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16. Abstract (Limit: 250 words)

The use of recycled materials promotes sustainability in roadway construction by reducing the consumption of energy and emission of greenhouse gases associated with mining and the production of virgin aggregate (VA). Recycled asphalt pavement (RAP) and recycled concrete aggregate (RCA) have comparable characteristics to VA that have been used in roadway base course applications. This study developed a database for RAP and RCA materials' characteristics, including gradation, compaction, resilient modulus (M_r), California bearing ratio (CBR), and saturated hydraulic conductivity (K_{sat}). In addition, this study summarizes construction specifications provided by several departments of transportation (DOTs) regarding the use of recycled aggregates in pavement systems. The effects of the presence of RAP and RCA in aggregate matrices on the engineering and index properties of aggregates were investigated, and some trends were observed. For example, it was found that a higher RAP content reveals a higher summary M_r (SM_r), and a higher RCA content causes an increase in optimum moisture content (OMC) and a decrease in maximum dry unit weight (MDU). In addition, a series of AASHTOWare Pavement Mechanistic-Empirical (ME) Design (PMED) analyses was conducted for three traffic volumes [low (1,000 AADTT), medium (7,500 AADTT), and high (25,000 AADTT)] 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/rigid pavement designs. Results showed that M_r had a higher effect on pavement distress predictions compared to gradation and saturated hydraulic conductivity (K_{sat}).

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IMPROVE MATERIAL INPUTS INTO MECHANISTIC DESIGN PROPERTIES FOR RECLAIMED HMA & RECYCLED CONCRETE AGGREGATE (RCA) ROADWAYS

FINAL REPORT

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LIST OF ABBREVIATIONS

AADTT = annual average daily truck traffic

CBC = crushed base course

CBR = California bearing ratio

DGABC = dense-graded aggregate base course

DOT = department of transportation

IRI = international roughness index

JPCP = jointed plain concrete pavement

K_{sat} = saturated hydraulic conductivity

MDU = maximum dry unit weight

ME = mechanistic-empirical

OMC = optimum moisture content

PCC = Portland cement concrete

PD = permanent deformation

PMED = pavement mechanistic-empirical design

RAP = recycled asphalt pavement

RCA = recycled concrete aggregate

RCM = recycled concrete material

RPM = recycled pavement material

SM_r = summary resilient modulus

USCS = unified soil classification system

VA = virgin aggregate

EXECUTIVE SUMMARY

The lack of high-quality material availability and high cost of virgin aggregate (VA) have made engineers look for alternative sources, such as recycled concrete aggregate (RCA) and recycled asphalt pavement (RAP), to use as base layers. These recycled aggregates are prone to exhibiting different properties depending on several factors, such as the aggregate source and the type of milling/crushing operation. Therefore, their index and engineering properties should be well understood to design well-performing and long-lasting pavement systems. In this project, a detailed literature review was performed to create a database for such properties of RAP and RCA that can be used in aggregate base/subbase layers of pavement systems. In addition to the database development, the trends between different properties of RAP and RCA [e.g., California bearing ratio (CBR) vs. fines content] were investigated. Thanks to the database developed to summarize the properties of various RAP and RCA, more representative material inputs can be used in pavement design, so that more reliable performance prediction models can be obtained using a mechanistic-empirical (ME) pavement design approach. This database can allow more RAP and RCA to be used by more departments of transportation (DOTs) or local agencies in pavement foundation layers (particularly in aggregate base layers).

Based on ME simulations, summary resilient modulus (SM_r) values of RAP/RCA base layers had the highest influence on the pavement performance among other material inputs, including gradation and saturated hydraulic conductivity (K_{sat}). Higher fines content, lower SM_r value, and lower sand content for RAP/RCA base layers caused higher total rutting predictions in flexible pavement models. In addition, higher fines content for RAP/RCA base layers yielded higher international roughness index (IRI) predictions in these models. Acceptable IRI and total rutting performance was obtained for flexible pavements containing RAP/RCA base layers with the use of SM_r values presented in the developed database. However, it is recommended to take more considerations into account in some cases, such as high fines content, low sand, and high gravel content for RCA under medium/high traffic volumes (7,500/25,000 AADTT) as close-to-the-failure conditions were observed in such cases. Overall, there is a high chance that flexible pavements employing RAP/RCA base with a design life of 20 years can provide adequate performance in all types of traffic conditions in terms of IRI and total rutting according to the analyses performed in this study. However, it is recommended that before designing flexible pavement systems, one should collect data regarding gradation and SM_r as these parameters have an influence on total rutting and IRI.

ME simulations indicated that mean joint faulting and IRI were very critical for the rigid pavement design under high traffic volume (25,000 AADTT) and in some cases medium traffic volume (7,500 AADTT) since rigid pavement models containing RAP/RCA base layers failed under these conditions. All the cases under low traffic volume (1,000 AADTT) satisfied the failure criteria set for rigid pavements in terms of mean joint faulting and IRI. Therefore, it can be concluded that it is safe to design jointed plain concrete pavement (JPCP) systems with RAP/RCA base layers for low volume roads with minimal testing.”

CHAPTER 1: INTRODUCTION

In the United States, 4.3 million km (2.6 million miles) out of 6.6 million km (4.1 million miles) of total public roads are paved (BTS 2017). Flexible and rigid pavements are the two paved road systems, and more than 90% of the paved roads are flexible pavements (Copeland 2011). While most of vehicle loads are carried by a concrete surface layer in rigid pavements, the main function of flexible pavements is distributing the vehicle loads throughout the pavement structure. In rigid pavements, aggregate base/subbase layers are generally constructed to provide adequate drainage with less concern given to the structural benefits of these layers. On the other hand, in flexible pavements, the strength/stiffness properties of asphalt surface layers, as well as aggregate base, subbase, and subgrade layers, are very important for long-term pavement performance (Little and Nair 2009; Tutumluer et al. 2015). An aggregate base layer is a very critical component of a pavement structure. It is the first layer beneath an asphalt surface layer (Cosentino and Kalajian 2001; Yohannes et al. 2009). There are two primary functions of the aggregate base layer: (1) providing adequate mechanical support to the asphalt surface layer to prevent fatigue cracking and rutting, and (2) providing adequate drainage to evacuate the excessive water infiltrated from the pavement structure. Materials used in aggregate base layers are responsible for distributing the wheel loads uniformly to subbase and subgrade layers, so they can protect the sublayers from excessive loading and ultimately increase the service life of pavements (Yoder and Witzack 1975; Xiao et al. 2011). Aggregate base layers are made of coarse-grained aggregates to provide adequately stiff and permeable layers (Schuettpelz et al. 2010; Haider et al. 2014; Cetin et al. 2014; Edil and Cetin 2015). The majority of pavement failures occur due to the lack of required mechanical properties of the materials used in aggregate base layers (Tutumluer and Pan 2008; Xiao et al. 2011).

While large amounts of virgin aggregate (VA) are used in aggregate base layers (Perkins et al. 2005; Haider et al. 2014; Hatipoglu et al. 2020), the lack of availability and high cost of good-quality VA have made engineers look for alternative materials, such as recycled concrete aggregate (RCA) and recycled asphalt pavement (RAP). In addition to helping to reduce the need for good-quality VA, these alternative materials can additionally provide environmental benefits, such as reduced energy consumption and greenhouse gas emissions (Cetin et al. 2010; Lee et al. 2010).

