USE OF RECYCLED AGGREGATE MATERIALS CHARACTERISTICS IN PAVEMENT DESIGN ANALYSES

By

Ida Gheibi

A THESIS

Submitted to
Michigan State University
in partial fulfillment of the requirements
for the degree of

Civil Engineering—Master of Science

2020

ABSTRACT

USE OF RECYCLED AGGREGATE MATERIALS CHARACTERISTICS IN PAVEMENT DESIGN ANALYSES

By

Ida Gheibi

Use of recycled materials promotes sustainability in roadway construction by reducing consumption of energy and emission of greenhouse gases associated with mining and the production of natural aggregates. Recycled asphalt pavements (RAP) and recycled concrete aggregates (RCA) have comparable characteristics to natural aggregates that are currently used in roadway base course applications. This study has developed a database for RAP and RCA materials' characteristics including resilient modulus (M_r), California bearing ratio (CBR), gradations along with construction specifications. RAP and RCA relationships with different engineering and index properties were investigated and some trends were proposed such as higher RAP content reveals higher summary of resilient modulus (SM_r), higher RCA content causes higher optimum moisture content (OMC) and lower maximum dry unit (MDU).

In addition, pavement mechanistic-empirical (ME) analyses have been conducted with the material inputs collected for the database to determine whether different values of different characteristics of RCA and RAP can be used in flexible or rigid pavement designs. Results showed that M_r parameter had the highest impact on pavement distress predictions among gradations and hydraulic conductivity.

I dedicate this thesis to my parents who have always supported me to pursue my dreams. It is their unconditional love that motivates me to set higher goals.

ACKNOWLEDGEMENTS

I would like to first gratefully acknowledge Dr. Bora Cetin, my advisor, for his guidance, support, advice and extensive patience during my graduate studies at Michigan State University. I would like to thank Dr. Mustafa Hatipoglu as he helped me through early stages of my project. Thanks also to Dr. Timothy Gates and Dr. Syed Waqar Haider for being on my committee.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

INTRODUCTION

4.3 million km (2.6 million miles) out of 6.6 million km (4.1 million miles) of total public roads are paved roads in the United States of America (Bureau of Transportation Statistics (BTS 2017) and more than 90% of the paved roads are flexible pavements (Copeland 2011). The main function of flexible pavements is distributing the vehicle loads to the layers beneath the asphalt surface layer (e.g. base, subbase, and subgrade layers). Thus, the characteristics and properties of these layers are very important for the long-term pavement performance (Little and Nair 2009, Tutumluer et al. 2015). Aggregate base layer is the first layer beneath the asphalt surface (Cosentino and Kalajian 2001; Yohannes et al. 2009). It is made of coarse-grained aggregates to provide a stiff and highly permeable layer (Schuettpelz et al. 2010, Haider et al. 2014, Cetin et al. 2014, Edil and Cetin 2015). The high stiffness of aggregate base layer improves the stability of the sublayers by improving the vertical load distribution (Zornberg 2017). Therefore, stiffer base course has less permanent deformation and increases the lifespan of the pavement (Edil et al. 2012a). Highway base layer is a very critical component of pavement structures. There are two primary functions of highway base layers: (1) acting as a foundation to provide adequate mechanical support to the asphalt or concrete layer to prevent fatigue and occurrence of rutting failures, and (2) providing adequate drainage to move the excessive infiltrated water out and away from the pavement structure. Materials used in highway base layer construction are responsible for distributing the wheel loads uniformly to the subgrade layer so they can protect the subgrade layer from excessive loading at a single location and ultimately increase the service life of the pavements (Yoder and Witzack 1975, Xiao et al. 2011). It is very well known that majority of the rutting failures occur due to lack of required mechanical properties of the materials used in the highway base layer construction (Tutumluer and Pan 2008, Xiao et al. 2011).

While large amounts of virgin unbound granular materials are used for the aggregate base layers in pavement constructions (Perkins et al. 2005, Haider et al. 2014, Hatipoglu et al. 2020), the lack of good quality material availability and high cost of virgin aggregates (VA) have made engineers to look for alternative sources such as recycled concrete aggregates (RCAs) and recycled asphalt pavements (RAPs). These materials additionally provide environmental benefits for being recycled materials. As a result, in this project alternative materials are being investigated to see if they meet the essential pavement performance criteria. The main focus of this project is to collect index and performance data of these two types of recycled base materials. Therefore, this thesis reports the properties of these materials used in such applications.

Department of transportations (DOTs) nationwide have been trying to implement the mechanistic pavement design approach (Rahn and Biehler 2008) for pavement design and analyses. The Mechanistic-Empirical Pavement Design Guide (MEPDG) represents a major improvement over its predecessors. However, accurate and reliable data first must be collected to take the advantage of such improvements in the pavement design. In this pavement design approach, pavement performance is evaluated based on mechanistically determined critical stresses, strains, temperatures, and moisture levels that are in turn the inputs to empirical prediction models for specific pavement distresses such as rutting, fatigue cracking, thermal cracking, and roughness for flexible pavements and cracking, faulting, and roughness for rigid pavements. Accurate characterization of the traffic, climate, and material input parameters is therefore important to ensure that the theoretical computation of pavement stresses, strains, temperatures, and moisture levels are accurate at the critical locations within the system. Depending on the desired level of accuracy of input parameter, three levels of input are provided from Level 1 (highest level of accuracy) to level 3 (lowest level of accuracy). Depending on the criticality of the project and the available resources, the designer has the flexibility to choose any one of the input levels for the design as well as use a mix of levels.

The material parameters required for pavement foundation materials including unbound granular materials, subgrade, and bedrock may be classified in one of three major groups: (1) pavement response model material inputs, (2) Enhanced Integrated Climatic Model (EICM) material inputs, and (3) other material inputs. Pavement response model materials input required are resilient modulus (Mr) used for quantifying the stress dependent stiffness of unbound materials under moving wheel loads. Material parameters associated with EICM are those parameters that are required and used by the EICM models to predict the temperature and moisture conditions within a pavement system. These inputs include Atterberg limits, gradation, and saturated hydraulic conductivity.

Proper implementation of the PMED requires realistic values for the input parameters. The main inputs include general site and project information, allowable distress limits and associated reliability levels, traffic volumes and axle load distributions, pavement structure, material properties, groundwater depth, and climate. Pavement structures generally contain 3 layers: asphalt/Portland cement concrete (PCC) (often consisting of several sublayers or lifts), base/subbase, and subgrade. The layers beneath the asphalt/PCC usually consist of unbound materials, and their physical and engineering properties are very crucial for overall pavement performances and service life (Haider et al. 2014, Gopisetti et al. 2019, Hatipoglu et al. 2020, Gopisetti et al. 2020). This past literature clearly concluded that material properties are crucial parameters that must be taken into account during the design of pavements. Therefore, the properties of recycled asphalt pavement (RAP) and recycled concrete aggregate (RCA) materials should be well understood as they play an important role in pavement design as a base layer. To address this need, the research team created a large database summarizing the characteristics of RCA and RAP that had been used for such applications in pavements. At the end of this thesis, the impact of properties of these materials were evaluated on pavement distress predictions via use of the AASHTOware Pavement ME software version 2.6.0. These analyses were conducted using, the lowest, the highest and the median value of different properties of RAP and RCA. The "Inputs" chapter and "Distresses" chapter provide detailed information about input parameters and performance distress evaluations.

This thesis reports the collected data for RCA and RAPs used in highway base layers and provide recommendations about how to use this data for pavement design and analysis.

BACKGROUND

Use of recycled materials promotes sustainability in roadway construction by reducing consumption of energy and emission of greenhouse gases associated with mining and production of natural aggregates (Lee et al. 2010 and Lee et al. 2011). Recycled materials often manifest mechanical behavior that is distinct from that of natural aggregate due to the composition and the nature of particulate characteristics. The most widely used recycled aggregates in roadway construction are recycled asphalt pavement (RAP) and recycled concrete aggregate (RCA) (Edil et al. 2012a). The performance of a pavement system mostly depends on stiffness of the pavement structure under specified traffic loads and environmental conditions. RAP and RCA have comparable stiffness to natural aggregates that are currently used in roadway base course applications (FHWA 2008, Guthrie et al., 2007, Edil et al. 2012a). Hence, their performance should be evaluated based upon their relative engineering or index properties.

In order to obtain recycled asphalt pavement (RAP), old asphalt pavement surfaces are milled to a specific depth (depending on the asphalt course thickness) and then processed (Edil 2011). In simple terms, RAP is a mixture of aged bitumen and aggregate, which is obtained as a by-product of pavement milling (Taha et al. 1999). On the other hand, RCA was obtained via crushing of the existing hardened concrete of old pavement surfaces or other structures (e.g. buildings and bridges) (Edil et al. 2012a).

RAP and RCA can be either used at the same construction site or stockpiled for future applications. Producing and using them at the same construction site can help to reduce the cost and the duration of the construction. In fact, up to 30% of cost savings could be achieved by in-place recycling for a recycled aggregate generation (Edil 2011).

The properties of RAP or RCA as an unbound aggregate are certainly depending on several factors that relate back to the production of asphalt or concrete as well as the processes followed during the production of RAP or RCA. Some of these factors are listed below:

- The type of the road (interstate highway, arterial highway, or parking lot) that is milled may affect the binder content of the produced RAP since different bituminous content are used in different asphalt mixtures.
- The regional differences in location of the milled road may result in different RAPs and RCAs due to different geological composition and formation of aggregates.
- Processing operation that is used to create RAP and RCA may affect the grain size distribution of these materials due to the different opening sizes of the screens used by the different milling operation stations.
- The time of exposure of RAP and RCA to atmospheric conditions during stockpiling may affect the stiffness of the binder content of RAP as asphalt changes its properties when

exposed to drastic temperatures (cold or hot) for a long period of time (Ullah and Tanyu 2019) and carbonation of remaining cement content in RCAs (Bestgen et al. 2016).

- The type of concrete, quality of raw materials, water/cement ratio, coarse/fine aggregate ratio, age of concrete, compaction of concrete, temperature, relative humidity and curing of concrete can affect the strength of the recycled concrete all of which come from the origin of RCA.

DATA ON RCA AND RAP PROPERTIES

Material characteristics such as mineralogy, gradation, angularity, texture, and durability are different for each RCA and RAP materials, and these differences affect the engineering properties of them (Tutumluer 2013, Tan et al. 2014). The index properties of RAP and RCA are also highly affected by several factors such as the aggregate source, the aggregate type, and the type of crushing operations. While determining the properties of these recycled materials before the pavement design and construction is preferred, it may be costly and take a long time to be completed for DOTS. Therefore, it is important to establish a database with the information collected from previous studies which would provide some insight information about the boundaries and average properties of these materials and can be used by DOTs during pavement analyses and designs. Table 1 summarizes the list of the RCA/RAP data collected from the literature. It also shows the number of available data for each characteristic along with the corresponding data source. Approximately 50 different studies were examined to create Table 1. The RCA and RAP materials for the available data was captured for the states of Minnesota, Colorado, Michigan, California, Texas, Ohio, New Jersey, Wisconsin, Illinois, Montana, Virginia, Florida, Tennessee, Maryland, New Mexico, Washington, Utah and Rhode Island. The laboratory data of more than 40 different recycled samples were collected in terms of geomechanical properties. Most of the samples used in the studies were 100% recycled materials, while there were also some blended RCA-RAP materials with natural aggregates at different mixture ratios.

Table 1. LIST OF THE COLLECTED DATA AND CORRESPONDING RESOURCES

References	Loc	Type of Material	Grain Size Distribution	Atterberg Limits	Compaction	Hydraulic Conductivity	Shear Strength	CBR	Resilient Modulus	R Value
Edil et al. (2012a)	MN, MI, CO, CA, TX, OH, NJ, WI	Aggregate Class 5 (MN)	1		1	1			26	
		Blend (50%RCA 50% Class5)	1		1	1			2	
		RAP	7		7	7			96	
		RCA	7		7	7			96	
		RPM	2		2	2			4	
Edil et al. (2012b)	WI	RPM	1	1	1				1	
Edil et al. (2012c)	MN	RPM	1		1			1	1	
Tutumluer et al. (2015)	IL	60%RCA+40% RAP	1				6		6	
		100% RAP	1		1			1	6	
Locander (2009)	CO	RAP	11	11	11	11			45	11
Mokwa and Peebles (2005)	MT	RAP CBC#1	3		3	3				
		RAP CBC#2	3		3	3				
		RAP CBC#3	3		3	3	24			48
		RAP pitrun	3		3	3	24			48
Ullah and Tanyu (2019)	VA	RAP	4	5				16	21	
Saeed (2008)	FL	RAP	3		3		3			
Bennert et al. (2000)	NJ	DGABC	1		1				1	
		RAP	1		4		3		4	
		RCA	1		4				4	
Kim et al. (2005)	MN	RAP	4		4				16	

Table 1 (cont’d)

Huang and Dong (2014)	TN	RAP	1		3				9	
Mijic et al. (2019)	MD	RAP	7		7	7				
Ullah et al. (2018)	VA	RAP	4	4	4				PD=11	
Edil et al. (2017)	MN	RAP	1		1				2	
		RCA	2		2				4	
Hasan et al. (2018)	NM	RAP	3		1				16	
Abdelrahman and Noureldin (2014)	MN	RAP			3				9	
Cosentino and Bleakley (2013)	FL	RAP						3	PD=3	
Cosentino et al. (2013)	FL	RAP	1		8			8		
Wu et al. (2012)	WA	RAP	1		5	5			20	
Puppala et al. (2012)	TX	RAP	1		1				5	
Attia et al. (2013)	MN	RAP							PD= 6	
Soleimanbeigi and Edil (2015a)	WI	RAP	1		2				7	
Soleimanbeigi and Edil (2015b)	WI	RAP	1		1	1				
		RCA	1		1	1				

Table 1 (cont'd)

Soleimanbeigi et al. (2015)	CA, TX, NJ, MI, CO, MN	RAP	4		4					
		RCA	4		4					
Kang et al. (2011)	MN	RAP				4	4		4	
		RCM	4			4	4		4	
Camargo et al. (2013)	WI	RPM	1		1			1	1	
Attia and Abdelrahman (2010a)	MN	RAP			11				12	
Attia and Abdelrahman (2010b)	MN	RAP	6	6	12				11	
Guthrie et al. (2007)	UT	RAP	4	4	4					
Bradshaw et al. (2016)	RI	RAP	7		7				7	
Alam et al. (2010)	MN	RAP	5						5	
Attia and Abdelrahman (2011)	MN	RAP			7				4	

Table 1 (cont'd)

Bennert and Maher (2005)	NJ	RAP				8	1	8	4	
		RCA	1			8	1	8	4	
Bestgen et al. (2016)	Eastern USA	RCA	2		2			13	24	
Tutumluer et al. (2012)	IL	RCA	3		3			3	3	
Natarajan et al. (2019)	MN	RCA	4		4					
Mahedi and Cetin (2020)	TX, IA, MN	RCA	5		5					
Chen et al. (2013)	CA, CO, MI, MN, WI, TX	RCA	7		7	7				
Diagne et al. (2015)	WI	RCA	1		1	1			3	
Cetin et al. (2020)	MN	RCA	3	3	6	3			3	
Total	United States of America	RCA	47	3	47	32	5	24	153	0
		RAP	92	31	126	57	66	38	316*	107

Notes: *: It only represents resilient modulus of RAPs or RAP blends. Strains were not counted for this number. PM= Recycled pavement material; CBC= Crushed base course; RCM= Recycled concrete material; DGABC= Dense graded aggregate base course; PD= Permanent Deformation; CBR= California Bearing Ratio; R-Value= Measures the response of compacted aggregates to a vertically applied pressure under specific conditions.

CHAPTER 1. PHYSICAL PROPERTIES OF RAP AND RCA

1.1. GRADATION CHARACTERISTICS

Gradation of the aggregates affects the engineering properties of granular materials such as hydraulic conductivity, shear strength, stiffness, and frost-susceptibility (Saeed 2008). Original aggregate type, milling operations, and the crushing methods affect the gradation of RAP and RCA (Cosentino and Kalajian 2001).

The first material characteristics for the database was selected as the index properties, which mainly consists of the gradation of aggregates. Gradation characteristics database include gravel, sand, silt and clay contents, effective diameter sizes (D_{60}, D_{30} and D_{10}), and coefficient of uniformity (C_u) and coefficient of curvature (C_c). Approximately 190 different aggregate materials including blends with natural aggregate were included in the gradation database.

It was observed that the gradations of RAPs were generally similar to natural aggregates; however, depending on the method used, RAPs were likely to contain lower fines content (Chesner et al. 1998). Per MnDOT guideline and applications, RAPs can be considered as a Class 7 aggregate based on their comparable gradation curves whereas RCA can be considered as Class 5 aggregate based on their comparable gradation curves (LRRB 2016).

Asphalt content (~ 4.5% - 6%) and trapped air between asphalt coating and aggregate particles cause lower specific gravity values for RAP than that of natural aggregates (Cosentino et al. 2003). RCA also has a relatively lower specific gravity than natural aggregates due to the presence of mortar in RCA matrix (Snyder et al. 1994). This is well shown in our database when comparing the specific gravities of the recycled materials with natural aggregates (Appendix A and Table 2).

The G_s of RAP ranges from 2.19 to 2.87 with the median value of 2.4 while G_s of RCA is between 2.12 and 2.7 with the median value of 2.39.

94.1% is the highest gravel percent reported for RCA in Mahedi and Cetin (2020) while Edil et al. (2017) reported the lowest gravel content (31.8%) for RCA. On the other hand, Alam et al. (2010) showed 4% gravel content for a RAP material which was the lowest gravel percent in the database. Locander (2009) reported the highest gravel percent for a RAP material which contained 75% gravel. Finally, the median gravel percent of RAP and RCA is reported to be 45% (Guthrie et al. 2007) and 51% (Diagne et al. 2015), respectively.

The highest sand content in RCA was reported to be 64.9% (Edil et al. 2012a) while Mahedi and Cetin (2020) used a RCA with 4.9% sand which was the lowest sand percent in the database. The highest and lowest sand contents for RAP materials were 97% and 28.1%, respectively. At last, the median value of sand content is 54% and 46.3% for RAP and RCA, respectively.

12.8% is the highest fines percent for RCA (Edil et al 2012a) while 0.1% is the lowest fines percent as reported in Mahedi and Cetin (2020). On the other hand, RAP's lowest fines content is 0% in Alam et al. (2010) study while Camargo et al. (2013) reported highest fines content in RAP with 11%. In summary, the median value of fines percent is 1% and 2.8% for RAP and RCA, respectively.

Table 2. summarizes the gradation table in Appendix A by providing maximum, minimum and median value of RAPs and RCAs according to the database.

Appendix A reports the specific gravities and gradation characteristics (sand and fines percent are shown along with D_{10}, D_{30}, D_{60}, C_c and C_u) of the materials for each study. According to Appendix A, all of the RAPs and RCAs are classified as coarse-grained soils. Since most of the materials had fines content lower than 12%, they were all classified either well-graded gravel (GW) and poorly graded gravel (GP) or well-graded sand (SW) and poorly graded sand (SP)-SW.

Table 2. INDEX PROPERTY RANGES OF RCA AND RAP

Characteristics	RAP			RCA		
	Lower Limit	Median	Upper Limit	Lower Limit	Median	Upper Limit
% Gravel	3	45	68.1	31.8	51	94.1
% Sand	28.1	54	97	4.9	46.3	64.9
% Fines	0	1	11	0.1	2.8	12.8
D_{10} (mm/inch)	10^{-1}/ $3.9x10^{-3}$	$5x10^{-1}$/ $1.96x10^{-2}$	1/ $3.93x10^{-2}$	10^{-1}/ $3.9x10^{-3}$	$2.3x10^{-1}$/ $9x10^{-3}$	$4.3x10^{-1}$/ $1.7x10^{-2}$
D_{30} (mm/inch)	$8x10^{-2}$/ $3.1x10^{-3}$	1.5/ $6x10^{-2}$	4.9/ $1.9x10^{-1}$	$2x10^{-1}$/ $7.9x10^{-3}$	1.2/ $4.72x10^{-2}$	6.5/ $2.56x10^{-1}$
D_{60} (mm/inch)	$1.5x10^{-1}$/ $5.9x10^{-3}$	4.82/ $1.89x10^{-1}$	10.4/ $4.09x10^{-1}$	$6x10^{-1}$/ $2.36x10^{-2}$	6.8/ $2.67x10^{-1}$	16.3/ $6.42x10^{-1}$
C_u	5	10.65	40	2.1	32	66
C_c	0.21	1.2	8	0.14	1.4	6
G_s	2.19	2.395	2.87	2.12	2.39	2.7

Notes: 52 gravel, sand and fines contents data were collected for RAPs from different sources, while 30, 27 and 27 data were used to determine the lowest, median and highest values of D_{10}, D_{30} and D_{60} for RAPs, respectively. 35 C_u, 37 C_c, and 38 G_s data were available to derive lower,median and upper limits of RAPs. 34 gravel, sand and fines contents data were collected for RCAs from different sources, while 19, 17 and 17 data were used to determine the lowest, median and highest values of D_{10}, D_{30} and D_{60} for RCAs, respectively. 29 C_u, 29 C_c, and 32 G_s data were available to derive lower,median and upper limits of RCAs.

1.2. COMPACTION CHARACTERISTICS

The general trend of Proctor compaction tests shows that RAP and RCA have lower maximum dry unit weight than natural aggregates (Figure 1, Figure 2, Figure 3 and Figure 4). Table 3 summarizes the compaction characteristics (MDU and OMC) of RCA and RAP materials collected for the database. MDU of RAP ranges between 17.2 kN/m^3 (110 pcf) and 24.1 kN/m^3 (155 pcf) with the median value of 19.6 kN/m^3 (126 pcf). The limits of MDU for RCA is 18.3 kN/m^3 (118 pcf) and 21.7 kN/m^3 (140 pcf) with the median of 19.7 kN/m^3 (127 pcf). OMC of RAPs ranges between 4% and 10.7% with the median to be 6.05% while OMC of RCA ranges between 6.1% and 14.8% with the median of 10.8%. Figures 3 and 4 show that the voids in RAP matrix cannot be filled effectively because of low fines contents which yields to a lower maximum dry unit weight (Locander 2009; Blankenagel and Guthrie 2006).

RAP possesses hydrophobic properties due to the asphalt coating around aggregate particles and this contributes RAP materials to have lower optimum water content (Figure 5 and Figure 6). On the other hand, RCA shows hydrophilic properties due to concrete mortar residues thus a higher optimum moisture content is reported as a result of higher water absorption capacity of RCA (Figure 7 and Figure 8) (Edil et al. 2012a, Nokkaew et al. 2012, Sayed et al. 1993, Rahardjo et al. 2010). In addition, the hydration and cementation of dehydrated cement particles in RCA may cause a reduction in the dry unit weight of RCA. Furthermore, higher fines contents in RCA cause a higher optimum water content due to an increase in the surface area and absorption capacity of aggregate and cement particles (Jayakody et al. 2012). On the other hand, lower maximum dry unit weights of RAP may be due to their lower specific gravities caused by asphalt content and low fines contents (Guthrie et al. 2007, Locander 2009).

The reduction in the maximum dry unit weight (MDU) of recycled aggregate-natural aggregate mixtures is directly proportional to the RAP and RCA contents in the mixtures (Taha et al. 1999). Higher rate of reductions in the maximum dry unit weights were observed with an increase in recycled aggregate contents in the mixtures (Bennert et al. 2000). Moreover, using more RAP content in the RAP-natural aggregate mixtures caused further reductions in the optimum water content (Locander 2009) while use of a higher amount of RCA in the RCA-natural aggregate mixtures caused an increase in the optimum moisture content (OMC) (Bennert et al. 2000). According to our database, the compaction results were mostly obtained from materials compacted at modified Proctor compaction energy. Therefore, it is recommended to use modified Proctor compaction data for analyses.

Table 3. COMPACTION CHARACTERISTICS RANGES OF RCA AND RAP

Characteristics	RAP			RCA		
	Lower Limit	Median	Upper Limit	Lower Limit	Median	Upper Limit
MDU $(kN/m^3)/(pcf)$	17.2 (110)	19.6 (126)	24.1 (155)	18.3 (118)	19.7 (127)	21.7 (140)
OMC (%)	4	6.05	10.7	6.1	10.8	14.8

Notes: MDU=Maximum dry density, OMC=optimum moisture content. 46 and 35 MDU and OMC data were collected for RAPs and RCAs, respectively.

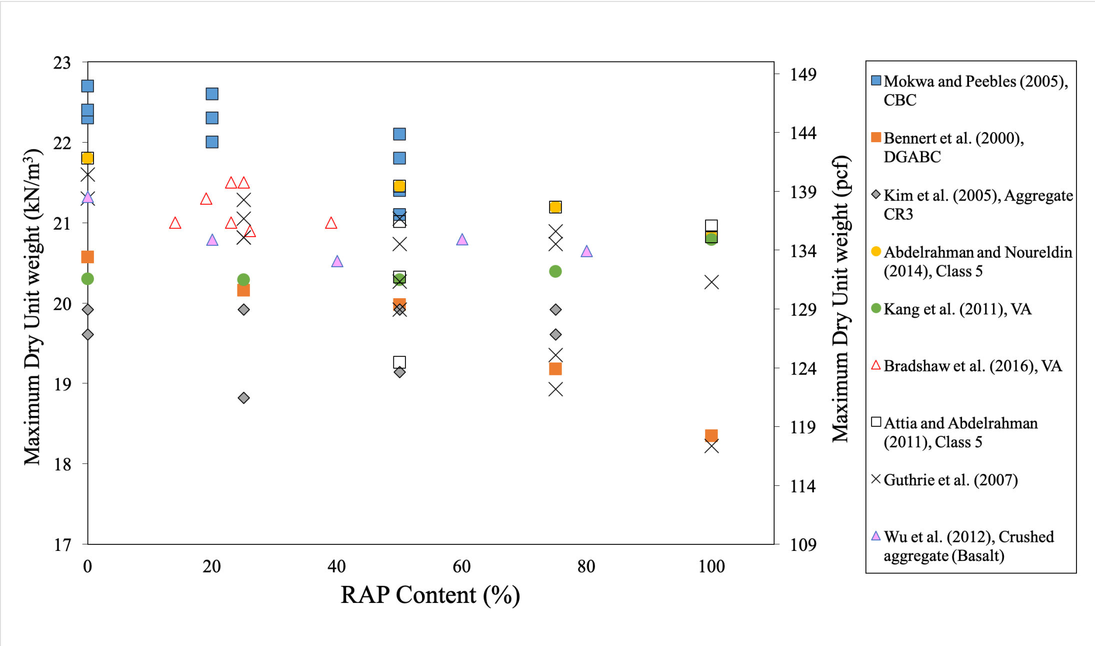

Figure 1. MAXIMUM DRY UNIT WEIGHT (MDU) VERSUS RAP CONTENT

Notes: CBC= Crushed base course; DGABC= Dense graded aggregate base course; CR3= County Road 3; VA= Virgin aggregate

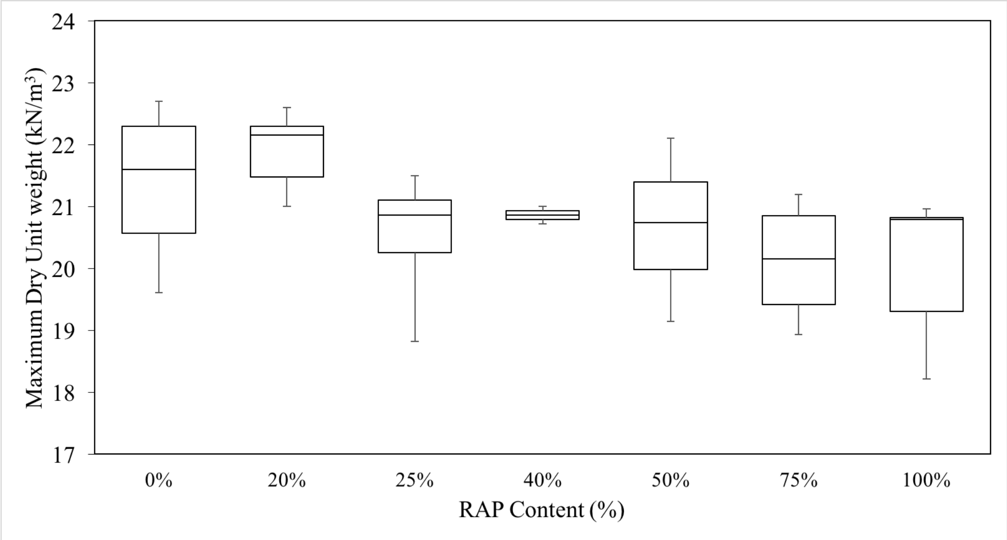

Figure 2. MAXIMUM DRY UNIT WEIGHT (MDU) VERSUS RAP CONTENT (WHISKER PLOT)

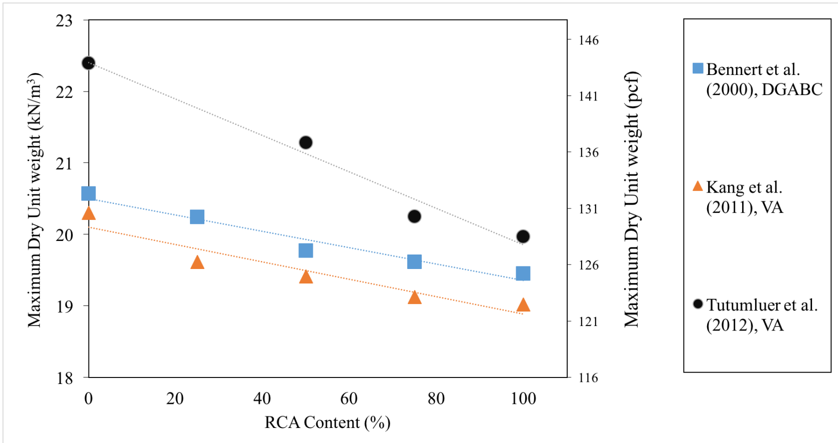

Figure 3. MAXIMUM DRY UNIT WEIGHT (MDU) VERSUS RCA CONTENT

Notes: DGABC= Dense graded aggregate base course; VA= Virgin aggregate

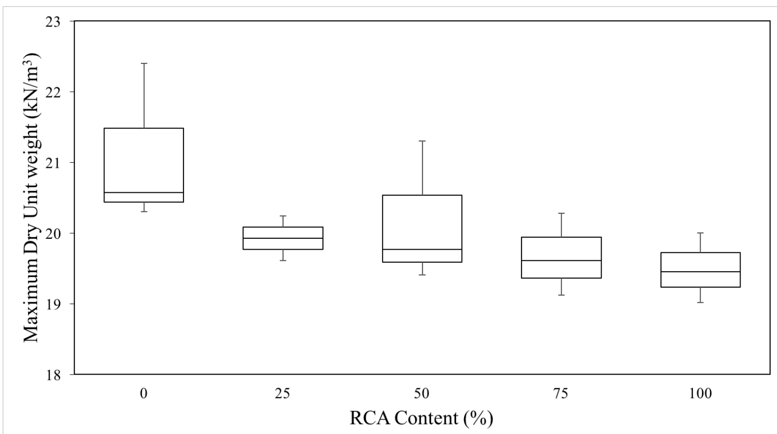

Figure 4. MAXIMUM DRY UNIT WEIGHT (MDU) VERSUS RCA CONTENT (WHISKER PLOT)

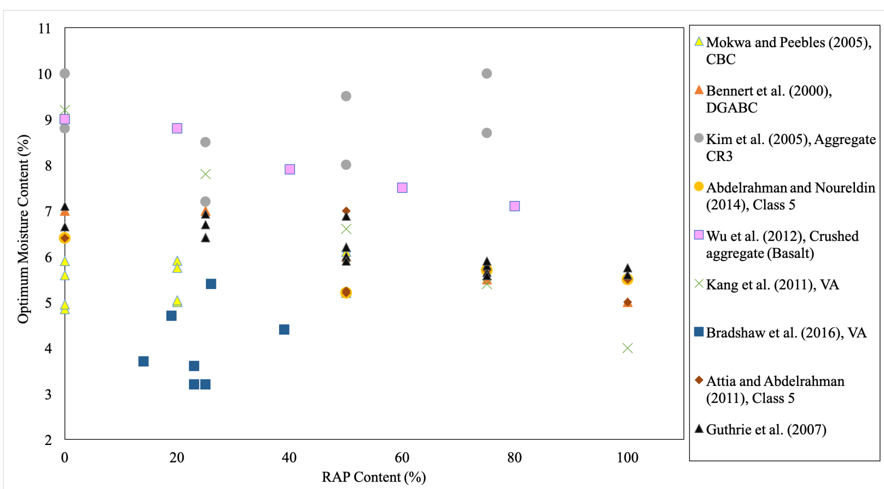

Figure 5. OPTIMUM MOISTURE CONTENT (OMC) VERSUS RAP CONTENT

Notes: CBC= Crushed Base Course; DGABC= Dense Graded Aggregate Base Course; CR3= County road 3; VA= Virgin aggregate

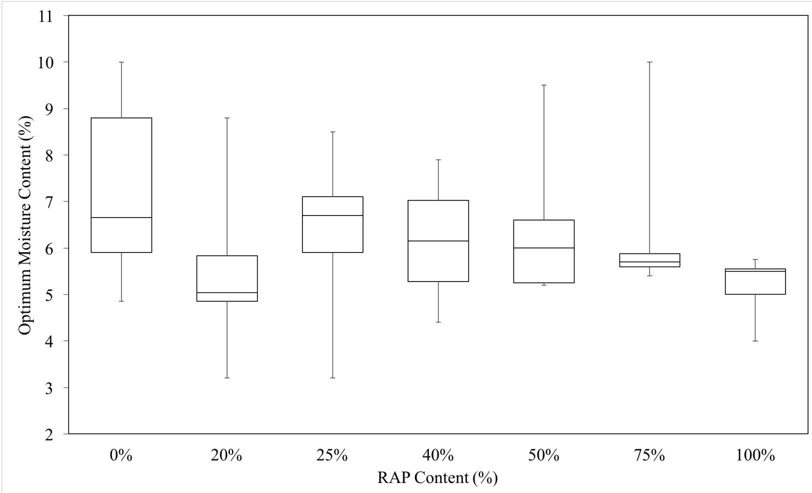

Figure 6. OPTIMUM MOISTURE CONTENT (OMC) VERSUS RAP CONTENT (WHISKER PLOT)

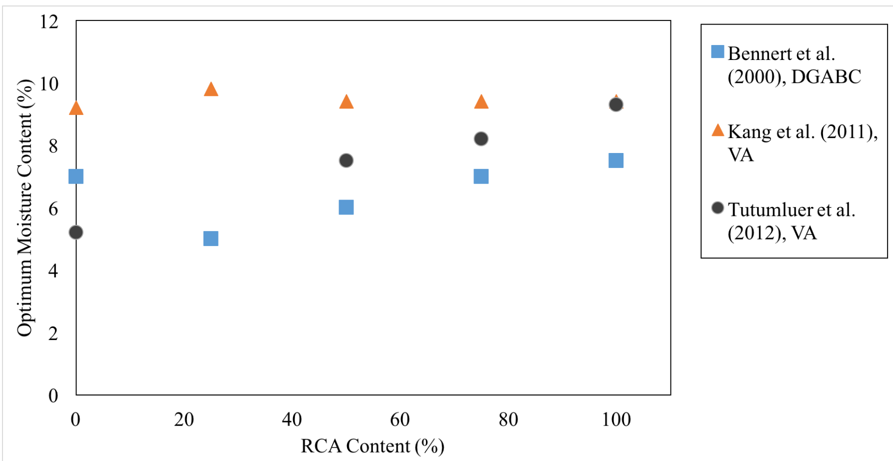

Figure 7. OPTIMUM MOISTURE CONTENT (OMC) VERSUS RCA CONTENT

Note: VA= Virgin aggregate; DGABC= Dense graded aggregate base course

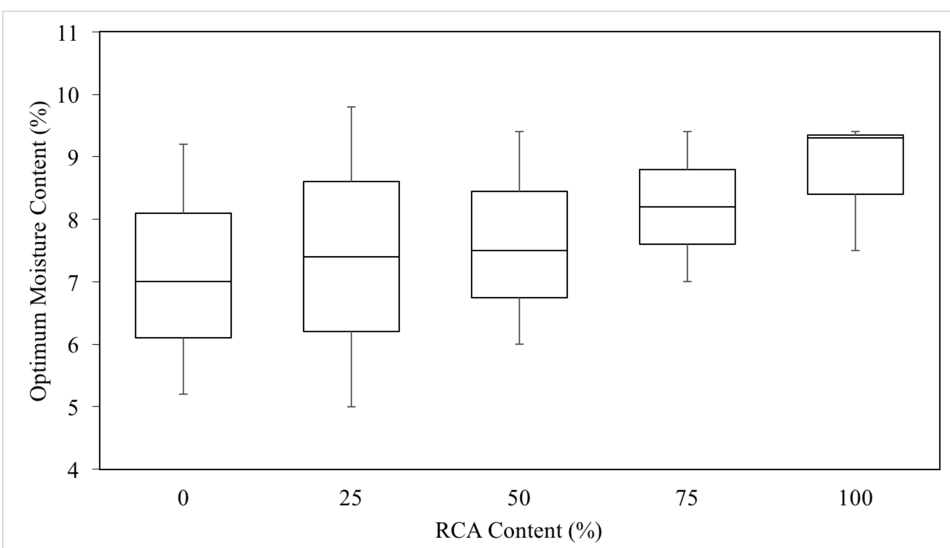

Figure 8. OPTIMUM MOISTURE CONTENT (OMC) VERSUS RCA CONTENT (WHISKER PLOT)

It is stated by Kim et al. (2007) that gyratory compactor provided better results to simulate the in-situ conditions. Figure 9 shows that the gratory compaction results simulate the actual field results better than the Proctor compaction results in terms of moisture content and dry unit weight (Kim et al. 2007).

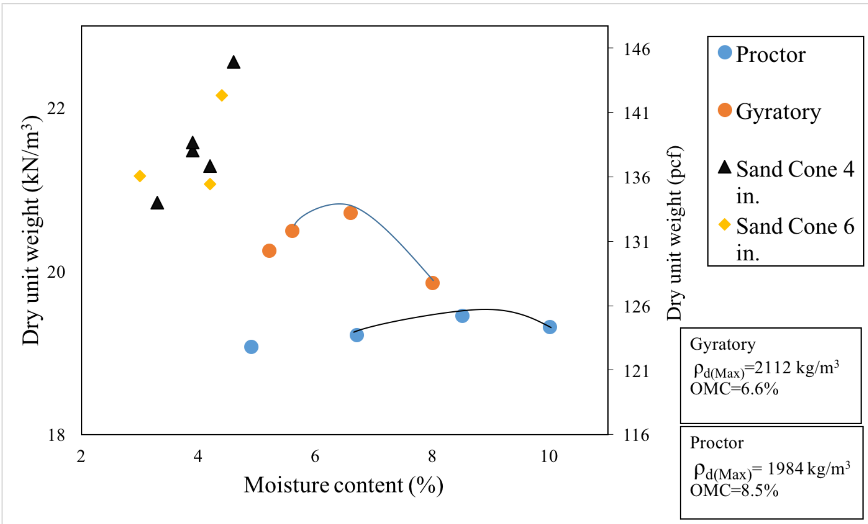

Figure 9. THE EFFECT OF COMPACTION METHOD ON THE DEGREE OF THE COMPACTION

Binding quality improves between aggregate particles due to softening of asphalt binder at higher temperatures. Therefore, the compaction characteristics of RAP changes with temperature (Soleimanbeigi and Edil 2015b). For example, the dry unit weight of the specimens increased about 3.5% when compacted at 49°C (120°F) than the ones compacted at 21°C (70°F) (Montemayor 1998, as cited in Cosentino and Kalajian 2001).

1.3. PLASTICITY CHARACTERISTICS

Most of the plasticity index of RAP and RCA were reported as non-plastic (NP) (Locander 2009, Ullah and Tanyu 2019, Edil et al. 2012a, Mijic et al. 2019, Ullah et al. 2018, Edil et al. 2012b, Guthrie et al. 2007, and Cetin et al. 2020. Attia and Abdelrahman (2010a) tested the liquid limit (LL) of 100% RAP and 75% RAP and reported LL to be 26 and 25, respectively. They also reported LL to be 19, 20, 25 and 30 for different 50% RAPs mixed with Class 5 aggregate. Class 5 is a typical base course material classification used in pavements by MnDOT. More detailed information about Class 5 can be found at MnDOT grading and base manual (MnDOT 2016).

CHAPTER 2. MECHANICAL AND HYDRAULIC PROPERTIES OF RAP AND RCA

Index properties, the aggregate type, and asphalt/mortar content of RAP and RCA affect their engineering properties significantly (Thakur and Han 2015 and Hiller et al. 2011). Thus, it is important to study the components and the engineering properties of the aggregates for constructing high-quality pavements as recycled aggregates are obtained from different sources (Gonzalez and Moo-Young 2004). Some specifications (AASHTO 2002, Greenbook 2009, ASTM 2016) limit the content of an impurity, e.g., crushed clay brick, unless its presence improves the engineering properties of aggregate base course (Edil et al. 2012a). Hydraulic conductivity, stiffness, strength, shear strength and permanent deformations are discussed and summarized in this section.

2.1. HYDRAULIC CONDUCTIVITY

One of the main functions of aggregate base layers is to provide an adequate drainage and prevent capillary action to increase the service life of pavements (Cedergren 1988). An increase in the pore water pressure in the aggregate base layers causes a reduction in the stiffness of aggregate base layers (Edil et al. 2012a). Hydraulic properties of aggregates are affected by gradation characteristics (e.g. sand, fines content and D_{10}). Fine particles fill up the voids and reduce drainage properties of aggregates (Cosentino et al. 2003). Saturated hydraulic conductivity (k_{sat}) and soil-water characteristics curve (SWCC) are the two parameters that should be evaluated for pavement designs (Nokkaew et al. 2012). Saturated hydraulic conductivity is a quantitative measure of a saturated soil's ability to transmit water when subjected to a hydraulic gradient and it is used as a parameter for drainage design while SWCC can be used to determine the matric suction of aggregates at different moisture contents then it can be used to predict the modulus of aggregate base layers (Gupta et al. 2004, Ba et al. 2013).

As mentioned in the previous section, RAP shows hydrophobic properties while RCA shows hydrophilic properties (Edil et al. 2012a; Rahardjo et al. 2010). Due to the hydrophobicity of RAP, it tends to have higher k_{sat} than RCA (Nokkaew et al. 2012). Thus, if the gradations are similar, RAP tends to provide a better drainage layer than RCA (Edil et al. 2012a; Hoppe et al. 2015).

