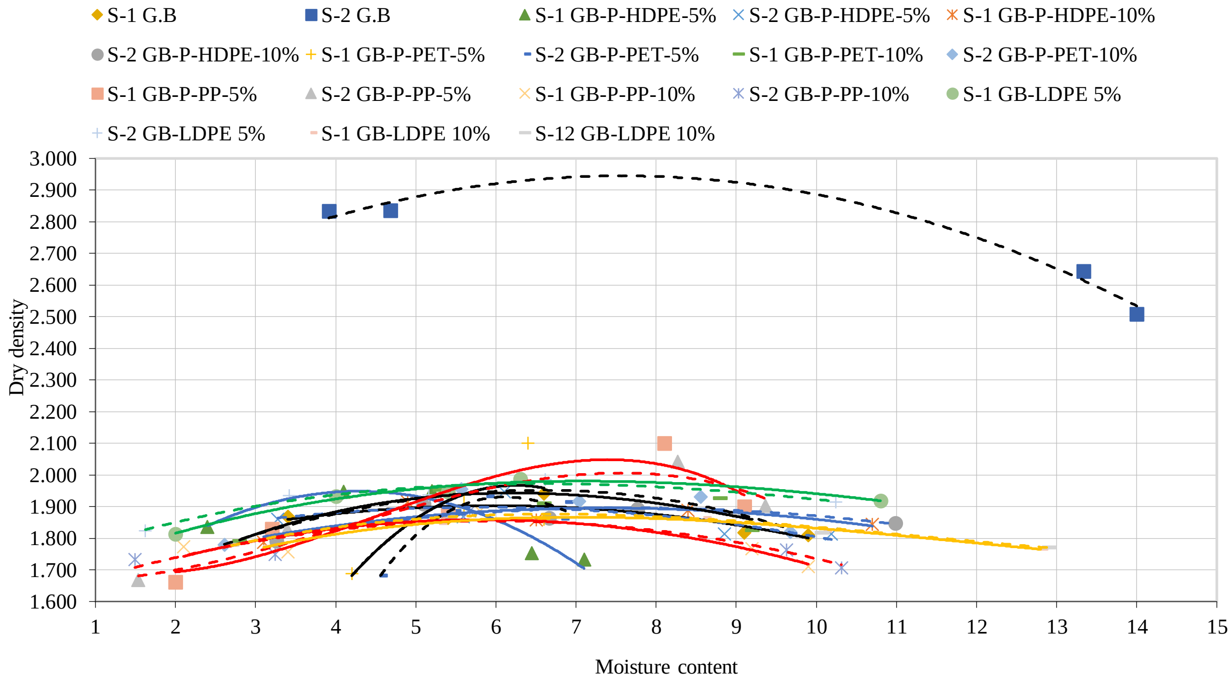


Figure 7.43. Changes in compaction properties for pellet types with granular B material