In order to obtain 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 by-product of pavement milling, and it is a mixture of aged asphalt binder and VA (Taha et al. 1999). On the other hand, RCA is obtained via crushing the existing hardened concrete recovered from 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 duration of construction. In fact, up to 30% of cost savings could be achieved by in-place recycling for a recycled aggregate generation (Edil 2011).

Material characteristics such as mineralogy, gradation, angularity, texture, and durability are different for each RAP and RCA materials, and these differences can significantly affect their engineering properties (Tutumluer 2013; Tan et al. 2014). The properties of RAP and RCA also depend on several factors that relate back to the production of asphalt or concrete as well as the processes followed during the production of RAP and RCA. Some of these factors are listed below:

- The type of road (e.g., interstate highway, arterial highway, parking lot, etc.) that is milled may affect the binder grade and binder content of the produced RAP since different bituminous materials and contents are used in different asphalt mixtures (aged binder can also affect the properties of RAP).
- The regional differences in location of the milled road may result in different RAP and RCA due to different geological composition and formation of aggregates.
- Processing operations used to create RAP and RCA may affect the gradation of these materials due to the different opening sizes of the screens used by 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 extreme temperatures (cold or hot) for a long period of time (Ullah and Tanyu 2019) and carbonation of remaining cement content in RCA (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.

One of the most important steps for constructing high-quality and long-lasting pavements is the determination of surface, aggregate base, and subbase layers' thicknesses. While there are methods and assumptions for using VA as aggregate base/subbase layers, designing pavements with recycled aggregates (RAP and RCA) can be challenging (Edil 2011). Recycled materials often manifest mechanical behavior that is distinct from that of VA due to the composition and the nature of particle characteristics. RAP and RCA have comparable stiffness to VA used in roadway base course applications (FHWA 2008; Guthrie et al. 2007; Edil et al. 2012a). The engineering properties of RAP and RCA should be well understood as they have become important in sustainable pavement design.

There are several parameters to be considered, such as the stiffness of layers, climate zone, traffic conditions, the designed service life of the pavement, and failure criteria, to create the most sustainable pavement design. The AASHTO pavement design guide (AASHTO 1993) and the pavement mechanistic-empirical (ME) design (PMED) approach are the two most commonly used design methods in flexible and rigid pavements (Edil 2011). The PMED approach represents a major improvement over its predecessors. The PMED method has been developed to take climate and traffic effects into account for pavement analyses since the AASHTO method did not consider these effects directly in pavement analyses. 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 for 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). Reliable characterizations of the traffic, climate, and material input parameters are therefore important to ensure that the theoretical computation of pavement stresses, strains, temperatures, and moisture levels are reliable at the critical locations within the system (Schwartz et al.

2015). Depending on the desired level of accuracy of input parameters, 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 the use of a mix of levels.

In this project, the AASHTOWare PMED software was used as the PMED approach. Proper implementation of the AASHTOWare PMED software requires realistic values for the input parameters. Pavement structures generally contain three layers: asphalt/Portland cement concrete (PCC) (often consisting of several sublayers or lifts), aggregate base/subbase, and subgrade. The layers beneath the asphalt/PCC layers usually consist of unbound materials, and their physical and engineering properties are very crucial for long-term pavement performance (Haider et al. 2014; Gopisetti et al. 2019; Hatipoglu et al. 2020; Gopisetti et al. 2020). The material parameters required for pavement foundation materials, including unbound granular materials, subgrade, and bedrock, can 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 inputs are resilient modulus (M_r), which is used for quantifying the stress-dependent stiffness of unbound materials under moving wheel loads. Material parameters associated with EICM include Atterberg limits, gradation, and saturated hydraulic conductivity (K_{sat}).

As previously stated, the properties of RAP and RCA should be well understood as they play an important role in pavement design as aggregate base layers. While determining the properties of RAP and RCA is preferred before designing pavement systems, it may be costly and take a long time to be completed. Therefore, it is important to establish a database with the information collected from previous studies, which would provide some insight into information about the boundaries and average properties of these materials and can be used by DOTs during pavement analysis and design. To address this need, in this project, a detailed literature review was performed to create a database for material characteristics of RAP and RCA that can be used in aggregate base/subbase layers of pavement systems. In addition, data for RAP-VA and RCA-VA blends were collected from the literature as well. Thanks to the database developed to summarize the properties of various RAP and RCA, more representative material inputs can be used in pavement design so that more reliable performance prediction models can be obtained using the AASHTOWare PMED software. This database can allow more RAP and RCA to be used by more departments of transportation (DOTs) or local agencies in pavement foundation layers (particularly in aggregate base layers). In addition to the database development, the trends between different properties of RAP and RCA [e.g., California bearing ratio (CBR) vs. fines content] were investigated. The AASHTOWare PMED software was used to conduct sensitivity analyses to determine how the material properties of RAP and RCA affect pavement performance predictions for both flexible and rigid pavement systems.

CHAPTER 2: PHYSICAL PROPERTIES OF RAP AND RCA

2.1 DATABASE

Table 2.1 summarizes the list of the RAP/RCA 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 2.1. RAP and RCA for the available data were captured for the states of Minnesota (MN), Colorado (CO), Michigan (MI), California (CA), Texas (TX), Ohio (OH), New Jersey (NJ), Wisconsin (WI), Illinois (IL), Montana (MT), Virginia (VA), Florida (FL), Tennessee (TN), Maryland (MD), New Mexico (NM), Washington (WA), Utah (UT), and Rhode Island (RI). 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 RAP-RCA materials with VA at different mixture ratios.

Table 2.1. List of the collected data and corresponding resources

Source	Location	Material	Gradation	Atterberg Limits	Compaction	K_{sat}	Shear Strength	CBR	M_r	R-value
Edil et al. (2012a)	MN, MI, CO, CA, TX, OH, NJ, WI	Class 5 (MN)	1		1	1			26	
		50% RCA + 50% Class 5	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

RPM = recycled pavement material; CBR = California bearing ratio; R-value = measures the response of compacted aggregates to a vertically applied pressure under specific condition. Class 5 is an aggregate base layer specified in Minnesota DOT (MnDOT) 2018 report.

Table 2.2. List of the collected data and corresponding resources (cont'd)

Source	Location	Material	Gradation	Atterberg Limits	Compaction	K_{sat}	Shear Strength	CBR	M_r	R-value
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	
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	

CBC = crushed base course; 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 condition. Class 5 is an aggregate base layer specified in Minnesota DOT (MnDOT) 2018 report.

Table 2.3. List of the collected data and corresponding resources (cont'd)

Source	Location	Material	Gradation	Atterberg Limits	Compaction	K_{sat}	Shear Strength	CBR	M_r	R-value
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	

PD = permanent deformation; CBR = California bearing ratio; R-value = measures the response of compacted aggregates to a vertically applied pressure under specific condition. Class 5 is an aggregate base layer specified in Minnesota DOT (MnDOT) 2018 report.