Mokwa and Peebles (2005) and Cosentino et al. (2003) reported an increase in hydraulic conductivity with higher RAP content in the RAP-natural aggregate mixtures. Kang et al. (2011) also showed 100% RAP had a higher hydraulic conductivity than natural aggregates. On the other hand, Wu et al. (2012) indicated that the hydraulic conductivity of base course materials decreased by the addition of RAP. After porosity analysis using X-ray scanning, it turned out that the 80% RAP had less air voids than the crushed aggregate specimens which may have been the cause for observing low hydraulic conductivity. According to Bennert and Maher (2005), RAP-natural aggregate blends with an increase in RAP content from 25% to 75% lowered the hydraulic conductivity of the mixture to almost less than 3.5×10^{-6} m/s (4.2×10^{-2} ft/hr) while 100% RAP had a hydraulic conductivity value of approximately 5.64×10^{-5} m/s (6.7×10^{-1} ft/hr). Kang et al. (2011) showed that addition of 25% RAP in aggregates improved the saturated hydraulic conductivities of the aggregate mixtures since RAP was a coarser material than that of natural aggregates used in that particular study. However, with a further increase in RAP contents, the saturated hydraulic conductivities of the mixtures reduced. It was concluded that a reduction in the hydraulic conductivity may have been due to the that dense packing of the RAP-natural aggregate mixtures. Thus, it lowered the saturated hydraulic conductivity of the mixtures.

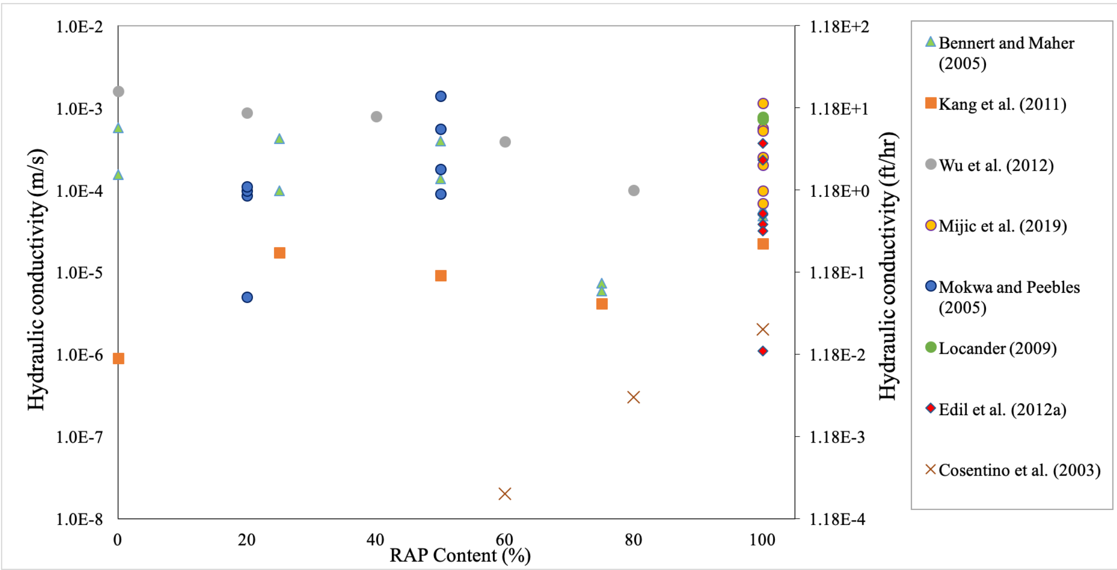

Figure 10. HYDRAULIC CONDUCTIVITY VERSUS RAP CONTENT

Bennert and Maher (2005) showed that RCA-natural aggregate blends with an increase in RCA content from 25% to 75% of total weight lowered the hydraulic conductivity of the blend to approximately 50% while the hydraulic conductivity of the RCA was 10^{-6} m/s (0.000145 ft/hr).

According to Kang et al. (2011), hydraulic conductivity of natural aggregates increased with addition of RCA up to 50% by weight. However, further addition of RCA caused a reduction in hydraulic conductivity. The RCA alone had higher hydraulic conductivity than that of natural aggregate, while the 50% RCA-50% natural aggregate mixture had the highest hydraulic conductivity with in all blends.

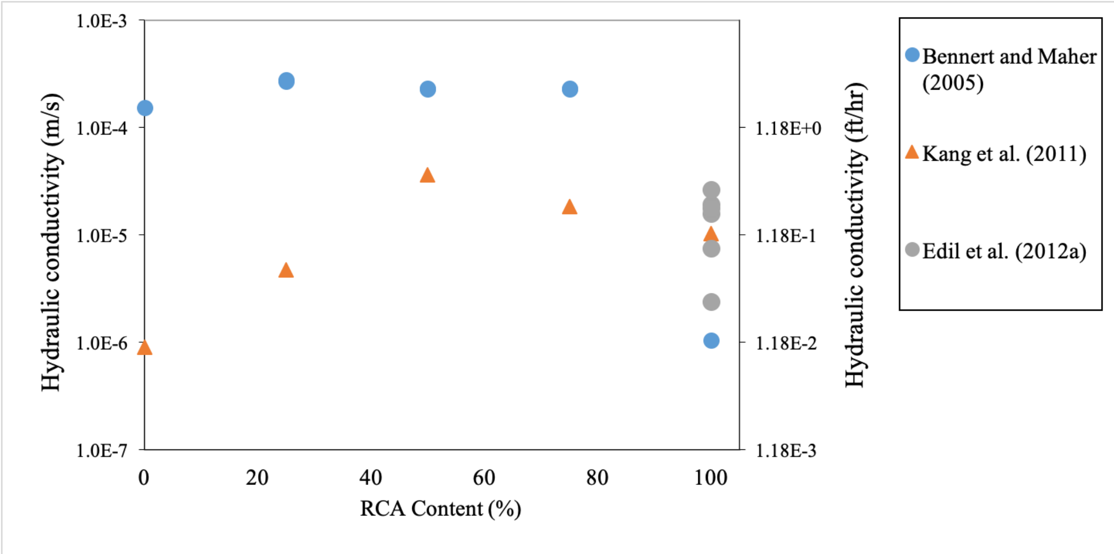

Figure 11. HYDRAULIC CONDUCTIVITY VERSUS RCA CONTENT

To evaluate the relationship between hydraulic conductivity and gradation of RAP materials, Figure 12 is presented with 8 data of hydraulic conductivity of 100% RAP from different studies. D_{10} and percent fines are expected to have major influence on hydraulic conductivity. Low D_{10} means higher fine particles, which is expected to clog the pores in the material matrix and reduce air voids. Thus, it causes lower hydraulic conductivity. However, Figure 12 confirms that hydraulic conductivity of RAP increases when D_{10} values increases with few exceptions (e.g. D_{10} = 0.4 mm).

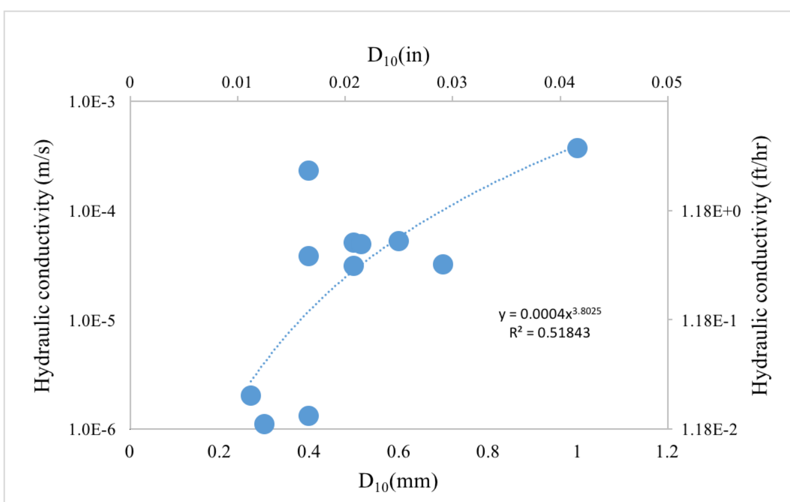

Figure 12. HYDRAULIC CONDUCTIVITY VERSUS D_{10} OF 100% RAP

According to Figure 13, there is a decreasing trend for the hydraulic conductivities of RAP materials as fines content increases.

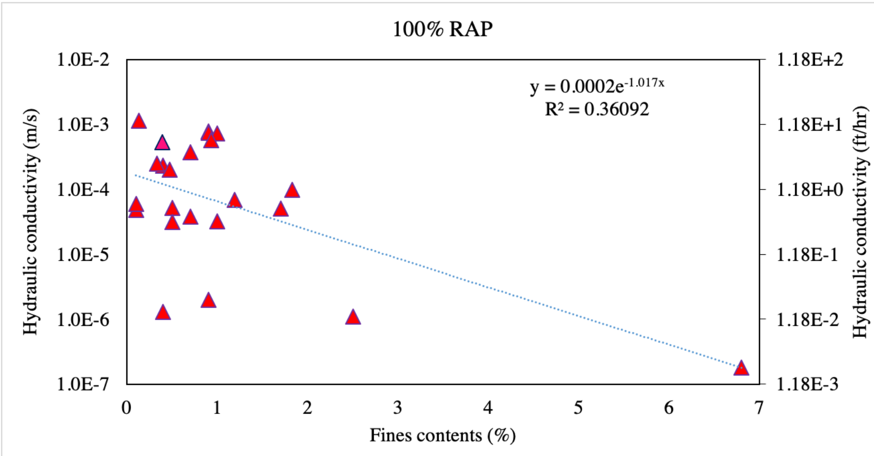

Figure 13. HYDRAULIC CONDUCTIVITY VERSUS FINES CONTENT OF 100% RAP

Figure 14 shows the relationship between hydraulic conductivity of different RAP blends and their corresponding fines contents. The hydraulic conductivities of different crushed base course materials mixed with RAP materials at 20% and 50% RAP were collected from Mokwa and Peebles (2005).

Hydraulic conductivity of 100% RAP ranges between 1.8×10^{-7} m/s (2.1×10^{-3} ft/hr) and 1.1×10^{-3} m/s (1.7×10^{-1} ft/hr) with the median of 6. 9×10^{-5} m/s (1×10^{-2} ft/hr) according to our database.

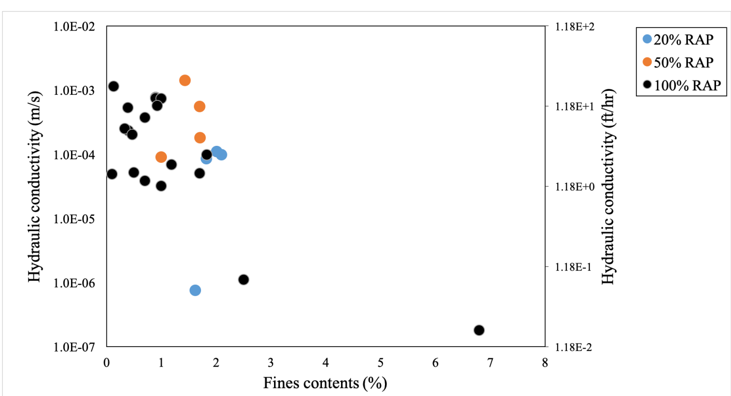

Figure 14. HYDRAULIC CONDUCTIVITY VERSUS FINES CONTENT OF DIFFERENT RAP BLENDS

In Figure 15, there are 11 different hydraulic conductivity data of 100% RCA with corresponding fines content. On the other hand, no trend was observed between hydraulic conductivity and fines content of the RAP materials. Hydraulic conductivities of RCAs ranged between 1.05×10^{-6} m/s (1.2×10^{-2} ft/hr) and 1.2×10^{-3} m/s (1.7×10^{-1} ft/hr) with the median of 1.7×10^{-5} m/s (2.5×10^{-3} ft/hr) according to our database.

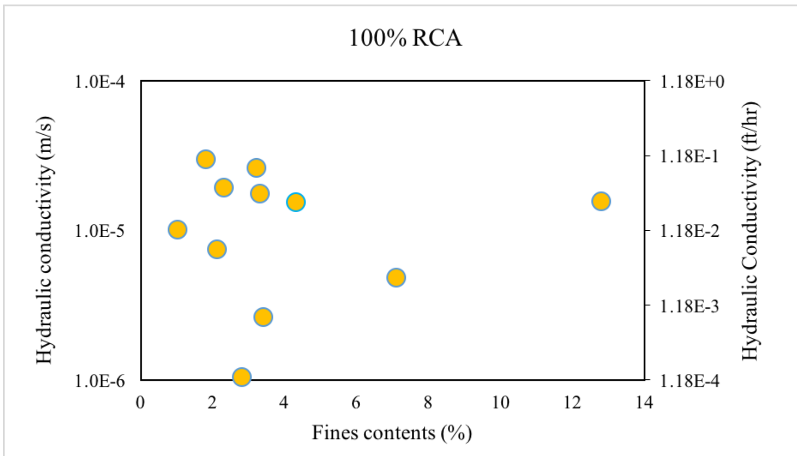

Figure 15. HYDRAULIC CONDUCTIVITY VERSUS FINE CONTENT OF 100% RCA

2.2. STRENGTH

The California Bearing Ratio (CBR) of base materials is an indication of their mechanical characteristics under vertical loading (traffic) and is determined as the ratio of the penetration resistance of the base material to that of a standard crushed stone. The CBR has been used by pavement engineers to characterize the strength of materials for designing pavements (Thakur and Han 2015). The minimum CBR values of the aggregate base and subbase layers should be 80 and 60, respectively (Jayakody et al. 2012; Ooi et al. 2010). In Florida, lime rock Bearing Ratio (LBR) which is a modified version of conventional CBR test, is commonly used (Cosentino et al. 2003).

In addition to the specified minimum CBR values, LBR should be at least 100 (LBR = 1.25 x CBR) for aggregate base layers (FDOT 2018).

The database showed that the CBR values of 100 % RAP ranged from 18 to 68 with the median to be 28 while CBR of 100% RCA was between 58 to 169 with the median 146.

2.2.1. IMPACTS OF SELECTED INDEX PROPERTIES ON CBR

Gravel-to-sand (G/S) ratio and fines contents were selected to investigate their effects on CBR of RAP and RCA materials. Figures 16 and 17 reveal that there is not a specific trend between CBR of 100% RAP versus fines content and/or gravel to sand ratio; however, it is not possible to draw any conclusion due to lack of the data.

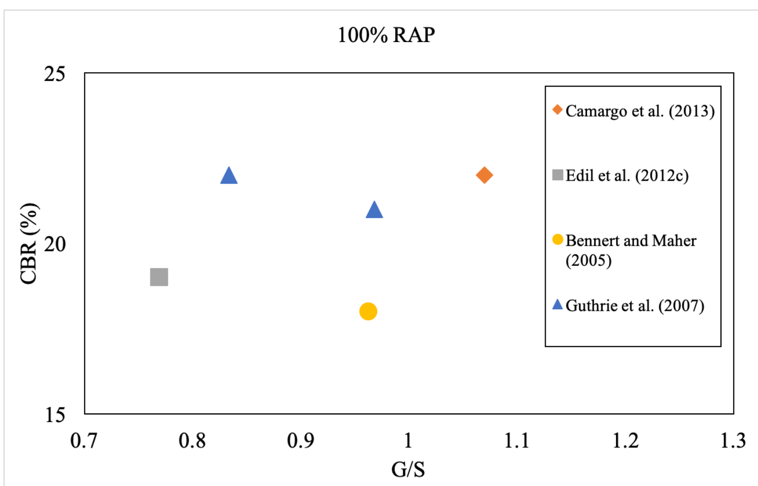

Figure 16. CBR VERSUS GRAVEL TO SAND RATIO OF 100% RAPS

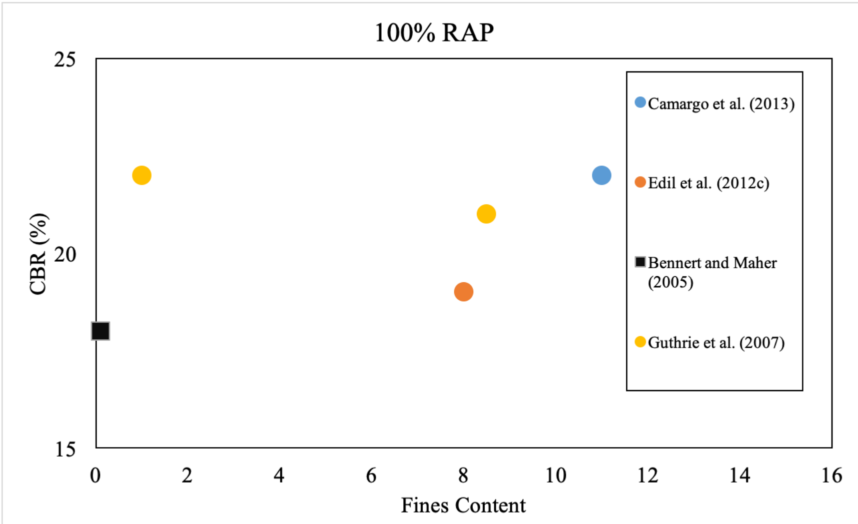

Figure 17. CBR VERSUS FINES CONTENT OF 100% RAP SAMPLES

According to Figures 18 and 19, there is also not a specific trend between CBR of 100% RCA and fines content or gravel to sand ratio; however, it is not possible to draw any conclusion due to lack of data.

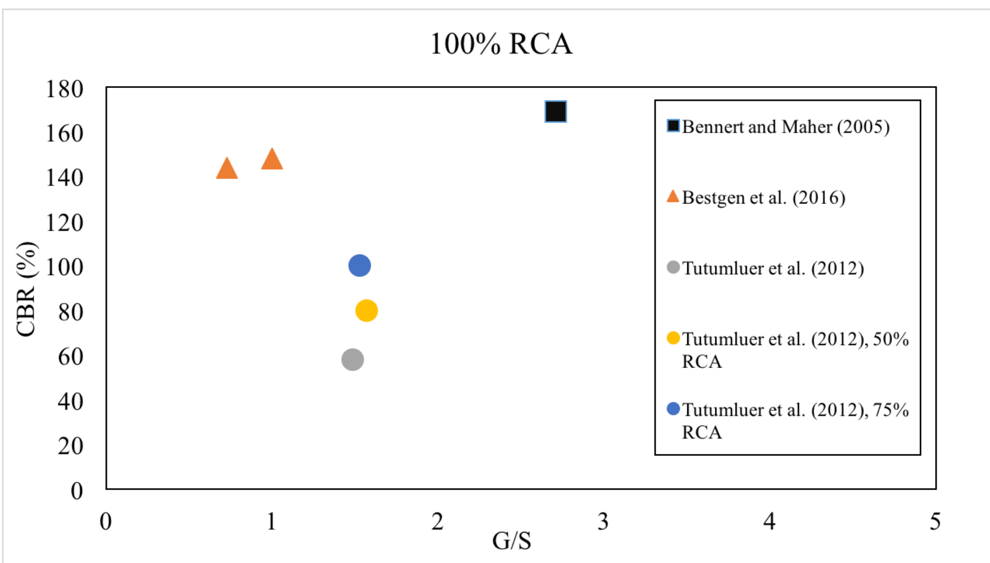

Figure 18. CBR VERSUS GRAVEL TO SAND RATIO OF RCA SAMPLES

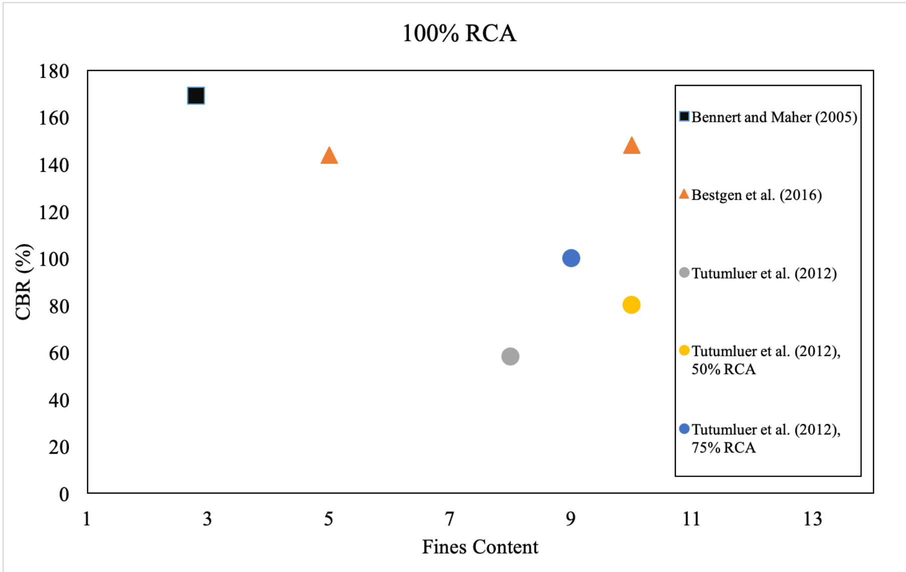

Figure 19. CBR VERSUS FINES CONTENT OF RCA SAMPLES

2.2.2. IMPACTS OF RAP/RCA CONTENTS ON CBR

In general, RAP has lower CBR than natural aggregates. In addition, increasing the RAP content in the RAP-natural aggregate mixtures reduces the CBR (Bennert and Maher 2005; Guthrie et al. 2007). Figure 20 clearly shows that CBR of RAP-natural aggregates decreases with higher RAP contents in the mixture. The asphalt coating around the particles may be the reason for CBR reduction in the presence of RAP since asphalt coating reduces the particle bonding and interlocking mechanism of aggregate particles (Ooi et al. 2010; Taha et al. 1999). In addition, a lower fines content of RAP may leave unfilled voids (open-graded structure), which may result in lower CBR (Sayed et al. 1993). Cosentino et al. (2003), Bennert and Maher (2005), Ullah and Tanyu (2019), Cosentino and Bleakley (2013) and Guthrie et al. (2007) conducted CBR tests on blended RAP-natural aggregate specimens, all of which except Cosentino et al. (2003), reported a decrease in CBR with an increase in RAP content. On the other hand, Cosentino et al. (2003) observed that the CBR of the mixtures first increased with an increase in RAP content in the blend up to a certain level (~RAP content is 80%) and then started decreasing.

Depending on the physical, chemical and morphological characteristics of RAP and/or moisture contents used for blends, different trends could be observed in different applications (Thakur and Han 2015). Figure 20 reports the type of each material that is blended with RAP. Natural aggregate, lime rock (LR), base material, dense graded aggregate base course (DGABC) and fine sand were used to blend RAPs.

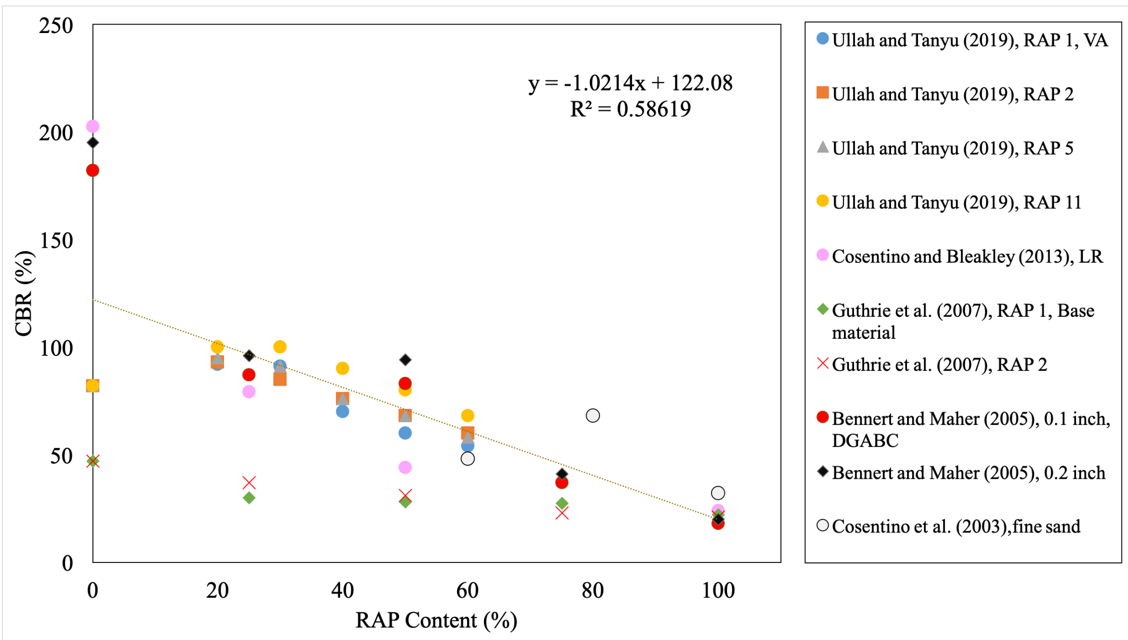

Figure 20. CBR VERSUS RAP CONTENT
Notes: VA= Virgin aggregate; LR= Limerock; DGABC= Dense graded aggregate base course

The literature review showed that compacting RAP at a relatively higher temperature increased its dry unit weight which led to an increase in the LBR values. For instance, RAP that was compacted at 49°C (120°F), the range of LBR was increased from 25-50 to 42-125 (Montemayor 1998). On the other hand, higher ambient temperature decreases the LBR of RAP after compaction, while higher LBR values were observed at lower ambient temperatures due to asphalt material hardening (Cosentino and Kalajian 2001).

It was observed that the CBR of RCA materials (either soaked or unsoaked) had different trends in different studies. While lower CBR values were seen for unsoaked RCA materials compared to natural aggregates, this trend was opposite when they were soaked (Jayakody et al. 2012). The reason of different behaviors of RCA could be due to the presence of dehydrated cement content. Relatively higher CBR values can be observed with longer soaking period since more cementitious reactions could occur with longer curing periods (Poon et al. 2006; Garach et al. 2015; Bestgen et al. 2016).

To investigate the CBR behaviors of natural aggregate-RCA mixtures, Figure 21 and 22 are presented. These Figures show that there is not a discernible trend between CBR and RCA contents. Figure 22 shows normalized CBR versus RCA content. Normalized CBR is obtained by dividing the CBR of each RCA blend by the CBR of 100% RCA of the same study.

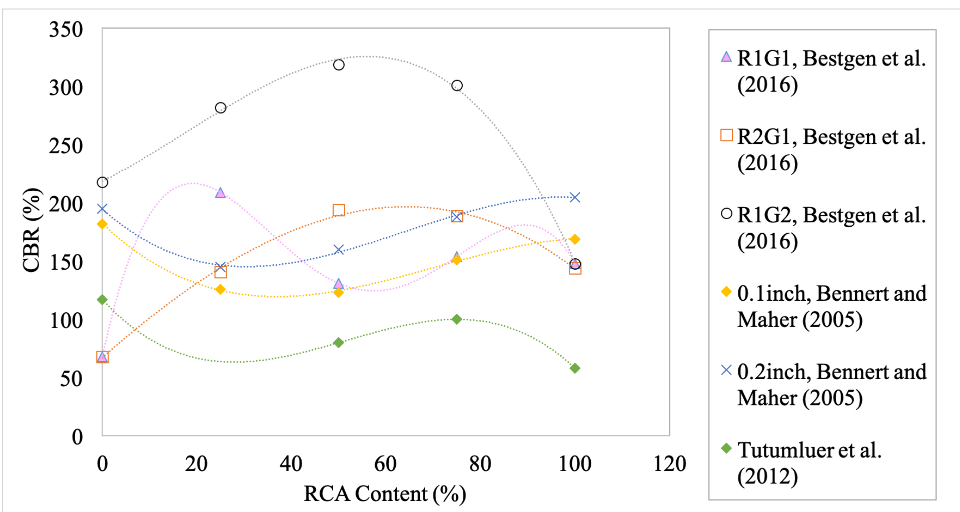

Figure 21. CBR VERSUS RCA CONTENT

Notes: The CBR value corresponding to 0.1 and 0.2 inches of penetration was used in Bennert and Maher (2005) study

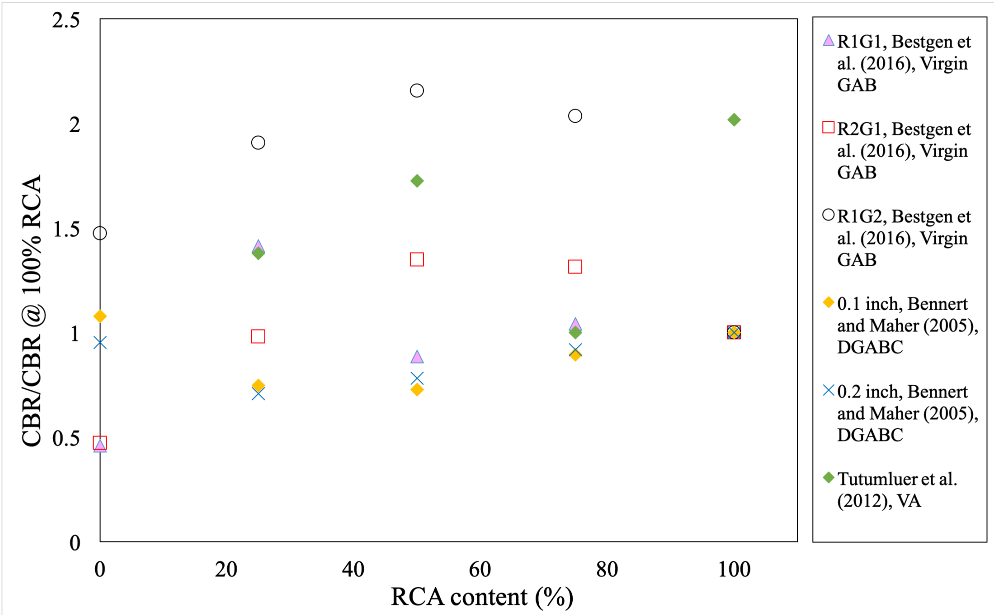

Figure 22. NORMALIZED CBR VERSUS RCA CONTENT

Notes: Virgin GAB= Virgin graded aggregate base; DGABC= Dense graded aggregate base course; VA= Virgin aggregate

2.3. STIFFNESS

Resilient modulus (M_r) is a fundamental material property used to analyze stiffness of materials under different conditions such as moisture, density, and stress level. The 1993 American Association of State Highway and Transportation Officials (AASHTO) flexible pavement design method and the current Mechanistic-Empirical Pavement Design Guide (MEPDG) use M_r to define subgrade and base stiffness for pavement systems. Mr is defined as a ratio of applied axle deviator stress and axle recoverable strain. M_r of RAP and/or RCA materials depends on several factors including moisture content, freeze-thaw cycles, density, stress history, aggregate type, RAP or RCA type, gradation, temperature, asphalt content in RAP, type of stabilizing agent and curing time (Thakur and Han 2015, Bestgen et al. 2016).

M_r plays an important role in pavement design. Therefore, most of the studies have reported M_r as a stiffness characteristic. In addition to the summary resilient moduli (SM_R) values, k_1, k_2, k_3, k_6

and k_7 were provided in the database. Summary resilient moduli (SM_R) were reported at a bulk stress of 208 kPa and octahedral stress of 48.6 kPa for base materials as recommended by NCHRP 1-28A.

As RAP content increases, M_r gets higher while the plastic strain increases. More than 400 M_r data investigating the resilient modulus of RAP, RCA or blends were collected for the database. It also includes the M_r data of these materials that were tested at different environmental conditions including different temperatures, freeze-thaw cycles, and moisture contents.

Overall, it was observed that RAP and RCA in the base course had higher M_r than that of well-graded natural aggregates. The SM_r reported in the database for RAP was between 168 MPa (24366 psi) and 400 MPa (58015 psi) with the median value to be 261.5 MPa (37927 psi). The SM_r of RCA ranged between 123.4 MPa (17897 psi) and 370 MPa (53664 psi) with the median value of 183 MPa (26541 psi) according to the database.

2.3.1. EFFECTS OF INDEX PROPERTIES ON STIFFNESS OF RCA AND RAP

Figure 23 presents summary resilient modulus of RAP versus gravel-to-sand (G/S) ratio to investigate the effects of index properties on corresponding stiffness. Figure 24 shows that SM_r of RAP is lower at higher G/S ratios. This indicates that RAP with higher sand content would have higher SM_r in general (Figure 24).

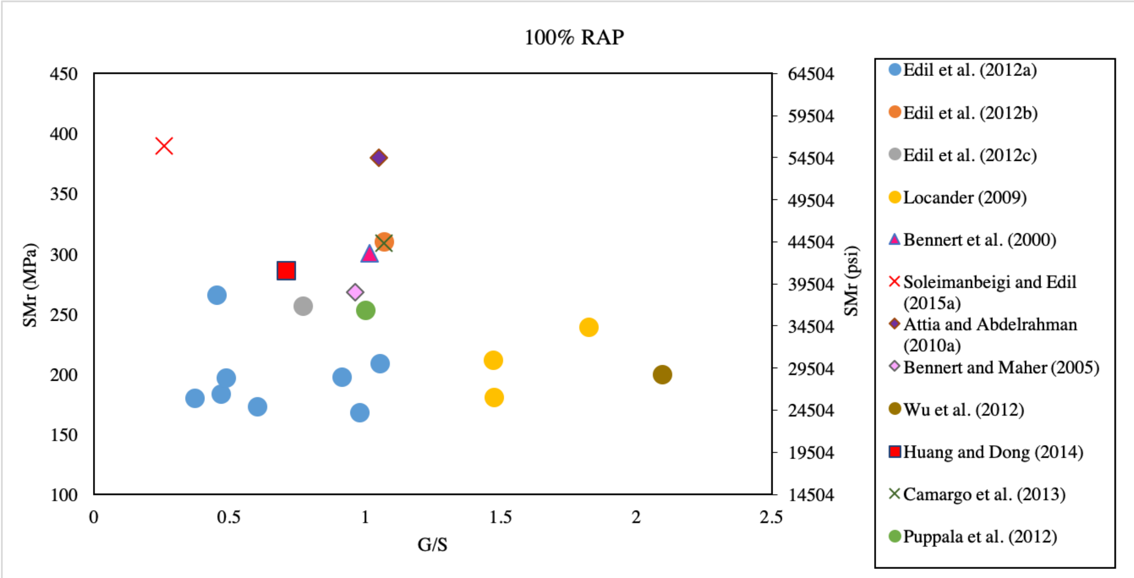

Figure 23. SM_R AND GRAVEL TO SAND RATIO OF 100% RAPS

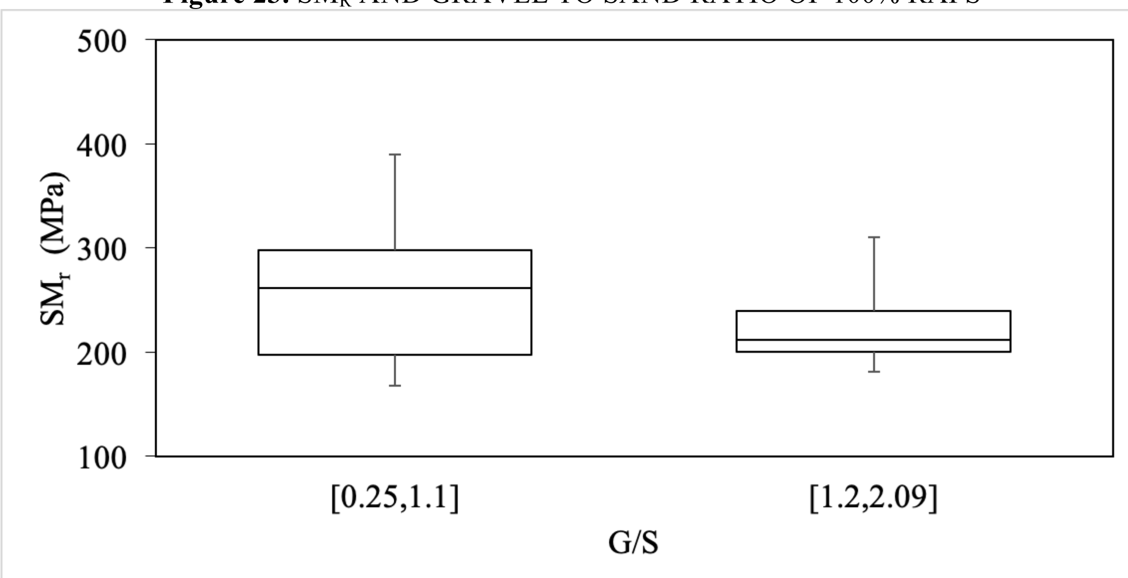

Figure 24. SM_R VERSUS GRAVEL TO SAND RATIO OF 100% RAPS (WHISKER PLOT)

Figure 25 and Figure 26 show that there is no correlation or trend between SM_r and fines content of RAPs (within the typical ranges observed in this study).

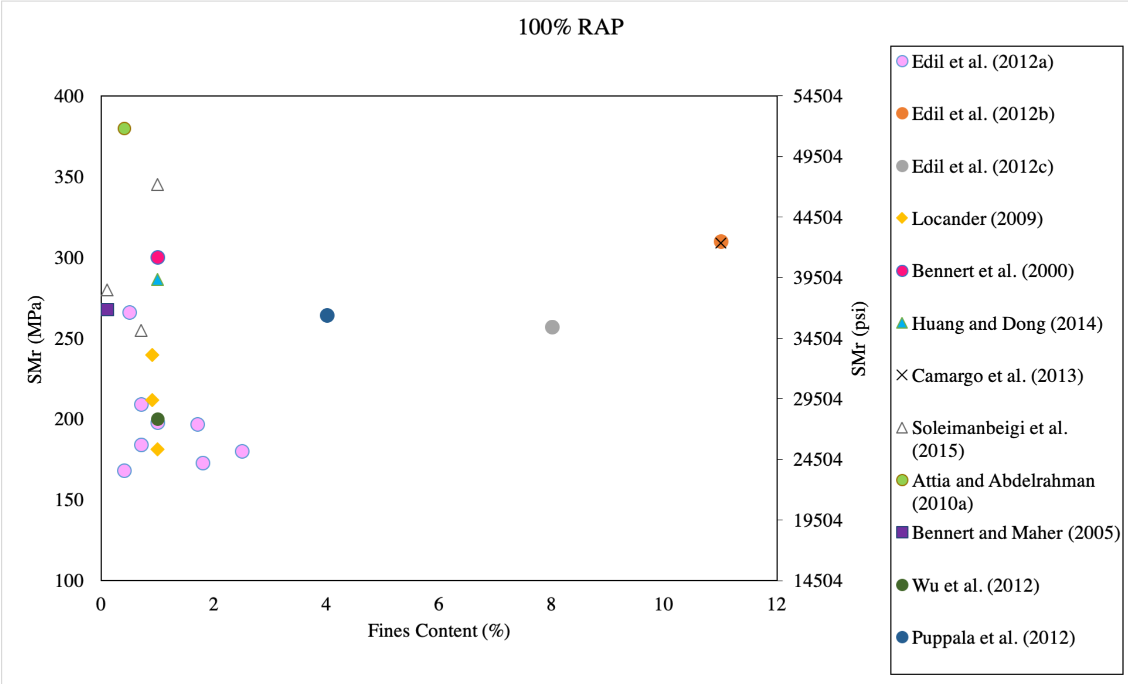

Figure 25. SM_R VERSUS FINES CONTENT OF 100% RAPS

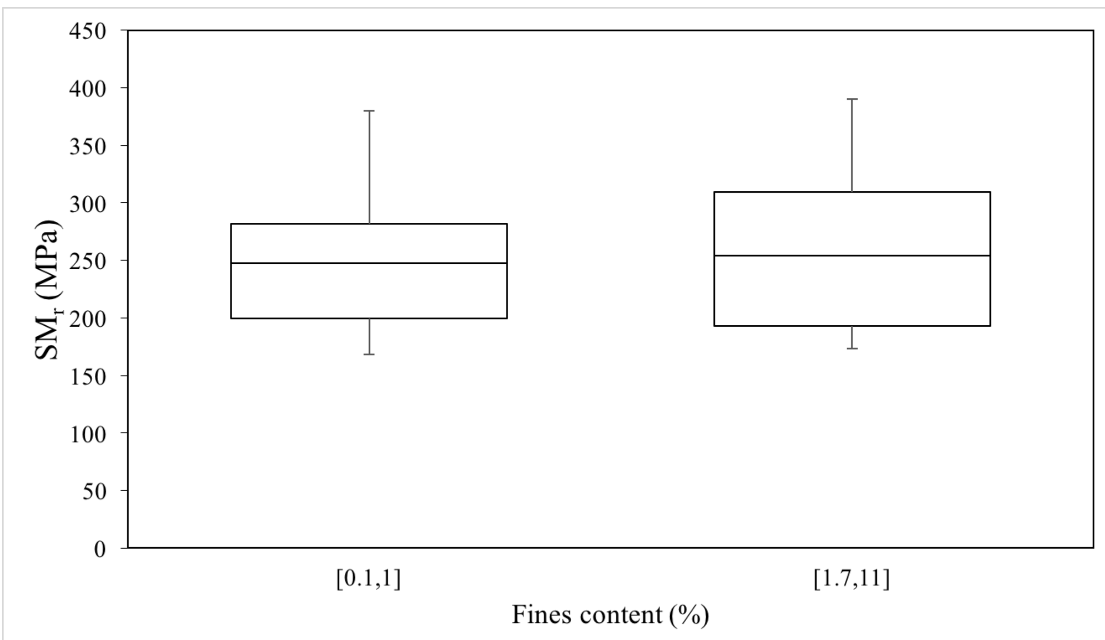

Figure 26. SM_R VERUS FINES CONTENT OF 100% RAPS (WHISKER PLOT)

The trend between SM_r of RCA and corresponding G/S ratios were not as significant as the ones observed for RAPs. However, it was observed that an increase in SM_r of RCAs when G/S ratio was higher.