Table 2.4. List of the collected data and corresponding resources (cont'd)

Source	Location	Material	Gradation	Atterberg Limits	Compaction	K_{sat}	Shear Strength	CBR	M_r	R-value
Soleimanbeigi and Edil (2015b)	WI	RAP	1		1	1				
		RCA	1		1	1				
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					

RPM = recycled pavement material; RCM = recycled concrete material; CBR = California bearing ratio; R-value = measures the response of compacted aggregates to a vertically applied pressure under specific condition. Class 5 is an aggregate base layer specified in Minnesota DOT (MnDOT) 2018 report.

Table 2.5. List of the collected data and corresponding resources (cont'd)

Source	Location	Material	Gradation	Atterberg Limits	Compaction	K_{sat}	Shear Strength	CBR	M_r	R-value
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	
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					

CBR = California bearing ratio; R-value = measures the response of compacted aggregates to a vertically applied pressure under specific condition. Class 5 is an aggregate base layer specified in Minnesota DOT (MnDOT) 2018 report.

Table 2.6. List of the collected data and corresponding resources (cont'd)

Source	Location	Material	Gradation	Atterberg Limits	Compaction	K_{sat}	Shear Strength	CBR	M_r	R-value
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	US	RCA	47	3	47	32	5	24	153	0
		RAP	92	31	126	57	66	38	316	107

Total = it is the total of all the data provided in Table 2.1, Table 2.2, Table 2.3, Table 2.4, Table 2.5, and Table 2.6. CBR = California bearing ratio; R-value = measures the response of compacted aggregates to a vertically applied pressure under specific condition. Class 5 is an aggregate base layer specified in Minnesota DOT (MnDOT) 2018 report.

2.2 GRADATION

The gradation properties of aggregates affect their engineering properties, such as K_{sat}, shear strength, stiffness, and frost-susceptibility (Saeed 2008); therefore, such properties of aggregates must be well understood. Original aggregate type, milling operations, and crushing methods can significantly affect the gradation of RAP and RCA (Cosentino and Kalajian 2001).

The first material characteristics to be considered to develop the database were selected as the index properties, which mainly consist of the gradation of aggregates. Gradation database includes the following parameters: gravel content, sand content, fines (silt and clay) content, specific diameter sizes [D_{10} (diameter at which 10% of the particles are finer – effective diameter), D_{30} (diameter at which 30% of the particles are finer), and D_{60} (diameter at which 60% of the particles are finer)], coefficient of uniformity (C_u), coefficient of curvature (C_c), and specific gravity (G_s). Approximately 190 different recycled aggregates, including their blends with VA, were included in the gradation database.

Appendix A reports the gradation and G_s database for RAP and RCA. According to Appendix A, all RAP and RCA were classified as coarse-grained soils in the previous studies. Since most of the materials had fines content lower than 12%, they were all classified as either well-graded gravel (GW) and poorly graded gravel (GP) or well-graded sand (SW) and poorly graded sand (SP)-SW. Table 2.7 summarizes the gradation and G_s database shown in Appendix A and provides the lower limit, median, and upper limit of gravel content, sand content, fines content, D_{10}, D_{30}, D_{60} C_u, C_c, and G_s for RAP and RCA.

Mahedi and Cetin (2020) reported the highest gravel content (94.1%) for RCA, while Edil et al. (2017) reported the lowest gravel content (31.8%) for RCA. On the other hand, Alam et al. (2010) showed 3% gravel content for RAP, which was the lowest gravel content reported for RAP in the database. Locander (2009) reported the highest gravel content for RAP, which was 75%. Finally, the median gravel contents for RAP and RCA is reported to be 45 and 51%, respectively.

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

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

Asphalt binder content (∼4.5-6%) and trapped air between asphalt coating and aggregate particles cause lower G_s values for RAP compared to VA (Cosentino et al. 2003). RCA also tends to have relatively lower G_s values than VA due to the presence of residual mortar in their matrices (Snyder et al. 1994). G_s values of RAP range from 2.19 to 2.87 with a median value of 2.4, while G_s values of RCA are between 2.12 and 2.7 with a median value of 2.39.

Table 2.7. Summary of gradation and specific gravity (G_s) database for recycled asphalt pavement (RAP) and recycled concrete aggregate (RCA)

Parameter	RAP			RCA		
	Lower Limit	Median	Upper Limit	Lower Limit	Median	Upper Limit
Gravel, %	3 *[52]*	45 *[52]*	75 *[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} ($3.9\text{x}10^{-3}$) *[30]*	$5\text{x}10^{-1}$ ($1.96\text{x}10^{-2}$) *[30]*	1 ($3.93\text{x}10^{-2}$) *[30]*	10^{-1} ($3.9\text{x}10^{-3}$) *[19]*	$2.3\text{x}10^{-1}$ ($9\text{x}10^{-3}$) *[19]*	$4.3\text{x}10^{-1}$ ($1.7\text{x}10^{-2}$) *[19]*
D_{30}, mm (in)	$8\text{x}10^{-2}$ ($3.1\text{x}10^{-3}$) *[27]*	1.5 ($6\text{x}10^{-2}$) *[27]*	4.9 ($1.9\text{x}10^{-1}$) *[27]*	$2\text{x}10^{-1}$ ($7.9\text{x}10^{-3}$) *[17]*	1.2 ($4.72\text{x}10^{-2}$) *[17]*	6.5 ($2.56\text{x}10^{-1}$) *[17]*
D_{60}, mm (in)	$1.5\text{x}10^{-1}$ ($5.9\text{x}10^{-3}$) *[27]*	4.82 ($1.89\text{x}10^{-1}$) *[27]*	10.4 ($4.09\text{x}10^{-1}$) *[27]*	$6\text{x}10^{-1}$ ($2.36\text{x}10^{-2}$) *[17]*	6.8 ($2.67\text{x}10^{-1}$) *[17]*	16.3 ($6.42\text{x}10^{-1}$) *[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]*	2.4 *[38]*	2.87 *[38]*	2.12 *[32]*	2.39 *[32]*	2.7 *[32]*

D_{10} = diameter at which 10% of the particles are finer – effective diameter; D_{30} = diameter at which 30% of the particles are finer; D_{60} = diameter at which 60% of the particles are finer; C_u = coefficient of uniformity; C_c = coefficient of curvature; G_s = specific gravity. Italic numbers provided in square brackets represent the corresponding sample size.

2.3 COMPACTION

The general trend with respect to the Proctor compaction properties of RAP and RCA is that these recycled aggregates are prone to showing lower maximum dry unit weight (MDU) values than VA. Lower MDU values of RAP may be due to their lower G_s values caused by asphalt content and low fines contents (Guthrie et al. 2007; Locander 2009). In addition, the hydration and cementation of unhydrated cement particles in RCA may cause a reduction in MDU values of RCA. In addition, the literature shows that an increase in RAP or RCA content in recycled aggregate-VA blend matrices tends to cause a decrease in MDU values of blends (Bennert et al. 2000). The reduction in MDU values of recycled aggregate-VA blends is directly proportional to the RAP or RCA content of the blends (Taha et al. 1999). Figure 2.1 and Figure 2.2 show the effect of RAP content on MDU values of RAP-VA blends. In addition, Figure 2.3 and Figure 2.4 show the effect of RCA content on MDU values of RCA-VA blends.

RAP generally possesses hydrophobic properties due to the presence of asphalt coating around aggregate particles, and this contributes RAP to have lower OMC values. On the other hand, RCA is prone to showing hydrophilic properties due to the presence of porous residual mortar in their matrices (in addition, RCA generally shows higher water absorption than VA due to their porous structure caused by residual mortar); thus, higher OMC values are generally reported for RCA (Edil et al. 2012a; Nokkaew et al. 2012; Sayed et al. 1993; Rahardjo et al. 2010). Moreover, using higher RAP contents in RAP-VA blends can cause further reductions in OMC values (Locander 2009), while the use of higher amounts of RCA contents in RCA-VA blends can cause an increase in OMC values (Bennert et al. 2000). Figure 2.5 and Figure 2.6 show the effect of RAP content on OMC values of RAP-VA blends. In addition, Figure 2.7 and Figure 2.8 show the effect of RCA content on OMC values of RCA-VA blends.

According to the MDU and OMC database, the compaction results were mostly obtained from materials compacted with modified Proctor compaction effort (ASTM D1557 & AASHTO T 180). Therefore, it is recommended to use modified Proctor compaction data for analyses. Table 2.8 summarizes the MDU and OMC database and provides the lower limit, median, and upper limit of MDU and OMC values for RAP and RCA. MDU values of RAP range between 17.2 and 24.1 kN/m^3 (110 and 155 pcf) with a median value of 19.6 kN/m^3 (126 pcf). MDU values of RCA range from 18.3 to 21.7 kN/m^3 (118 to 140 pcf) with a median value of 19.7 kN/m^3 (127 pcf). OMC values of RAP range between 4 and 10.7% with a median value of 6.05%, while OMC values of RCA range between 6.1 and 14.8% with a median value of 10.8%.

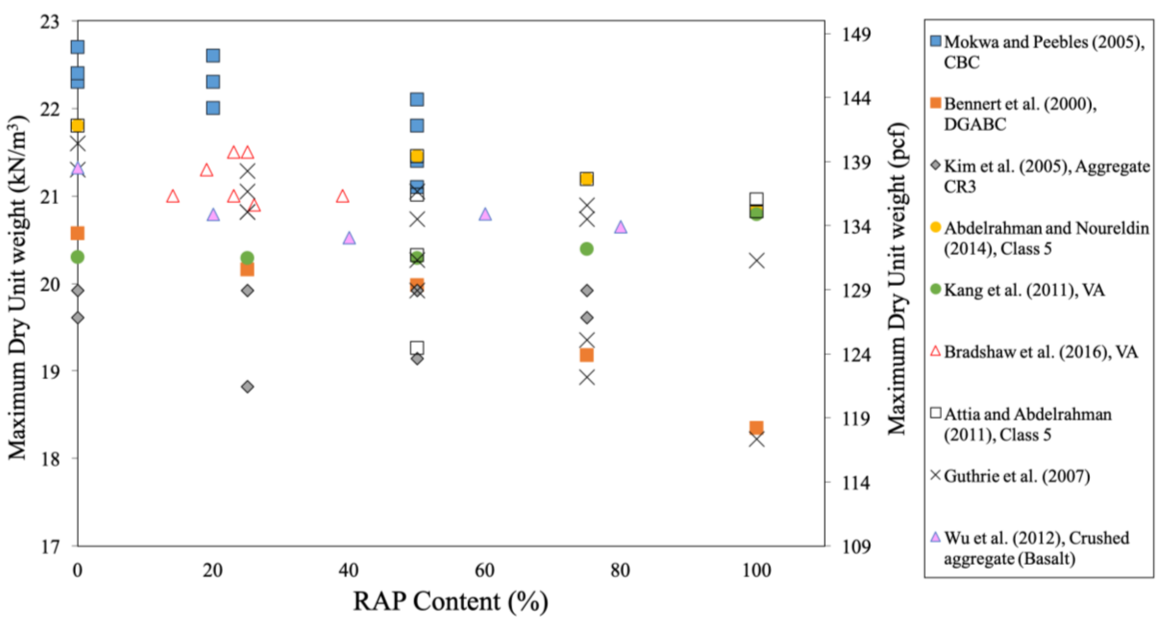

Figure 2.1. Maximum dry unit weight (MDU) vs. recycled asphalt pavement (RAP) content (CBC = crushed base course; DGABC = dense-graded aggregate base course; CR3 = County Road 3; VA = virgin aggregate)

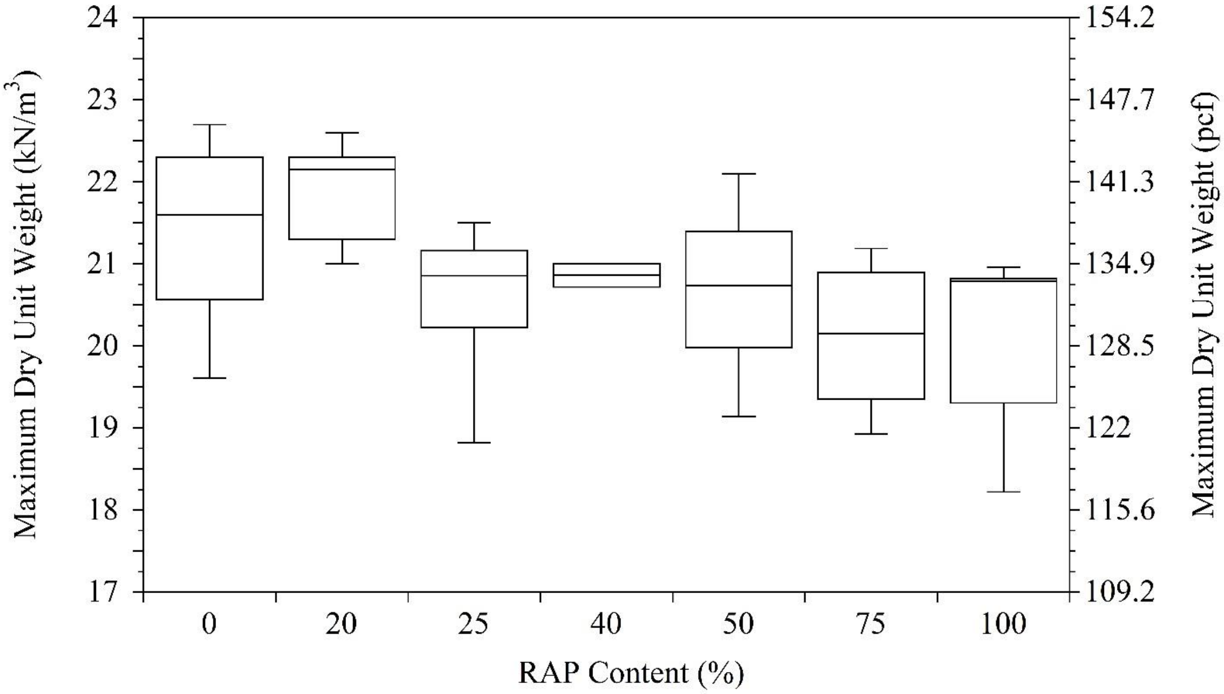

Figure 2.2. Maximum dry unit weight (MDU) vs. recycled asphalt pavement (RAP) content (box and whisker plot)