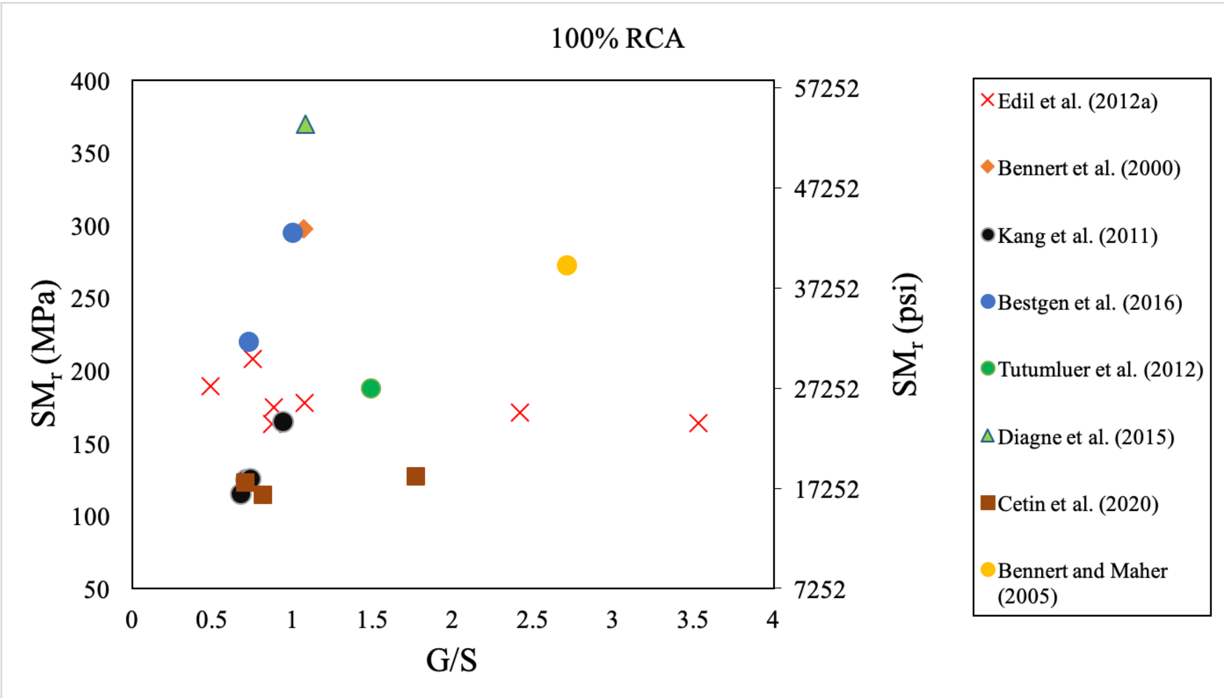

Figure 27. SM_R VERSUS GRAVEL TO SAND RATIO OF 100% RCAS

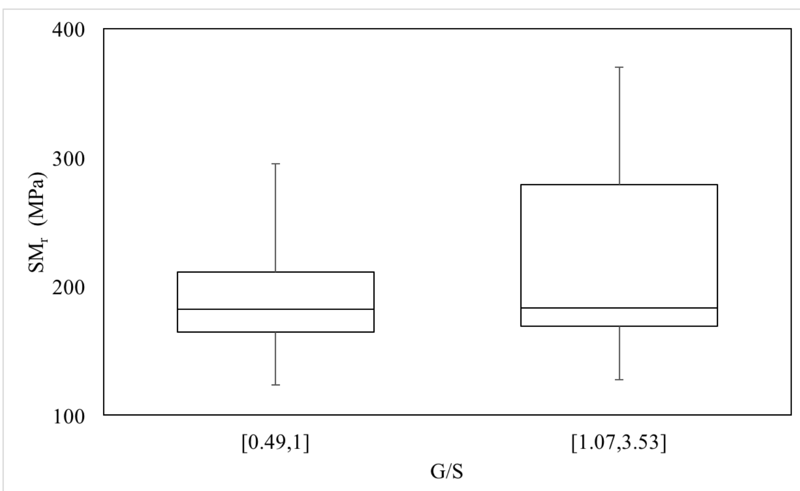

Figure 28. SM_R VERSUS GRAVEL TO SAND RATIO OF 100% RCAS (WHISKER PLOT)

According to Figures 29 and 30, there is a slight decrease in SM_r of RCAs as fines content increases. However, this was not consistent with some other studies such as Bestgen et al. (2016).

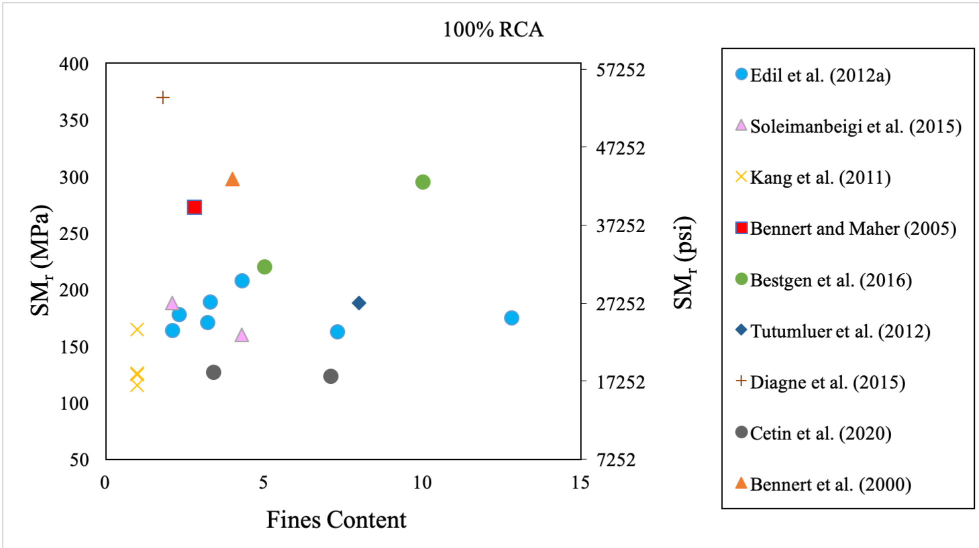

Figure 29. SM_R VERSUS FINES CONTENT OF 100% RCAS

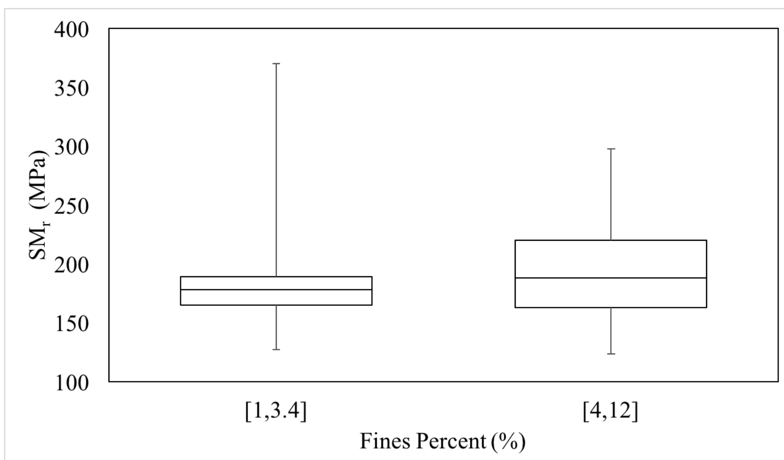

Figure 30. SM_R VERSUS FINES CONTENT OF 100% RCAS (WHISKER PLOT)

The maximum dry unit weight (MDU) and optimum moisture content (OMC) of RAP from different studies were plotted against the corresponding summary resilient modulus (SM_r) of the corresponding RAP materials (Figures 31 and 33). To better understand these scatter plots, Figures 32 and 34 are shown as whisker plots. However, no significant trend was observed between MDU and SM_r of RAP materials (Figure 32). There was a slight decrease in SM_r of RAPs with an increase in OMC (Figure 33).

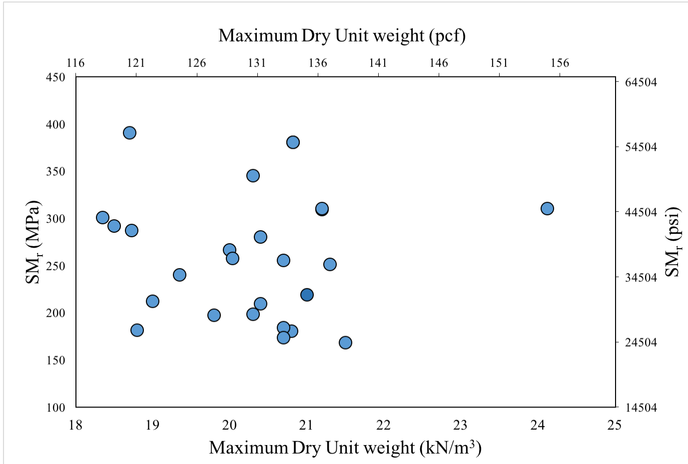

Figure 31. SM_R VERSUS MAXIMUM DRY UNIT WEIGHT OF 100% RAP

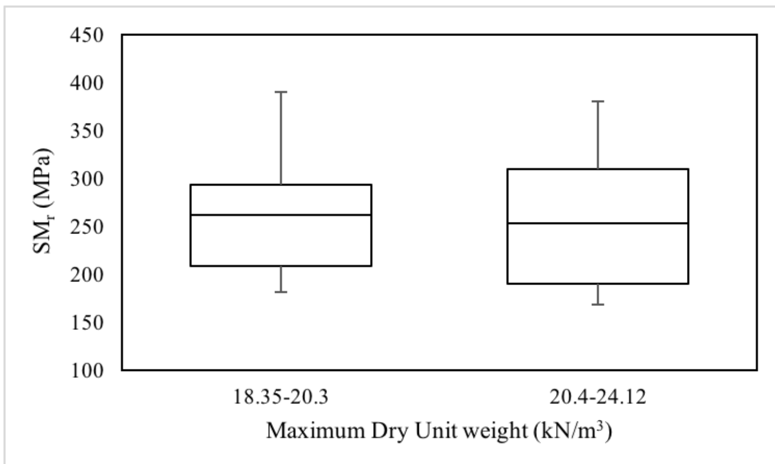

Figure 32. SM_R VERSUS MAXIMUM DRY UNIT WEIGHT OF 100% RAP (WHISKER PLOT)

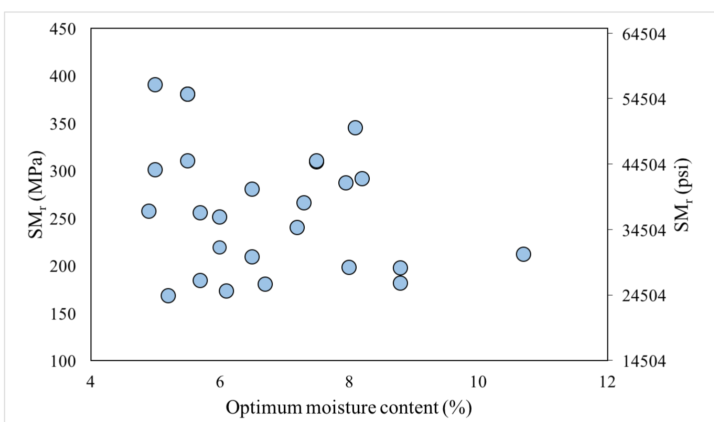

Figure 33. SM_R VERSUS OPTIMUM MOISTURE CONTENT OF 100% RAP

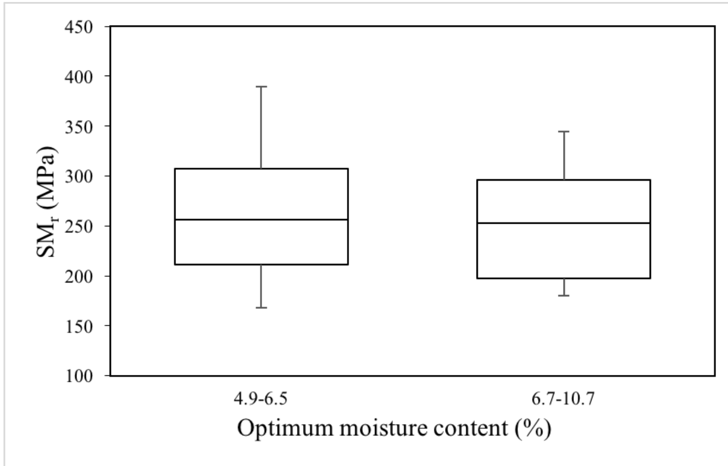

Figure 34. SM_R VERSUS OPTIMUM MOISTURE CONTENT OF 100% RAP (WHISKER PLOT)

D_{30} and D_{60} of RAPs were plotted against their corresponding SM_r values in Figures 35 and 37, respectively. According to Figure 36, RAPs with higher D_{30} tend to have higher SM_r. Similar trend was also observed in Figure 38. It shows that SM_r of RAPs are higher when they have higher D_{60} values.

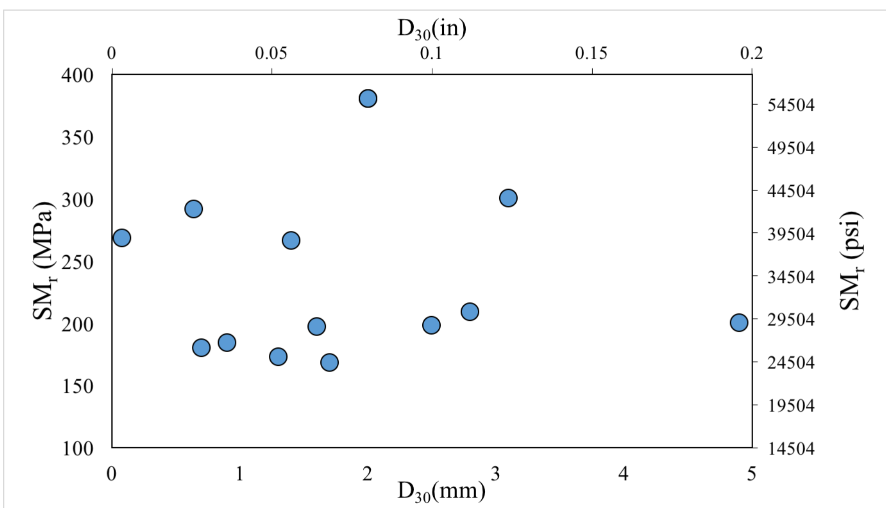

Figure 35. SM_R VERSUS D_{30} OF 100% RAP

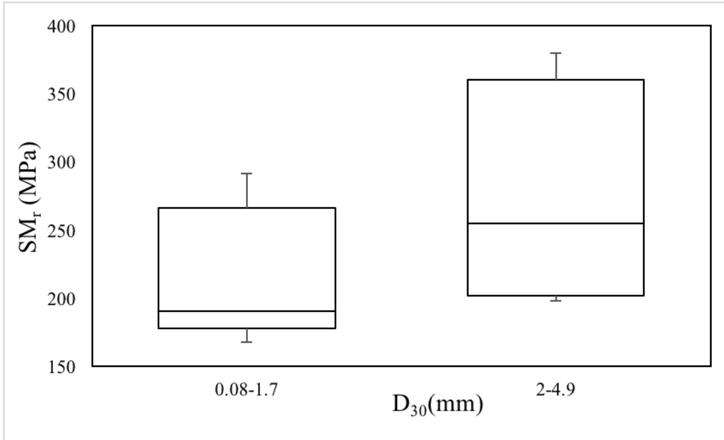

Figure 36. SM_R VERSUS D_{30} OF 100% RAP (WHISKER PLOT)

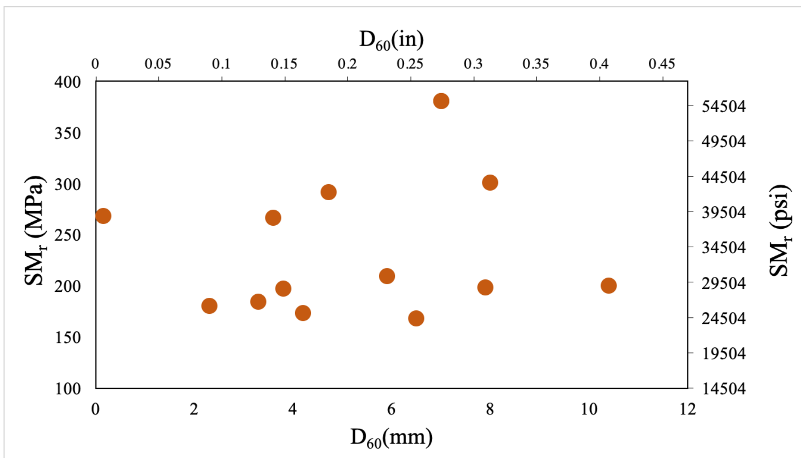

Figure 37. SM_R VERSUS D_{60} OF 100% RAP

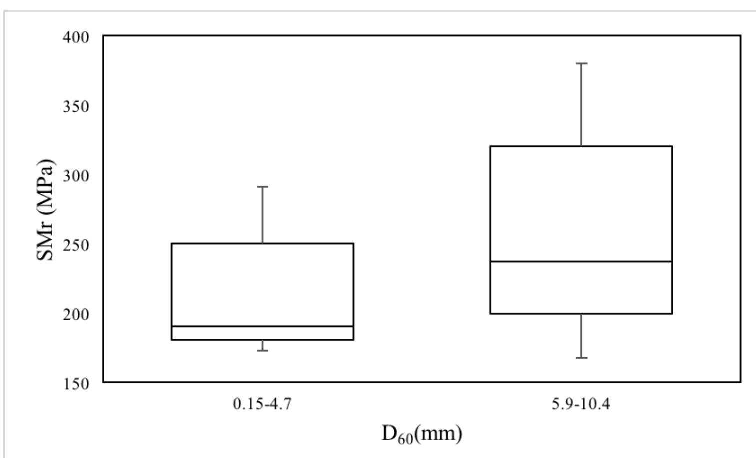

Figure 38. SM_R VERSUS D_{60} OF 100% RAP (WHISKER PLOT)

Coefficient of curvature (C_c) should be between 1 and 3 for well-graded gravel and sand. Figures 39 and 41 show variation of SM_r of RAPs with their corresponding C_c values. Based on the data collected, it was observed that SM_r of RAPs were higher when C_c of RAPs were lower than 1 ($1>C_c$) and higher than 3 ($C_c>3$). It means that poor graded RAP may have higher SM_r. Figure 42 shows that an increase in C_u yields some increase in SM_r).

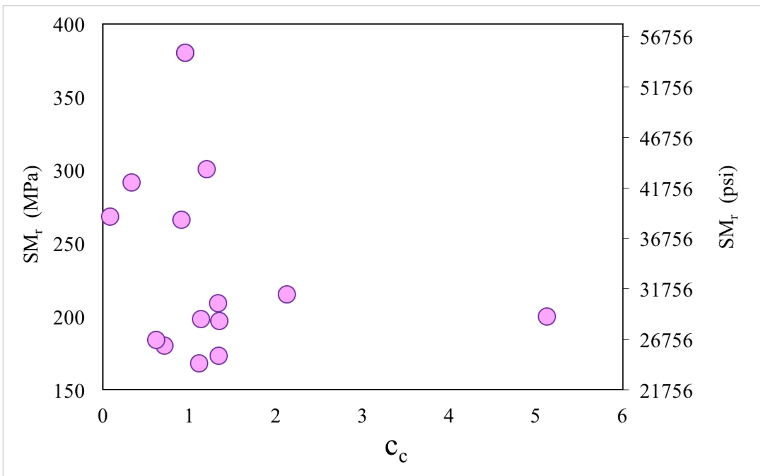

Figure 39. SM_R VERSUS C_C OF 100% RAP

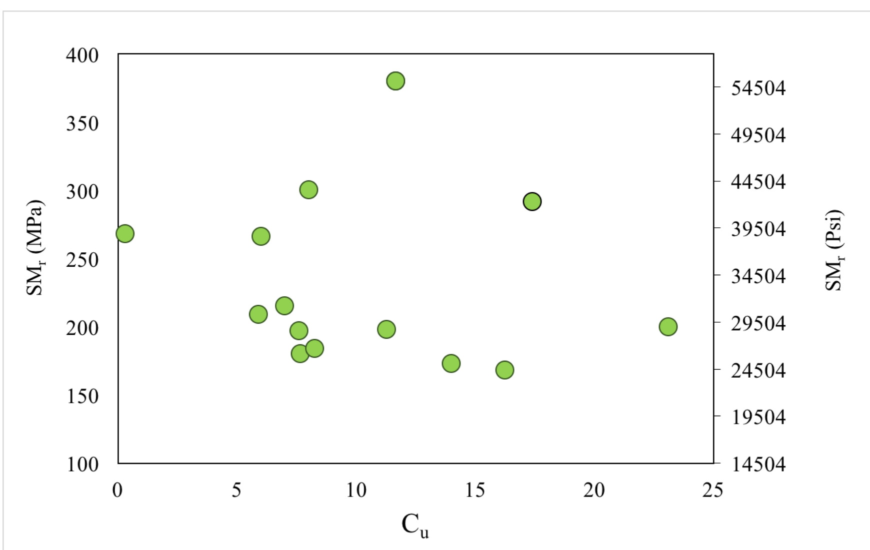

Figure 40. SM_R VERSUS C_U OF 100% RAP

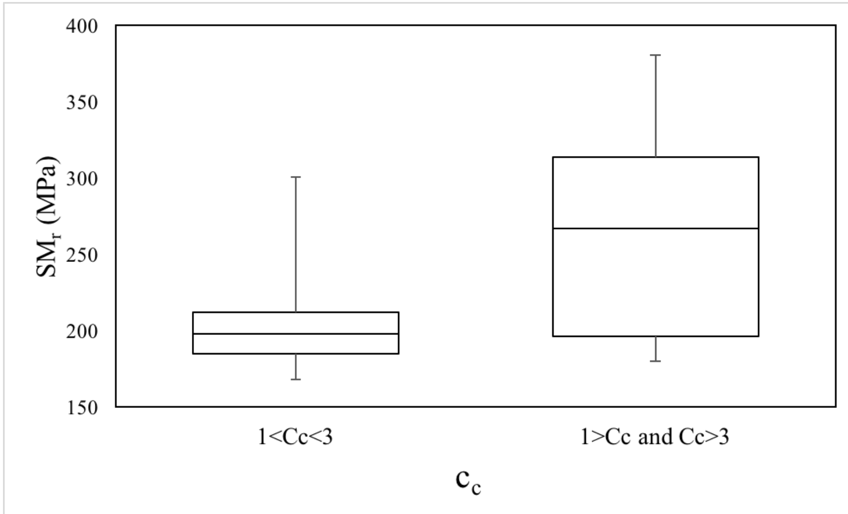

Figure 41. SM_R VERSUS C_C OF 100% RAP (WHISKER PLOT)

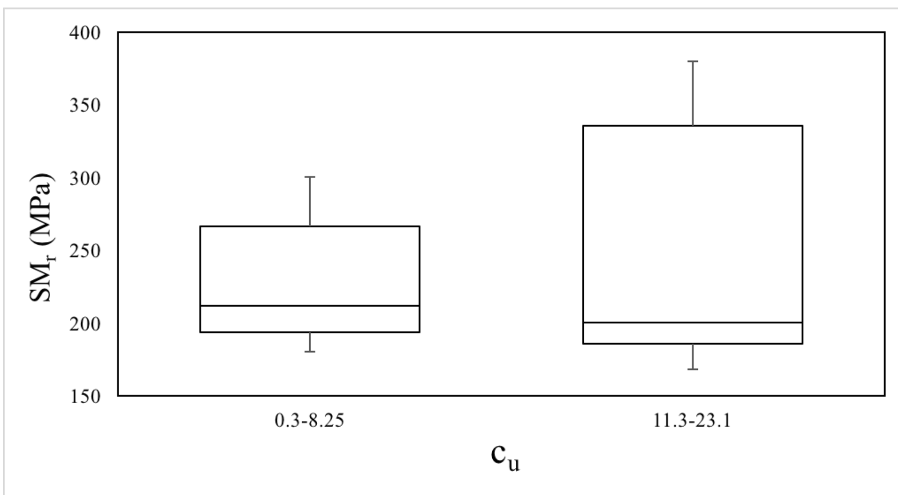

Figure 42. SM_R VERSUS C_U OF 100% RAP (WHISKER PLOT)

RCA materials compacted at higher MDU are likely to have higher SM_r values(Figures 43 and 44). Higher OMC results in a reduction in SM_r of RCAs. The OMC of RCA ranged from 6.1% to 11.9% while their corresponding SM_r changed between 370 MPa (53664 psi) and 124MPa (17984.7 psi) (Figures 45 and 46). Figures 47, 48, 49 and 50 shows that no correlations are observed between D_{30} and D_{60} characteristics of RCAs and the corresponding SM_r values.

The MDU of RCA ranged from 18.9 kN/m^3 (121.4 pcf) to 20.9 kN/m^3 (134.4 pcf) while their corresponding SM_r changed between 370 MPa (53664 psi) and 124MPa (17985psi) (Figures 43 and 44).

According to Figure 51 and 52, well-graded RCA yields higher SM_r than that of poorly-graded ones. The SM_r of RCA with C_c between 1 and 3 tended to be higher than the ones with C_c ($1>C_c$) smaller than 1 or higher than 3 ($C_c>3$). Higher C_u values in RCAs could result in higher SM_r (Figure 54).

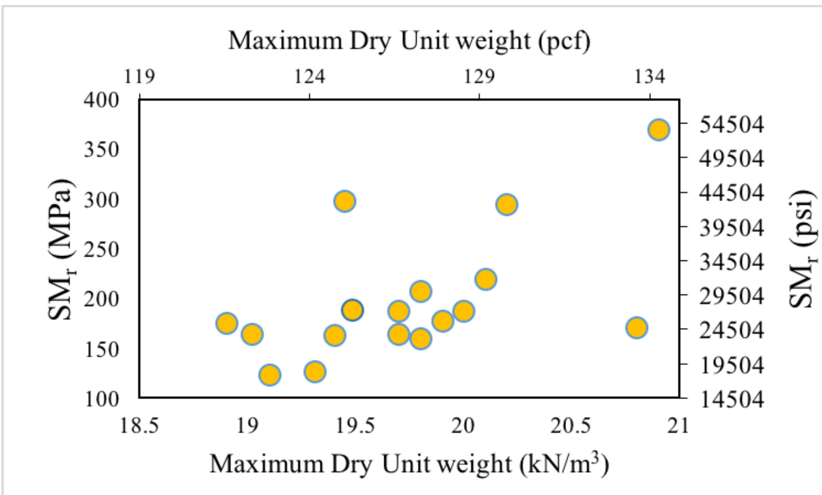

Figure 43. SM_R VERSUS MDU OF 100% RCA

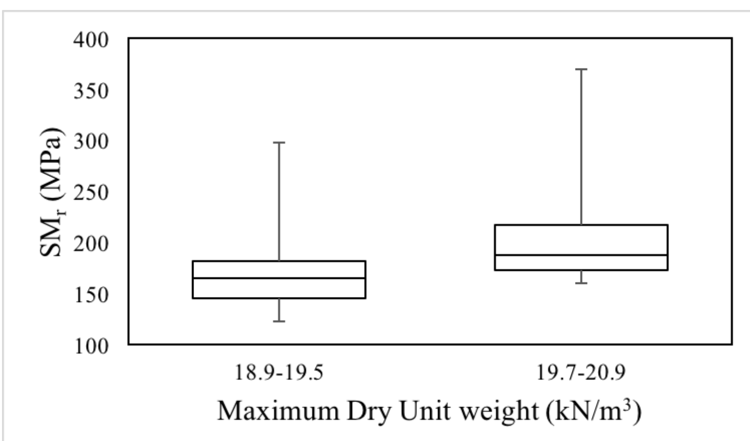

Figure 44. SM_R VERSUS MDU OF 100% RCA (WHISKE PLOT)

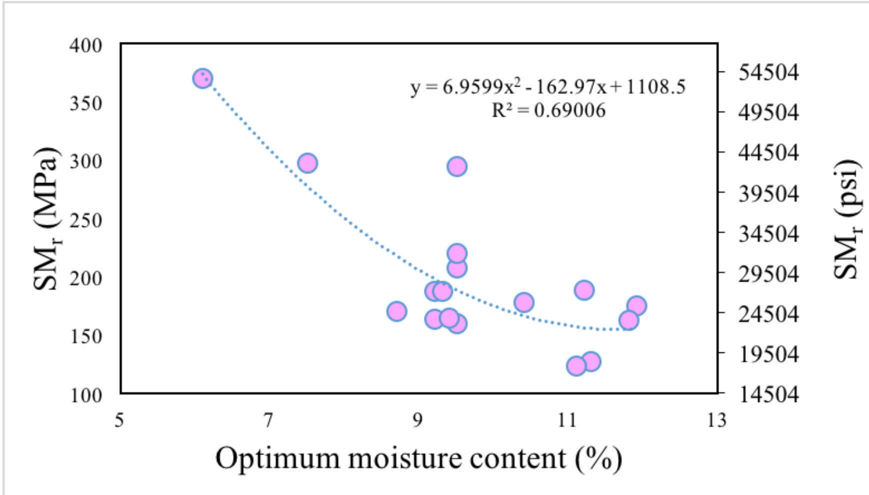

Figure 45. SM_R VERSUS OPTIMUM MOISTURE CONTENT OF 100% RCA

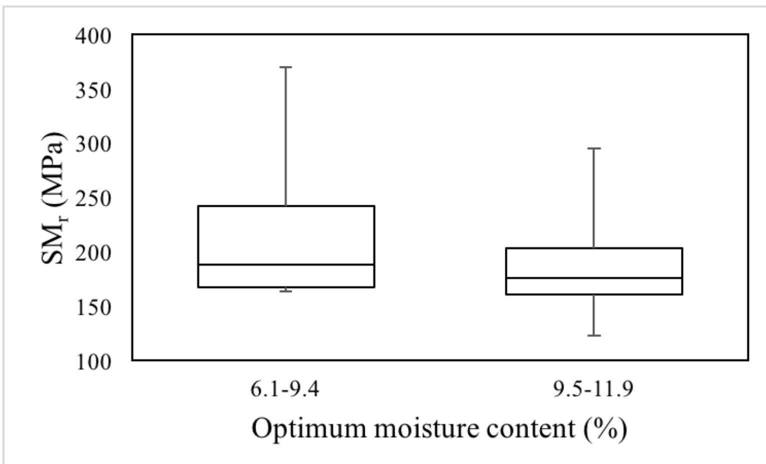

Figure 46. SM_R VERSUS OPTIMUM MOISTURE CONTENT OF 100% RCA (WHISKER PLOT)

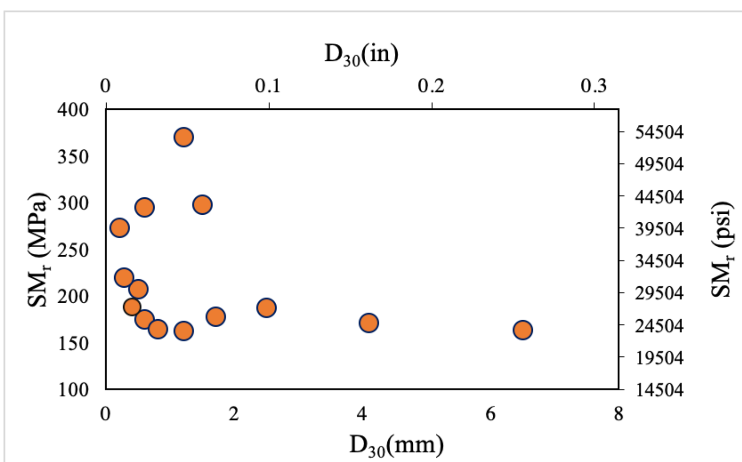

Figure 47. SM_R VERSUS D_{30} OF 100% RCA

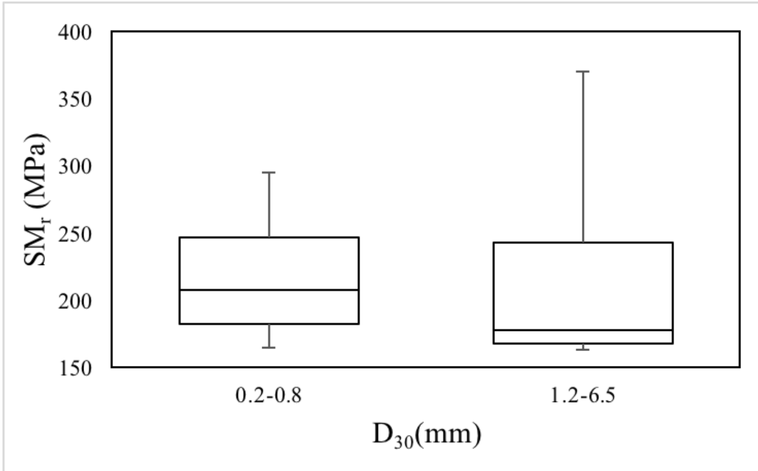

Figure 48. SM_R VERSUS D_{30} OF 100% RCA (WHISKER)

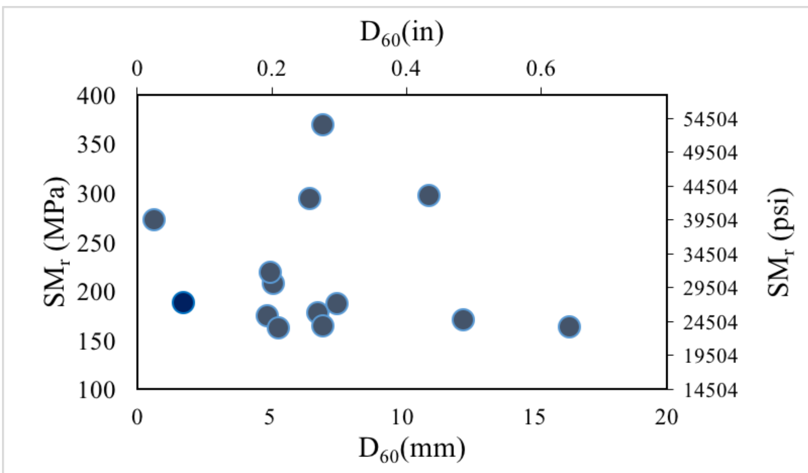

Figure 49. SM_R VERSUS D_{60} OF 100% RCA

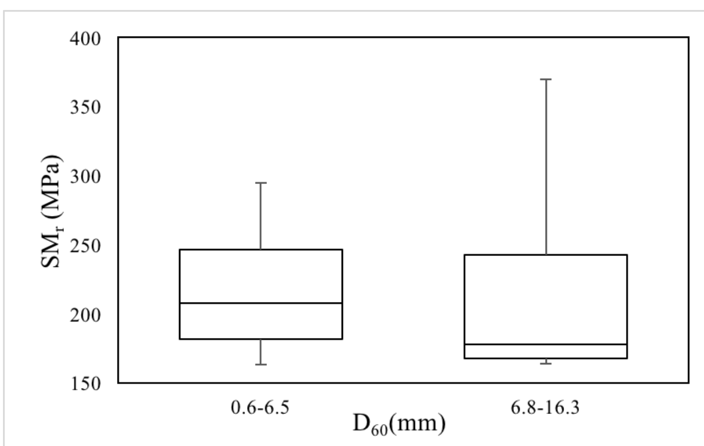

Figure 50. SM_R VERSUS D_{60} OF 100% RCA (WHISKER PLOT)

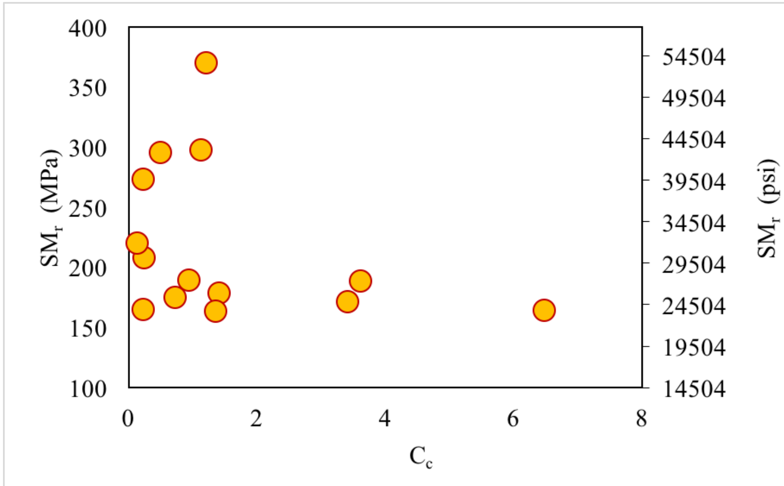

Figure 51. SM_R VERSUS C_C OF 100% RCA

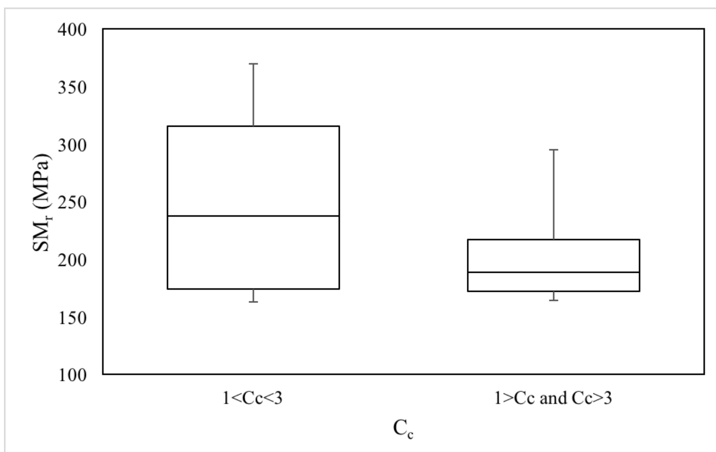

Figure 52. SM_R VERSUS C_C OF 100% RCA (WHISKER PLOT)

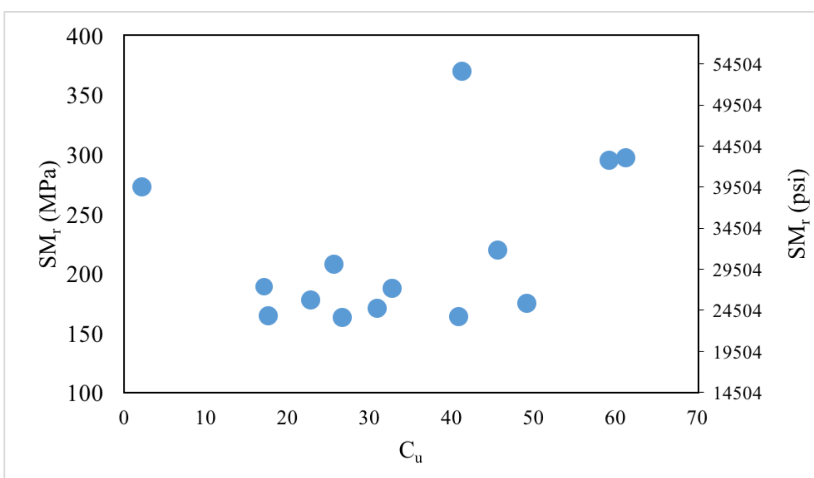

Figure 53. SM_R VERSUS C_U OF 100% RCA

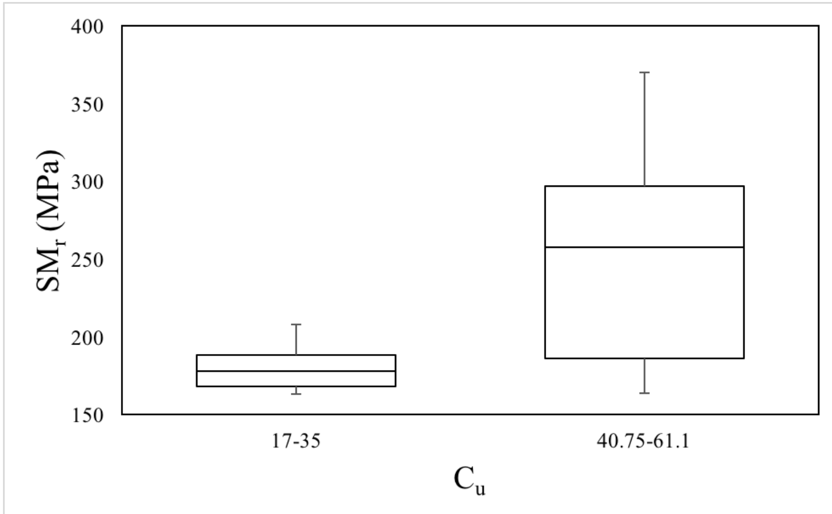

Figure 54. SM_R VERSUS C_U OF 100% RCA (WHISKER PLOT)

2.3.2. TEMPERATURE EFFECTS ON RAP AND RCA STIFFNESS

RAP is sensitive to temperature due to its asphalt content, which is a temperature-sensitive material. On the other hand, RCA and natural aggregates are not as sensitive to temperature changes as RAP (Wen et al. 2011; Soleimanbeigi et al. 2015). The SM_r of RAP-natural aggregate mixtures reduces as temperature increases (Soleimanbeigi et al. 2015). However, Soleimanbeigi and Edil (2015b) claimed that RAP could undergo a thermal preloading process. Thus, it would have higher stiffness at higher temperatures (Read and Whiteoak 2003, Wen et al. 2011). Thermal conditioning in this context means inducing elevated temperature to RAP during compaction process. The induced elevated temperature increases compressibility of RAP, thus reducing the void space in the material. When the temperature drops, the compacted RAP is expected to have higher stiffness and strength due to reduction in void space. Therefore, it is important to know the proper thermal conditioning for RAP during compaction when used as a base course (Soleimanbeigi and Edil 2015b).

Edil et al. (2012a) and Soleimanbeigi et al. (2015) conducted M_r tests on Colorado, Texas and New Jersey RAP at 7°, 23°, 35°, 50°C (44.6°, 73.4°, 95° and 122°F) and it was observed that SMr of all RAP materials decreased with an increase in temperature (Figure 55). Soleimanbeigi and Edil (2015b) showed that SM_R of RAP was also affected by the compaction temperature. SM_r of RAPs increased when RAPs were compacted at higher temperatures and then cooled and tested for resilient modulus as shown in Figure 55. Figure 56 shows the normalized SM_r of RAP versus different temperatures. SM_r of RAP at different temperature was divided by the SM_r of RAP at 23°C (73.4°F) of each RAP study. There was a decreasing trend with higher temperature except for the New Jersey and Colorado RAP from Soleimanbeigi et al. (2015) which had a slight increasing trend.