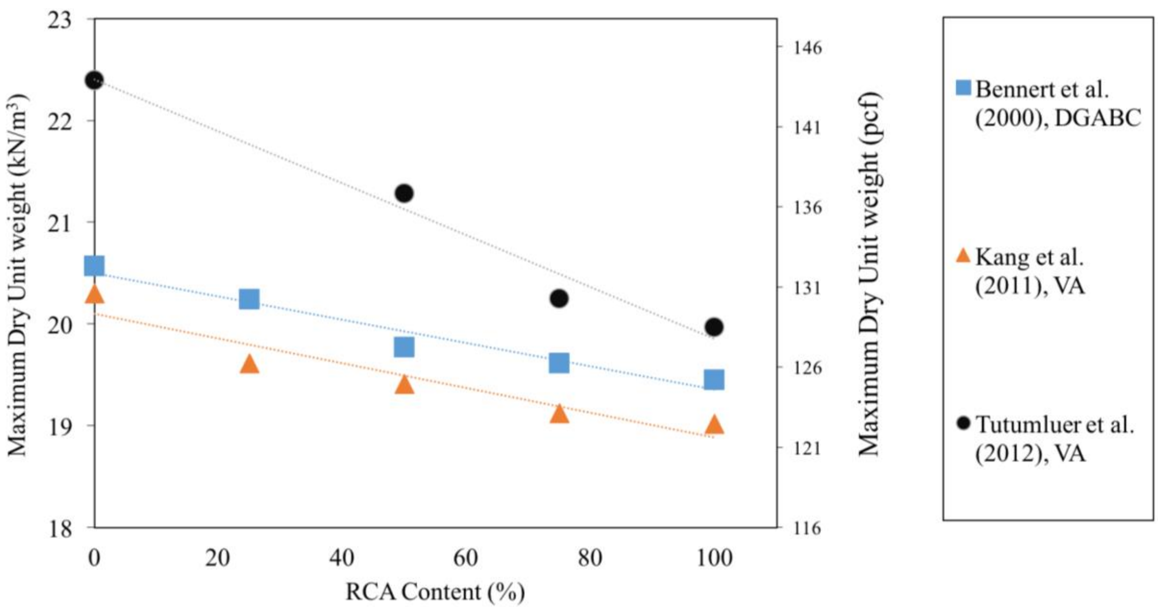

Figure 2.3. Maximum dry unit weight (MDU) vs. recycled concrete aggregate (RCA) content (DGABC = dense-graded aggregate base course; VA = virgin aggregate)

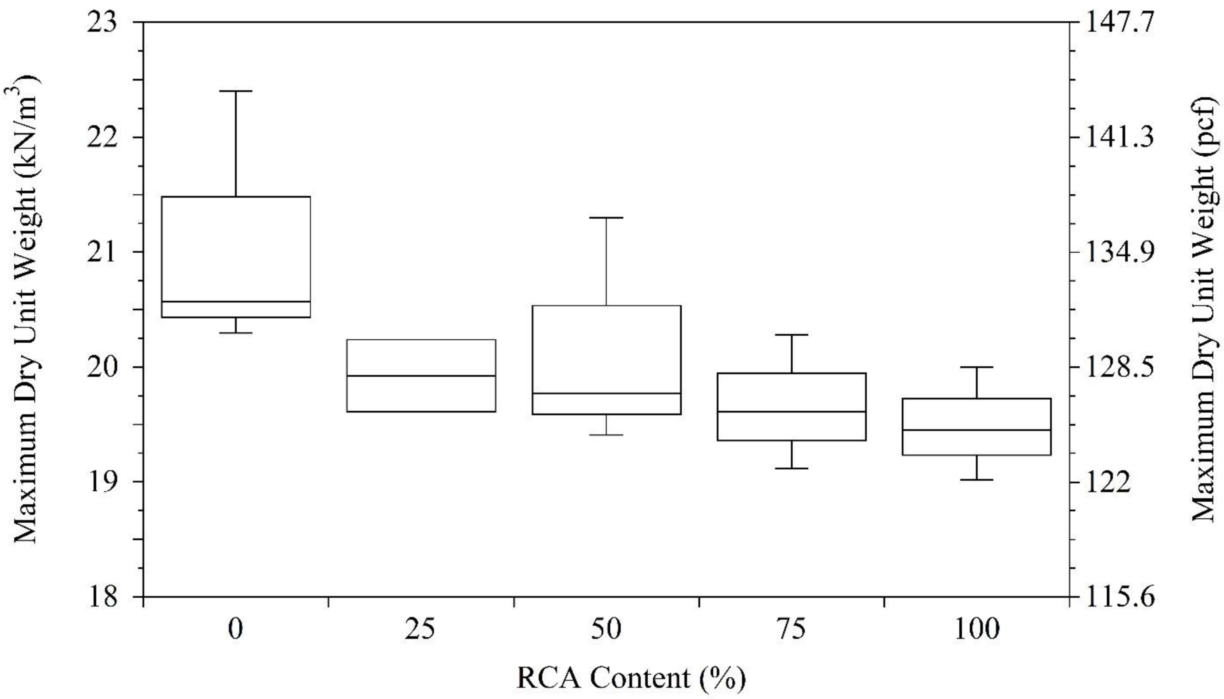

Figure 2.4. Maximum dry unit weight (MDU) vs. recycled concrete aggregate (RCA) content (box and whisker plot)

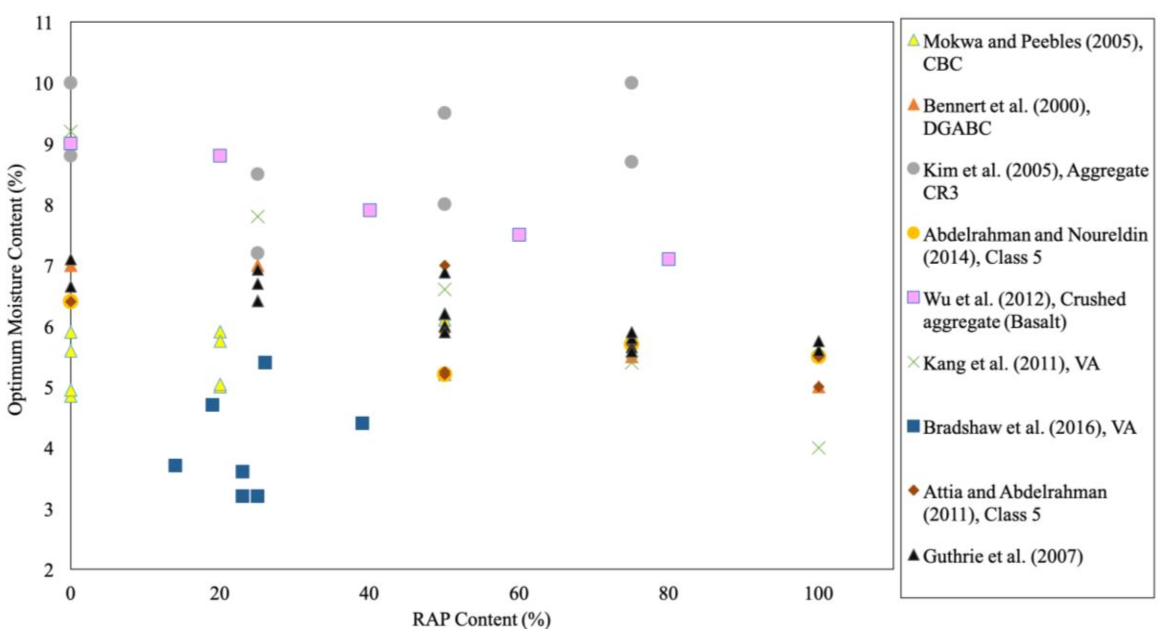

Figure 2.5. Optimum moisture content (OMC) vs. recycled asphalt pavement (RAP) content (CBC = crushed base course; DGABC = dense-graded aggregate base course; CR3 = County Road 3; VA = virgin aggregate)