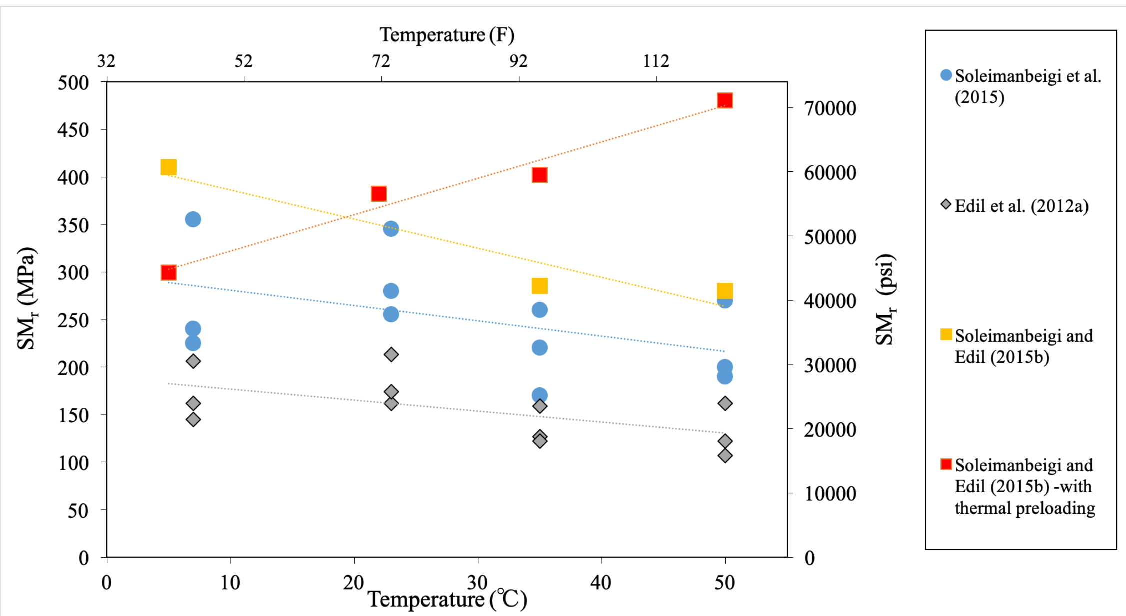

Figure 55. SM_R VERSUS TESTING TEMPERATURE OF 100% RAPS

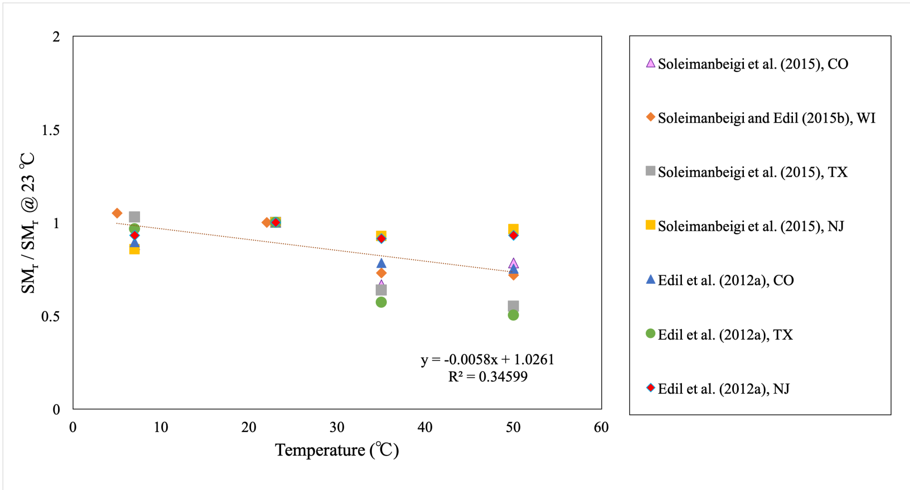

Figure 56. NORMALIZED SM_R VERSUS TEMPERATURE OF 100% RAPS

On the other hand, Figure 57 shows that RCA is not temperature-dependent. Moreover, Figure 58 shows the normalized SM_r–Temperature data that were tested at different temperatures. The SM_r of RCA at different temperature was divided by the SM_r of the same RCA at 23°C (73.4°F). The low R^2 value in Figure 58 reveals that there is no relationship between SM_r of RCA and temperature conditions during testing. Figure 59 also shows that SM_r of RCA is independent from the temperature as the whisker plots do not indicate any visible trend. However, the SM_r of RCA at 50°C (122°F) had generally lower SM_r than that of tested at 7°C (44.6°F).

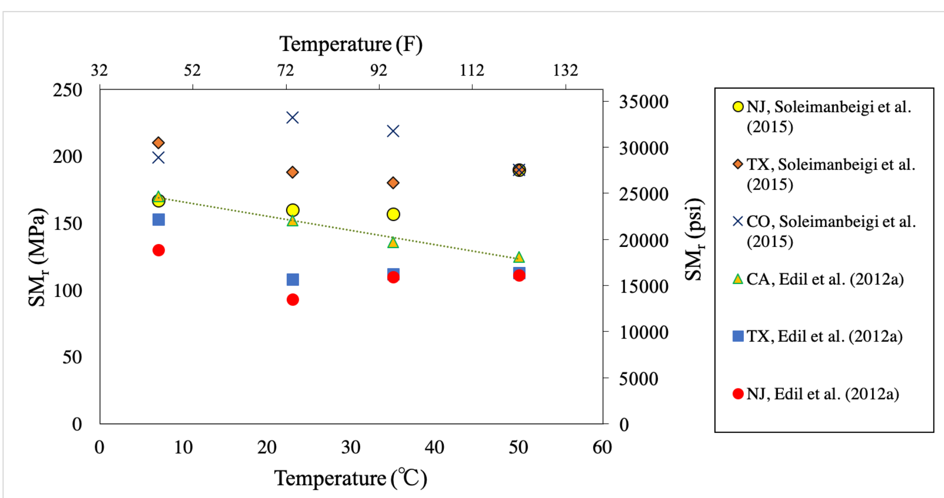

Figure 57. SM_R VERSUS TEMPERATURE OF 100% RCA

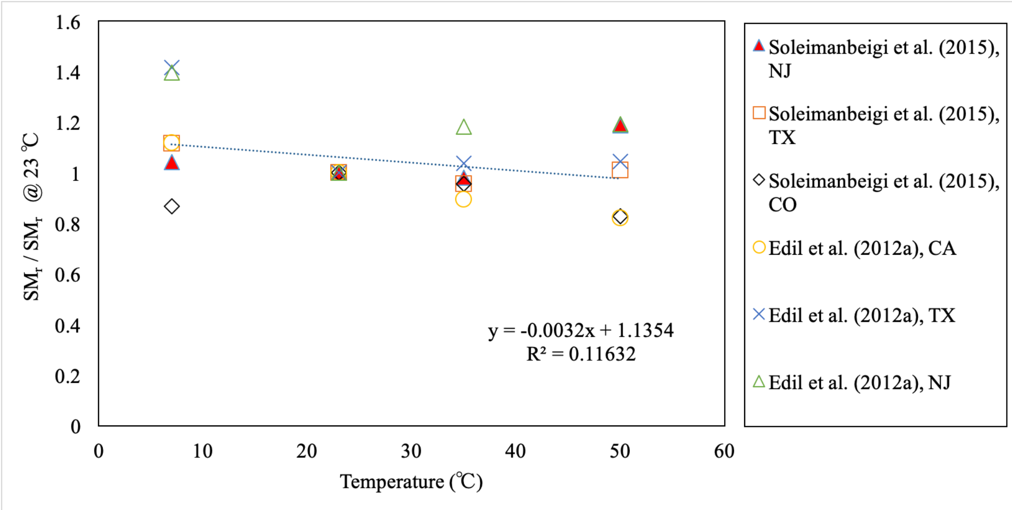

Figure 58. NORMALIZED SM_R VERSUS TEMPERATURE OF 100% RCA

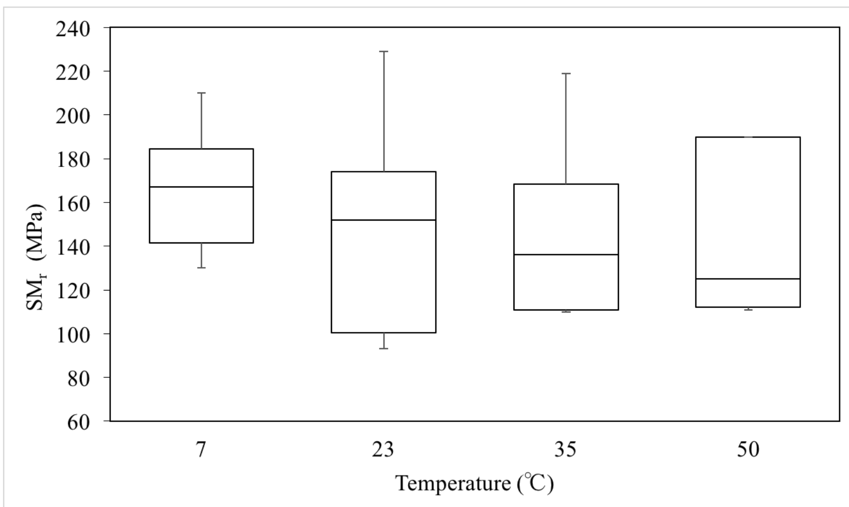

Figure 59. SM_R VERSUS TEMPERATURE OF 100% RCA (WHISKER PLOT)

According to Figure 60, an increase in moisture content of RAPs causes a decrease in SM_R. Attia and Abdelrahman (2010b) conducted research on Minnesota's 100% RAP and 50% RAP-natural aggregate mixture. In both cases a reduction in SM_r was observed as the OMC of the specimens increased. In addition, Edil et al. (2012a) tested the RAP samples from Texas and Ohio and concluded that as OMC increased, SM_r of these materials decreased. Resilient modulus test following the NCHRP 1-28A, procedure 1A was conducted with test samples at different moisture contents, as defined by testing matrix.

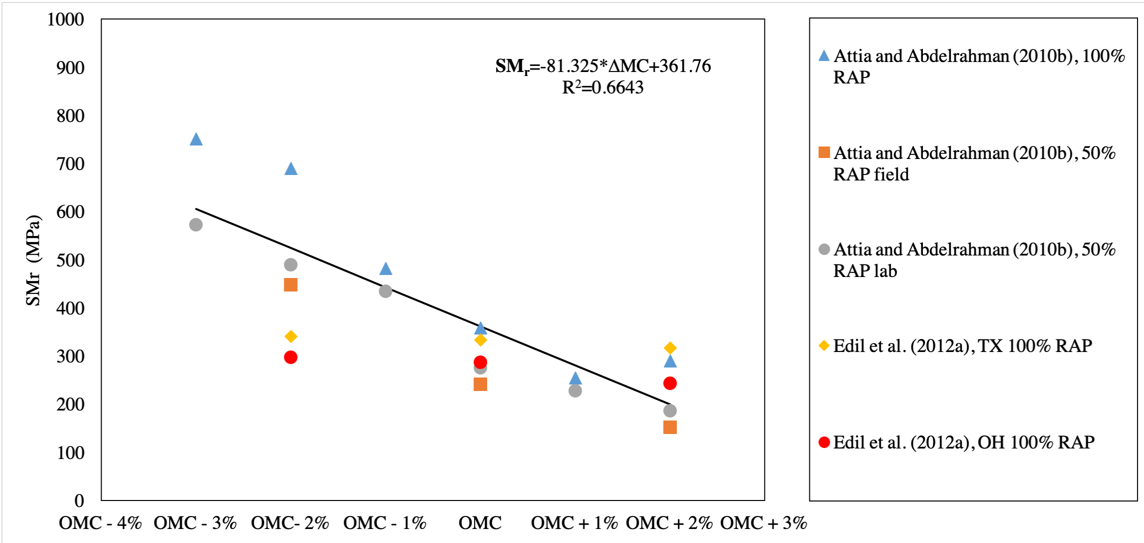

Figure 60. SM_R VERSUS OMC OF 100% RAPS

Notes: Samples were compacted at different moisture contents to achieve maximum dry density. OMC-2%= 2% dry of OMC; OMC+2%= 2% wet of OMC; ΔMC= Percent change in the OMC, for example, ΔMC=+2 in the equation for SM_r of OMC+2%

2.3.3. RESILIENT MODULUS BLENDS OF RAP AND RCA WITH NATURAL AGGREGATES

The use of a higher amount of RAP and RCA increased the stiffness of the RAP-natural aggregates and RCA-natural aggregates mixtures (Bennert et al. 2000). However, there were several exceptions. For instance, Bestgen et al. (2016) did not observe a consistent increase in the SM_r of the natural aggregates when they were mixed with RCA until RCA contents reached to 75% by weight in the mix design.

An increase in RAP content lead to an increase in SM_r (Figure 61). The SM_r of 100% RAP ranged between 170 MPa (24656.4 psi) (Kim et al. 2005) and 417 MPa (60480.7 psi) (Attia and Abdelrahman 2010a), while the SM_r of 100% RCA varied from 164 MPa (23786.2 psi) to 297 MPa (43076.2 psi).

Ullah and Tanyu (2019) conducted resilient modulus tests on 3 different RAP (RAP 1, RAP2, and RAP3) that were mixed with natural aggregates at 20%, 30%, 40%, 50%, 60% by weight. The SM_r of natural aggregate in this study was 141.1 MPa (20450 psi). The minerals in parent rock of all the RAPs were plagioclase and pyroxene. This study showed that RAP 1 with a high binder content (5.6-5.8%) resulted in the highest SM_r in RAP-natural aggregate mixture, while RAP 2 with low binder content (4.5-4.7%) had the lowest SM_r in the blend. Overall, this study claimed that the binder contents of RAP samples had a slight impact on the SM_R of blends when the blends had low RAP percentages and low binder contents. This paper indicated that, parent rock had the most impact on SM_r on the RAP and RAP-natural aggregate mixtures. For instance, RAP materials with plagioclase and pyroxene had a higher SMr than those made of quartz, muscovite, biotite, and amphibole.

Attia and Abdelrahman (2010a) tested Minnesota Class 5 (Class 5) and RAP-natural aggregate blends consisting of 50% RAP + 50% Class 5, 75% RAP + 25% Class 5, and 100% RAP material. Attia and Abdelrahman (2011) determined that RAP-natural aggregate blends were generally less sensitive to bulk stress and more sensitive to confining pressure. They showed that materials with 50% RAP would have SM_r of 265 MPa (38435 psi) while SM_r of 75% RAP blends was 210 MPa (30458 psi). 100% RAP had a SM_r of 400 MPa (58015.1 psi) which was higher than any blends.

Bulk stress in this study was calculated as it is defined in the previous studies. It is the sum of the σ_d and 3 times σ_3.

Alam et al. (2010) collected the RAP materials from millings of the 2001 rehabilitation project on Mn/ROAD Cell-26 constructed in 1994. This cell was located on the low volume roadway which had been subjected to 20,000 vehicles daily (Mulvaney and Worel 2002). These RAP materials were mixed with natural aggregates at the following rates by weight 30%, 50%, 70% and 100% and were subjected to M_r tests. This study determined that SM_r increased with an increase in RAP content from 154 MPa (22335.8 psi) at 30% RAP to 270 MPa (39160.2 psi) at 100% RAP.

Bradshaw et al. (2016) studied two different types of RAP and natural aggregate blends. The materials included cold recycled RAP blends that were prepared off site and RAP blends that were generated in situ during full depth reclamation (FDR). The SM_r of the cold recycled RAP blends (14– 39% RAP content) was between 120 MPa (17404.5 psi) and 502 MPa (72808.9 psi). The possible reason for slightly higher SM_r of these blends that those of RAP blends in the literature could be the differences in aggregate composition and/or particle shape. The SM_r of the FDR-RAP blends (57–71% RAP content) ranged from 171 MPa (24802 psi) to 578 MPa (83832 psi) which were higher than the SM_r of the cold recycled RAP blends. This could be due the higher RAP contents used in these blends. In addition, more shear softening and permanent strains were observed for the FDR-RAP mixtures as compared to the cold recycled RAP blends.

Kang et al. (2011) tested recycled materials and natural aggregates. RAP was collected from highway 61 in Minneapolis. Natural aggregates were collected from a pit south of Jordan, MN. RAP and RCA were mixed with natural aggregates at 25%, 50% and 100% by weight. SM_r of blends in this study increased with an increase in RAP content. However, generally the reported

SM_r was smaller than any other study ranging from 90 MPa (13053 psi) at 25% RAP blend to 192 MPa (27847 psi) at 100% RAP.

Abdelrahman and Noureldin (2014) conducted research on one source of RAP supplied by the Minnesota Department of Transportation (DOT) from a trunk highway. This RAP was blended with Class 5 base aggregates (Minnesota DOT) at 50%, 75%, and 100% RAP. According to the results, SM_r values changed from 289 MPa (41916 psi) at 50% to 262 MPa (38000 psi) at 75% RAP mixture. It was reported that the SM_r of 100% RAP was 330 MPa (47863 psi) which was the highest among all the blends.

Kim et al. (2005) obtained the reclaimed materials from County Road (CR) 3 in central Minnesota. An in situ blend, a mixture of 25%, 50% and 75% RAP and crushed aggregate, pure RAP and pure aggregate materials were taken separately during FDR. This was the only study that reported similar SM_r for different blends. 100% RAP had SM_r of 170 MPa (24656 psi).

Wu et al. (2012) obtained crushed aggregate (basalt) from POE Asphalt Paving, Inc. (Pullman, Wash.) and RAP from the Fairmount Road construction site in Pullman. To eliminate the effect of gradation, one single gradation was selected meeting the Washington State Department of Transportation (DOT) specifications for crushed surfacing base course material for all percentages of RAP used in the study. This study showed a constant value for blends from 20% to 60% around 200 MPa (29008 psi) whereas 80% RAP blend had a SM_r of 550 MPa (79771 psi) which was considered an outlier.

Bennert and Maher (2005), conducted M_r tests on RAP and RCA blended with dense graded aggregate base course (DGABC) material from the Central region of New Jersey since the quarried material did not exist naturally in southern New Jersey. The ratios of blends for testing were 25%, 50% and 75% along with the 100% RAP or RCA and 100% DGABC. It reported an increasing trend with higher RAP contents from 25% (202 MPa- 29225 psi) to 50% (234 MPa- 33895 psi) and from 75% (214 MPa- 31009 psi) to 100% RAP (268 MPa- 38870 psi). SM_r of 50% RAP was observed to be higher than that of blend mixed with 75% RAP.

Hasan et al. (2018) collected the subgrade soils and the RAP from the interstate 40 (I-40) construction site at the mile post of 141 near Albuquerque, New Mexico and the RAP material was supplied from the stockpile. They reported SM_r of 175MPa (25382 psi) at 25% RAP and SM_r of 290 MPa (42061 psi) at 75% RAP.

Bennert et al. (2000) conducted research on 25%, 50%, 75% and 100% RAP blended with dense graded aggregate base course (DGABC) in the state of New Jersey. This study reported an increasing trend in SM_r as RAP content increased. It also showed that the 25% RAP blend had 187 MPa (27122 psi) stiffness which was 300 MPa (43555 psi) for 100% RAP.

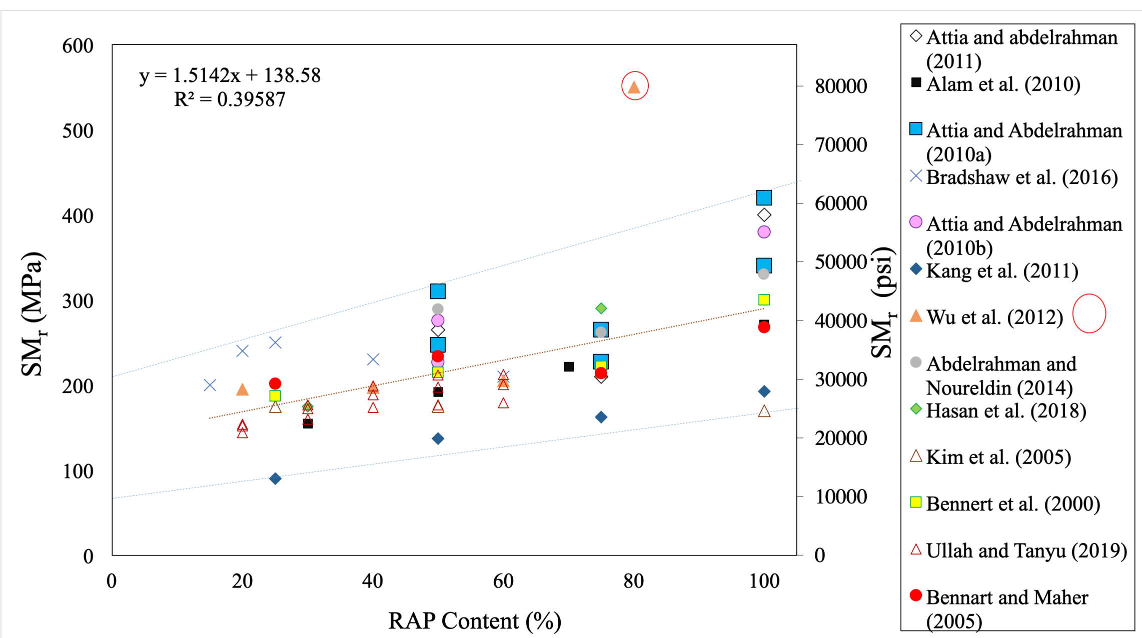

Figure 61. SM_R VERSUS RAP CONTENT

Figure 62 shows the normalized SM_r of each RAP blend to the SM_r of the corresponding RAP alone and it is observed that SM_r of RAP mixtures increase with an increase in RAP contents. Figure 63 also confirms this trend more clearly.

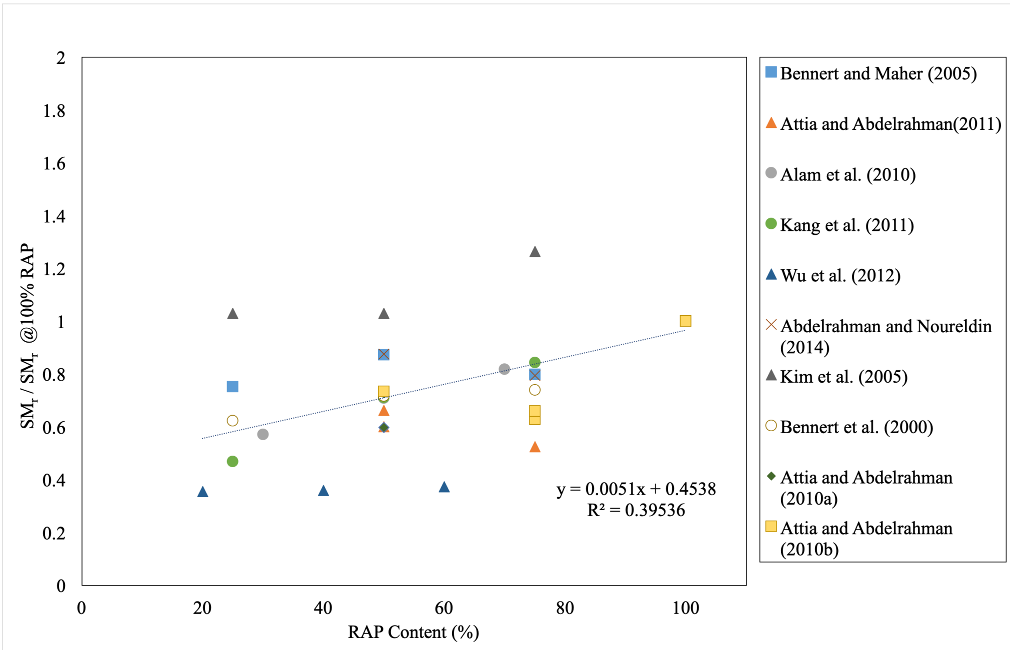

Figure 62. NORMALIZED SM_R VERSUS RAP CONTENT

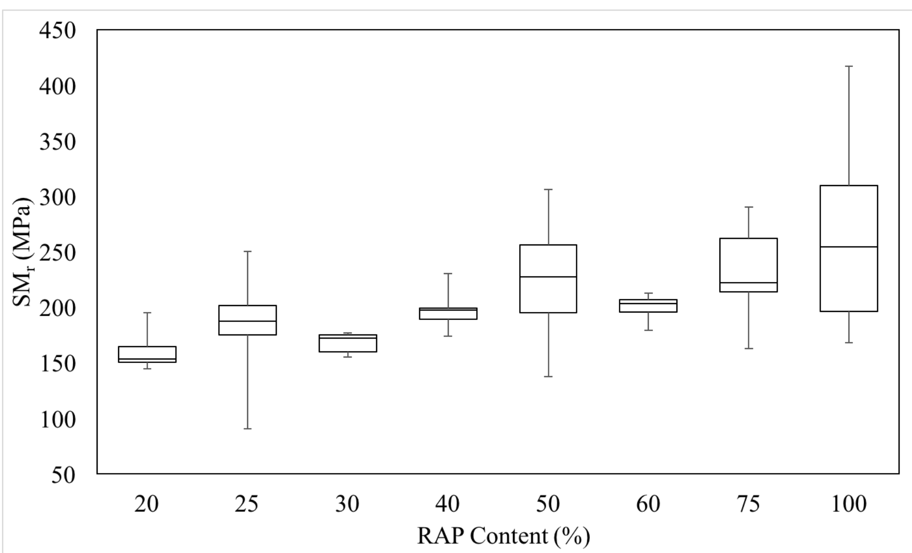

Figure 63. SM_R VERSUS RAP CONTENT (WHISKER PLOT)

Furthermore, Figure 64 shows that RCA content and SM_r relationship is not very straight forward as the one observed between the RAP content and SM_r. According to Bestgen et al. (2016), the presence of higher CaO content in RCA materials led to higher SM_r values than the natural aggregates. CaO initiates the cementitious reaction in the aggregate matrix which can improve the mechanical properties of RCA materials. However, database contained 18 different RCA-natural aggregate mixtures and it was observed that each of these mixtures had different trends. Bestgen et al. (2016) tested four different natural aggregate materials with two different RCAs. RCAs were mixed with natural aggregates at 25%, 50%, 75% and 100% ratios by weight. Overall, the mixtures presented a slight increase in SM_r when 25% RCA was increased to 50%. All of the mixtures showed an increase trend in SM_r with RCA content going from 25% to 75% with a few exceptions. RCA had higher SM_r than the RCA-natural aggregate mixtures and the natural aggregates alone. Nevertheless, overall trend was that 100% RCA had a higher summary resilient modulus than any other mixtures and natural aggregates (Figure 65). Figure 66 shows the normalized SM_r values of each blend of RCA (Normalized SM_r means the ratio of the SM_r value of each RCA blend to the corresponding SM_r value of 100% RCA).

Figure 67 indicates that no specific trends are observed between the SM_r of 50% and 75% RCA-natural aggregate blends. The number of available data for SM_r of RCA was lower than those available for RAP. For 100% RCA and 50% RCA, 23 and 11 data points were available, respectively. The lowest number of available data was 10 for 25% and 75% RCA blends. Overall, Figure 67 also confirms that 100% RCA has a higher resilient modulus than any blends and natural aggregates.

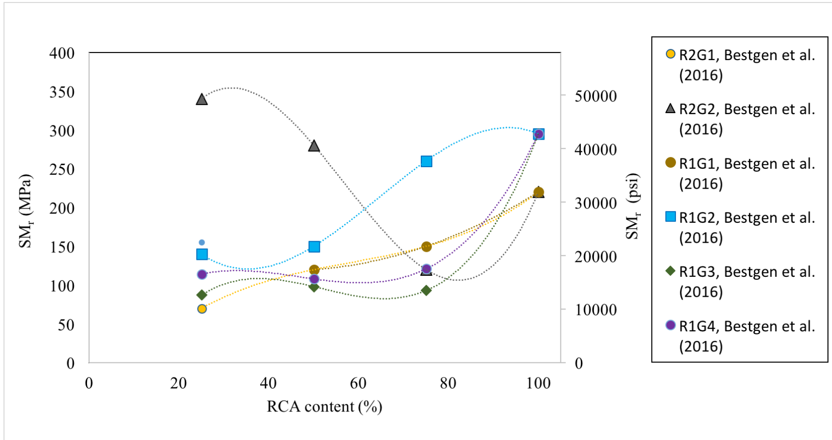

Figure 64. SM_R VERSUS RCA CONTENT

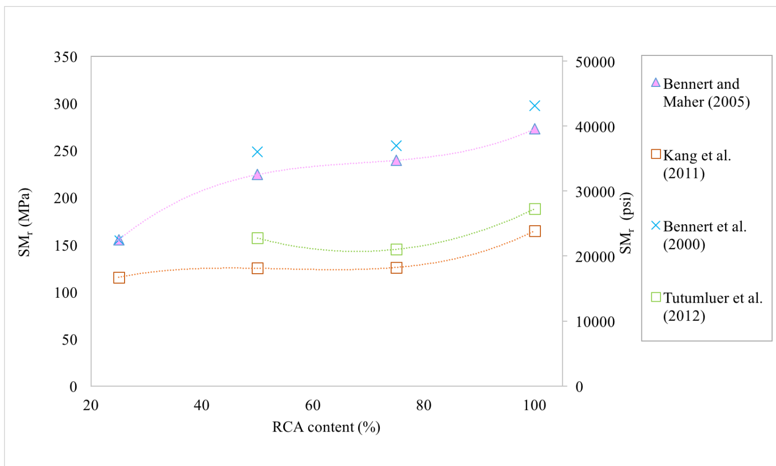

Figure 65. SM_R VERSUS RCA CONTENT

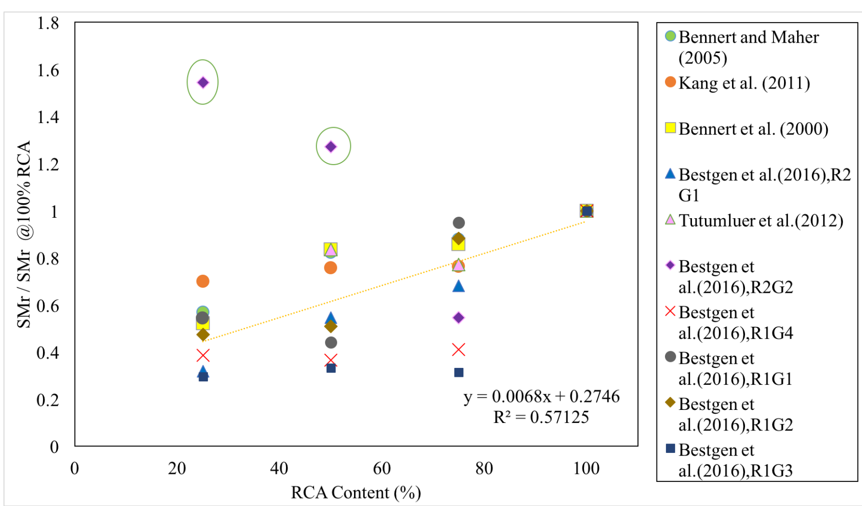

Figure 66. NORMALIZED SM_R AND RCA CONTENT

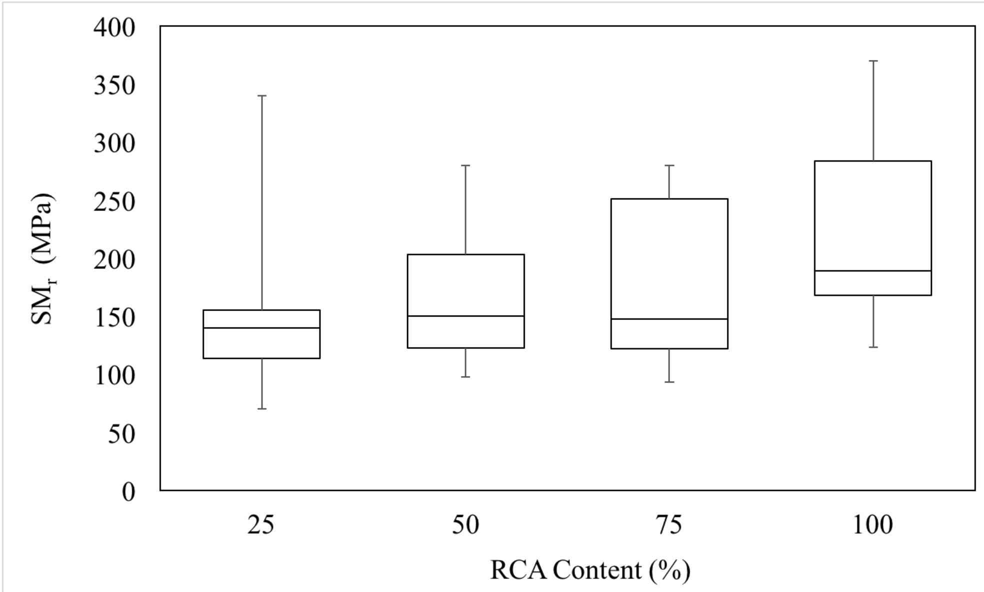

Figure 67. SM_R VERSUS RCA CONTENT (WHISKER PLOT)

2.4. PERMANENT DEFORMATION

The permanent deformation failure is attributed to the vertical compressive strains of geomaterials under repeated loading conditions which lead to failure mechanisms in the flexible pavement systems (Bennert et al. 2000, Thompson and Smith 1990). Permanent deformation is determined by performing a cyclic triaxial test in which the confining pressure, deviatoric stress and the number of cycles are predetermined.

Increasing the number of loading cycles leads to an increase in the permanent deformation of pavement foundation materials regardless of the aggregate type. A relatively higher permanent deformations were observed with an increase in the RAP content of the RAP-natural aggregate mixtures (Kim and Labuz 2007, Thakur and Han 2015). On the other hand, an increase in the RCA content of the RCA-natural aggregate mixtures caused a relatively lower permanent deformation (Bennert et al. 2000). In general, RCA showed the lowest permanent deformation among RCA, RAP and natural aggregate while RAP showed the highest permanent deformation (Bennert et al. 2000; Edil et al. 2012a). Having the highest permanent deformation in RAP may have been the progressive breakdown of its asphalt binder (Bennert et al. 2000). Moreover, viscous creep behavior of asphalt material could be one of the reasons for the high plastic deformation of RAP (Edil et al. 2012a). The permanent deformation of 100% RAPs ranged from 1.05% (Attia 2010) to 5.63% (Bennert and Maher 2005) while these values were between 0.1% (Bestgen et al. 2016) and 0.83% for RCAs (Edil et al. 2012a).

Different trends have been observed between RAP, RCA, and natural aggregate due to their different gradation characteristics (e.g. fines contents). Virgin aggregate could show lower permanent deformation than RCA due to its lower fines content (Bestgen et al. 2016). Fines content can significantly affect the permanent deformation while it has no considerable effect on the resilient moduli of aggregates. A relatively higher fines content leads to a higher permanent deformation of aggregates (Mishra and Tutumluer 2012). Moreover, repetitive load may break hydrated cement particles thus reduce the angularity of RCA which finally leads to a higher permanent deformation in RCA than those observed for natural aggregates (Bestgen et al. 2016).

According to Thompson and Smith (1990), permanent deformation plays an important role in determining the pavement performance. Bennert et al. (2000), Attia (2010), Garg and Thompson (1996), Kim and Labuz (2007) and Wen and Wu (2011) showed that permanent deformation of RAP-natural aggregate mixtures increased with an increase in the RAP content (Figure 68).

Particle sizes of the RCAs and RAPs are important since the presence of larger aggregates in the material matrix tend to lead to higher strength and the resistance against deformation (Gray 1962, Kazmee et al. 2016). Moreover, the thicker base layers result in the lower permanent deformation due to the improved stress distribution (Cetin et al. 2010, Schaertl 2010).

It was also observed that temperature is very crucial for RAPs permanent deformation performances. An increase in temperature yields an increase in the permanent deformation or RAP materials because of the temperature-sensitivity of RAP (Edil et al. 2012a; Soleimanbeigi et al. 2015). On the other hand, the temperature has little to no effect on the permanent deformation performances of RCA and natural aggregates (Edil et al. 2012a).

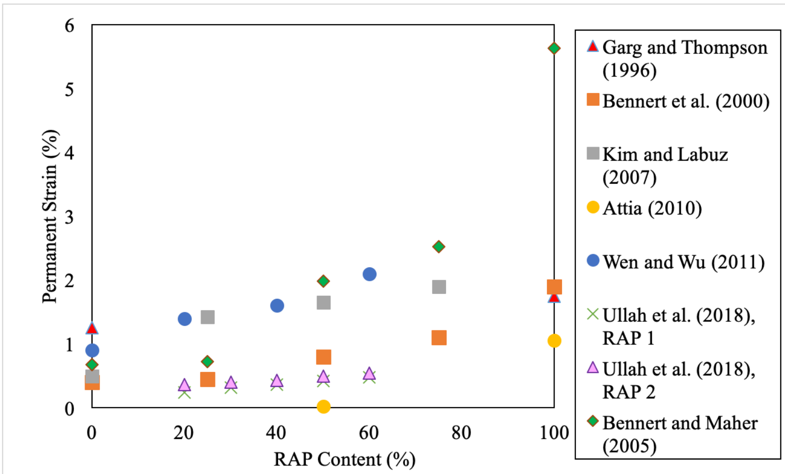

Figure 68. PERMANENT STRAIN VERSUS RAP CONTENT

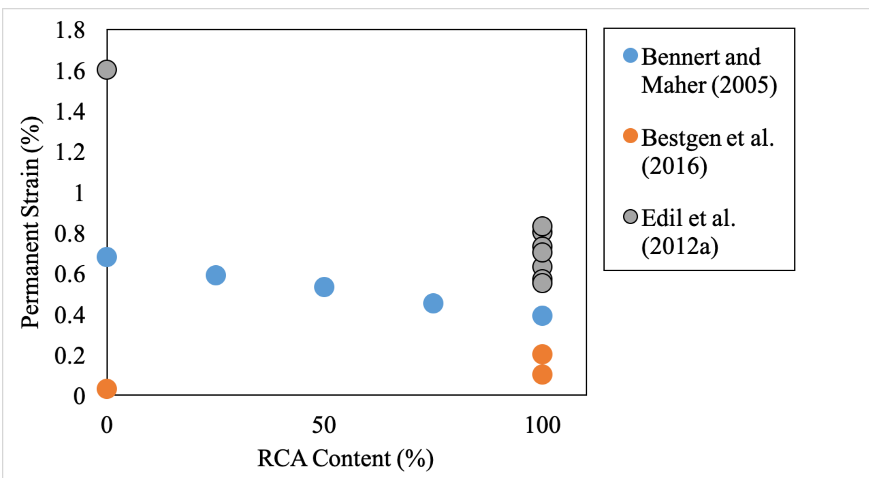

Figure 69. PERMANENT STRAIN VERSUS RCA CONTENT

2.5. SHEAR STRENGTH

Shear strength is the maximum shear stress that a soil can sustain. Attia (2010) identified shear strength as an important property for unbound materials when used under a thin HMA layer that is subjected to high shear stresses. Shear strength is a function of normal or confining stress, friction angle, and cohesion for a particular material. Cosentino et al. (2003), Bennert and Maher (2005), Attia (2010), Bejarano (2001), Garg and Thompson (1996), and Kim and Labuz (2007) evaluated shear strength parameters (friction angle and cohesion) of the RAP-blended natural aggregate materials. Results of this study showed that the friction angle (φ) and the cohesion (c) of 100 % RAP specimen varied from 44° to 52º and 0 kPa (0 psi) to 131 kPa (19 psi), respectively. The large variation in the cohesion of RAP may resulted from the variation in the asphalt binder content of the RAP used by different researchers. No correlations or trends were observed between the φ and the c parameters of RAP-natural aggregate mixtures and the corresponding RAP content (Figures 70 and 71). There were less available data regarding shear strength of RCAs. The "c" of RCAs ranged from 24.13 kPa (3.5 psi) to 191 kPa (27.7 psi) and the φ of RCAs ranged from 19° to 52.7° (Figures 72 and 73).

The typical ranges for angle of friction of granular soil materials for GW, GP, SW, SP are 33-40°, 32-44°, 33-43°, 30-39°, respectively (Swiss Standard SN 670 010b and Koloski et al. 1989) while they are between 35° and 51° for muddy shale and stone Mt. granite rocks (Goodman 1980).