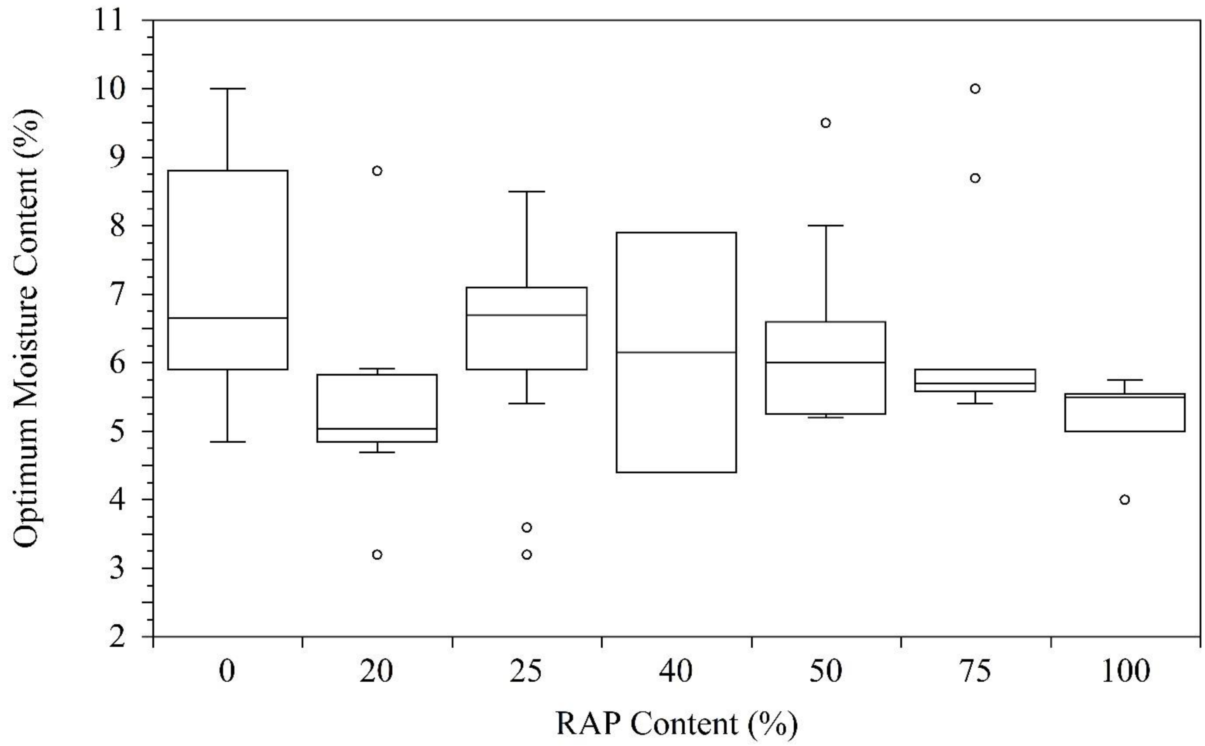

Figure 2.6. Optimum moisture content (OMC) vs. recycled asphalt pavement (RAP) content (box and whisker plot)

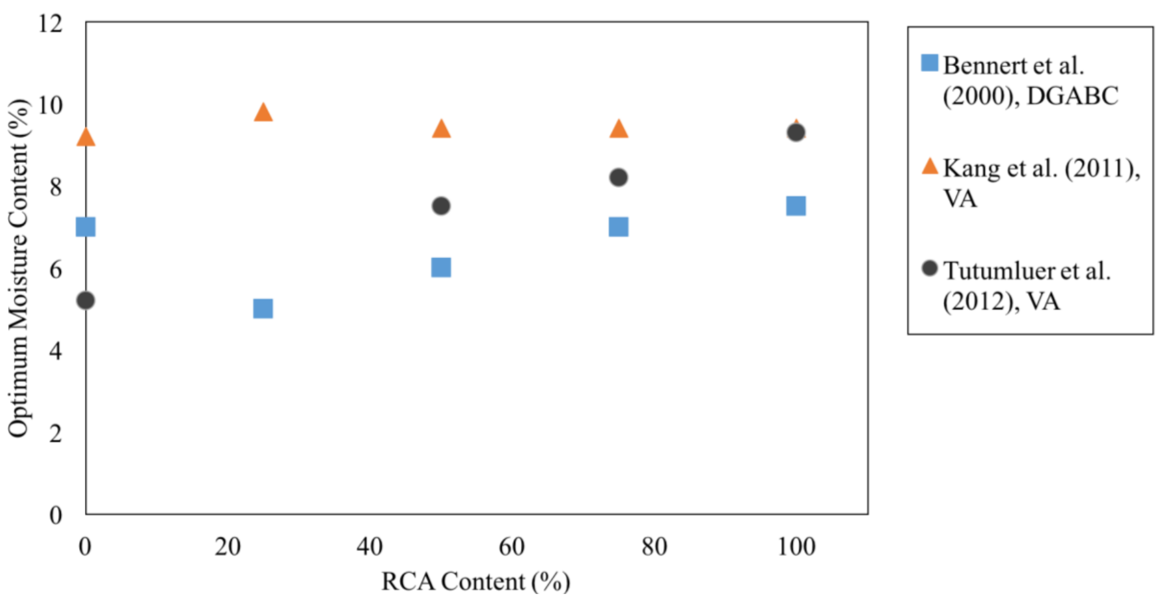

Figure 2.7. Optimum moisture content (OMC) vs. recycled concrete aggregate (RCA) content (VA = virgin aggregate; DGABC = dense-graded aggregate base course)