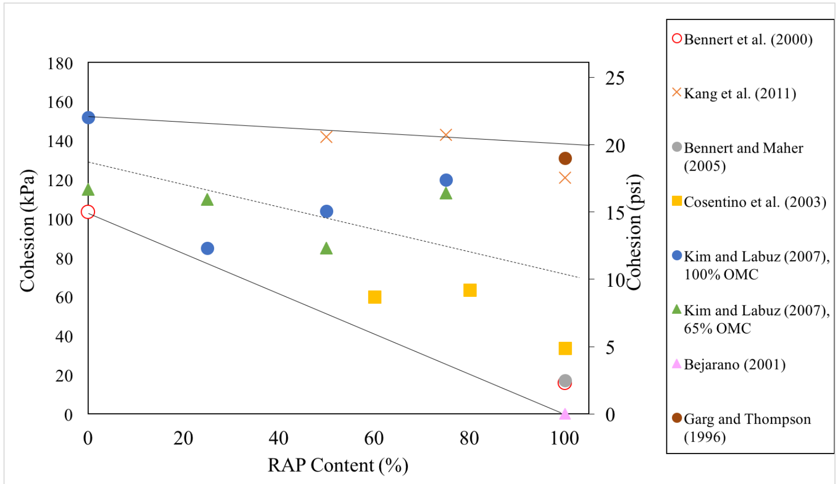

Figure 70. COHESION VERSUS RAP CONTENT

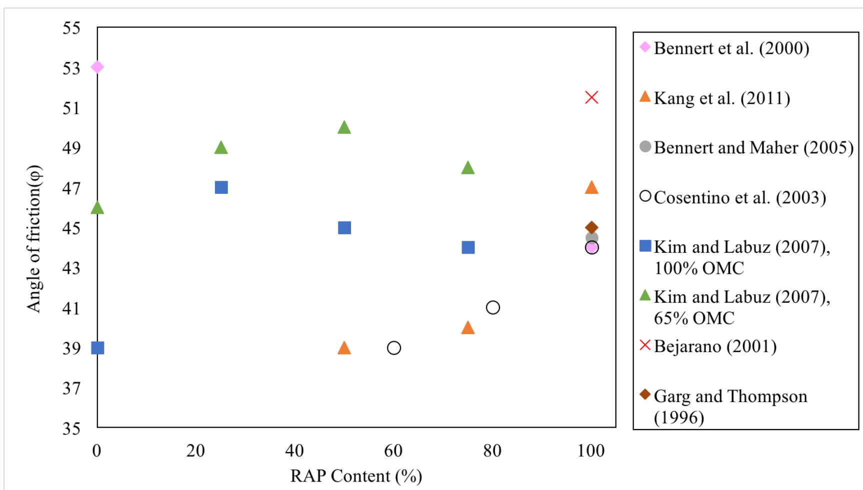

Figure 71. FRICTION ANGLE VERSUS RAP CONTENT

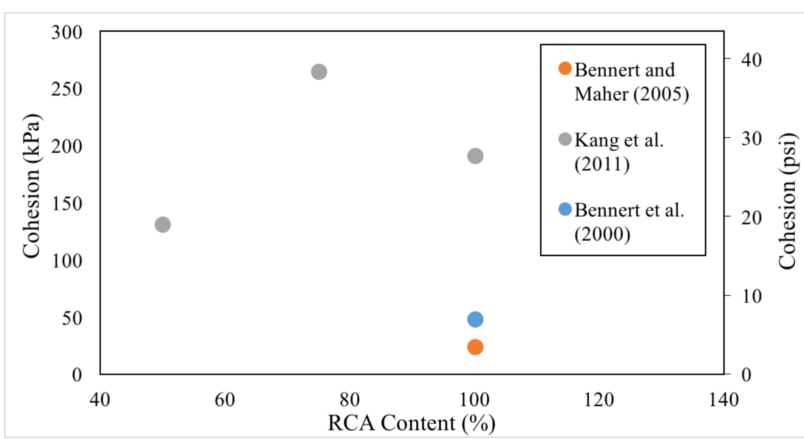

Figure 72. COHESION VERSUS RCA CONTENT

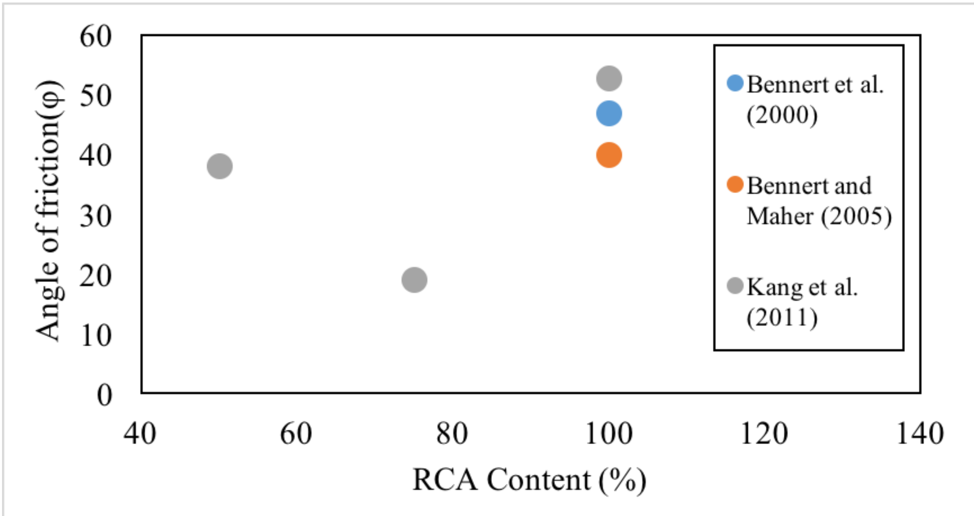

Figure 73. FRICTION ANGLE VERSUS RCA CONTENT

CHAPTER 3. DESIGN METHODS

One of the most important steps for constructing high-quality and long-lasting pavement systems is the determination of surface, aggregate base, and subbase layers' thickness. While there are methods and assumptions for using natural aggregates as an aggregate base or subbase layer, designing pavement systems with recycled (RAP and RCA) can be challenging (Edil 2011). The engineering properties of RAP and RCA should be well understood as they play an important role in design. AASHTO (1993) and Mechanistic-Empirical Pavement Design Guide (MEPDG) are the most commonly used design methods for flexible and rigid pavements (Edil 2011). The focus of this research is to improve the inputs for MEPDG design method which is now called Pavement ME design.

3.1. PAVEMENT MECHANISTIC EMPIRICAL DESIGN

AASHTO (1993) and pavement mechanistic-empirical (PMED) are the two most commonly used design methods for flexible and rigid pavements (Edil 2011). One of the main reasons to develop PMED method was the lack of using the impact of climate and traffic impacts in the most widely used empirical AASHTO design methods. In the PMED approach, pavement performance is evaluated based on mechanistically determined critical stresses, strains, temperatures, and moisture levels that are in turn the inputs to empirical prediction models for specific pavement distresses such as rutting, fatigue cracking, thermal cracking, and roughness for flexible pavements and cracking, faulting, and roughness for rigid pavements. Accurate characterization of the traffic, climate, and material input parameters is therefore important to ensure that the theoretical computation of pavement stresses, strains, temperatures, and moisture levels are accurate at the critical locations within the system (Schwartz et al. 2015).

Plastic deformation is taken into account in the Pavement ME, which is a mechanistic-empirical approach in contrast to the AASHTO (1993) design method which is an empirical approach. There are several parameters such as the modulus values of layers, climate zone, traffic conditions, the designed service life of the pavement, and failure criteria to be considered to create the most suitable design. Resilient modulus values should be obtained from laboratory or field tests for conventional and recycled aggregates. Finally making iterations for specific materials along with other related conditions leads to determining the design thicknesses (Edil 2011). For performance evaluation of pavement systems, required parameters for the analysis can be obtained for RAP, RCA, natural aggregates, and the RAP-natural aggregate and RCA-natural aggregate mixtures.

CHAPTER 4. SELECTED PRACTICES OF STATE DOTS

The materials which do not meet the specifications, which state DOTs have established, cannot be used due to high failure risk (NCHRP-838). As more DOTs understand the importance of RAPs and RCAs, they tend to develope guidelines for RAP and RCA usage in pavements as they can be more economical and readily available. This literature review illustrates the practical aspects of the use of RAP and RCA in pavement design by different state DOTs and how each guideline slightly differs from each other. Caltrans, MnDOT, MoDOT, WiDOT allow RAP and RCA to be used as a base course material in pavements if they meet the requirements for gradation and quality characteristics. MDOT and IDOT only allow RCA in base applications eventhough IDOT recently starts considering the use of RAP in such applications as well. More detailed information about DOT spesifications is discussed below.

4.1. CALIFORNIA DOT

In California, RAP and RCA base applications are allowed up to 100% since 2006 but before then their usage was limited to 50%. Recycled aggregates must meet the grading and quality specifications stated for natural aggregate in the Caltrans Standard Specifications (CalRecycle 2014).

Aggregate base and subbase applications of the recycled aggregates are discussed in Sections 25 and 26 of the Caltrans Standard Specifications published in 2015 (Caltrans 2015). Clean broken stone, crushed gravel, natural rough surfaced gravel, sand, and reclaimed processed Portland cement concrete (PCC) can be used as subbases and aggregate bases. The subbase aggregates must meet the gradation ranges of Class 1, Class 2, or Class 3 as shown in Table 4 (section 25 of Caltrans 2015). In addition, the aggregates must have adequate quality characteristics presented in Table 5 depending on its class. The aggregates used as base materials should meet the requirements of gradations and quality characteristics of Class 2 or Class 3 aggregates shown in Tables 6, 7, 8, and 9.

Contract compliance is a larger range than the Operating Range and is used to adjust fort not having to shut the job down or pay a fine. If the gradation is outside of the Operating Range but within the Contract Compliance requirements, this material can continue to be used for the remainder of the day. It should be noted, that even if within the Contract Compliance requirements, changes still need to be made by the next day to ensure the material is within Operating Range, or construction will be stopped until requirements are met. If a test results indicate the material is still outside the Contract Compliance requirements, Caltrans generally has the right to ask for removal or a payment deduction.

Table 4. AGGREGATE GRADATION FOR SUBBASE APPLICATIONS (CALTRANS 2015)

Sieve size	Percentage passing					
	Class 1		Class 2		Class 3	
	Operating range	Contract compliance	Operating range	Contract compliance	Operating range	Contract compliance
3"	100	100	100	100	100	100
2 1/2"	90–100	87–100	90–100	87–100	90–100	87–100
No. 4	35–70	30–75	40–90	35–95	50–100	45–100
No. 200	0–20	0–23	0–25	0–29	0–30	0–34

Table 5. AGGREGATE QUALITY CHARACTERISTICS FOR SUBBASE APPLICATIONS (CALTRANS 2015)

Quality characteristic	Requirement					
	Class 1		Class 2		Class 3	
	Operating range	Contract compliance	Operating range	Contract compliance	Operating range	Contract compliance
Sand equivalent, (min)	21	18	21	18	21	18
Resistance, (R-value, min)	--	60	--	50	--	40

Table 6. CLASS 2 AGGREGATE GRADATION FOR AGGREGATE BASE APPLICATIONS (CALTRANS 2015)

Sieve size	Percentage passing			
	1-1/2 inch maximum		3/4 inch maximum	
	Operating range	Contract compliance	Operating range	Contract compliance
2"	100	100	--	--
1-1/2"	90–100	87–100	--	--
1"	--	--	100	100
3/4"	50–85	45–90	90–100	87–100
No. 4	25–45	20–50	35–60	30–65
No. 30	10–25	6–29	10–30	5–35
No. 200	2–9	0–12	2–9	0–12

Table 7. CLASS 2 AGGREGATE QUALITY CHARACTERISTICS FOR AGGREGATE BASE APPLICATIONS (CALTRANS 2015)

Quality characteristic	Requirement	
	Operating range	Contract compliance
Resistance (R-value, min)	--	78
Sand equivalent (min)	25	22
Durability index (min)	--	35

Table 8. CLASS 3 AGGREGATE GRADATION FOR AGGREGATE BASE APPLICATIONS (CALTRANS 2015)

Sieve size	Percentage passing			
	1-1/2 inch maximum		3/4 inch maximum	
	Operating range	Contract compliance	Operating range	Contract compliance
2"	100	100	--	--
1-1/2"	90–100	87–100	--	--
1"	--	--	100	100
3/4"	50–90	45–95	90–100	87–100
No. 4	25–60	20–65	40–70	35–75
No. 30	10–35	6–39	12–40	7–45
No. 200	3–15	0–19	3–15	0–19

Table 9. CLASS 3 AGGREGATE QUALITY CHARACTERISTICS FOR AGGREGATE BASE APPLICATIONS (CALTRANS 2015)

Quality characteristic	Requirement	
	Operating range	Contract compliance
Resistance (R-value) (min)	--	50
Sand equivalent (min)	21	18

4.2. ILLINOIS DOT

Sections 311 and 351 of the IDOT Standard Specifications for Road and Bridge Construction published in 2016 allows crushed concrete produced from Portland cement concrete, crushed gravel and crushed stone for the aggregate base and subbase courses (IDOT 2016). According to section 1004, 20 different aggregate classes are defined for different applications (Table 10).

Crushed concrete must have adequate gradation requirements of CA6 or CA10 aggregates for aggregate base applications (Table 11) (IDOT 2016).

As stated in Section 1004, coarse aggregate quality control specifications are established by Illinois DOT (Table 12). Crushed concrete should be evaluated as class D for checking its quality in terms of Illinois Test Procedure (ITP) 96 (LA abrasion test) and must be evaluated as a class C for Illinois Test Procedure (ITP) 203 which is used for the determination of deleterious particles in coarse aggregate. According to the Los Angeles (LA) abrasion limit, abrasion loss should be less than 45% and instead of the given limit for deleterious materials (2%), the content of other deleterious should be limited to 7% with no more than 5% RAP (IDOT - Bureau of Materials and Physical Research). The California bearing ratio should be 80 for the aggregate base applications of typical materials but there is no requirement for crushed concrete (IDOT 2016).

Per Section 303, IDOT allows RAP usage in constructing an aggregate subgrade improvement which can contain coarse aggregate or reclaimed asphalt pavement. Crushed RAP , from either full depth or single lift removal, may be mechanically blended with aggregate gradations CS 01, CS 02 and RR 01 but the total product must contain RAP at 40% or less. The size of RAP particles must be less than 4 inches and well graded. RAP with 100% passing 1-1/2 inch sieve and being well graded, may be used as a capping aggregate on the top 3 inches when aggregate gradations CS 01, CS 02 or RR 01 are used in lower lifts. The RAP used for aggregate subgrade improvement shall be selected according to the current Bureau of Materials and Physical Research Policy Memorandum, “Reclaimed asphalt pavement (RAP) for aggregate applications.

Table 10. GRADATION RANGES OF DIFFERENT AGGREGATES (IDOT 2016)

Grad No.	COARSE AGGREGATE GRADATIONS												
	Sieve Size and Percent Passing												
	3 in.	2 1/2 in.	2 in.	1 1/2 in.	1 in.	3/4 in.	1/2 in.	3/8 in.	No. 4	No. 8	No. 16	No. 50	No. 200 [1/]
	75 mm	63 mm	50 mm	37.5 mm	25 mm	19 mm	12.5 mm	9.5 mm	4.75 mm	2.36 mm	1.18 mm	300 µm	75 µm [1/]
CA 1	100	95±5	60±15	15±15	3±3								
CA 2		100	95±5		75±15		50±15		30±10		20±15		8±4
CA 3		100	93±7	55±20	8±8		3±3						
CA 4			100	95±5	85±10		60±15		40±10		20±15		8±4
CA 5				97±3 [2/]	40±25		5±5		3±3				
CA 6				100	95±5		75±15		43±13		25±15		8±4
CA 7				100	95±5		45±15 [7/]		5±5				
CA 8				100	97±3	85±10	55±10		10±5		3±3 [3/]		
CA 9				100	97±3		60±15		30±15		10±10		6±6
CA 10					100	95±5	80±15		50±10		30±15		9±4
CA 11					100	92±8	45±15 [4/7/]		6±6		3±3 [3/ 5/]		
CA 12						100	95±5	85±10	60±10		35±10		9±4
CA 13						100	97±3	80±10	30±15		3±3 [3/]		
CA 14							90±10 [6/]	45±20	3±3				
CA 15							100	75±15	7±7		2±2		
CA 16							100	97±3	30±15		2±2 [3/]		
CA 17	100								65±20		45±20	20±10	10±5
CA 18	100				95±5				75±25		55±25	10±10	2±2
CA 19	100				95±5				60±15		40±15	20±10	10±5
CA 20							100	92±8	20±10	5±5	3±3		

Table 11. TYPICAL AGGREGATES FOR VARIOUS APPLICATIONS (IDOT 2016)

Use	Gradation
Granular Embankment, Special	CA 6 or CA 10 [1/]
Granular Subbase:	
Subbase Granular Material, Ty. A	CA 6 or CA 10 [2/]
Subbase Granular Material, Ty. B	CA 6, CA 10, CA 12, or CA 19 [2/]
Subbase Granular Material, Ty. C	CA 7, CA 11, or CA 5 & CA 7 [3/]
Stabilized Subbase	CA 6 or CA 10 [4/]
Aggregate Base Course	CA 6 or CA 10 [2/]
Aggregate Surface Course:	
Type A	CA 6 or CA 10 [1/]
Type B	CA 6, CA 9, or CA 10 [5/]
Aggregate Shoulders	CA 6 or CA 10 [2/]

Table 12. COARSE AGGREGATE QUALITY CONTROL SPECIFICATIONS (IDOT 2016)

COARSE AGGREGATE QUALITY				
QUALITY TEST	CLASS			
	A	B	C	D
Na_2SO_4 Soundness 5 Cycle, ITP 104 [1/], % Loss max.	15	15	20	25 [2/]
Los Angeles Abrasion, ITP 96, % Loss max.	40 [3/]	40 [4/]	40 [5/]	45
Minus No. 200 (75 µm) Sieve Material, ITP 11	1.0 [6/]	---	2.5 [7/]	---
Deleterious Materials [10/]				
Shale, % max.	1.0	2.0	4.0 [8/]	---
Clay Lumps, % max.	0.25	0.5	0.5 [8/]	---
Coal & Lignite, % max.	0.25	---	---	---
Soft & Unsound Fragments, % max.	4.0	6.0	8.0 [8/]	---
Other Deleterious, % max.	4.0 [9/]	2.0	2.0 [8/]	---
Total Deleterious, % max.	5.0	6.0	10.0 [8/]	---

4.3. MINNESOTA DOT

RAP and RCA are both allowed in Section 2211 of the MnDOT Standard Specifications for construction published in 2018 to be used as aggregate base course (MnDOT 2018). In Section 3138, aggregates are classified based on their quality characteristics and they should meet the quality requirements of one of those classes (Table 13). In addition, RAP and RCA should meet the quality requirements, which are the same for all aggregate classes (Table 14) (MnDOT 2018). When the RAP content is more than 10% of the blend by volume, the gradation of RAP and aggregate blend must meet the specified gradation for the aggregate class (McGarrah 2007). RAP and natural aggregate must be blended at the crushing site, not at the job site with stockpiles aggregates (McGarrah 2007).

Almost all concrete pavements in Minnesota are recycled as dense-graded base aggregate material (Gonzalez and Moo-Young 2004). Fine-grained (< #4 sieve) RCA particles must be removed to avoid the drainage issues. Moreover, open-graded RCA can be mixed with natural aggregates to reduce the heavy metal leaching (Snyder 1995, as cited in Gonzalez and Moo-Young 2004).

Per Section 3138, depending on the project, the blends of natural aggregates and recycled aggregates with less than 25% recycled aggregates used as a pavement aggregate base material should meet the gradations specified for different aggregate classes (Table 15) (MnDOT 2018). If 25% or more up to 75% recycled aggregates are used in the blends, the mixture should meet the gradation criteria provided in Table 16. In addition, if 75% or more recycled concrete is used, the mixture should meet the gradation criteria shown in Table 17 (MnDOT 2018).

Table 13. QUALITY REQUIREMENTS FOR VIRGIN AGGREGATES (MNDOT 2018)

Requirement	**Class**			
	1 and 2	**3 and 4**	**5 and 5Q**	**6**
Max Shale, if No. 200 ≤ 7% by mass	NA	10.0%	10.0%	7.0%
Max Shale, if No. 200 > 7% by mass	NA	7.0%	7.0%	7.0%
Minimum Crushing Requirements *	NA	NA	10%	15%
Maximum Los Angeles Rattler (LAR) loss from carbonate quarry rock	40%	40%	40%	35%
Maximum Insoluble residue for the portion of quarried carbonate aggregates passing the No. 200 sieve	10%	10%	10%	10%
* Material crushed from quarries is considered crushed material.				

Table 14. QUALITY REQUIREMENTS FOR RECYCLED AGGREGATES (MNDOT 2018)

Requirement	**Classes 1, 3, 4, 5, 5Q, and 6**
Maximum Bitumen Content of Composite	3.5%
Maximum Masonry block %	10%
Maximum percentage of glass *	10%
Maximum size of glass *	¾ in
Crushing (Class 1, 5, 5Q, and 6) ‖	10% for Class 1 & 5 †, 60% for Class 5Q †, and 15% for Class 6 †
Maximum amount of Brick	1.0% #
Maximum amount of other objectionable materials including but not limited to: wood, plant matter, plastic, plaster, and fabric	0.3% #
* Glass must meet certification requirements on the Grading and Base website. Combine glass with other aggregates during the crushing operation. † If material ≥ 20% RAP and/or Concrete, Class 5 crushing requirement is met. † If material ≥ 60% RAP and/or Concrete, Class 5Q crushing requirement is met. † If material ≥ 30% RAP and/or Concrete, Class 6 crushing requirement is met. ‖ Material crushed from quarries is considered crushed material. # The Contractor/Supplier may not knowingly allow brick and other objectionable material and must employ a QC process to screen it out, before it becomes incorporated into the final product.	

Table 15. GRADATION OF BASE AGGREGATE CONTAINING LESS THAN 25% RECYCLED AGGREGATES (MNDOT 2018)

Sieve Size	Class 1 (Surfacing £)	Class 2 (Surfacing β)	Class 3 (Subbase)	Class 4 (Subbase)	Class 5 (Base)	Class 5Q (Base)	Class 6 (Base)
2 in	—	—	100	100	—	100	—
1½ in	—	—	—	—	100	—	100
1 in	—	—	—	—	—	65 - 95	—
¾ in	100	100	—	—	70 - 100	45 - 85	70 - 100
⅜ in	65 - 95	65 - 90	—	—	45 - 90	35 - 70	45 - 85
No. 4	40 - 85	35 - 70	35 - 100	35 - 100	35 - 80	15 - 45	35 - 70
No. 10	25 - 70	25 - 45	20 - 100	20 - 100	20 - 65	10 - 30	20 - 55
No. 40	10 - 45	12 - 35	5 - 50	5 - 35	10 - 35	5 - 25	10 - 30
No. 200	8.0 - 15.0	5.0 - 16.0	5.0 - 10.0	4.0 - 10.0	3.0 - 10.0	0.0 - 10.0	3.0 - 7.0

***** If product contains recycled aggregate, add letters in parentheses for each aggregate blend designating the type of recycled products included in the mixture.
(B) = Bituminous, (C) = Concrete, (G) = Glass
(BC) = Bituminous and Concrete, (BG) = Bituminous and Glass
(CG) = Concrete and Glass, (BCG) = Bituminous, Concrete, and Glass
£ Recycled concrete when used for surfacing is only allowed for shoulders
β Class 2 must be composed of 100% crushed quarry rock per 3138.2.B.2.

Table 16. GRADATION OF BASE AGGREGATE CONTAINING 25% OR MORE RECYCLED AGGREGATES and 75% OR LESS RECYCLED CONCRETE (MNDOT 2018)

Sieve Size	Class 1 (Surfacing £)	Class 3 (Subbase)	Class 4 (Subbase)	Class 5 (Base)	Class 5Q (Base)	Class 6 (Base)
2 in	—	100	100	—	100	—
1½ in	—	—	—	100	—	100
1 in	—	—	—		65 - 95	
¾ in	100	—	—	70 - 100	45 - 85	70 - 100
⅜ in	65 - 95	—	—	45 - 90	35 - 70	45 - 85
No. 4	40 - 85	35 - 100	35 - 100	35 - 80	15 - 45	35 - 70
No. 10	25 - 70	20 - 100	20 - 100	20 - 65	10 - 30	20 - 55
No. 40	10 - 45 † 5 - 45	5 - 50	5 - 35	10 - 35	5 - 25	10 - 30
No. 200	5.0 - 15.0 † 0 - 15.0	0 - 10.0	0 - 10.0	0 - 10.0	0 - 10.0	0 - 7.0

* Add letters in parentheses for each aggregate blend designating the type of recycled products included in the mixture.
(B) = Bituminous, (C) = Concrete, (G) = Glass
(BC) = Bituminous and Concrete, (BG) = Bituminous and Glass
(CG) = Concrete and Glass, (BCG) = Bituminous, Concrete, and Glass
† Note: For Class 1, if the bitumen content is ≥ 1.5%, the gradation requirement is modified to 5 – 45% for the #40 sieve and 0 – 15.0% for the #200 sieve.
£ Recycled concrete is only allowed for shoulders

Table 17. GRADATION OF BASE AGGREGATE CONTAINING MORE THAN 75% RECYCLED CONCRETE (MNDOT 2018)

Sieve Size	Class 1 (Surfacing £)	Class 3 (Subbase)	Class 4 (Subbase)	Class 5 (Base)	Class 5Q (Base)	Class 6 (Base)
2 in	—	100	100	100	100	100
1½ in	—	—	—	—		—
1 in	—	—	—	—	65 - 95	—
¾ in	100	—	—	45 - 100	45 - 85	45 - 100
⅜ in	65 - 95	—	—	25 - 90	35 - 70	25 - 85
No. 4	40 - 85	35 - 100	35 - 100	15 - 65	15 - 45	15 - 65
No. 10	25 - 70	20 - 100	20 - 100	10 - 45	10 - 30	10 - 45
No. 40	10 - 45	0 - 20	0 - 20	0 - 20	0 - 20	0 - 20
No. 200	5.0 - 15.0	0 - 6.0	0 - 6.0	0 - 6.0	0 - 6.0	0 - 6.0

* Add letters in parentheses for each aggregate blend designating the type of recycled products included in the mixture.
(B) = Bituminous, (C) = Concrete, (G) = Glass, (BC) = Bituminous and Concrete,
(BG) = Bituminous and Glass, (CG) = Concrete and Glass, (BCG) = Bituminous, Concrete, and Glass
£ Recycled concrete is only allowed for shoulders

4.4. MISSOURI DOT

The use of reclaimed asphalt and concrete aggregates as base aggregates are allowed in Sections 304 and 1007 of the MoDOT Standard Specifications for Highway Construction published in 2018 if they meet the gradation specifications of Type 1 (Table 18), Type 5 (Table 19), and Type 7 (Table 20) (MoDOT 2018). Section 1007 limits deleterious materials of Type 1, Type 5, and Type 7 aggregates to be less than 15%. Deleterious materials should be distributed uniformly along with sand, silt, and clay contents. Plasticity index (PI) of particles passing No. 40 sieve should not be more than 6 (MoDOT 2018). In addition to Types 1, 5, and 7 aggregates, durable stones containing no more than 10% (by weight) of earth, sand, shale, and non-durable rock are allowed for aggregate base applications according to Section 303. The maximum size depends on the layer thickness. For example, the maximum size should be about 12 and 9 inches for 18-inch and 12-inch rock base respectively (MoDOT 2018).

Table 18. GRADATION CRITERIA OF TYPE 1 AGGREGATE (MODOT 2018)

Sieve	Percent by Weight
Passing 1-inch	100
Passing 1/2-inch	60-90
Passing No. 4	35-60
Passing No. 30	10-35

Table 19. GRADATION CRITERIA OF TYPE 5 AGGREGATE (MODOT 2018)

Sieve	Percent by Weight
Passing 1-inch	100
Passing 1/2-inch	60-90
Passing No. 4	35-60
Passing No. 30	10-35
Passing No. 200	0-15

Table 20. GRADATION CRITERIA OF TYPE 7 AGGREGATE (MODOT 2018)

Sieve	Percent by Weight
Passing 1 1/2-inch	100
Passing 1-inch	70-100
Passing No. 8	15-50
Passing No. 200	0-12

4.5. WISCONSIN DOT

Aggregates, breaker run, crushed gravel, crushed stone, pit run, reclaimed asphalt, and crushed concrete can be used for different aggregate base applications according to Section 301 of the WisDOT Standard Specifications published in 2018 (Table 21). Reclaimed asphalt is only suitable for dense 1 ¼-inch aggregate base type while crushed concrete is suitable for dense ¾-inch, dense 1 1/4 -inch, and dense 3-inch aggregate base types (WisDOT 2018). Base course materials cannot contain any deleterious materials such as shale, soft or porous rock fragments, coal, and organic particles.

Per section 301, reclaimed asphalt aggregates should contain at least 75% of reclaimed asphaltic pavement or surfacing. Crushed concrete aggregate should contain at least 90% crushed concrete without any steel reinforcements or any other impurities. In addition, asphaltic pavement and surfacing material content should be lower than 10% in crushed concrete aggregate.

Crushed natural aggregates and recycled aggregates can be mixed at various percentages to create reprocessed materials or blended materials. Every single aggregate of blended materials must satisfy the specified aggregate base physical properties criteria (Table 22), and final blend must meet the specified gradation (WisDOT 2018). Per section 305, dense graded aggregates such as crushed stone, crushed gravel and crushed concrete (except reclaimed asphalt) should meet the gradations provided in Table 23. For reclaimed asphalt, gradation is primarily assessed visually, e.g., reclaimed asphalt 100% passing 1 ¼-inch sieve may be used for 1 ¼-inch aggregate base application (WisDOT 2018).

Per section 301, crushed concrete can contain up to 12% of glass, 7% of foundry slag, 75% of steel mill slag, 8% of bottom ash, and 7% of pottery cull (by weight). However, all of the by-products should not have any deleterious materials (WisDOT 2018).

Table 21. SUITABILITY OF VARIOUS AGGREGATE BASE MATERIALS (WISDOT 2018)

BASE TYPE	CRUSHED STONE	CRUSHED GRAVEL	CRUSHED CONCRETE	RECLAIMED ASPHALT	REPROCESSED MATERIAL	BLENDED MATERIAL
Dense 3/4-inch	Yes	Yes	Yes	No	Yes[1]	Yes[1]
Dense 1 1/4-inch	Yes	Yes	Yes	Yes	Yes	Yes
Dense 3-inch	Yes	Yes	Yes	No	Yes[2]	Yes[2]
Open-graded	Yes	Yes	No	No	No	No

[1] The contractor may provide reprocessed material or blended material as 3/4-inch base only if the material contains 50 percent or less reclaimed asphalt, by weight.

[2] Ensure that material is substantially free of reclaimed asphalt.

Table 22. AGGREGATE BASE PHYSICAL PROPERTIES (WISDOT 2018)

PROPERTY	CRUSHED STONE	CRUSHED GRAVEL	CRUSHED CONCRETE	RECLAIMED ASPHALT	REPROCESSED MATERIAL	BLENDED MATERIAL
Gradation AASHTO T27						
dense	305.2.2.1	305.2.2.1	305.2.2.1	305.2.2.2	305.2.2.1	305.2.2.1[1]
open-graded	310.2	310.2	not allowed	not allowed	not allowed	not allowed
Wear AASHTO T96 loss by weight	≤50%	≤50%	note[2]	___	note[2]	note[3]
Sodium sulfate soundness AASHTO T104 loss by weight						
dense	≤18%	≤18%	___	___	___	note[3]
open-graded	≤12%	≤12%	not allowed	not allowed	not allowed	not allowed
Freeze/thaw soundness AASHTO T103 loss by weight						
dense	≤18%	≤18%	___	___	___	note[3]
open-graded	≤18%	≤18%	not allowed	not allowed	not allowed	not allowed
Liquid limit AASHTO T89	≤25	≤25	≤25	___	___	note[3]
Plasticity AASHTO T90	≤6[4]	≤6[4]	≤6[4]	___	___	note[3]
Fracture ASTM D5821[6] min one face by count						
dense	58%	58%	58%	___	note[5]	note[3]
open-graded	90%	90%	not allowed	not allowed	not allowed	not allowed

[1] The final aggregate blend must conform to the specified gradation.

[2] No requirement for material taken from within the project limits. Maximum of 50 percent loss, by weight, for material supplied from a source outside the project limits.

[3] Required as specified for the individual component materials defined in columns 2 - 6 of the table before blending.

[4] For base placed between old and new pavements, use crushed stone, crushed gravel, or crushed concrete with a plasticity index of 3 or less.

[5] ≥75 percent by count of non-asphalt coated particles.

[6] as modified in CMM 8-60.

Table 23. GRADATION REQUIREMENTS OF DENSE-GRADED AGGREGATE BASE MATERIALS EXCEPT FOR RECLAIMED ASPHALT (WISDOT 2018)

	PERCENT PASSING BY WEIGHT		
SIEVE	3-INCH	1 1/4-INCH	3/4-INCH
3-inch	90 - 100	___	___
1 1/2-inch	60 - 85	___	___
1 1/4-inch	___	95 - 100	___
1-inch	___	___	100
3/4-inch	40 - 65	70 - 93	95 - 100
3/8-inch	___	42 - 80	50 - 90
No. 4	15 - 40	25 - 63	35 - 70
No. 10	10 - 30	16 - 48	15 - 55
No. 40	5 - 20	8 - 28	10 - 35
No. 200	2.0 - 12.0	2.0 - 12.0[1][3]	5.0 - 15.0[2]

4.6. MICHIGAN DOT

Sections 302 and 902 of the MDOT Standard Specifications for Construction published in 2012 allows the crushed concrete along with natural aggregate and iron blast furnace slag as base materials if they meet the gradation (Table 24) and quality (Table 25) specifications for Class 21AA, 21A, 22A, and 23A dense-graded aggregates. Dense-graded aggregates can be mixed with fine-grained aggregates to meet the specifications. Crushed concrete should not contain more than 5% of brick, wood, plaster or asphalt by particle count but steel reinforcement pieces are allowed as long as they meet the specified gradation of stated dense-graded aggregate Classes.

Crushed concrete can be used as long as there is an additional granular layer of at least 12 inches (with class I, II, IIA, or IIAA aggregates – Table 26) between the dense-graded aggregate base and an underdrain, which the dense-graded aggregate base drains into. In addition, a geotextile liner or geomembrane can be used as an alternative to granular layer between the dense-graded aggregate base and the underdrain (MDOT 2012).

Table 24. GRADING REQUIREMENTS FOR DENSE-GRADED AGGREGATES (MDOT 2012)

Series/Class	Sieve Analysis (MTM 109) Total Percent Passing								Loss by Washing (MTM 108) % Passing
	1½ in	1 in	¾ in	½ in	⅜ in	No. 4	No. 8	No. 30	No. 200
21 AA	100	85-100	-	50-75	-	-	20-45	-	4-8
21 AA	100	85-100	-	50-75	-	-	20-45	-	4-8
22 A	-	100	90-100	-	65-85	-	30-50	-	4-8
23 A	-	100	-	-	60-85	-	25-60	-	9-16

Table 25. PHYSICAL REQUIREMENTS FOR DENSE-GRADED AGGREGATES (MDOT 2012)

Series/Class	Crushed Material, % min (MTM 117)	Loss, % max, Los Angeles Abrasion (MTM 102)
21 AA	95	50
21 AA	25	50
22 A	25	50
23 A	25	50

Table 26. GRADING REQUIREMENTS FOR GRANULAR MATERIALS (MDOT 2012)

Material	Sieve Analysis (MTM 109), Total % Passing (a)									Loss by Washing % Passing No. 200 (a), (b)
	6 in	3 in	2 in	1 in	½ in	⅜ in	No. 4	No. 30	No. 100	
Class I	—	—	100	—	45–85	—	20–85	5–30		0–5
Class II (c)	—	100	—	60–100	—	—	50–100	—	0–30	0–7
Class IIA (c)	—	100	—	60–100	—	—	50–100	—	0–35	0–10
Class IIAA	—	100	—	60–100	—	—	50–100	—	0–20	0–5
Class III	100	95–100	—	—	—	—	50–100	—	—	0–15
Class IIIA	—	—	—	—	—	100	50–100	—	0–30	0–15

a. Test results based on dry weights.
b. Use test method MTM 108 for Loss by Washing.
c. Except for use in granular blankets, Class IIA granular material may be substituted for Class II granular material for projects located in the following counties: Arenac, Bay, Genesee, Gladwin, Huron, Lapeer, Macomb, Midland, Monroe, Oakland, Saginaw, Sanilac, Shiawassee, St. Clair, Tuscola, and Wayne counties.

CHAPTER 5. INPUTS FOR PAVEMENT MECHANISTIC-EMPIRICAL DESIGN (PMED)

In order to produce reliable and accurate results, the PMED relies on a high level of detailed information about input parameters for materials, traffic and climate. Determining all these parameters requires extensive testing and data collection efforts, and it can be difficult to devote the resources to that if the information is not part of an already existing data set. As an alternative, the PMED software allows users to enter this information in a hierarchical fashion, meaning that the user has the option to provide different levels of detail, then the program adjusts these inputs accordingly. Level 1 input needs more precise information from field and laboratories which should lead to the most accurate pavement distress analyses while level 3 input provides the least precise pavement distress predictions.

For instance, traffic data in its simplest form could simply be an estimate of vehicle traffic volumes. Since the PMED process relies on traffic data to calculate pavement loads, the software would need to convert this into a load factor by assuming a typical distribution of vehicle types. However, if you had actual traffic counts for a project site, including vehicle class information, this would allow an additional level of input in the hierarchy. Assumptions would still need to be made about the spectrum of actual loads based on equivalency factors (ESALs or Equivalent Single Axle Loads). At the top of the hierarchy, you would need to collect vehicle weight data near the site to determine the actual load distribution, in addition to monitoring vehicle counts. This can be achieved by detailed analyses of Weigh-In-Motion (WIM) data. This is the one example of the most comprehensive data and increases the precision of the design assumptions. However, the PMED process can still function at lower levels of detail.

During PMED analyses in this study, the design inputs of pavement surface layers and subgrade layers are kept constant to be able to investigate impact of the properties of RCA and RAP base layers on predicted pavement distresses. All analyses were conducted at 90% reliability level. Table 27 summarizes the general inputs used for PMED analyses.

Table 27. GENERAL INPUTS

Input	Value
Design Period	20 years
SM_r of Subgrade	15000 psi
Subgrade Gradation	A-1-b
Groundwater Depth (ft)	10
Flexible Pavement Input	
Binder Grade	Super Pave PG 58-34
Base Poisson's Ratio	0.35
HMA Poisson's Ratio	0.35
Rigid Pavement Input	
PCC Unit Weight (pcf)	150
PCC Poisson's Ratio	0.15

Notes: SM_r=Summary resilient modulus, HMA= Hot mix asphalt, PCC= Portland cement concrete.

Three different traffic volumes were considered for pavement design analyses (e.g. low, medium, and high traffic). Table 28 shows the traffic data used in Pavement ME analyses along with surface layer and base layer thicknesses which were selected per recommendations of Schwartz et al. (2011).

Table 28. TRAFFIC INPUTS

Inputs	Low Traffic	Medium Traffic	High Traffic
AADTT	1000	7500	25000
Number of Lanes in Design Direction	2	3	3
Percent of Trucks in Design Direction (%)	50	50	50
Percent of Trucks in Design Lane (%)	75	55	50
Operational Speed (mph)	50	50	50
Asphalt Thickness in flexible pavement (in)	2	3	4
Base Thickness in flexible pavement (in)	8	10	12
PCC Thickness for rigid pavement (in)	8	9	11
Base Thickness in rigid pavement (in)	4	6	8

Notes: AADTT= Average Annual Daily Truck Traffic, PCC=Portland cement concrete.

In order to investigate the effects of RAP and RCA properties on pavement distress predictions when used as base layer materials, the lowest, the highest and median values of summary resilient modulus (SM_r), gradation, hydraulic conductivity, optimum moisture content (OMC) and maximum dry unit weight (γ_{dmax}) of these materials were collected from the database. A summary of these input is also shown in Appendix C and D. The highest and the lowest values are obtained from the database for each characteristic shown in Tables 29 while the median values are calculated from all the available data for each characteristic in the database. For instance, the lowest SM_r of RAP was reported to be 24,366 psi by Edil et al. (2012a) thus other inputs shown in Table 3 were chosen from that paper accordingly. On the other hand, the highest SM_r of RAPs was 58,015 psi from Attia and Abdelrahman (2010a) and other inputs were collected from the same paper as well.

Table 29. BASE INPUTS INVESTIGATING SM_R EFFECT OF RAP

Data Value	Varied Parameter (SM_r, psi)	Gravel Percent (%)	Sand Percent (%)	Fines Content (%)	MDU (pcf)	OMC (%)	Hydraulic conductivity (ft/hr)
Lowest*	24366	49.3	50.4	0.4	138	5.2	2.73
Median	37927	45	54	1	126	6.1	0.71
Highest**	58015	51	48.6	0.4	134	5.5	-

Notes: SM_r=Summary resilient modulus, MDU= Maximum dry unit weight, OMC= Optimum moisture content. *Edil et al. (2012a), **Attia and Abdelrahman (2010a)

The lowest SM_r of RCA was reported to be 17,898 psi by Cetin et al. (2020) thus other inputs shown in Table 30 are chosen from that report accordingly. The highest SM_r of RCAs was 53,664 psi from Diagne et al. (2015) and other inputs were collected from the same paper as well.

Table 30. BASE INPUTS INVESTIGATING SM_R EFFECT OF RCA

Data Value	Varied Parameter (SM_r, psi)	Gravel Percent (%)	Sand Percent (%)	Fines Content (%)	MDU (pcf)	OMC (%)	Hydraulic conductivity (ft/hr)
Lowest*	17898	38.3	54.6	7.1	123	11.1	0.06
Median	26542	50.8	45.5	3	127	10.8	0.2
Highest**	53664	47.2	48.6	1.8	134	6.1	0.35

Notes: SM_r=Summary resilient modulus, MDU= Maximum dry unit weight, OMC= Optimum moisture content. *Cetin et al. (2020), **Diagne et al. (2015)

The lowest fines content of RAP was reported to be 0% by Alam et al. (2010) thus other inputs shown in Table 31 are chosen from that report accordingly. The highest fines content of RAPs was 11% from Camargo et al. (2013) and other inputs were collected from the same paper as well.

Table 31. BASE INPUTS INVESTIGATING FINES CONTENT EFFECT OF RAP

Data Value	Varied Parameter (Fines content, %)	Gravel Percent (%)	Sand Percent (%)	MDU (pcf)	OMC (%)	Hydraulic conductivity (ft/hr)	SM_r (psi)
Lowest*	0	3	97	-	-	-	39349
Median	1	45	54	126	6.1	0.71	37927
Highest**	11	46	43	136	7.5	-	44962

Notes: SM_r=Summary resilient modulus, MDU= Maximum dry unit weight, OMC= Optimum moisture content. *Alam et al. (2010), **Camargo et al. (2013)

The lowest fines content of RCA was reported to be 0.1% by Mahedi and Cetin (2020) thus other inputs shown in Table 32 are chosen from that report accordingly. The highest fines content of RCAs was 15% from Chen et al. (2013) and other inputs were collected from the same paper as well.

Table 32. BASE INPUTS INVESTIGATING FINES CONTENT EFFECT OF RCA

Data Value	Varied Parameter (Fines content, %)	Gravel Percent (%)	Sand Percent (%)	MDU (pcf)	OMC (%)	Hydraulic conductivity (ft/hr)	SM_r (psi)
Lowest*	0.1	68.8	31.1	127	14.4	-	-
Median	3	50.8	45.5	127	10.8	0.2	26542
Highest**	15	41	44	121	11.9	-	27412

Notes: SM_r=Summary resilient modulus, MDU= Maximum dry unit weight, OMC= Optimum moisture content. *Mahedi and Cetin (2020), **Chen et al. (2013)

The lowest gravel content of RAP was reported to be 3% by Alam et al. (2010) thus other inputs shown in Table 33 are chosen from that report accordingly. The highest gravel content of RAPs was 68.1% from Garg and Thompson (1996) and other inputs were collected from the same paper as well.

Table 33. BASE INPUTS INVESTIGATING GRAVEL CONTENT EFFECT OF RAP

Data Value	Varied Parameter (Gravel Percent, %)	Sand Percent (%)	Fines content (%)	MDU (pcf)	OMC (%)	Hydraulic conductivity (ft/hr)	SM_r (psi)
Lowest*	3	97	0	-	-	-	39349
Median	45	54	1	126	6.1	0.71	37927
Highest**	68.1	28.1	3.8	135	6	-	31702

Notes: SM_r=Summary resilient modulus, MDU= Maximum dry unit weight, OMC= Optimum moisture content. *Alam et al. (2010), **Garg and Thompson (1996)

The lowest gravel content of RCA was reported to be 31.8% by Edil et al. (2012a) thus other inputs shown in Table 34 are chosen from that report accordingly. The highest gravel content of RCAs was 94.1% from Mahedi and Cetin (2020) and other inputs were collected from the same paper as well.

Table 34. BASE INPUTS INVESTIGATING GRAVEL CONTENT EFFECT OF RCA

Data Value	Varied Parameter (Gravel Percent, %)	Sand Percent (%)	Fines content (%)	MDU (pcf)	OMC (%)	Hydraulic conductivity (ft/hr)	SM_r (psi)
Lowest*	31.8	64.9	3.3	125	11.2	-	27412
Median	50.8	45.5	3	127	10.8	0.2008	26542
Highest**	94.1	4.9	1	118	12.6	-	-

Notes: SM_r=Summary resilient modulus, MDU= Maximum dry unit weight, OMC= Optimum moisture content. *Edil et al. (2012a), **Mahedi and Cetin (2020)

The lowest sand content of RAP was reported to be 28.1% by Garg and Thompson (1996) thus other inputs shown in Table 35 are chosen from that report accordingly. The highest sand content of RAPs was 97% from Alam et al. (2010) and other inputs were collected from the same paper as well.

Table 35. BASE INPUTS INVESTIGATING SAND CONTENT EFFECT OF RAP

Data Value	Varied Parameter (Sand Percent, %)	Gravel Percent (%)	Fines content (%)	MDU (pcf)	OMC (%)	Hydraulic conductivity (ft/hr)	SM_r (psi)
Lowest*	28.1	68.1	3.8	135	6	-	31702
Median	54	45	1	126	6.1	0.71	37927
Highest**	97	3	0	-	-	-	39349

Notes: SM_r=Summary resilient modulus, MDU= Maximum dry unit weight, OMC= Optimum moisture content. *Garg and Thompson (1996), **Alam et al. (2010)

The lowest sand content of RCA was reported to be 4.9% by Mahedi and Cetin (2020) thus other inputs shown in Table 36 are chosen from that report accordingly. The highest sand content of RCAs was 64.9% from Edil et al. (2017) and other inputs were collected from the same paper as well.

Table 36. BASE INPUTS INVESTIGATING SAND CONTENT EFFECT OF RCA

Data Value	Varied Parameter (Sand Percent, %)	Gravel Percent (%)	Fines content (%)	MDU (pcf)	OMC (%)	Hydraulic conductivity (ft/hr)	SM_r (psi)
Lowest*	4.9	94.1	1	118	12.6	-	-
Median	45.5	50.8	3	127	10.8	0.2	26542
Highest**	64.9	31.8	3.5	125	11.2	-	27412

Notes: SM_r=Summary resilient modulus, MDU= Maximum dry unit weight, OMC= Optimum moisture content. *Mahedi and Cetin (2020), **Edil et al. (2017)

The lowest D_{60} of RAP was reported to be 2.3 mm by Edil et al. (2012a) in a RAP sample from Minnesota thus other inputs shown in Table 37 are chosen from that report accordingly. The highest D_{60} of RAPs was 10.4 mm from Wu et al. (2012) and other inputs were collected from the same paper as well.

Table 37. BASE INPUTS INVESTIGATING D_{60} EFFECT OF RAP

Data Value	Varied Parameter (D_{60}, in)	Gravel Percent (%)	Sand Percent (%)	Fines content (%)	MDU (pcf)	OMC (%)	Hydraulic conductivity (ft/hr)	SM_r (psi)
Lowest*	0.090	26.3	71.2	2.5	134	6.7	0.013	26107
Median	0.19	45	54	1	126	6.05	0.71	37927
Highest**	0.409	67	32	1	-	-	-	29008

Notes: SM_r=Summary resilient modulus, MDU= Maximum dry unit weight, OMC= Optimum moisture content. *Edil et al. (2012a), **Wu et al. (2012)

The lowest D_{60} of RCA was reported to be 1.7 mm by Edil et al. (2012a) thus other inputs shown in Table 38 are chosen from that report accordingly. The highest D_{60} of RCAs was 16.3 mm from Edil et al. (2012a) and other inputs were collected from the same paper as well.

Table 38. BASE INPUTS INVESTIGATING D_{60} EFFECT OF RCA

Data Value	Varied Parameter (D_{60}, in)	Gravel Percent (%)	Sand Percent (%)	Fines content (%)	MDU (pcf)	OMC (%)	Hydraulic conductivity (ft/hr)	SM_r (psi)
Lowest*	0.067	31.8	31.8	3.3	125	11.2	-	27412
Median	0.268	50.8	45.5	3	127	10.8	0.2	26542
Highest*	0.642	76.3	21.6	2.1	127	9.2	-	23786

Notes: SM_r=Summary resilient modulus, MDU= Maximum dry unit weight, OMC= Optimum moisture content. *Edil et al. (2012a)

CHAPTER 6. DISTRESSES

The following pavement distresses were analyzed via PMED software: 1) for flexible pavements- International Roughness Index (IRI), rutting, and fatigue distresses, 2) for rigid pavements-IRI, joint faulting, transverse cracking.

Target failure values at a reliability level of 90% for different pavement distresses for flexible pavements are summarized in Table 39. IRI values greater than 170 in/mile were marked as a failure in this study per suggestions of Elbheiry et al. (2011) and this value was determined as the terminal IRI. 0.75 inches were determined as a target values for failure for total rutting distresses (Ceylan et al. 2015). Table 40 represents the target values of distresses for rigid pavements. Terminal IRI and joint faulting distresses for rigid pavements were chosen as 172 in/mile and 0.12 inches, respectively.

Table 39. PAVEMENT DISTRESS TYPES AND TARGET VALUES FOR FLEXIBLE PAVEMENT

Parameter	Target value	Reliability (%)
Terminal IRI (in/mile)	170	90
Total Pavement Rutting (in)	0.75	90

Notes: IRI= International Roughness Index; AC= Asphalt Concrete

Table 40. PAVEMENT DISTRESS TYPES AND TARGET VALUES FOR JCPC

Parameter	Target value	Reliability (%)
Terminal IRI (in/mile)	172	90
Mean Joint Faulting (in)	0.12	90

Notes: IRI= International Roughness Index

In this section, distress analysis is done using the inputs indicated in Section 2 focusing on two distresses which might be affected by the base properties, IRI and total pavement deformation.

6.1. INTERNATIONAL ROUGHNESS INDEX (IRI) FOR FLEXIBLE PAVEMENTS

The international roughness index (IRI) is a standard measure of pavement smoothness and ride quality (Izevbekhai and Akkari 2011). The terminal IRI value was defined to be 170 in/mile (Elbheiry et al. 2011). The initial IRI value was determined to be 63 in/mile which was in accordance with the suggestions provided by Izevbekhai and Akkari (2011); Ceylan et al. (2015).

6.1.1. IMPACT OF SUMMARY RESILIENT MODULUS (SM_R) ON IRI

The predicted IRI values using the inputs mentioned in Table 29 and Table 30 are shown in Figure 74 for RAP and Figure 75 for RCA in flexible pavements. Both Figure 74 and Figure 75 show that higher traffic and base layers with lower SM_r yield higher IRI damage to the flexible pavements indicating that stiffness of the base layers have an impact on IRI distress. However, it does not seem to cause high differences in terms of IRI performance and none of the results exceeded the terminal IRI values. Thus, it can be claimed that any SM_r values presented in the database can be used comfortably for pavement design analyses if IRI is the main design criteria.

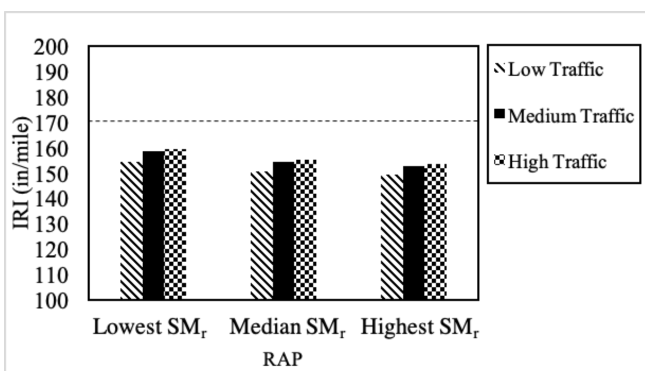

Figure 74. IRI VERSUS DIFFERENT SM_R OF RAP

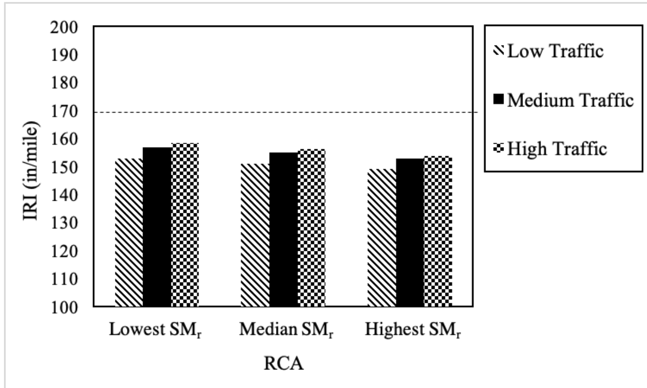

Figure 75. IRI VERSUS DIFFERENT SM_R OF RCA

6.1.2. IMPACT OF FINES CONTENT ON IRI

The predicted IRI values using the inputs mentioned in Table 31 and Table 32 are shown in Figure 76 for RAP and Figure 77 for RCA in flexible pavements. Results showed that higher fines contents in RAP (ranging between 0%-11%) and RCA (ranging between 0.1%-15%) used as a base course material had higher IRI values in flexible pavements (Figure 76 and Figure 77). However, none of the results exceeded the terminal IRI values indicating that any fines content values presented in the database can be used comfortably for pavement design analyses if IRI is the main design criteria. In addition, higher volume of traffic yielded higher IRI values regardless of fines content of RAP and RCA materials.

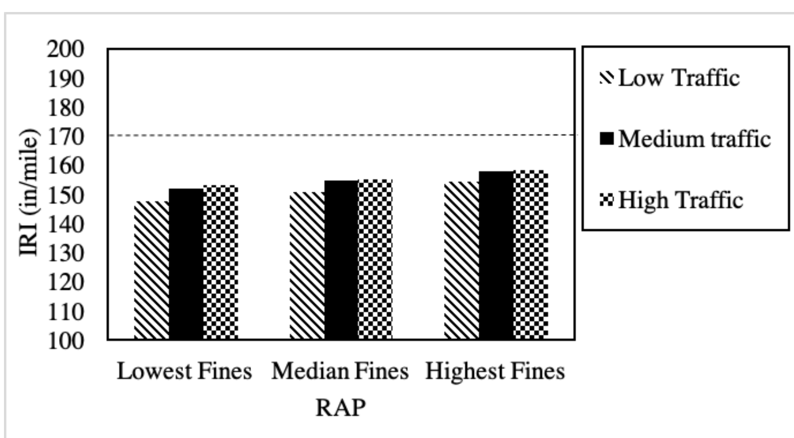

Figure 76. IRI VERSUS DIFFERENT FINES CONTENT OF RAP IN FLEXIBLE PAVEMENT

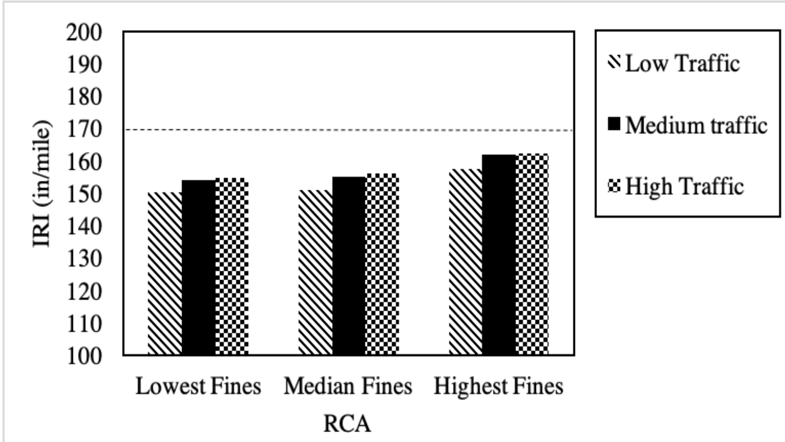

Figure 77. IRI VERSUS DIFFERENT FINES CONTENT OF RCA IN FLEXIBLE PAVEMENT

6.1.3. IMPACT OF GRAVEL CONTENT ON IRI

The predicted IRI values using the inputs mentioned in Table 33 and Table 34 are shown in Figure 78 for RAP and Figure 79 for RCA in flexible pavements. Results showed that higher gravel content in RAP (ranging between 3%-68.1%) and RCA (ranging between 31.8%-94.1%) materials seemed to increase IRI values slightly (almost negligible). As expected, higher traffic volume resulted in higher IRI values. Moreover, none of the results exceeded the terminal IRI values indicating that any gravel content values presented in the database can be used comfortably for pavement design analyses if IRI is the main design criteria.

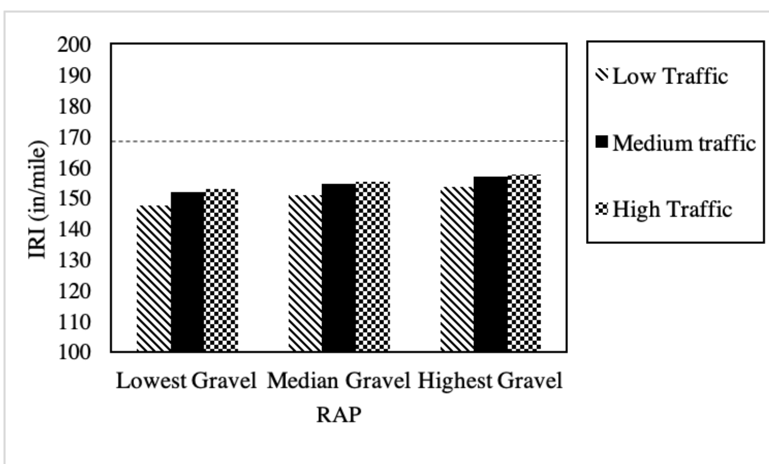

Figure 78. IRI VERSUS DIFFERENT GRAVEL CONTENT OF RAP IN FLEXIBLE PAVEMENT

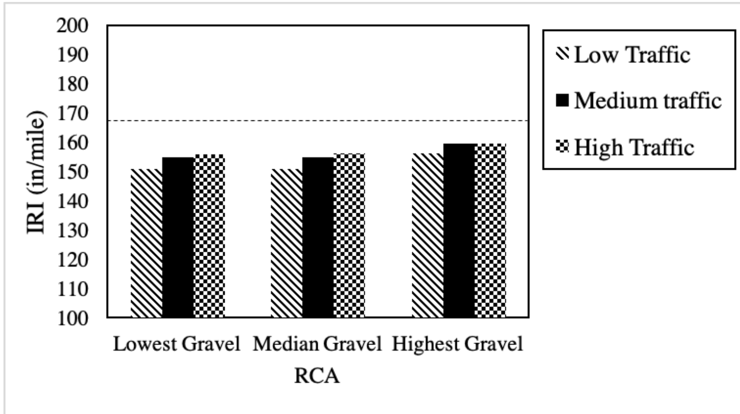

Figure 79. IRI VERSUS DIFFERENT GRAVEL CONTENT OF RCA IN FLEXIBLE PAVEMENT

6.1.4. IMPACT OF SAND CONTENT ON IRI

The predicted IRI values using the inputs mentioned in Table 35 and Table 36 are shown in Figure 80 for RAP and Figure 81 for RCA in flexible pavements. Figure 80 and Figure 81 show that there is a small decrease in IRI values when RAP (ranging between 28.1%-97%) and RCA (ranging between 4.9%-64.9%) base materials have relatively higher sand contents However, this change was very small and can be assumed negligible. Moreover, none of the results exceeded the terminal IRI values indicating that any sand content values presented in the database can be used comfortably for pavement design analyses if IRI is the main design criteria.

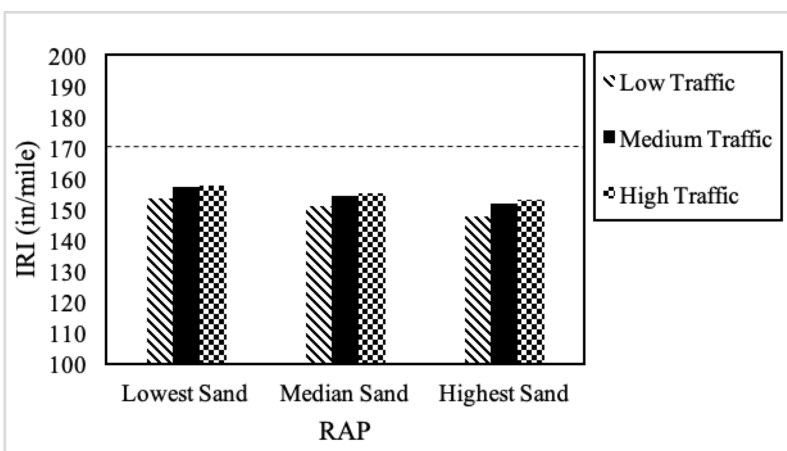

Figure 80. IRI VERSUS DIFFERENT SAND CONTENT OF RAP IN FLEXIBLE PAVEMENT

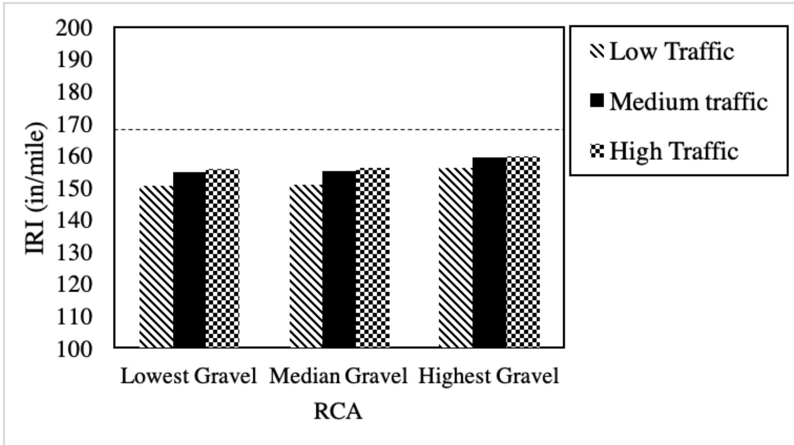

Figure 81. IRI VERSUS DIFFERENT SAND CONTENT OF RCA IN FLEXIBLE PAVEMENT

6.1.5. IMPACT OF D_{60} ON IRI

The predicted IRI values using the inputs mentioned in Table 37 and Table 38 are shown in Figure 82 for RAP and Figure 83 for RCA in flexible pavements. Impacts of D_{60} of the RAP and RCA materials were also investigated to determine whether there was a relationship between D_{60} of these materials and predicted IRI. As shown in Figures 82 and 83, no trend is observed between D_{60} and IRI values while higher traffic volume causes higher IRI values as expected.

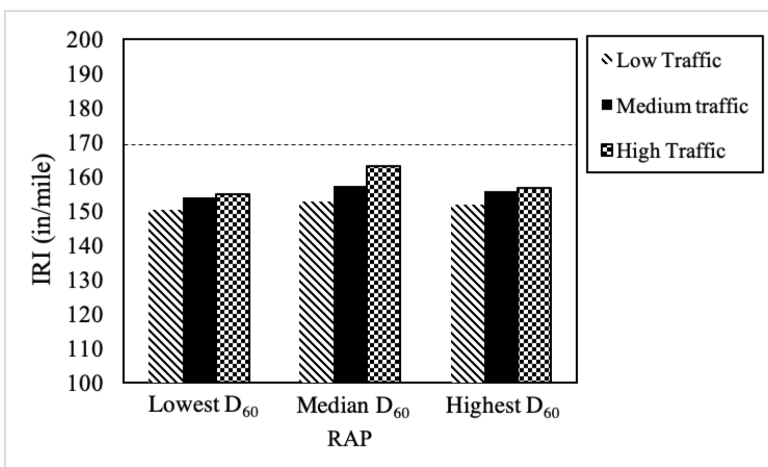

Figure 82. IRI VERSUS DIFFERENT D_{60} OF RAP IN FLEXIBLE PAVEMENT

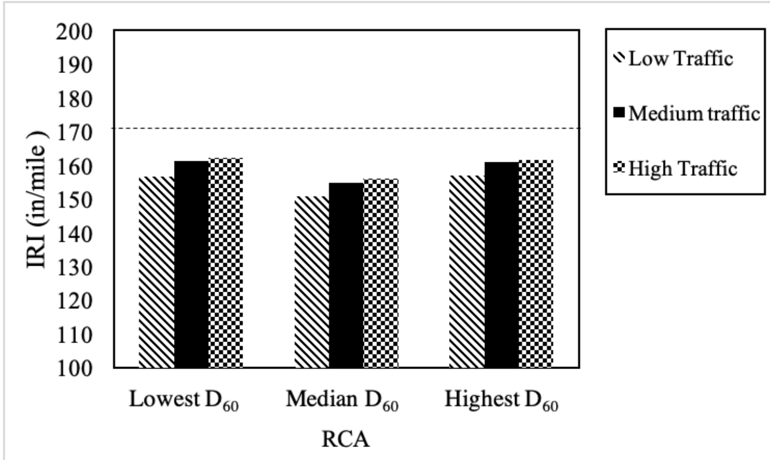

Figure 83. IRI VERSUS DIFFERENT D_{60} OF RCA IN FLEXIBLE PAVEMENT

6.2. INTERNATIONAL ROUGHNESS INDEX (IRI) FOR RIGID PAVEMENTS

6.2.1. IMPACT OF SMR ON IRI

The predicted IRI values using the inputs mentioned in Table 29 and Table 30 are shown in Figure 84 for RAP and Figure 85 for RCA in rigid pavements. Results showed that traffic volume had a significant impact on IRI of rigid pavements while SM_r of the RAP and RCA materials did not seem to impact the rigid pavements IRI performances.

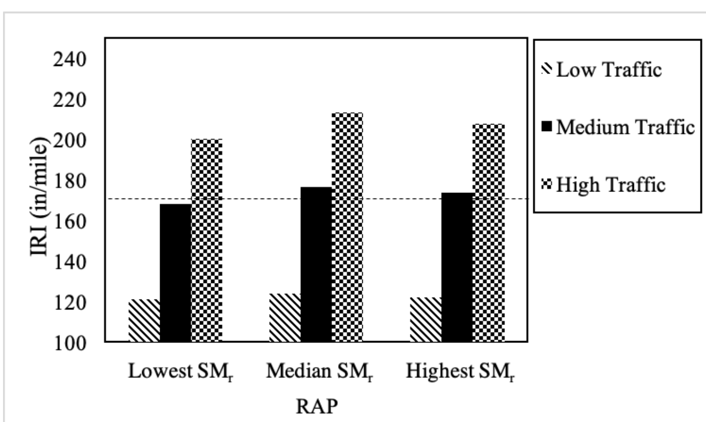

Figure 84. IRI VERSUS DIFFERENT SM_R OF RAP IN RIGID PAVEMENT

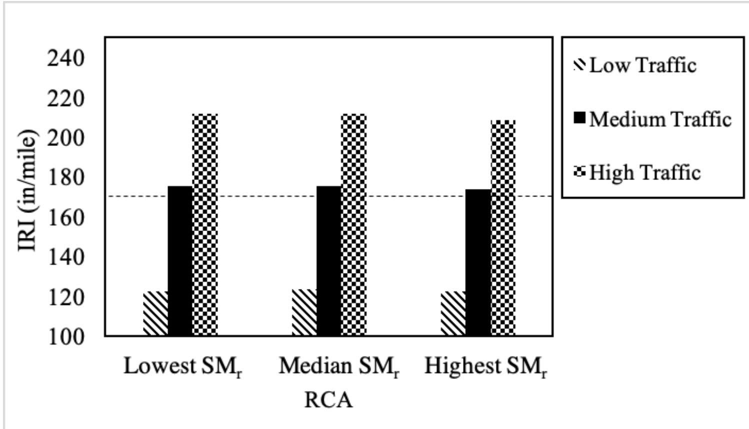

Figure 85. IRI VERSUS DIFFERENT SM_R OF RCA IN RIGID PAVEMENT

6.2.2. IMPACT OF FINES CONTENT ON IRI

The predicted IRI values using the inputs mentioned in Table 31 and Table 32 are shown in Figure 86 for RAP and Figure 87 for RCA in rigid pavements. Figure 86 shows that an increase in fines content in RAP material (ranging between 0%-11%) caused a slight decrease in IRI values for rigid pavements. On the other hand, an opposite trend was observed for RCA material as an increase in fines content (ranging between 0.1%-15%) resulted in higher IRI values. Moreover, Figures 86 and 87 show that all IRI values exceeded the terminal IRI value for RAP except the ones subjected to lower traffic volume while RCA IRI values satisfied this threshold performance for medium traffic level as well.

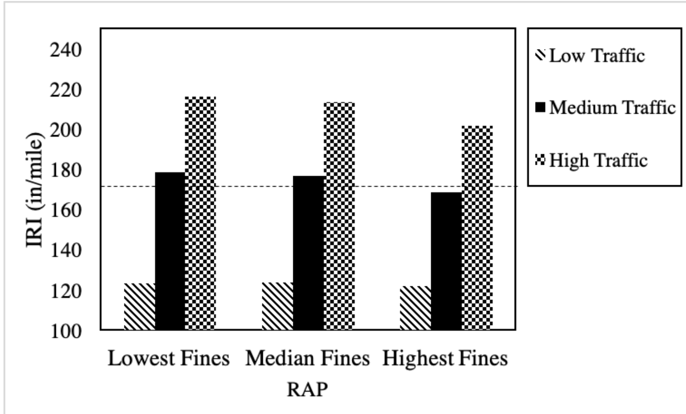

Figure 86. IRI VERSUS DIFFERENT FINES CONTENT OF RAP IN RIGID PAVEMENT

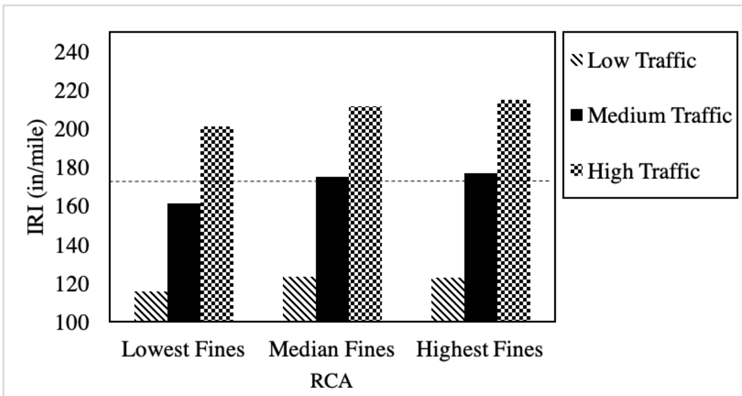

Figure 87. IRI VERSUS DIFFERENT FINES CONTENT OF RCA IN RIGID PAVEMENT

6.2.3. IMPACT OF GRAVEL CONTENT ON IRI

The predicted IRI values using the inputs mentioned in Table 33 and Table 34 are shown in Figure 88 for RAP and Figure 89 for RCA in rigid pavements. Figures 88 and 89 show that IRI values decrease when RAP (ranging between 3%-68.1%) and RCA (ranging between 31.8%-94.1%) with higher gravel contents are used as base materials. In addition, it was observed that terminal IRI values were exceeded when the lowest and median gravel contents were used under high and medium level traffic volumes. This suggests determining the gravel content of RAP and RCA materials before their use as a base material for rigid pavement design.

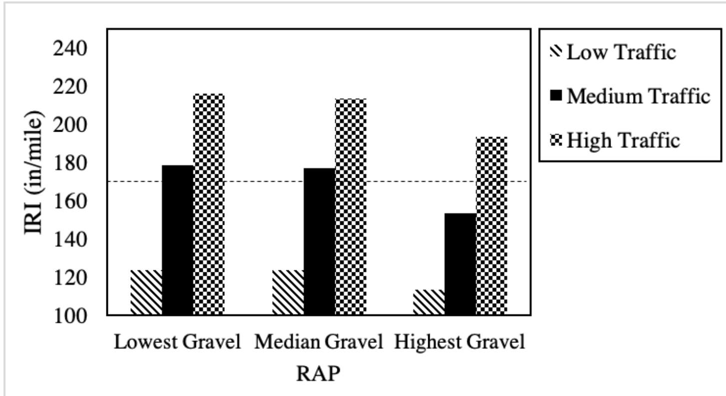

Figure 88. IRI VERSUS DIFFERENT GRAVEL CONTENT OF RAP IN RIGID PAVEMENT

Figure 89. IRI VERSUS DIFFERENT GRAVEL CONTENT OF RCA IN RIGID PAVEMENT

6.2.4. IMPACT OF SAND CONTENT ON IRI

The predicted IRI values using the inputs mentioned in Table 35 and Table 36 are shown in Figure 90 for RAP and Figure 91 for RCA in rigid pavements. Figure 90 shows that IRI values of RAP are significantly impacted when sand content is increased from the lowest (28.1%) to median sand content while such impact is not observed between IRI values predicted via median and the highest sand contents (97%). Such a trend was not observed for RCA (Figure 91). These results claim that sand contents of RAP bases could be a critical parameter to be checked before conducting rigid pavement design.

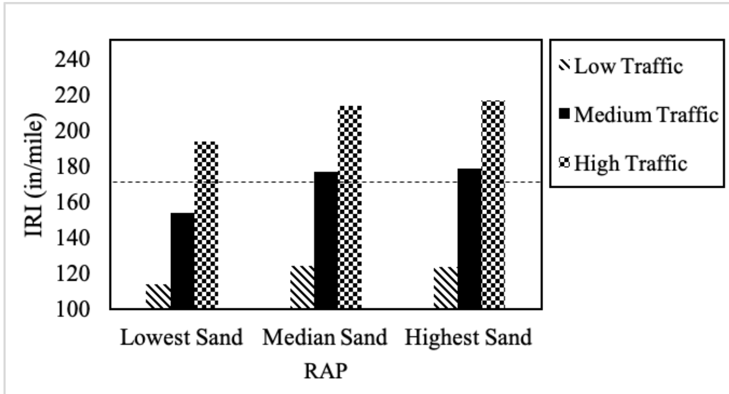

Figure 90. IRI VERSUS DIFFERENT SAND CONTENT OF RAP IN RIGID PAVEMENT

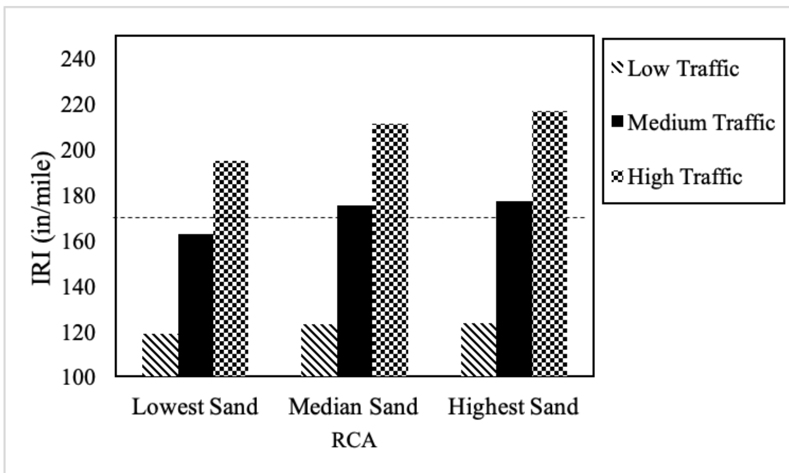

Figure 91. IRI VERSUS DIFFERENT SAND CONTENT OF RCA IN RIGID PAVEMENT

6.2.5. IMPACT OF D_{60} ON IRI

The predicted IRI values using the inputs mentioned in Table 37 and Table 38 are shown in Figure 92 for RAP and Figure 93 for RCA in rigid pavements. Results for both RAP and RCA showed that an increase in D_{60} from the lowest (0.090 inch in RAP and 0.067 inch in RCA) to median value (0.19 inch in RAP and 0.268 inch in RCA) did not seem to impact the IRI performance of rigid pavements while it was improved significantly when D_{60} was the highest value presented in the database. These results indicated that agencies may prefer to use RAP and RCA materials with higher D_{60} values.

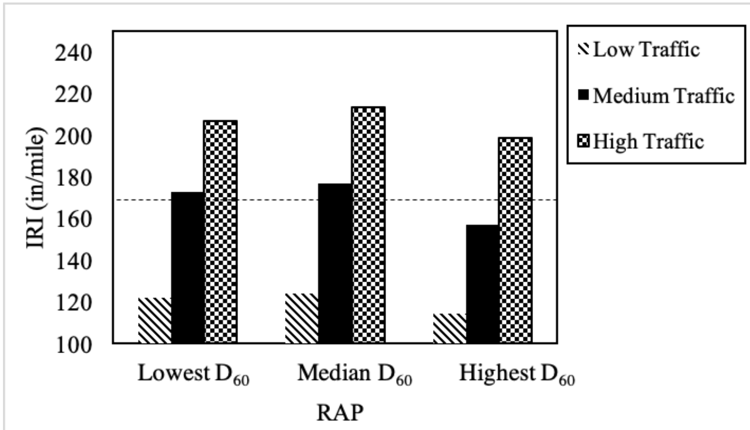

Figure 92. IRI VERSUS DIFFERENT D_{60} OF RAP IN RIGID PAVEMENT

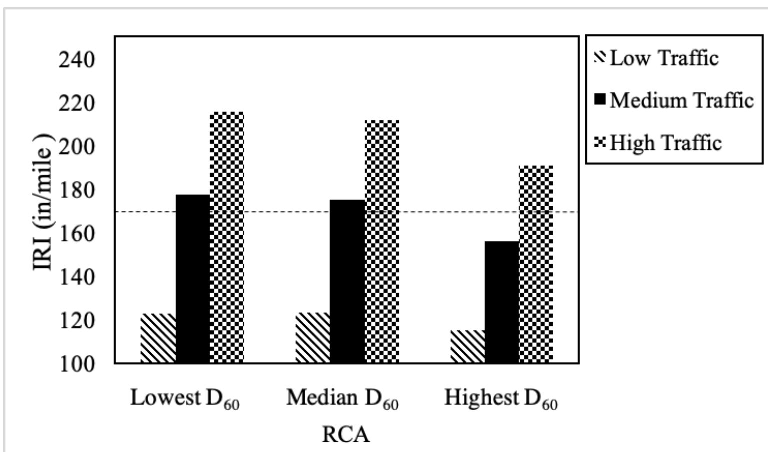

Figure 93. IRI VERSUS DIFFERENT D_{60} OF RCA IN RIGID PAVEMENT

6.3. Total Rutting on Flexible Pavements

6.3.1. IMPACT OF SMR ON TOTAL RUTTING

Figure 94 and Figure 95 show that the summary resilient modulus (SM_r) of RCA and RAP has an impact on the total rutting of the pavement system. It was observed that changes in SM_r of both

RAP (ranging between 24366 psi-58015 psi) and RCA (ranging between 17898 psi-53664 psi) had a same rate of decrease in total rutting distress predictions.

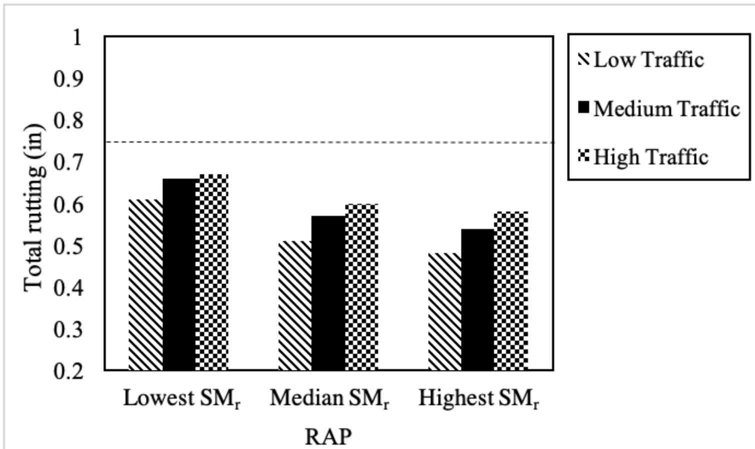

Figure 94. TOTAL RUTTING VERSUS DIFFERENT SM_R OF RAP

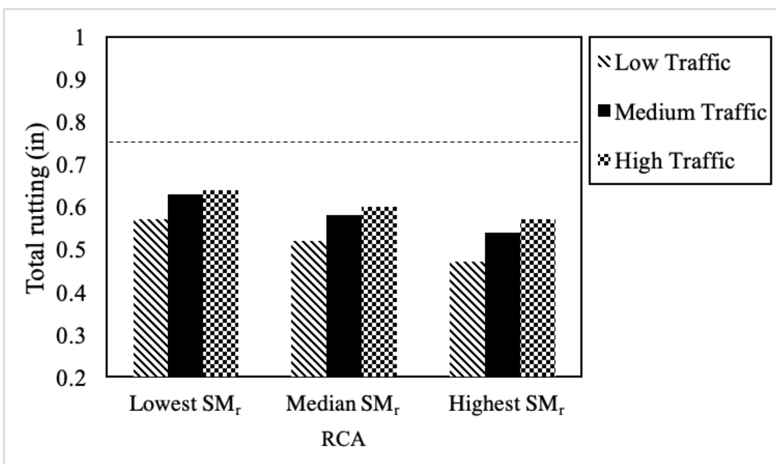

Figure 95. TOTAL RUTTING VERSUS DIFFERENT SM_R OF RCA

6.3.2. IMPACT OF FINES CONTENT ON TOTAL RUTTING

According to Figure 96 and Figure 97, total rutting of pavements (running Pavement ME with input shown in Tables 31 and 32) increase significantly with an increase in fines contents of both RAP (ranging between 0%-11%) and RCA (ranging between 0.1%-15%) materials. This indicates that extra attention should be paid for fines content of these materials even though none of the cases exceeded the terminal total rutting thresholds.

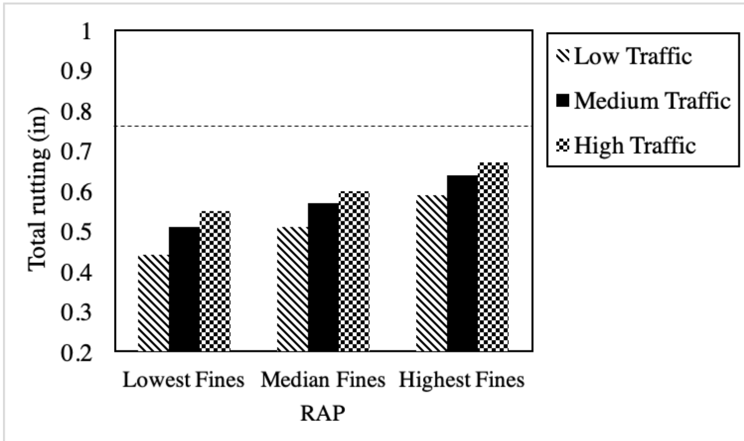

Figure 96. TOTAL RUTTING VERSUS DIFFERENT FINES CONTENT OF RAP

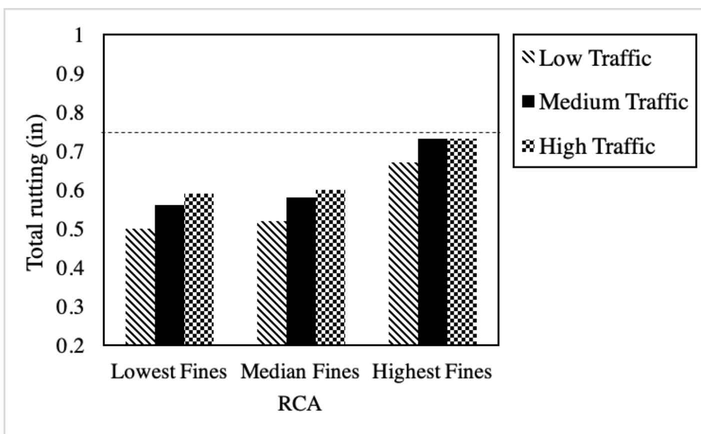

Figure 97. TOTAL RUTTING VERSUS DIFFERENT FINES CONTENT OF RCA

6.3.3. IMPACT OF GRAVEL CONTENT ON TOTAL RUTTING

Figures 98 and 99 show that RCAs and RAPs with higher gravel content resulted in higher total rutting distresses in both RAP (ranging between 3%-68.1%) and RCA (ranging between 31.8%-

94.1%). However, all the cases were below the total rutting criteria of 0.75 in (input data used for Pavement ME is shown in Tables 33 and 34).

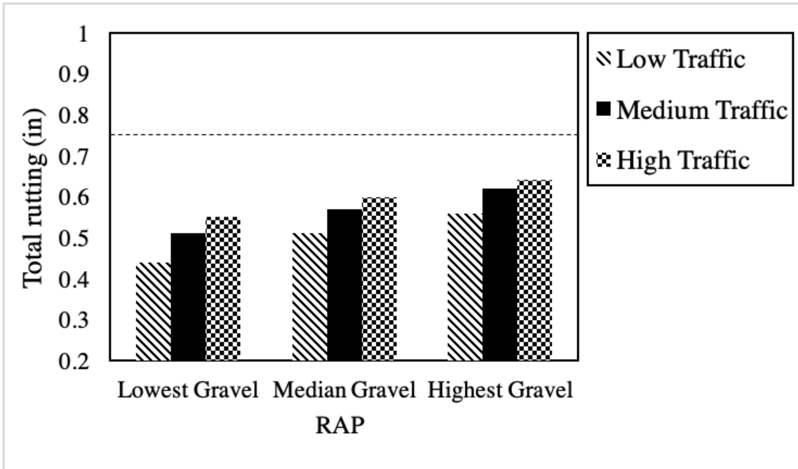

Figure 98. TOTAL RUTTING VERSUS DIFFERENT GRAVEL CONTENT OF RAP

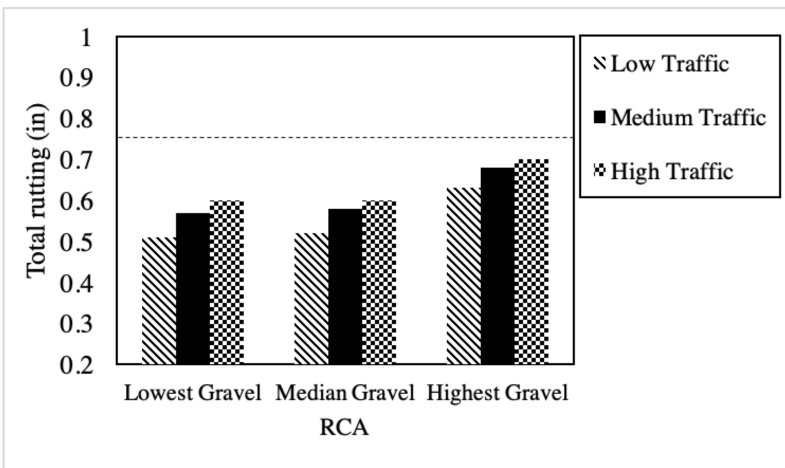

Figure 99. TOTAL RUTTING VERSUS DIFFERENT GRAVEL CONTENT OF RCA

6.3.4. IMPACT OF SAND CONTENT ON TOTAL RUTTING

Unlike gravel and fines content, both RAPs (ranging between 28.1%-97%) and RCAs (ranging between 4.9%-64.9%) with higher sand content yielded lower total rutting distresses (Figure 100 and Figure 101). In addition, all cases were below the rutting failure criteria for these analyses (input data used for Pavement ME is shown in Tables 35 and 36).