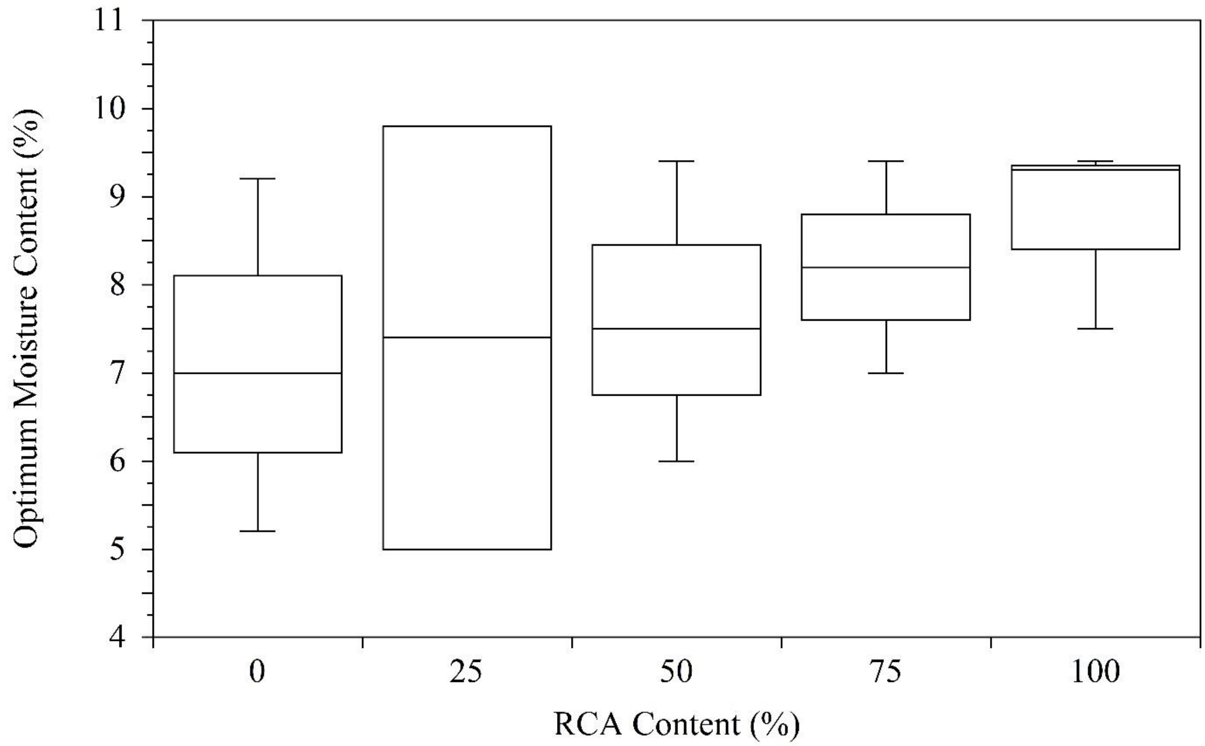

Figure 2.8. Optimum moisture content (OMC) vs. recycled concrete aggregate (RCA) content (box and whisker plot)

Table 2.8. Summary of maximum dry unit weight (MDU) and optimum moisture content (OMC) database for recycled asphalt pavement (RAP) and recycled concrete aggregate (RCA)

Parameter	RAP			RCA		
	Lower Limit	Median	Upper Limit	Lower Limit	Median	Upper Limit
MDU, kN/m^3 (pcf)	17.2 (110) [*46*]	19.6 (126) [*46*]	22.8 (146) [*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*]

MDU = maximum dry unit weight; OMC = optimum moisture content. Italic numbers provided in square brackets represent the corresponding sample size.

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

2.4 PLASTICITY

Most of the 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 their LL to be 26 and 25, respectively. They also reported LL values of 19, 20, 25, and 30 for different 50% RAP mixed with Class 5 aggregate. Class 5 aggregate is a typical base layer material used in pavement systems by Minnesota DOT (MnDOT). More detailed information about Class 5 aggregate can be found at MnDOT grading and base manual (MnDOT 2016).

CHAPTER 3: HYDRAULIC AND MECHANICAL PROPERTIES OF RAP AND RCA

Index properties, aggregate type, and asphalt binder/residual mortar contents of RAP and RCA can affect their engineering properties significantly (Thakur and Han 2015; Hiller et al. 2011). Thus, it is important to study the index and engineering properties of recycled aggregates for constructing high-quality and long-lasting pavement systems as recycled aggregates are obtained from different sources and have the potential to show very different properties (Gonzalez and Moo-Young 2004). Saturated hydraulic conductivity (K_{sat}), stiffness, shear strength, and permanent deformation properties of RAP and RCA and their relationships with the index properties of these materials are discussed and summarized in this chapter.

3.1 SATURATED HYDRAULIC CONDUCTIVITY (K_{SAT})

One of the main functions of aggregate base layers is to provide adequate drainage and prevent capillary action to increase the service life of pavements (Cedergren 1988). An increase in the pore water pressure in aggregate base layers can cause a reduction in the stiffness of aggregate base layers (Edil et al. 2012a). Saturated hydraulic conductivity (K_{sat}) 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 (Gupta et al. 2004; Ba et al. 2013). Hydraulic properties of aggregates are affected by their gradation properties (e.g., sand content, fines content, D_{10}, etc.). Fine particles can fill up the voids and reduce drainage properties of aggregates (Cosentino et al. 2003).

As previously mentioned, RAP tends to show hydrophobic properties, while RCA tends to show 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).

The relationships observed in the literature between K_{sat} values and RAP contents of RAP-VA blends are summarized in Figure 3.1. Mokwa and Peebles (2005) and Cosentino et al. (2003) reported an increase in K_{sat} values with higher RAP contents of RAP-VA blends. Kang et al. (2011) also showed 100% RAP had a higher K_{sat} value than VA. On the other hand, Wu et al. (2012) indicated that K_{sat} values of base layer aggregates decreased by the addition of RAP. After porosity analysis using X-ray scanning, it turned out that the 80% RAP had fewer air voids than the crushed aggregate specimens, which may have been the cause for observing low K_{sat} value. According to Bennert and Maher (2005), RAP-VA blends with an increase in RAP content from 25 to 75% lowered K_{sat} values of the blends to almost less than 3.5×10^{-6} m/s (4.2×10^{-2} ft/hr) while 100% RAP had a K_{sat} value of approximately 5.64×10^{-5} m/s (6.7×10^{-1} ft/hr). Kang et al. (2011) showed that the addition of 25% RAP in aggregates improved K_{sat} values of the blends since RAP was coarser than VA used in that particular study. However, with a further increase in RAP contents, K_{sat} values of the blends reduced. It was concluded that a reduction in K_{sat} values might have been due to the dense packing of the RAP-VA blends.