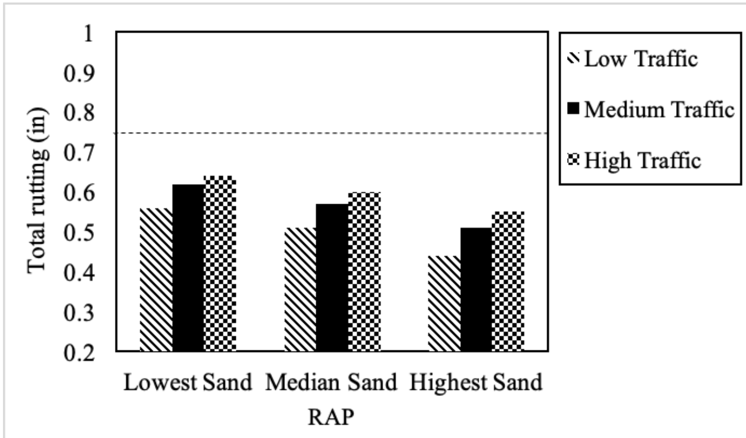

Figure 100. TOTAL RUTTING VERSUS DIFFERENT SAND CONTENT OF RAP

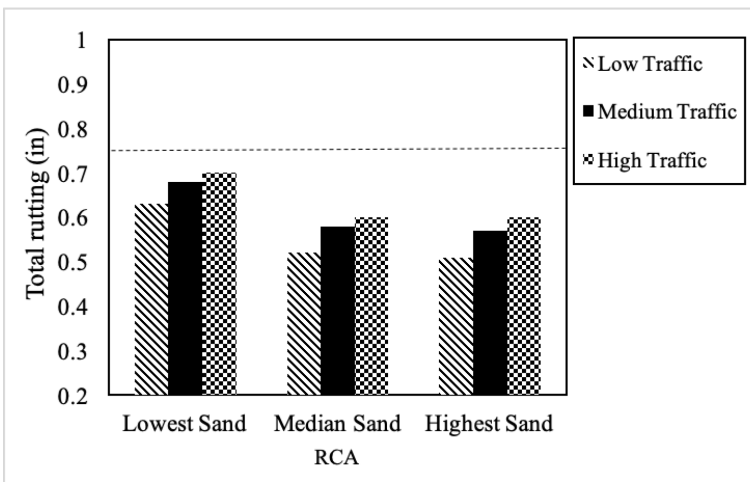

Figure 101. TOTAL RUTTING VERSUS DIFFERENT SAND CONTENT OF RCA

6.3.5. IMPACT OF D_{60} ON TOTAL RUTTING

Figure 102 shows that higher D_{60} for RAP materials tend to slightly increase total rutting of pavements (input data used for Pavement ME is shown in Table 37). On the other hand, the median

D_{60} value presented in the database yielded to the lower total rutting distress predictions for RCA material (Figure 103) (input data used for Pavement ME is shown in Table 38). Overall, both Figures 102 and 103 show that any data used from the database in the analyses resulted in total rutting values lower than that of total rutting failure criteria.

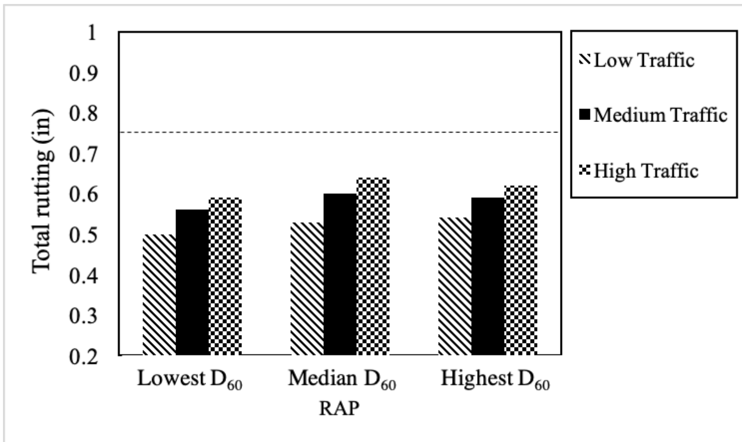

Figure 102. TOTAL RUTTING VERSUS DIFFERENT D_{60} OF RAP

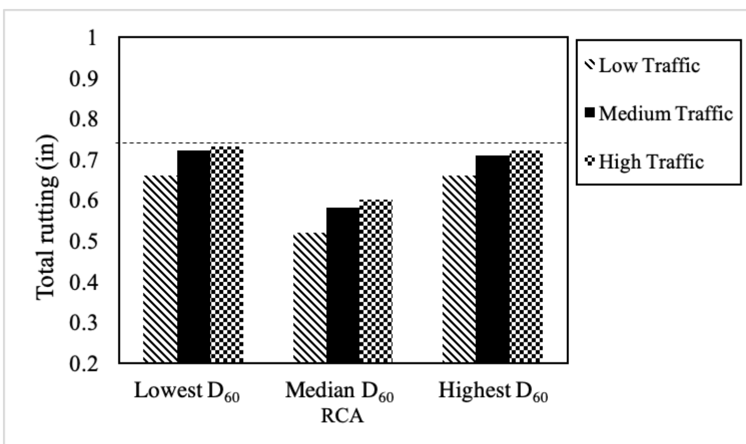

Figure 103. TOTAL RUTTING VERSUS DIFFERENT D_{60} OF RCA

6.4. MEAN JOINT FAULTING

Transverse joint faulting is one of the main types of distresses in rigid pavements affecting its serviceability. Joint faulting is defined as the difference in elevation between adjacent joints at a transverse joint and it is developed due to a combination of repeated heavy axle loads, insufficient load transfer between the adjacent slabs, free moisture in the pavement structure, and erodible base or subgrade material. When there is an excess moisture in a pavement with an erodible base or underlying fine-grained subgrade material, repeated vehicle loadings will cause the mixture of water and fines materials to be removed from beneath the leave slab corner and ejected to the surface through the transverse joint or along the shoulder. This process is called pumping which will eventually cause a void below the leave slab corner. Additionally, some of the fines that are not ejected will be deposited under the approach slab corner, making the approach slab to rise. This material building up beneath the approach corner and losing support due to a void under the leave corner can result in significant faulting at the joint (especially for rigid pavement without dowels). As mentioned above it is clear that properties of base materials may have a great impact on joint faulting distresses of rigid pavements. Therefore, sensitivity analyses were conducted to determine whether the values collected in the database provide results that are under threshold limits for joint faulting distress (0.12 inches) for rigid pavement design analyses.

6.4.1. IMPACT OF SMR ON MEAN JOINT FAULTING

Figures 104 and 105 show that SM_r of either RAP or RCA materials have any impact on joint faulting distress predictions for rigid pavements while they are directly impact by an increase in traffic volumes. (input data used for Pavement ME is shown in Tables 29 and 30).

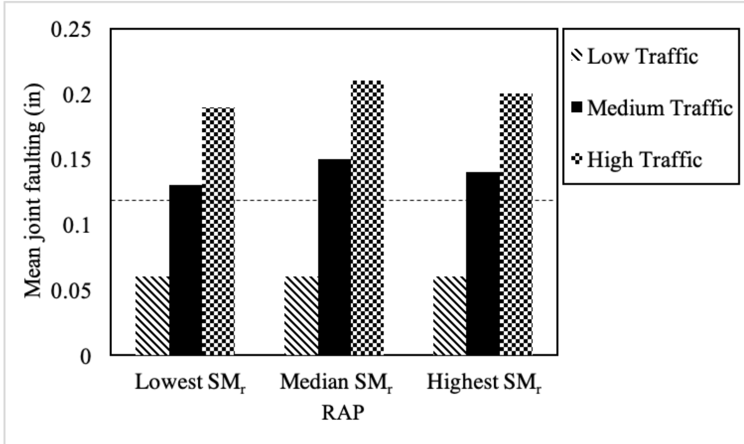

Figure 104. MEAN JOINT FAULTING VERSUS DIFFERENT SM_R OF RAP

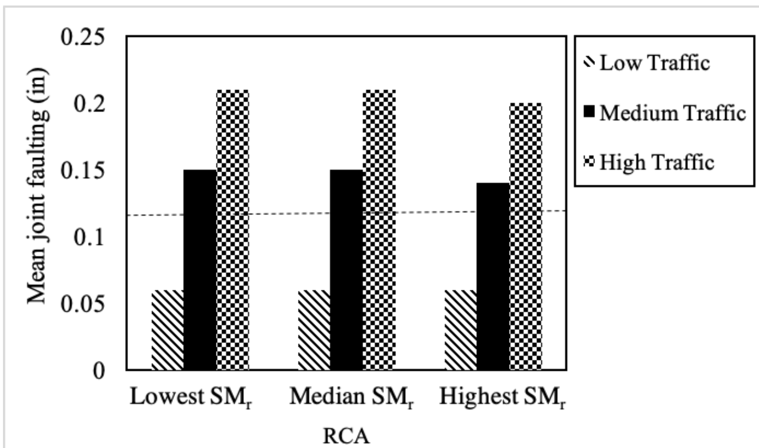

Figure 105. MEAN JOINT FAULTING VERSUS DIFFERENT SM_R OF RCA

6.4.2. IMPACT OF FINES CONTENT ON MEAN JOINT FAULTING

Figure 106 shows that RAP material with the highest fines content (11%) resulted in a slight decrease in joint faulting distresses under medium and high traffic volumes. On the other hand, Figure 107 shows that joint faulting distresses increased slightly when fines content of RCA increased from the lowest (0.1%) fines content values to medium fines content (input data used for Pavement ME is shown in Tables 31 and 32).

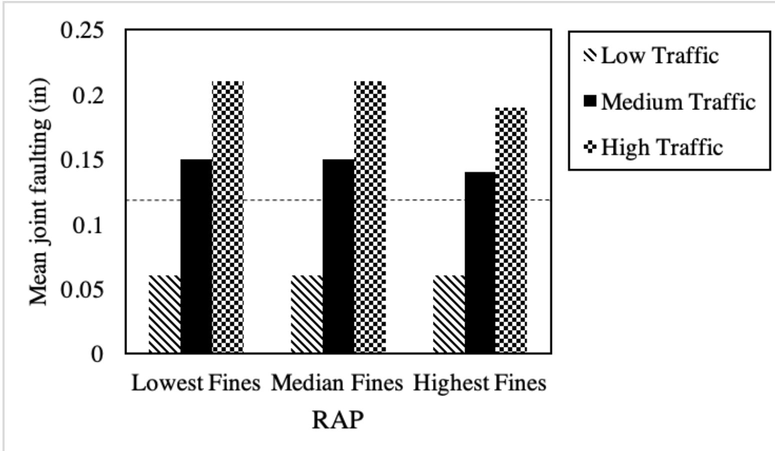

Figure 106. MEAN JOINT FAULTING VERSUS DIFFERENT FINES CONTENT OF RAP

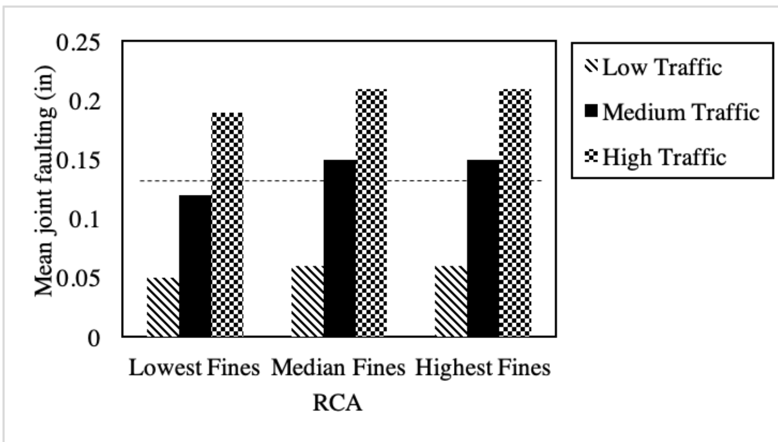

Figure 107. MEAN JOINT FAULTING VERSUS DIFFERENT FINES CONTENT OF RCA

6.4.3. IMPACT OF GRAVEL CONTENT ON MEAN JOINT FAULTING

Both Figures 108 and 109 show that using the highest gravel content for both RAP (68.1%) and RCA (94.1%) materials yielded a slight decrease in joint faulting distresses for rigid pavements under all traffic conditions (input data used for Pavement ME is shown in Tables 33 and 34).

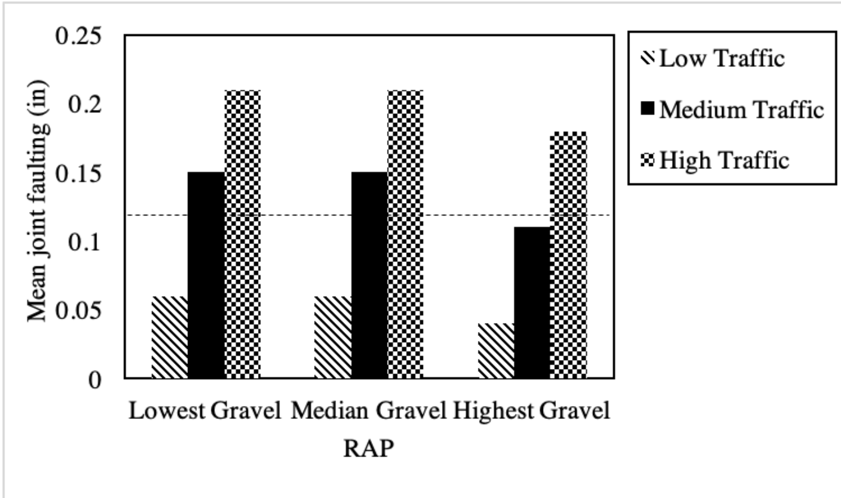

Figure 108. MEAN JOINT FAULTING VERSUS DIFFERENT GRAVEL CONTENT OF RAP

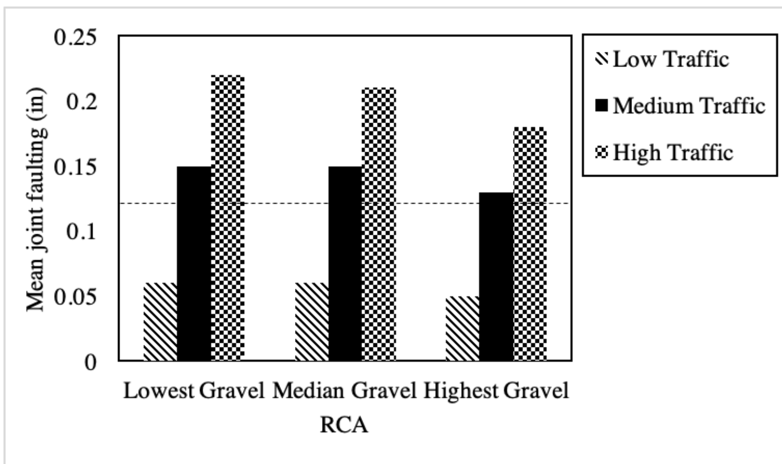

Figure 109. MEAN JOINT FAULTING VERSUS DIFFERENT GRAVEL CONTENT OF RCA

6.4.4. IMPACT OF SAND CONTENT ON MEAN JOINT FAULTING

For both RAP and RCA materials, it was observed that an increase in sand content (ranging between 28.1%-97.0% for RAP and 4.90%-64.9% for RCA) in these materials caused a consistent increase in joint faulting distresses under all traffic conditions (input data used for Pavement ME is shown in Tables 35 and 36). Figure 110 and 111 are derived from running Pavement ME using inputs from Table 35 and 36.

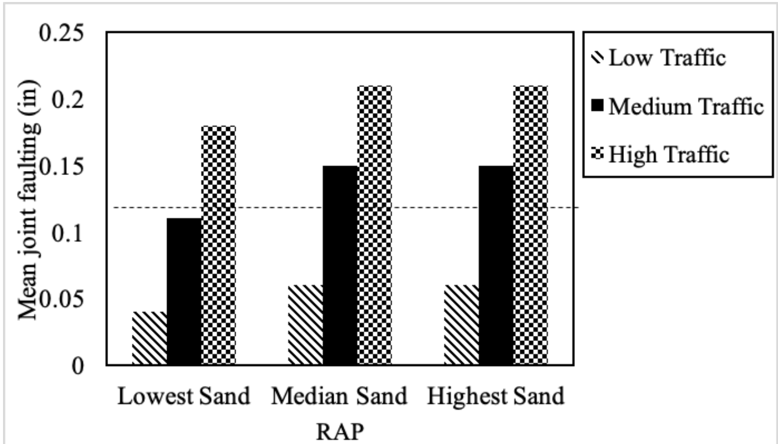

Figure 110. MEAN JOINT FAULTING VERSUS DIFFERENT SAND CONTENT OF RAP

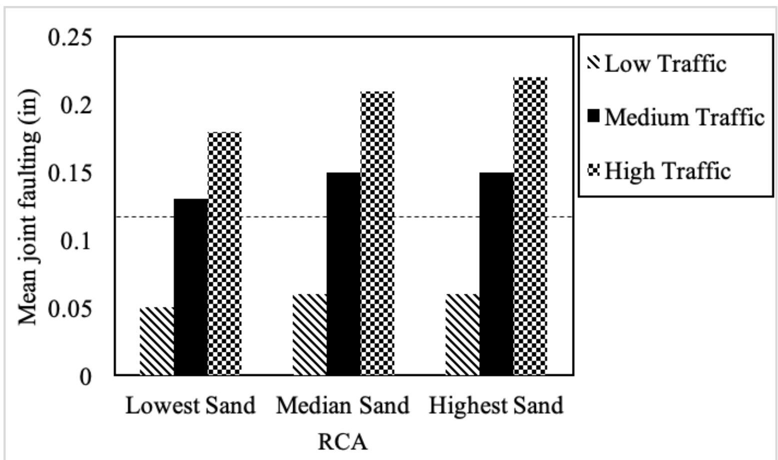

Figure 111. MEAN JOINT FAULTING VERSUS DIFFERENT SAND CONTENT OF RCA

6.4.5. IMPACT OF D_{60} ON MEAN JOINT FAULTING

Both Figures 112 and 113 show that joint faulting distresses decrease slightly when the highest D_{60} values from the database are used for both RAP (0.409 inch) and RCA (0.642 inch) materials (input data used for Pavement ME is shown in Tables 37 and 38).

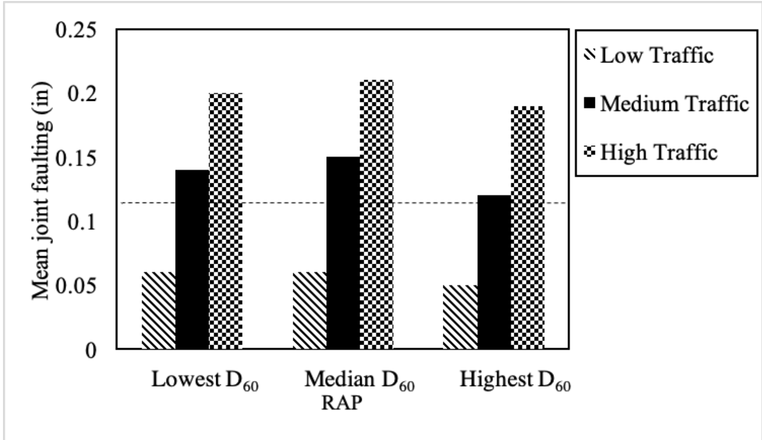

Figure 112. MEAN JOINT FAULTING VERSUS DIFFERENT D_{60} OF RAP

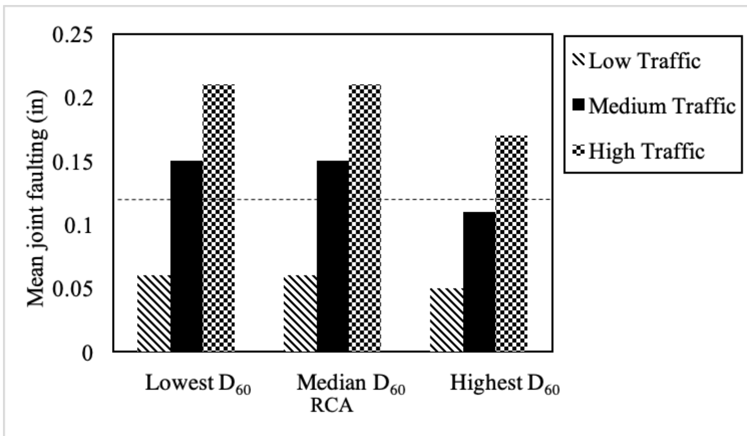

Figure 113. MEAN JOINT FAULTING VERSUS DIFFERENT D_{60} OF RCA

CHAPTER 7. CONCLUSIONS/RECOMMENDATIONS

Extensive literature review was conducted on RAP and RCA used as base or subbase materials. In addition, data for the RAP and RCA mixtures with natural aggregates was collected as well. The relationships between summary resilient modulus (SM_r), California Bearing Ratio (CBR), hydraulic conductivity, permanent deformation and index characteristics of these materials were investigated. Sensitivity analysis was conducted to determine how these input properties of the 100% RAP and RCA could affect pavement distress predictions for both flexible and rigid pavement via using AASHTOware Pavement ME software. Based on the analyses of dataset and the results of PMED the following conclusions can be drawn:

- Gravel contents of RAPs range from 3% to 68% with the median of 45% while the gravel contents of RCA are between 32 % and 94 % with the median to be 51%. Thus, RCA tends to be slightly coarser than RAP.
- Sand contents of RAP are between 28% and 97% with the median to be 54%. The lower limit of sand content is 4.9% and the upper limit of sand content is 65% in RCA with the median value of 46%.
- Fines contents in most of the RAP and RCA contents are below 12%. Fines content of RAP ranged between 0% and 11% with the median of 1% while it ranged from 0.1% to 13% with the median of 2.8% for RCA.
- Specific gravities of RAPs fall between 2.19 to 2.87 with the median of 2.395 while these values are between 2.12 and 2.7 with the median of 2.39 for the RCAs.
- Maximum dry unit weight (MDU) of RAPs ranges from 17.2 kN/m^3 (110 pcf) to 24.1 kN/m^3 (155 pcf) with the median of 19.6 kN/m^3 (126 pcf). MDU of RCAs falls between

18.3 kN/m^3 (118 pcf) and 21.7 kN/m^3 (139 pcf) with the median value to be 19.7 kN/m^3 (127 pcf).

- Optimum moisture content (OMC) of RAP and RCAs ranges between 4-10.7% and 6.1-14.8%, respectively.
- Summary resilient modulus (SM_r) of RAPs ranges from 168 MPa (24366 psi) to 400 MPa (58015 psi) with the median value of 262 MPa (37927 psi). The SM_r of RCA is between 123 MPa (17897 psi) and 370 MPa (53664 psi) with the median value of 183 MPa (26542 psi).
- The permanent deformation of 100% RAPs ranges from 1.05% to 5.63%. The permanent deformation of 100% RCAs is between 0.1% and 0.83%. RCA shows the lowest permanent deformation among RCA, RAP and natural aggregates while RAP shows the highest permanent deformation.
- CBR of RAP is between 18 and 68 with the median of 28 while CBR of RCA ranges between 58 and 169 with the median to be 146.
- Angle of friction (φ) and the cohesion (c) of 100 % RAP specimen vary from 44° to 52º and 0 kPa to 131 kPa, respectively.
- The cohesion (c) of RCAs range from 24 kPa to 191 kPa and angle of friction (φ) of RCAs range from 19° to 52.7°.
- No trend was observed between CBR and gravel-to-sand ratio (G/S) or fines content in both RCA and RAP.
- Hydraulic conductivity of RAPs falls between 1.8x10^{-7} m/s (2.12x10^{-3} ft/hr) and 1.14x10^{-3} m/s (13.46 ft/hr) with the median value to be 6.89x10^{-5} m/s (8.14x10^{-1} ft/hr). Hydraulic

conductivity of RCA ranges from $1.05\text{x}10^{-6}$ m/s ($1.24\text{x}10^{-2}$ ft/hr) to $1.2\text{x}10^{-3}$ m/s (14.17 ft/hr) with the median value to be $1.7\text{x}10^{-5}$ m/s ($2.00\text{x}10^{-1}$ ft/hr).

- It is observed that higher D_{10} values results in higher hydraulic conductivity and higher fines contents yield smaller hydraulic conductivity for RAP and RCA materials.
- SM_r of RAPs and RCAs increases with higher G/S ratio while fines contents have no effect on SM_r of RAPs and RCAs.
- There is an increasing trend in SM_r of RAPs with higher D_{30} and D_{60} values.
- RAPs with higher values of C_c and C_u will have higher SM_r.
- RCAs with higher C_u tends to have higher SM_r.
- As temperature increases SM_r of RAPs decreases except when thermal preloading is applied. On the other hand, SM_r of RCA is independent from both compaction and testing temperature conditions.
- SM_r of RAPs decrease as optimum moisture content (OMC) of RAPs increases.
- Higher permanent deformation of natural aggregates is observed with addition of RAP which needs to be considered when designing a pavement with RAP.
- As RCA content increases in RCA-natural aggregate blends, OMC increases as well while MDU of the blends decreases.
- According to PMED results, summary resilient modulus (SM_r) of base has the highest influence on the pavement performance among other material inputs for base.
- While with higher traffic volume, higher base and asphalt layer or PCC thicknesses were applied, more damage was observed with higher AADTT values.
- There is an increasing trend in total rutting with higher fines contents in RAP and RCA.
- There is a decreasing trend in rutting with higher SM_r in both RAP and RCA.

- As sand content increases, the rutting of the pavement decreases in both RAP and RCA.
- While fines content gets higher, the IRI increases as well in both RCA and RAP in flexible pavements.
- No special trend was observed in different D_{60} values with total rutting.
- No trend was observed between D_{60} and IRI in RCA and RAP in both flexible and rigid pavements.
- Mean joint faulting and IRI control the rigid pavement design located in high traffic and in some cases medium traffic volume as they always fail in all cases for both RAP and RCA.
- All cases with low traffic volume satisfy the target value defined for rigid pavements so it is safe to design JCPC pavements with RAP and RCA used as base course materials for low traffic roads without much experiment or data needed.
- All cases in flexible pavement pass the IRI and total rutting criteria. However, in some cases such as high fines content, low sand, and high gravel content for RCA in medium and high traffic areas, they come close to the target values defined.
- Overall, there is a high chance that flexible pavements with a period design of 20 years can provide adequate performance in all types of traffic volume. However, it is recommended that before designing flexible pavement, one can measure gradations and then summary resilient modulus as they have the major influence on total rutting and IRI.

The main part of the project was to collect engineering and index properties data from RAP and RCA and the whole database is summarized in the Table 41.

Table 41. SUMMARY OF DATABASE

Characteristics	**RAP**			**RCA**		
	Lower Limit	**Median**	**Upper Limit**	**Lower Limit**	**Median**	**Upper Limit**
% Gravel	3 (52)	45 (52)	68.1 (52)	31.8 (34)	51 (34)	94.1 (34)
% Sand	28.1 (52)	54 (52)	97 (52)	4.9 (34)	46.3 (34)	64.9 (34)
% Fines	0 (52)	1 (52)	11 (52)	0.1 (34)	2.8 (34)	12.8 (34)
D_{10} (mm/in)	10^{-1}/ $3.9x10^{-3}$ (30)	$5x10^{-1}$/ $1.96x10^{-2}$ (30)	1/ $3.93x10^{-2}$ (30)	10^{-1}/ $3.9x10^{-3}$ (19)	$2.3x10^{-1}$/ $9x10^{-3}$ (19)	$4.3x10^{-1}$/ $1.7x10^{-2}$ (19)
D_{30} (mm/in)	$8x10^{-2}$/ $3.1x10^{-3}$ (27)	1.5/ $6x10^{-2}$ (27)	4.9/ $1.9x10^{-1}$ (27)	$2x10^{-1}$/ $7.9x10^{-3}$ (17)	1.2/ $4.72x10^{-2}$ (17)	6.5/ $2.56x10^{-1}$ (17)
D_{60} (mm/in)	$1.5x10^{-1}$/ $5.9x10^{-3}$ (27)	4.82/ $1.89x10^{-1}$ (27)	10.4/ $4.09x10^{-1}$ (27)	$6x10^{-1}$/ $2.36x10^{-2}$ (17)	6.8/ $2.67x10^{-1}$ (17)	16.3/ $6.42x10^{-1}$ (17)
C_u	5 (35)	10.65 (35)	40 (35)	2.1 (29)	32 (29)	66 (29)
C_c	0.21 (37)	1.2 (37)	8 (37)	0.14 (29)	1.4 (29)	6 (29)
G_s	2.19 (38)	2.395 (38)	2.87 (38)	2.12 (32)	2.39 (32)	2.7 (32)
MDU (kN/m^3/pcf)	17.2/ 110 (46)	19.61/ 126 (46)	24.12/ 155 (46)	18.3/ 118 (35)	19.7/ 127 (35)	21.7/ 140 (35)
OMC(%)	4 (46)	6.05 (46)	10.7 (46)	6.1 (35)	10.8 (35)	14.8 (35)
SM_r (MPa/psi)	168/ 24366.3 (32)	261.5/ 37927.368 (32)	400/ 58015.1 (32)	123.4/ 17897.657 (18)	183/ 26541.9 (18)	370/ 53664 (18)
CBR (%)	18 (12)	28 (12)	68 (12)	58 (4)	146 (4)	169 (4)
Hydraulic conductivity (m/s/ft/hr)	$1.8x10^{-7}$/ $2.12x10^{-3}$ (23)	$6.89x10^{-5}$/ $8.14x10^{-1}$ (23)	$1.14x10^{-3}$/ 1.35x10 (23)	$1.05x10^{-6}$/ $1.24x10^{-2}$ (12)	$1.7x10^{-5}$/ $2.01x10^{-1}$ (12)	$1.2x10^{-3}$/ 1.42x10 (12)

Notes: Numbers provided in parantheses for each data represent the corresponding sample size

APPENDICES

APPENDIX A.

TABLE 42. GRADATION CHARACTERISTICS OF RCA-RAP DATABASE

Ref	Loc	Type of Material	Gravel (%)	Sand (%)	Fine (%)	Classification - USCS	Classification - AASHTO	D_{10} (mm)	D_{30} (mm)	D_{60} (mm)	C_u	C_c	G_s
Edil et al. (2012a)	MN	Aggregate Class 5	22.9	67.6	9.5	GW-GM	A-1-b	0.1	0.4	1.7	21	1.4	2.57
		Blend	32.7	63.8	3.4	SP	A-1-b	0.2	0.6	2.8	13	0.5	
		RAP	26.3	71.2	2.5	SP	A-1-a	0.3	0.7	2.3	7	0.7	2.41
		RCA	31.8	64.9	3.3	SW	A-1-a	0.1	0.4	1.7	21	1.4	2.39
	MI	RCA	68.5	28.3	3.2	GP	A-1-a	0.4	4.1	12.3	35	3.9	2.37
		RPM	49.3	50.4	0.4	SW	A-1-b	0.4	1.7	6.5	17	1.1	2.39
	CO	RCA	40.9	46.3	12.8	SC	A-1-b	0.1	0.6	4.9	66	1.1	2.28
		RAP	31.7	67.7	0.7	SP	A-1-a	0.4	0.9	3.3	9	0.7	2.23
	CA	RCA	50.6	47.1	2.3	GW	A-1-a	0.3	1.7	6.8	22	1.4	2.32
		RAP	36.8	61.4	1.8	SW	A-1-a	0.3	1.3	4.2	13	1.2	2.56
	TX	RCA	76.3	21.6	2.1	GW	A-1-a	0.4	6.5	16.3	38	6	2.27
		RAP	41	44.9	1	SW	A-1-a	0.7	2.5	7.9	11	1.1	2.34
	OH	RCA	43.2	49.5	7.3	SW-SM	A-1-a	0.2	1.2	5.3	34	1.7	2.24
		RAP	32.1	66.2	1.7	SW	A-1-a	0.5	1.6	3.8	7	1.3	2.43
	NJ	RCA	41.2	54.6	4.3	SP	A-1-b	0.2	0.5	5.1	28	0.3	2.31
		RAP	50.9	48.4	0.7	GW	A-1-a	1	2.8	5.9	6	1.3	2.37
		RMP	55.7	43.6	0.6	GW	A-1-b	0.5	2.1	8.7	18	1	2.35
	WI	RAP	30.9	68.5	0.5	SP	A-1-b	0.6	1.4	3.6	6	0.9	2.37

Appendix A. (cont'd)

Edil et al. (2012b)	WI	RPM	46	43	11	GW-GM	A-1-a						
Edil et al. (2012c)	MN	RPM	40	52	8	SW-SM	A-1-a						
Tutumluer et al. (2015)	IL	blend	73	25	2	GW		1.2	4.9	20	16.6	1	
		RAP	49	50	1	SW		0.9	2.8	5.5	6.1	1.5	
Locander (2009)	CO	RAP	55	43.6	1.4	GW-GP							2.25
			64	35.1	0.9	GW-GP							2.36
			54	43.6	2.4	GW-GP							2.3
			59	40.1	0.9	GW-GP							2.33
			45	54.4	0.6	SW-SP							2.39
			56	43	1	GW-GP							2.39
			59	40.2	0.8	GW-GP							2.37
			59	40	1	GW-GP							2.34
			67	32.2	0.8	GW-GP							2.36
			67	31.8	1.2	GW-GP							2.26
			75	24.1	0.9	GW-GP							2.29
Mokwa and Peebles (2005)	MT	CBC#1 unmixed	52.46	41.58	5.96	GW-GP	A-1-a (6A)						2.67
		CBC#1 20%RAP	54.98	42.39	1.82	GW-GP	A-1-a(5A)						2.67
		CBC#1 50%RAP	49.28	49.01	1.71	SW-SP	A-1-a(5A)						2.59

Appendix A. (cont’d)

		CBC#2 unmixed	55.8	41.59	2.61	GW-GP	A-1-a (6A)						2.7
		CBC#2 20%RAP	54.35	43.55	2.1	GW-GP	A-1-a (6A)						2.66
		CBC#2 50%RAP	53.74	42.37	1.7	GW-GP	A-1-a(5A)						2.59
		CBC#3 unmixed	55.5	39.35	5.15	GW-GP	A-1-a(5A)						2.68
		CBC#3 20%RAP	52.31	45.68	2.01	GW-GP	A-1-a(5A)						2.66
		CBC#3 50%RAP	58.48	40.09	1.43	GW-GP	A-1-a(5A)						2.59
		Pitrun unmixed	41.79	40.74	1.05	SP	Spec. Borrow	0.4	1.6	17	42.5	0.37	2.72
		Pitrun 20%RAP	57.66	38.23	1.62	GP	Spec. Borrow	0.4	2	15	37.5	0.66	2.63
		Pitrun 50%RAP	53.08	38.04	1	GW	Spec. Borrow	0.53	2.5	12	22.6	0.98	2.61
Saeed (2008)	FL	FL RAP unprocessed				GW/SW	A-1-a	0.28-0.32	1.3-2	5.1-6	17.1	1.2-2.2	
		FL RAP Hammermill				SW	A-1-a	0.35	1.9	3.75-5	10-14.3	1.5-2.1	
		FL RAP Tubgrinder				SP	A-1-a	0.35	0.9	5	14-14.3	0.5	
Ullah and Tanyu (2019)	VA	Virgin aggregate	45	43	12	SM-SC			0.7	7			2.95
		RAP1 (Plagioclase and Pyroxene)	46	53	1	SW		0.5	2	5.1	10.2	1.5	2.85
		20%RAP1	45	45.6	9.4	SW-SP							
		30%RAP1	44	47.8	8.2	SW-SP							

Appendix A. (cont'd)

		40%RAP1	45	47.8	7.2	SW-SP							
		50%RAP1	46	47.8	6.2	SW-SP							
		60%RAP1	46	48.8	5.2	SW-SP							
		RAP2 (Plagioclase and Pyroxene)	39	60	1	SW		0.5	1.5	4.5	9	1	2.82
		RAP5 (Plagioclase and Pyroxene)	26	73	1	SW		0.32	1.1	3	9.3	1.26	2.87
		RAP11 (Muscovite, Quartz, Biotite and Amphibo)	42	57	1	SW		0.5	1.7	5	10	1.1	2.6
Bennert et al. (2000)	NJ	DGABC	60	33	7	GW		0.18	2.1	9	50	2.7	
		RAP	60	59	1	GW		1	3.1	8	8	1.2	
		RCA	60	56	4	GW		0.18	1.5	11	61	1.1	
Kim et al. (2005)	MN	100% aggregate CR 3	17	74.5	8.5	SW-SP		0.14	0.42	2.6	18.5		
		25% RAP from CR 3	27	67	6	SW-SP		0.19	0.85	3.5	18.4		
		50% RAP from CR 3	35	61.5	3.5	SW		0.36	2.3	4.3	11.9		

Appendix A. (cont'd)

		75% RAP from CR 3	40	58	2	SW		0.7	2.7	4.9	7		
Huang and Dong (2014)	TN	RAP	41	58	1	SW-SP							
Mijic et al. (2019)	MD	RAP 1	46.3	51.8	1.83	SW	A-1-a				14	1.79	2.25
		RAP2	37.8	61.3	0.93	SW	A-1-a				10.6	1.26	2.36
		RAP3	45.7	54.1	0.13	SP	A-1-a				5.6	1.03	2.25
		RAP4	40.7	59	0.33	SW	A-1-a				8.28	1.58	2.44
		RAP5	44	54.8	1.19	SW	A-1-a				11.7	1.36	2.29
		RAP6	45.3	54.2	0.47	SW	A-1-a				11.2	1.32	2.48

Appendix A. (cont'd)

		RAP7	47.6	52	0.39	SW	A-1-a				6.87	1.26	2.4
Ullah et al. (2018)	VA	RAP 1 as is	45	53.5	1.5	SW	A-1-a				10.65	1.43	2.43
		RAP 2 as is	40	57.8	2.2	SW	A-1-a				9	1.36	2.6
		Virgin aggregate as is	46	42	12	SW-SM	A-1-a				93	1.1	2.85
		Virgin aggregate Eng.	48	45.7	6.5	SW-SM	A-1-a				31	2.6	2.81
Edil et al. (2017)	MN	Natural aggregate	22.9	67.6	9.5	GW-GM	A-1-b				21	1.4	2.57
		RCA	31.8	64.9	3.3	SW	A-1-a				21	1.4	2.39
		RCA blend	32.7	63.8	3.4	SP	A-1-b				13	0.5	

Appendix A. (connt'd)

		RAP	26.3	71.2	2.5	SP	A-1-a				7	0.7	2.41
Hasan et al. (2018)	NM	Subgrade soil	4	91.5	4.5	SW	A-2-6	0.2	0.8	1.8	9	1.7	
		RAP	48	51.7	0.3	SP		0.5	0.98	9	18	0.2	
		30% RAP	44.8	50.7	4.50	SP		0.4	0.9	9	22.5	0.2	
Puppala et al. (2012)	TX	RAP	48	48	4	GP					5	0.98	
Soleimanbeigi and Edil (2015a)	WI	RAP	20	78	2	SW							2.39
Camargo et al. (2013)	WI	RPM	46	43	11	GW-GM							

Appendix A. (cont’d)

Soleimanbeigi et al.(2015)	CA	RCA			2.3		A-1-a	0.31			22	1.4	2.32
	TX				2.1		A-1-a	0.43			38	6	2.27
	NJ				4.3		A-1-b	0.18			28	0.3	2.31
	MI				3.2		A-1-a	0.4			35	3.9	2.37
	CO	RAP			0.7		A-1-a	0.35			9	0.7	2.23
	TX				1		A-1-a	0.72			11	1.1	2.34
	NJ				0.7		A-1-a	1			6	1.3	2.37
	MN				2.5		A-1-a	0.3			7	0.7	2.41

Appendix A. (cont'd)

Kang et al (2011)	MN	25%RCM	40	59	1	SP		0.6	1	5	8.3	0.3	
		50% RCM	41	58	1	SP		0.5	0.9	5	10	0.3	
		75% RCM	42	57	1	SP		0.42	0.9	5	11	0.3	
		100% RCM	48	51	1	SP		0.4	0.8	7	17.5	0.22	
Attia and Abdelrahman (2010a)	MN	RAP Trunk highway 10	51	48.6	0.4	GP	A-1-b	0.6	2	7	11.7	0.95	
		RAP TH 19-MM 101 field 50-50	22	76.6	1.4	SP	A-1-b	0.32	0.6	1.8	5.6	0.6	
		RAP TH 19-MM 104 field 50-50	24	73.9	2.1	SP	A-1-b	0.25	0.6	2	8	0.72	
		RAP TH 22 field 50-50	41	57.7	1.3	SP	A-1-b	0.42	1.3	5	11.9	0.8	

Appendix A. (cont'd)

		50% RAP TH 10 +50% Class 5 lab	41.5	56.85	1.65	SP	A-1-b	0.32	0.95	5	15.6	0.56		
		75% RAP TH 10+25% Class 5 lab	46.25	52.72	1.03	SP	A-1-b	0.4	1.3	6.5	16.2	0.065		
Guthrie et al. (2007)	UT	RAP 1	45	46.5	8.5	SW-SM	A-1-a	0.127	0.889	5.08			2.47
		RAP 2	45	54	1	SW	A-1-a	0.508	1.651	4.826			2.47
		Base1	55	35.5	9.5	GWGM	A-1-a	0.08382	1.016	9.652			2.64
		Base2	44	46.5	9.5	SP-SM	A-1-a	0.08382	1.27	4.826			2.68
Bradshaw et al (2016)	RI	RAP1, 23%	3	97	0	SW-SP	A-1-a						
		RAP2, 14%	3	97	0	SW-SP	A-1-a						

Appendix A. (cont'd)

		RAP3, 23%	9	91	0	SW-SP	A-1-a						
		RAP FDR no treat	7	93	0	SW-SP	A-1-a						
		RAP4, 26%	5	95	0	SW-SP	A-1-a						
		Rap 5, 19%	15	85	0	SW-SP	A-1-a						
		RAP 6 , 39%	8	92	0	SW-SP	A-1-a						
Bennert and Maher (2005)	NJ	RAP	49	50.9	0.1	SW		0.516	0.08	0.15	10.85	1.22	
		RCA	71	26.2	2.8	GW		0.29	0.2	0.6	52.95	4.71	

Appendix A. (cont'd)

Bestgen et al. (2016)	Eastern USA	G1					A-1-a	0.1	1.8	10			
		G2					A-1-a(0)	0.05	0.3	5			
		G3					A-1-a(0)	0.08	1	10			
		G4					A-1-a(0)	0.1	0.3	6.8			
		RCA 1	45	45	10	SP	A-1-a(0)	0.11	0.6	6.5	59	0.5	
		RCA 2	40	55	5	SP	A-1-a(0)	0.11	0.28	5	45	0.14	
Tutumluer et al. (2012)	IL	RCA	55	37	8	GP		0.23	2.5	7.5	32	3.6	2.41
		75% RCA	55	36	9	GP		0.1	2.5	7.5	75	8.3	
		50% RCA	55	35	10	GP		0.075	2.5	7.5		11	
Natarajan et al. (2019)	MN	RCA				GW							2.7
		RCA Passing lane	55	43	2	GW		0.4	1.9	8			2.26
		RCA Center line	37	61	2	GW		0.35	0.8	4			2.13
		RCA Driving lane	52	46	2	GW		0.32	1.4	8			2.5
Mahedi and Cetin (2020)	TX	RCA1	93.4	5.8	0.8	GP	A-1-a				2.1	1.1	2.44
		RCA2	68.8	31.1	0.1	GP	A-1-a				32	3.6	2.41
	IA	RCA1	48.8	51.1	0.1	SP	A-1-a				7.9	0.6	2.33
		RCA 2	82	17.8	0.2	GW	A-1-a				7.6	1.8	2.36
	MN	RCA	94.1	4.9	1	GP	A-1-a				2.1	1.4	2.12

Appendix A. (cont'd)

Chen et al. (2013)	CA	RCA				SP							2.6
	CO	RCA				SM							2.6
	MI	RCA				GP							2.7
	MN	RCA				SP							2.7
	TX	RCA				GP-GM							2.6
	WI Fresh	RCA				GP							2.7
	WI Stock Pile	RCA				SP							2.6
Diagne et al. (2015)	WI	RCA	51	47.2	1.8	GW		0.17	1.2	7	41.67	1.25	2.41
Cetin et al. (2020)	MN	Coarse RCA	61.7	34.9	3.4	GW	A-1-a				34.49	1.75	2.64
		Fine RCA	38.3	54.6	7.1	SW-SM	A-1-a				33.93	1.12	2.64
		RCA+ RAP	41	50.4	8.6	SP-SM	A-1-a				49.41	0.98	2.52
Wu et al. (2012)	WA	RAP	67	32	1	GP		0.45	4.9	10.4	23	5.13	

Appendix A. (cont'd)

Alam et al. (2010)	MN	RAP 100%	4	96	0	SP-SW						
Cosentino et al. (2012)	FL	APAC Melbourne Crushed	24.2	75.2	0.6	SP	A-1-a	0.3	0.91	3.1	10.7	0.9	2.508
		APAC Melbourne Milled	41.9	57.6	0.5	SW	A-1-a	0.5	2	5	9.6	1.9	2.524
		Whitehurst Gainesville Milled	54	45.6	0.4	SP	A-1-a	0.4	1.5	4.8	11.2	0.8	2.576
		APAC Jacksonville Crushed	26.6	66.6	6.8	SP	A-1-b	0.1	0.3	3	26.2	0.4	2.604
		75% milled mel and LR	43	56	1			0.39	2	5			
		50% milled Melbourne and 50% LimeRock	45	53	2			0.3	1.8	6			

Appendix A. (cont'd)

		25% milled Melbourne and 75% LimeRock	50	47	3			0.2	1.3	7.1			
Cosentino et al. (2003)	FL	100% RAP modified	40		0.9	SP	A-1-a	0.27	0.65	4.7	17	0.3	2.19
		80% RAP-20% fine sand			3.1		A-1-b	0.17	0.35	3.3	19	0.2	2.25
		60% RAP-40% fine sand			4			0.15	0.25	0.62	4.1	0.7	2.37
Kim and Labuz (2007)	MN	25% RAP from CR 3	28	66	6			0.2	0.85	3.5			
		50% RAP from CR 3	36	60	4			0.35	2.3	4.3			

Appendix A. (cont'd)

		75% RAP from CR 3	40	58	2			0.7	2.7	4.9			
Bejarano (2001)	CA	RAP	54	45	1			0.46	2.1	7			
Garg and Thompson (1996)	IL	RAP	68.1	28.1	3.8								
Ba et al. 2013	CO, TX	TX RAP	54	45.01	0.99	SW		0.8	2.5	8			2.34
		CO RAP	31	68.31	0.69	SP		0.4	0.9	3.1			2.23

Notes: CBC= Crushed base aggregate; DGABC= Dense graded aggregate base course ; CR= County Road ; RPM= Recycled pavement material ; RCM= Recycled concrete material; RAP TH= RAP Trunk Highway ; RAP FDR= Full-depth reclamation. Pyroxene is a group of important rock-forming silicate minerals of variable composition including calcium-, magnesium-, and iron-rich varieties predominate, while Plagioclase contains calcium and sodium and is a mixture of albite (Ab), or sodium aluminosilicate ($NaAlSi_3O_8$), and anorthite (An), or calcium aluminosilicate ($CaAl_2Si_2O_8$).

APPENDIX B.

TABLE 43. RAP DATABASE

Ref.	Loc.	Gravel (%)	Sand (%)	Fine (%)	D_{10} (mm)	D_{30} (mm)	D_{60} (mm)	SM_r (MPa)	CBR	Density (kN/m^3)	OMC (%)	HC (m/s)
Edil et al. (2012a)	MN	26.3	71.2	2.5	0.3	0.7	2.3	180		20.8	6.7	0.0000011
	MI	49.3	50.4	0.4	0.4	1.7	6.5	168		21.5	5.2	0.000231
	CO	31.7	67.7	0.7	0.4	0.9	3.3	184		20.7	5.7	0.0000382
	CA	36.8	61.4	1.8	0.3	1.3	4.2	173		20.7	6.1	
	TX	41	44.9	1	0.7	2.5	7.9	198		20.3	8	0.0000318
	OH	32.1	66.2	1.7	0.5	1.6	3.8	197		19.8	8.8	0.0000503
	NJ	50.9	48.4	0.7	1	2.8	5.9	209		20.4	6.5	0.000369
	WI	30.9	68.5	0.5	0.6	1.4	3.6	266		20	7.3	0.0000519
Edil et al. (2012b)	WI	46	43	11				310		21.2	7.5	
Edil et al. (2012c)	MN	40	52	8				257	19	20.04	4.9	
Locander (2009)	CO	64	35.1	0.9				239.64		19.35	7.2	7.7x10-4
		59	40.1	0.9				211.8		19	10.7	0.00074
		59	40	1				181.13		18.8	8.8	0.00073

Appendix B. (cont'd)

Bennert et al. (2000)	NJ, RAP	60	59	1	1	3.1	8	300.3		18.35	5	
Huang and Dong (2014)	TN	41	58	1				286.5		18.73	7.95	
Puppala et al. (2012)	TX	48	48	4				251		21.3	6	
Soleimanbeigi and Edil (2015a)	WI	20	78	2				390		18.7	5	
Camargo et al. (2013)	WI	46	43	11				309	22	21.2	7.5	
Soleimanbeigi et al. (2015)	CO			0.7	0.35			255		20.7	5.7	
	TX			1	0.72			345		20.3	8.1	
	NJ			0.7	1			280		20.4	6.5	

Appendix B. (cont'd)

Attia and Abdelrahman (2010a)	MN	51	48.6	0.4	0.6	2	7	380		20.82	5.5	
Attia and Abdelrahman (2010b)	MN	51	48.6	0.4	0.6	2	7	380		20.82	5.5	
Bennert and Maher (2005)	NJ	49	50.9	0.1	0.516	0.08	0.15	268	18			0.00004865
Wu et al. (2012)	WA	67	32	1	0.45	4.9	10.4	200				
Guthrie et al. (2007)	UT RAP 1	45	46.5	8.5	0.127	0.889	5.08		21	20.26	5.6	
	UT RAP 2	45	54	1	0.508	1.651	4.826		22	18.22	5.75	
Hasan et al. (2018)	NM	48	51.7	0.3	0.5	0.98	9					
Alam et al. (2010)	MN	3	97	0				271.3				

Appendix B. (cont'd)

Cosentino et al. (2003)	FL	40	59.1	0.9	0.27	0.65	4.7	291.39	32	18.5	8.2	0.000002
Bejarano (2001)	CA	54	45	1				310		24.12	5.5	
Garg and Thompson (1996)	IL	68.1	28.1	3.8				218.58		21.04	6	
Mijic et al. (2019)	MD RAP 1	46.3	51.8	1.83						19.6	5.7	0.0000983
	MD RAP2	37.8	61.3	0.93						18.5	6.8	0.000566
	MD RAP3	45.7	54.1	0.13						17.2	6.3	0.00114
	MD RAP4	40.7	59	0.33						18.7	6.8	0.000251
	MD RAP5	44	54.8	1.19						19.2	7.5	0.0000689
	MD RAP6	45.3	54.2	0.47						19.1	6.4	0.000201
	MD RAP7	47.6	52	0.39						18.5	8.2	0.000527
Ullah and Tanyu (2019)	VA RAP1	46	53	1	0.5	2	5.1					
	VA RAP2	39	60	1	0.5	1.5	4.5					
	VA RAP 5	26	73	1	0.32	1.1	3					

Appendix B. (cont'd)

	VA RAP 11	42	57	1	0.5	1.7	5						
Ullah et al. (2018)	VA RAP 1 as is	45	53.5	1.5						19	5.5		
	VA RAP 2 as is	40	57.8	2.2						19.47	5.5		
Edil et al. (2017)	MN	26.3	71.2	2.5									
Ba et al. 2012	TX RAP	54	45.01	0.99	0.8	2.5	8						
	CO RAP	31	68.31	0.69	0.4	0.9	3.1						
Cosentino et al. (2012)	APAC Melbourne Crushed	24.2	75.2	0.6	0.3	0.91	3.1		62.4	19.16	5		
	APAC Melbourne Milled	41.9	57.6	0.5	0.5	2	5		60	19.03	6.2	0.000031	

Appendix B. (cont'd)

	Whitehurst Gainesville Milled	54	45.6	0.4	0.4	1.5	4.8		60	19.08	4	0.0000013
	APAC Jacksonville Crushed	26.6	66.6	6.8	0.1	0.3	3		68	19.63	4.5	0.00000018
Kang et al. (2011)	MN							193		20.79	4	0.000022152
Abdelrahman and Noureldin (2014)	MN							330		20.82	5.5	
Attia and Abdelrahman (2011)	MN							400				

APPENDIX C.

TABLE 44. RCA DATABASE

Ref.	Loc.	Gravel (%)	Sand (%)	Fine (%)	D_{10} (mm)	D_{30} (mm)	D_{60} (mm)	SM_r (MPa)	CBR	Density (kN/m^3)	OMC (%)	HC (m/s)
Edil et al. (2012a)	MN	31.8	31.8	3.3	0.1	0.4	1.7	189		19.5	11.2	
	MI	68.5	28.3	3.2	0.4	4.1	12.3	171		20.8	8.7	
	CO	40.9	46.3	12.8	0.1	0.6	4.9	175		18.9	11.9	
	CA	50.6	47.1	2.3	0.3	1.7	6.8	178		19.9	10.4	
	TX	76.3	21.6	2.1	0.4	6.5	16.3	164		19.7	9.2	
	OH	43.2	49.5	7.3	0.2	1.2	5.3	163		19.4	11.8	
	NJ	41.2	54.6	4.3	0.2	0.5	5.1	208		19.8	9.5	
Bennert et al. (2000)	NJ	60	56	4	0.18	1.5	11	297.6		19.45	7.5	
Soleimanbeigi et al. (2015)	Tx			2.1	0.43			188		19.7	9.2	
	Nj			4.3	0.18			160		19.8	9.5	
	CA			2.3						19.9	10.4	
	MI			3.2						20.8	8.7	
Kang et al. (2011)	MN RCM	48	51	1	0.4	0.8	7	164.8445312		19.02	9.4	0.000010212
Bennert and Maher (2005)	NJ	71	26.2	2.8	0.29	0.2	0.6	272.9	169			0.00000105

Appendix C. (cont'd)

Bestgen et al. (2016)	Eastern USA RCA 1	45	45	10	0.11	0.6	6.5	295	148	20.2	9.5	
	Eastern USA RCA 2	40	55	5	0.11	0.28	5	220	144	20.1	9.5	
Tutumluer et al. (2012)	IL	55	37	8	0.23	2.5	7.5	188	58	20	9.3	
Diagne et al. (2015)	WI	51	47.2	1.8	0.17	1.2	7	370		20.9	6.1	0.00003
Cetin et al. (2020)	Coarse RCA	61.7	34.9	3.4				127.3770752		19.31	11.3	2.67E-06
	Fine RCA	38.3	54.6	7.1				123.3858925		19.1	11.1	4.85E-06
Mahedi and Cetin (2020)	TX RCA 1	93.4	5.8	0.8						19	10.9	
	TX RCA 2	68.8	31.1	0.1						19.7	14.4	

Appedix C. (cont'd)

	IA RCA 1	48.8	51.1	0.1						19	14.8	
	IA RCA 2	82	17.8	0.2						18.4	14.3	
	MN RCA	94.1	4.9	1						18.3	12.6	
Natarajan et al. (2019)	MN RCA									19.5	11.2	
	MN RCA Passing lane	55	43	2	0.4	1.9	8			21.4	12	
	MN RCA Center line	37	61	2	0.35	0.8	4			21	11.7	
	MN RCA Driving lane	52	46	2	0.32	1.4	8			21.7	13.5	
Chen et al. (2013)	CA	50	47	3						19.8	10.9	0.000019
	CO	41	44	15						18.9	11.9	0.000016
	MI	69	28	3						20.8	8.7	0.000026
	MN	32	64	4						19.5	11.2	0.000018
	TX	76	21	3						19.7	9.2	0.000008

Appendix C. (cont'd)

	WI Fresh	48	50	2						19.4	10.8	0.0012
	WI Stockpile	65	32	3						19.9	9.9	0.00071
Edil et al. 2017	MN	31.8	64.9	3.3								

REFERENCES

REFERENCES

AASHTO (1993). Guide for Design of Pavement Structures, American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO (2002). Standard Specifications for Highway Bridges, American Association of State Highway and Transportation Officials, Washington, DC.

Abdelrahman, M., and Noureldin, E. (2014). Parametric Analysis of Resilient Modulus Modeling for Recycled Asphalt Pavement in Base Layer. *Transportation Research Record*, volume 2401, issue 1, pp 30-43.

Aggregate Subgrade Improvement, Provision of Illinois DOT specification of Section 5.2 http://www.idot.illinois.gov/Assets/uploads/files/Doing-Business/Specialty-Lists/Highways/Design-&-Environment/BDE-Special-Provisions/80274.pdf

ASTM D8038-16 (2016). Standard Practice for Reclamation of Recycled Aggregate Base (RAB) Material, *ASTM International*, West Conshohocken, PA.

Alam, T., Abdelrahman, M., and Schram, S. (2010). Laboratory Characterization of Recycled Asphalt Pavement As A Base Layer. *The International Journal of Pavement Engineering*, volume 11, issue 2, pp 123–131.

Attia, M. (2010). Characterization of the Structural Behavior of Reclaimed Asphalt Pavement as Pavement Base Layer (Ph.D. Dissertation). North Dakota State University.

Attia, M. and Abdelrahman, M. (2010a). Variability in Resilient Modulus of Reclaimed Asphalt Pavement as Base Layer and its Impact on Flexible Pavement Performance. *Transportation Research Record,* volume 2167, issue 1, pp 18–29.

Attia, M., and Abdelrahman, M. (2010b). Modeling the Effect of Moisture on Resilient Modulus of Untreated Reclaimed Asphalt Pavement. *Transportation Research Record: Journal of the Transportation Research Board*, volume 2167, issue 1, pp 30-40.

Attia, M. and Abdelrahman, M. (2011). Effect of State of Stress on the Resilient Modulus of Base Layer Containing Reclaimed Asphalt. *Road Materials and Pavement Design,* volume 12, issue 1, pp 79-97.

Attia, M., Abdelrahman, M., and Waldenmaier, A. (2013). Strain Response of Reclaimed Asphalt Pavement Material Blends under Extended Loading Testing. *Journal of Materials in Civil Engineering*, volume 25, issue 11, pp 1674-1681.

Ba, M., Nokkaew, K., Fall, M., and Tinjum, J. (2013). Effect of Matric Suction on Resilient Modulus of Compacted Aggregate Base Courses. *Geotechnical and Geological Engineering*, volume 31, issue 3, pp 1497-1510.

Bejarano, M. (2001). Evaluation of Recycled Asphalt Concrete Materials as Aggregate Base. Technical Memorandum TM-UCB-PRC-2001-4. *UC Davis: University of California Pavement Research Center*. Retrieved from https://escholarship.org/uc/item/66c0z5wm

Bennert, T., Papp Jr, W., Maher, A., and Gucunski, N. (2000). Utilization of Construction and Demolition Debris under Traffic-Type Loading in Base and Subbase Applications. *Journal of the Transportation Research Record,* volume 1714, issue 1, pp 33-39.

Bennert, T., and Maher, A. (2005). The Development of a Performance Specification for Granular Base and Subbase Material. No. FHWA-NJ-2005-003.

Bestgen, J. O., Hatipoglu, M., Cetin, B., and Aydilek, A. H. (2016). Mechanical and Environmental Suitability of Recycled Concrete Aggregate as a Highway Base Material. *Journal of Materials in Civil Engineering,* volume 28, issue 9, pp 04016067.

Blankenagel, B. and Guthrie, W. (2006). Laboratory Characterization of Recycled Concrete for Use as a Pavement Base Material. Transportation Research Record: *Journal of the Transportation Research Board*, volume 1952, issue 1, pp 21-27.

Bradshaw, A., Costa, J., and Giampa, J. (2016). Resilient Moduli of Reclaimed Asphalt Pavement Aggregate Subbase Blends. *Journal of Materials in Civil Engineering,* volume 28, issue 5, pp. 1-6.

BTS (2017). Bureau of Transportation Statistics. United States Department of Transportation, Washington, DC.

CalRecycle (2014). Construction and Demolition Debris Recycling, Caltrans Specifications for Aggregate Base and Subbase. California Department of Resources Recycling and Recovery, CA. http://www.calrecycle.ca.gov/Condemo/Specs/CaltransAgg.htm

CalTrans (2015). Standard Specifications. California Department of Transportation, Sacramento, CA.

Camargo, F., Edil, T., and Benson, C. (2013). Strength and Stiffness of Recycled Base Materials Blended with Fly Ash. *Road Materials and Pavement Design*, volume 14, issue 3, pp 504-517, doi: 10.1080/14680629.2013.779299.

Cedergren, H. R. (1988). Why All Important Pavements Should Be Well Drained. *Transportation Research Board,* issue 1188, ISSN: 0361-1981.

Cetin, B., Aydilek, A. H., and Guney, Y. (2010). Stabilization of Recycled Base Materials With High Carbon Fly Ash. Resources*, Conservation and Recycling*, volume 54, issue 11, pp 878-892.

Cetin, A., Kaya, Z., Cetin, B., and Aydilek, A. H. (2014). Influence of Laboratory Compaction Method on Mechanical and Hydraulic Characteristics of Unbound Granular Base Materials. *Road Materials and Pavement Design,* volume 15, issue 1, pp 220-235.

Cetin, B., Coban, H., Edil T. (2020). Determining Pavement Design Criteria for Recycled Aggregate Base and Large Stone Subbase. MnDOT Project TPF-5(341).

Ceylan, H., Gopalakrishnan, K., Kaya, O., and Kim, S. (2015). Investigation of AASHTOWare Pavement ME Design/DARWinME Performance Prediction Models for Iowa Pavement Analysis and Design. *InTrans Project Report.* 182. https://lib.dr.iastate.edu/intrans_reports/182

Chen, J., Tinjum, J., and Edil, T. (2013). Leaching of Alkaline Substances and Heavy Metals from Recycled Concrete Aggregate Used as Unbound Base Course. *Transportation Research Record: Journal of the Transportation Research Board*, volume 2349, issue 1, pp 81-90.

Chesner, W.H., Collins, R.J., and MacKay, M.H. (1998). User Guidelines for Waste and Byproduct Materials in Pavement Construction. FHWA-RD-97-148. Washington, DC.

Copeland, A. (2011). Reclaimed Asphalt Pavement in Asphalt Mixtures: State of the practice, No. FHWA-HRT-11-021.

Cosentino, P. J., and Kalajian, E. H. (2001). Developing Specifications for Using Recycled Asphalt Pavement as Base, Subbase or General Fill Materials Report. Contract BB-892. State Materials Office, FDOT.

Cosentino, P. J., Kalajian, E. H., Shieh, C. S., Mathurin, W. J. K., Gomez, F. A., Cleary, E. D., and Treeratrakoon, A. (2003). Developing Specifications for Using Recycled Asphalt Pavement as Base, Subbase or General Fill Materials, Phase II. Final Report. FL/DOT/RMC/06650-7754 BC 819, State Materials Office, FDOT.

Cosentino, P. J., Kalajian, E. H., Bleakley, A. M., Diouf, B. S., Misilo, T. J., Petersen, A. J., Krajcik, R. E., and Sajjadi, A. M. (2012). Improving the Properties of Reclaimed Asphalt Pavement for Roadway Base Applications. Final Report. FL/DOT/BDK81 97702, State Materials Office, FDOT.

Cosentino, P. J., Bleakley, A. M., Petersen, A. J., and Sajjadi, A. M. (2013). Evaluating Laboratory Compaction Techniques of Reclaimed Asphalt Pavement. *Transportation Research Record*, volume 2335, issue 1, pp 89-98.

Cosentino, P. J., and Bleakley, A. M. (2013). Improving Properties of Reclaimed Asphalt Pavement for Roadway Base Applications through Blending and Chemical Stabilization. *Transportation Research Record*, volume 2335, issue 1, pp 20-28.

Diagne, M., Tinjum, J., Nokkaew, K. (2015). The Effects of Recycled Clay Brick Content on the Engineering Properties, Weathering Durability, and Resilient Modulus of Recycled Concrete Aggregate. *Transportation Geotechnics,* volume 3, pp 15-23.

Edil, T., Cetin, B., Soleimanbeigi, A. (2017). Laboratory and Field Performance of Recycled Aggregate Base in a Seasonally Cold Region. *Sciences in Cold and Arid Regions*, volume 9, issue 3, pp 183-191.

Edil, T. B., and Cetin, B. (2015). Freeze-thaw Performance of Chemically Stabilized Natural and Recycled Highway Materials. *Sciences in Cold and Arid Regions*, volume 7, issue 5, pp 0482 0491.

Edil, T. B., Tinjum, J. M., and Benson, C. H. (2012a). Recycled Unbound Materials. Report No. 2012-35. Minnesota Department of Transportation. St. Paul, MN.

Edil, T., Ebrahimi, A., and Son. Y (2012b). Effectiveness of Cement Kiln Dust in Stabilizing Recycled Base Materials.

Edil, T., Wen, H., Camargo, F., and Son. Y (2012c). Comparative Assessment of Crushed Aggregates and Bound/Unbound Recycled Asphalt Pavement as Base Materials (Laboratory Evaluation of Sustainable Materials at MNroad). *International Journal of Pavement Engineering,* volume 14, issue 3, ISSN 1029-8436, pp 223-230.

Edil, T. (2011). Specifications and Recommendations for Recycled Materials Used as Unbound Base Course, Recycled Materials Resource Center, Univ. of Wisconsin-Madison, Madison, WI.

Elbheiry, M. R., Kandil, K. A., and Kotb, A. S. (2011). *Investigation of Factors Affecting Pavement Roughness* (Doctoral dissertation, Arab Academy for Science and Technology and Maritime Transport).

FHWA. (2008). User Guideline for Byproducts and Secondary Use Materials in Pavement Construction. FHWA Report FHWA-RD-97-148, FHWA, VA.

FHWA (2016). Highway Statistics 2016, Office of Highway Policy Information, Federal Highway Administration, Washington, DC.Garach, L., López, M., Agrela, F., Ordóñez, J., Alegre, J., and Moya, J. A. (2015). Improvement of Bearing Capacity in Recycled Aggregates Suitable for Use as Unbound Road Sub-Base. *Materials*, volume 8, issue 12, pp 8804-8816.

Garg, N., and Thompson, M. (1996). Lincoln Avenue Reclaimed Asphalt Pavement Base Project. *Transportation Research Record: Journal of the Transportation Research Board*, volume 1547, pp 89-95.

Gonzalez, G. P., and Moo-Young, H. K. (2004). Transportation Applications of Recycled Concrete Aggregate. FHWA State of the Practice National Review.

Goodman, R.E., (1980), Introduction to Rock Mechanics, John Wiley and Sons, New York.

Gray, J. E. (1962). Characteristics of Graded Base Course Aggregates Determined by Triaxial Tests. National Crushed Stone Association. *Engineering Bulletin*, issue 12.

Gopisetti, LSP., Ceylan, H., Kim, S., Cetin, B. and Kaya, O. (2020). Sensitivity Index Comparison of Pavement Mechanistic-Empirical Design Input Variables to Reflect Cracking Model for Different Climatic Zones, *Road Materials and Pavement Design*, pp 1-16.

Gopisetti, LSP., Cetin, B., Forman, B., Durham, S., Schwartz, C. and Ceylan, H. (2019). Evaluation of Four Different Climate Sources on Pavement Mechanistic-Empirical Design and Impact of Surface Shortwave Radiation. *International Journal of Pavement Engineering*, pp 1-14.

Greenbook (2009). Construction Materials, Section 200-Rock Materials. In Greenbook Standard Specifications for Public Works Construction.

Gupta, S., Singh, A., and Ranaivoson, A. (2004). Moisture Retention Characteristics of Base and Sub-base Materials. Report No. MN/RC-2005-06. Minnesota Department of Transportation, St. Paul, MN.

Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures, Final Document, Appendix JJ: Transverse Joint Faulting Model, NCHRP, Submitted by ARA, Inc., ERES Division 505 West University Avenue Champaign, Illinois 61820

Guthrie, W., Cooley, D., and Eggett, D. (2007). Effects of Reclaimed Asphalt Pavement on Mechanical Properties of Base Materials. *Transportation Research Record,* volume 2005, issue 1, pp 44-52.

Haider, I., Kaya, Z., Cetin, A., Hatipoglu, M., Cetin, B., and Aydilek, A. H. (2014). Drainage and Mechanical Behavior of Highway Base Materials. *Journal of Irrigation and Drainage Engineering*, volume 140, issue 6, pp 04014012.

Hasan, M., Islam, R., and Tarefder, R. (2018). Characterization of Subgrade Soil Mixed with Recycled Asphalt Pavement. *Journal of Traffic and Transportation Engineering*, volume 5, issue 3, pp 207-214.

Hatipoglu, M., Cetin, B., and Aydilek, A.H. (2020). Effects of Fines Content on Hydraulic and Mechanical Performance of Unbound Granular Base Aggregates, *Journal of*

Transportation Engineering, *Part B: Pavements*, volume 146, issue 1, article no. 04019036.

Hiller, J. E., Deshpande, Y. S., Qin, Y., Shorkey, C. J., and Peterson, K. (2011). Efficient Use of Recycled Concrete in Transportation Infrastructure, No. RC-1544. Michigan Technological University.

Hoppe, E. J., Lane, D. S., Fitch, G. M., and Shetty, S. (2015). Feasibility of Reclaimed Asphalt Pavement (RAP) Use as Road Base and Subbase Material, No. VCTIR 15-R6.

Huang, B., and Dong, Q. (2014). Laboratory Evaluation on Resilient Modulus and Rate Dependencies of RAP Used as Unbound Base Material. *Journal of Materials in Civil Engineering*, volume 26, issue 2, pp 379.

IDOT (2016). Standard Specifications for Road and Bridge Construction. Illinois Department of Transportation, Springfield, IL.

Izevbekhai, B. I., and Akkari, A. (2011). Pervious concrete test cells on MnROAD low-volume road. Minnesota Department of Transportation, Research Services Section.

Jayakody, S., Gallage, C., and Kumar, A. (2012). Assessment of Recycled Concrete Aggregate for Road Base and Sub-Base. In Proceedings of the Second International Conference on Geotechnique, *Construction Materials and Environment*. The GEOMATE International Society, pp 575-579.

Kang, D. H., Gupta, S. C., Bloom, P. R., Ranaivoson, A. Z., Roberson, R., and Siekmeier, J. (2011). Recycled Materials as Substitutes for Virgin Aggregates In Road Construction: II. Inorganic Contaminant Leaching. *Soil Science Society of America Journal*, volume 75, issue 4, pp 1276-1284.

Kazmee, H., Tutumluer, E., and Beshears, S. (2016). Pavement Working Platforms Constructed with Large-Size Unconventional Aggregates. *Transportation Research Record: Journal of the Transportation Research Board*, volume 2578, pp 1-11.

Kim, W. H., Edil, T., Benson, C., and Tanyu, B. (2005). Structural Contribution of Geosynthetic-Reinforced Working Platforms in Flexible Pavement. *Transportation Research Record: Journal of the Transportation Research Board*, volume 1936, pp 43-50.

Kim, W., and Labuz, J. F. (2007). Resilient Modulus and Strength of Base Course with Recycled Bituminous Material, No. MN/RC-2007-05.

Kim, W., Labuz, J., and Dai, S. (2007). Resilient Modulus of Base Course Containing Recycled Asphalt Pavement. *Transportation Research Record: Journal of the Transportation Research Board*, volume 2005, pp 27-35.

Koloski, J., Schwarz, S. and Tubbs, D. (1989) Geotechnical Properties of Geologic Materials, *Engineering Geology in Washington*, volume 1, Washington Division of Geology and Earth Resources Bulletin 78.

Lee, J., Edil, T., Tinjum, J., and Benson, C. (2010). Quantitative Assessment of Environmental and Economic Benefits of Recycled Materials in Highway Construction. *Transportation Research Record: Journal of the Transportation Research Board*, volume 2158, pp 138-142.

Lee, J., Edil, T., Benson, C. and Tinjum, J. (2011). Evaluation of Variables Affecting Sustainable Highway Design with BE^{2}ST-in-Highways System. *Transportation Research Record*, volume 2233, issue 1, pp 178-186.

Little, D. N., and Nair, S. (2009). Recommended Practice for Stabilization of Subgrade Soils and Base Materials, National Cooperative Highway Research Program, Project No. 20-07, *Transportation Research Board of the National Academies.*

Locander, R. (2009). Analysis of Using Reclaimed Asphalt Pavement (RAP) as a Base Course Material. Colorado Department of Transportation DTD Applied Research and Innovation Branch, Report No. CDOT-2009-5, pp 1-68.

LRRB (2016). Recycled Materials in Unbound Aggregate Base Layers in Minnesota. Transportation Research Synthesis, TRS 1604. Local Road Research Board, Minnesota Department of Transportation.

Mahedi, M., and Cetin, B., (2020). Carbonation Based Leaching Assessment of Recycled Concrete Aggregates. *Chemosphere*, volume 250, 126307.

McGarrah, E. J. (2007). Evaluation of Current Practices of Reclaimed Asphalt Pavement/Virgin Aggregate as Base Course Material. Washington Department of Transportation, No. WA-RD 713.1.

Mijic, Z., Dayioglu, A., Hatipoglu, M., and Aydilek, A. (2019). Hydraulic and Environmental Impacts of Using Recycled Asphalt Pavement on Highway Shoulders. *Construction and Building Materials,* volume 234, no 20, 117226.

Mishra, D., and Tutumluer, E. (2012). Aggregate Physical Properties Affecting Modulus and Deformation Characteristics of Unsurfaced Pavements. *Journal of Materials in Civil Engineering*, volume 24, issue 9, pp 1144-1152.

MnDOT (2016). Grading and Base Manual developed by Geotechnical section. http://www.dot.state.mn.us/materials/manuals/GBase/2016gbmanual3132016.pdf.

MnDOT (2018). Standard Specifications for Construction (2018 Edition). Minnesota Department of Transportation, St. Paul, MN.MoDOT (2018). Missouri Standard Specifications for Highway Construction. Missouri Highways and Transportations Comission.

Mokwa, R., and Peebles, C. (2005). Evaluation of the Engineering Characteristics of Rap/Aggregate Blends. FHWA/MT-05-008/8117-24. Prepared for State of Montana Department of Transportation, doi: 10.21949/1518189.

Montemayor, T. A. (1998). Compaction and Strength-Deformation Characteristics of Reclaimed Asphalt Pavement (MS thesis). Florida Institute of Technology, Melbourne, FL.

Mulvaney, R. and Worel, B., (2002). MnROAD Cell 26 Forensic Investigation Final Report, Minnesota Road Research Project.

Natarajan, B., Kanavas, Z., Sanger, M., Rudolph, J., Chen, J., Edil, T., and Ginder-Vogel, M. (2019). Characterization of Recycled Concrete Aggregate after Eight Years of Field Deployment. *Journal of Materials in Civil Engineering,* volume 31, no 6, pp 04019070.

NCHRP-838 (2017). Optimizing Materials QA to Improve Construction Save Costs. NCHRP Research Report. National Cooperative Highway Research Program.

NCHRP 1-28a (2003). Harmonized Test Methods for Laboratory Determination of Resilient Modulus for Flexible Pavement Design. Witczak, M. W., Final Report, volume 1, Unbound Granular Material.Nokkaew, K., Tinjum, J. M., and Benson, C. H. (2012). Hydraulic Properties of Recycled Asphalt Pavement and Recycled Concrete Aggregate. In GeoCongress 2012: State of the Art and Practice in Geotechnical Engineering, pp 1476-1485.

Ooi, P. S., Archilla, A. R., Song, Y., and Sagario, M. L. Q. (2010). Application of Recycled Materials in Highway Projects, No. HWY-L-2005-04.

Perkins, S. W., Bowders, J. J., Christopher, B. R., and Berg, R. R. (2005). Geosynthetic Reinforcement for Pavement Systems, US Perspectives. In International Perspectives on Soil Reinforcement Applications, pp 1-13.

Poon, C. S., Qiao, X. C., and Chan, D. (2006). The Cause and Influence of Self-Cementing Properties of Fine Recycled Concrete Aggregates on the Properties of Unbound Sub-Base. *Waste Management*, volume 26, issue10, pp 1166-1172.

Puppala, A., Saride, S., and Williammee, R. (2012). Sustainable Reuse of Limestone Quarry Fines and RAP in Pavement Base/Subbase Layers. *Journal of Materials in Civil Engineering,* volume 24, no 4, pp 418-429.

Rahardjo, H., Vilayvong, K., and Leong, E. C. (2010). Water Characteristic Curves of Recycled Materials. *Geotechnical Testing Journal*, volume 34, issue 1, pp 89-96.

Rahn, P. and Biehler, A. (2008). American Association of State Highway and Transportation Officials Executive Committee 2007/2008. Mechanistic–Empirical Pavement Design Guide ;A Manual of Practice.

Read, J. and Whiteoak, D. (2003). The Shell Bitumen Handbook, Fifth Edition. Thomas Telford Publishing, London, UK.

Saeed, A. (2008). NCHRP Report 598. Performance-Related Tests of Recycled Aggregates for Use in Unbound Pavement Layers. *Transportation Research Board of the National Academies*. Washington, D.C.

Sayed, S. M., Pulsifer, J. M., and Schmitt, R. C. (1993). Construction and Performance of Shoulders Using UNRAP Base. *Journal of Materials in Civil Engineering*, volume 5, issue 3, pp 321-338.

Schaertl, G.J. (2010). Scaling and Equivalency of Bench-scale Tests to Field Scale Conditions. MS Thesis, University of Wisconsin-Madison, WI.

Schwartz, C., Li, R., Kim, S., Ceylan, H. and Gopalakrishnan, K. (2011). Sensitivity Evaluation of MEPDG Performance Prediction, National Cooperative Highway Research Program Transportation Research Board Of The National Academies (NCHRP 1-47)

Schwartz, C., Elkins, G., Li, R., Visintine, B., Forman, B., Rada, G. and Groeger, J. (2015). Evaluation of Long-Term Pavement Performance (LTPP) Climatic Data for Use in Mechanistic-Empirical Pavement Design Guide (MEPDG) Calibration and Other Pavement Analysis (FHWA-HRT 15-019)

Schuettpelz, C. C., Fratta, D., and Edil, T. B. (2010). Mechanistic Corrections for Determining the Resilient Modulus of Base Course Materials Based on Elastic Wave Measurements. *Journal of Geotechnical and Geoenvironmental Engineering*, volume 136, issue 8, pp 1086-1094.

Snyder, M., Smith, K. D., Vandenbossche, J. M., and Wade, M. J. (1994). Physical and Mechanical Properties of Recycled PCC Aggregate Concrete. Interim Report–Task A, DTFH61-93C-00133, US Department of Transportation, Federal Highway Administration.

Snyder, M. (1995). Use of Crushed Concrete Products in Minnesota Pavement Foundations. Final Report. Report No. MN/RC-96/12. Minnesota Department of Transportation.

Soleimanbeigi, A., and Edil, T. B. (2015a). Compressibility of Recycled Materials for Use as Highway Embankment Fill. *Journal of Geotechnical and Geoenvironmental Engineering*, volume 141, issue 5, 04015011.

Soleimanbeigi, A., and Edil, T. B. (2015b). Thermal Conditioning to Improve Geotechnical Properties of Recycled Asphalt Pavements. *Geotechnical Testing Journal,* volume 38, issue 4, pp 537-548.

Soleimanbeigi, A., Shedivy, R. F., Tinjum, J. M., and Edil, T. B. (2015). Climatic Effect on Resilient Modulus of Recycled Unbound Aggregates. *Road Materials and Pavement Design*, volume 16, issue 4, pp 836-853.

Swiss Standard SN 670 010b, Characteristic Coefficients of Soils, Association of Swiss Road and Traffic Engineers Swiss Standard SN 670 010b, Characteristic Coefficients of Soils, Association of Swiss Road and Traffic Engineers.

Taha, R., Ali, G., Basma, A., and Al-Turk, O. (1999). Evaluation of Reclaimed Asphalt Pavement Aggregate in Road Bases and Subbases. *Transportation Research Record: Journal of the Transportation Research Board*, volume 1652, pp 264-269.

Tan, D., Hill, K., and Khazanovich L. (2014). Quantifying Moisture Effects in DCP and LWD Tests Using Unsaturated Mechanics. Report No. MN/RC 2014-13. Minnesota Department of Transportation.

Thakur, J. K., and Han, J. (2015). Recent Development of Recycled Asphalt Pavement (RAP) Bases Treated for Roadway Applications. *Transportation Infrastructure Geotechnology*, volume 2, issue 2, pp 68-86.

Thompson, M. R., and Smith, K. L. (1990). Repeated Triaxial Characterization of Granular Bases. *Transportation Research Record*, volume 1278, pp 7-17.

Tutumluer, E., Xiao, Y., and Wilde, W. J. (2015). Cost-Effective Base Type and Thickness for Long-Life Concrete Pavements. Report No. MN/RC 2015-42. Minnesota Department of Transportation, St. Paul, MN.

Tutumluer, E. (2013). Practices for Unbound Aggregate Pavement Layers: A Synthesis of Highway Practice. *Transportation Research Board of the National Academies*, Washington, DC, Rep. NCHRP Synthesis, 445.

Tutumluer, E., Kazmee, H., Mishra, D., Boler, H., and Roesler, J. (2012). Effects of Material Blending on Strength, Modulus and Deformation Characteristics of Recycled Concrete Aggregates (It was received from Dr. Erol Tutumluer directly as this paper was never published.)

Tutumluer, E. and Pan, T. (2008). Aggregate Morphology Affecting Strength and Permanent Deformation Behavior of Unbound Aggregate Materials. *Journal of Materials in Civil Engineering*, volume 20, issue 9, pp 617-627.TxDOT (2017). Pavement Manual. Texas Department of Transportation, Austin, TX.

Ullah, S., Tanyu, B., and Hoppe, E. (2018). Optimizing the Gradation of Fine Processed Reclaimed Asphalt Pavement and Aggregate Blends for Unbound Base Courses. *Transportation Research Record,* volume 2672, issue 52, pp 57-66.

Ullah, S., and Tanyu, B. (2019). Methodology to Develop Design Guidelines to Construct Unbound Base Course with Reclaimed Asphalt Pavement (RAP). *Construction & Building Materials,* volume 223, pp 463-476.

Wen, H., and Wu, M. (2011). Evaluation of High Percentage Recycled Asphalt Pavement as Base Materials, No. TNW2011-15.

Wen, H., Wu, M., and Uhlmeyer, J. (2011). Evaluation of the Effects of Climatic Conditions on Modulus of Base Materials with Recycled Asphalt Pavement. *Journal of ASTM International*, volume 8, issue 10, pp 1-13.

WisDOT (2018). Standard Specifications. Wisconsin Department of Transportation, WI.

Wu, M., Wen, H., Muhunthan, B., and Manahiloh, K. (2012). Influence of Recycled Asphalt Pavement Content on Air Void Distribution, Permeability, and Modulus of Base Layer. *Transportation Research Record,* volume 2267, issue 1, pp 65-71.

Xiao, F., Amirkhanian, S.N. and Wu, B. (2011). Fatigue and Stiffness Evaluations of Reclaimed Asphalt Pavement in Hot Mix Asphalt Mixtures. *Journal of Testing and Evaluation,* volume 39, issue 1, pp 1-9.

Yoder, E.J. and Witczak, M.W. (1975). Principle of Pavement Design. 2nd Edition, John Wiley and Sons, Hoboken, p 711.

Yohannes, B., Hill, K., and Khazanovich, L. (2009). Mechanistic Modeling of Unbound Granular Materials. Report No. MN/RC 2009-21. Minnesota Department of Transportation, St. Paul, MN.

Zornberg, J. G. (2017). Functions and Applications of Geosynthetics in Roadways. *Procedia Engineering*, volume 189, pp 298-306.