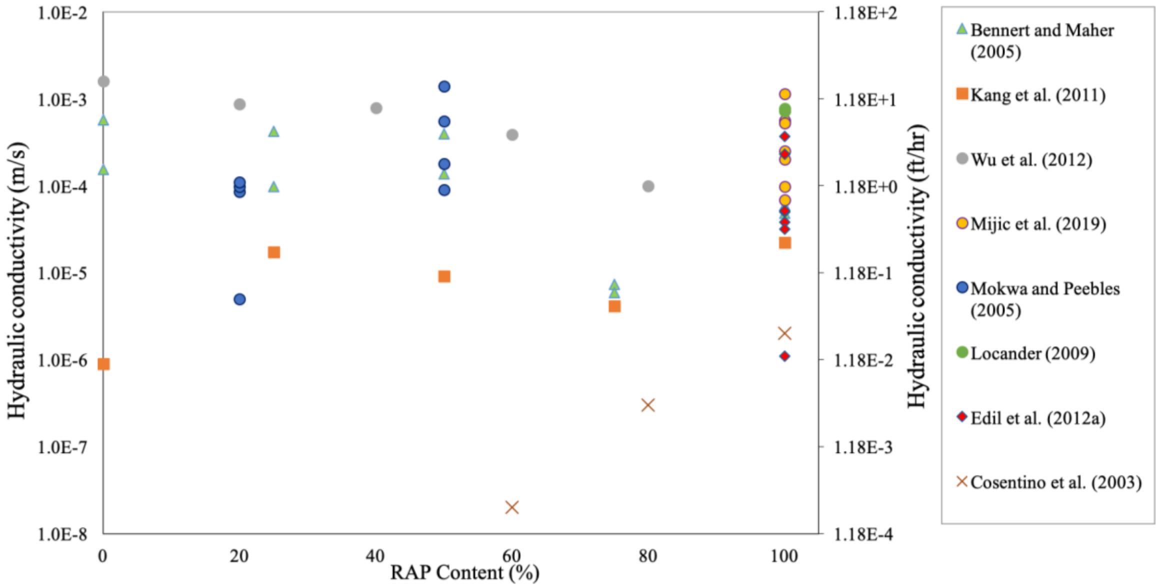

Figure 3.1. Saturated hydraulic conductivity (K_{sat}) vs. recycled asphalt pavement (RAP) content

The relationships observed in the literature between K_{sat} values and RCA contents of RCA-VA blends are summarized in Figure 3.2. Bennert and Maher (2005) showed that RCA-VA blends with an increase in RCA content from 25 to 75% of total weight lowered K_{sat} values of the blends to approximately 50%, while the K_{sat} value of the RCA was around $1x10^{-6}$ m/s ($1.2x10^{-2}$ ft/hr). According to Kang et al. (2011), K_{sat} values of VA increased with the addition of RCA up to 50% by weight. However, further addition of RCA caused a reduction in K_{sat} values. RCA alone had a higher K_{sat} value than VA, while the 50% RCA-50% VA blend had the highest K_{sat} value among all the blends.

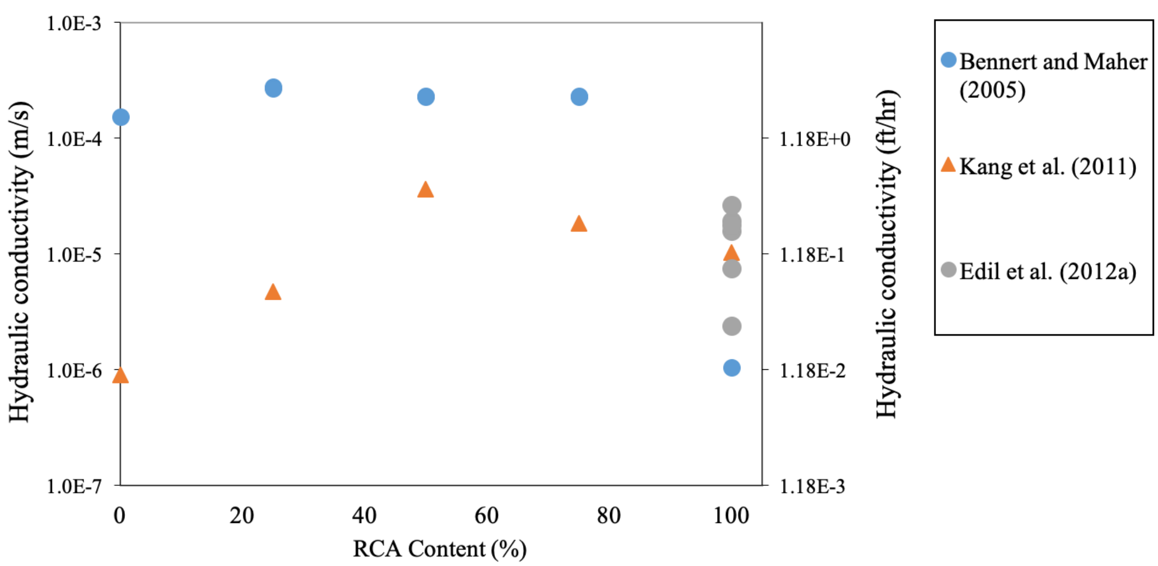

Figure 3.2. Saturated hydraulic conductivity (K_{sat}) vs. recycled concrete aggregate (RCA) content

Effective diameter (diameter at which 10% of the particles are finer – D_{10}) values and fines contents of RAP are expected to have a major influence on their K_{sat} values. Figure 3.3 and Figure 3.4 summarize the influence of D_{10} values and fines contents of RAP on their K_{sat} properties, respectively. A low D_{10} value means higher fine particles, which are expected to clog the pores present in the material matrix and reduce air voids that water can move through. Therefore, overall, it can be concluded that K_{sat} values increase with an increase in D_{10} values. Figure 3.3 confirms this relationship between K_{sat} and D_{10} values of RAP with few exceptions. According to Figure 3.4, K_{sat} values of RAP and their fines contents are inversely related to each other, as expected.

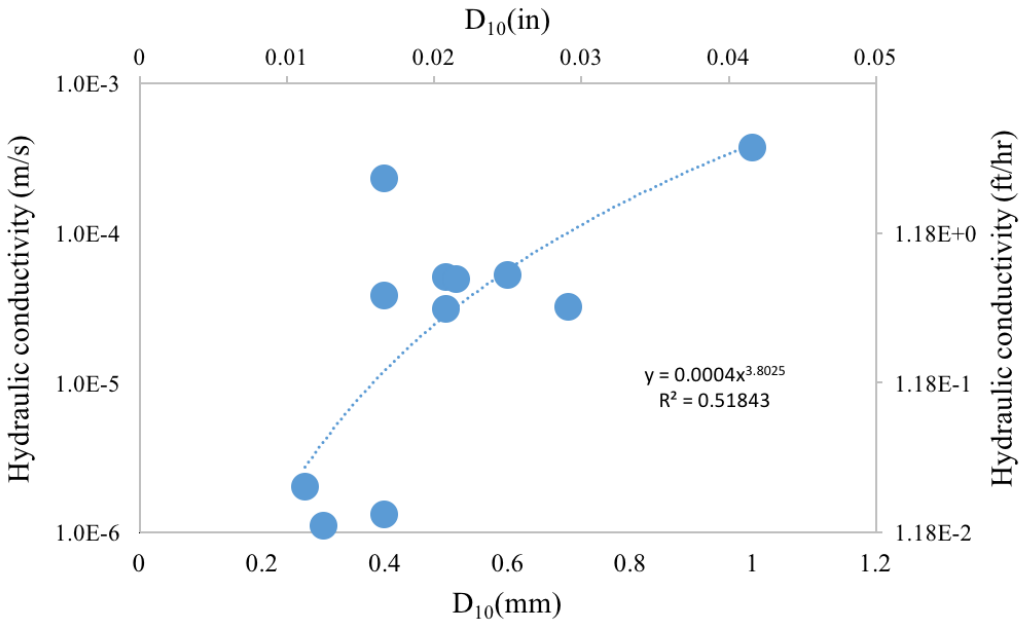

Figure 3.3. Saturated hydraulic conductivity (K_{sat}) vs. D_{10} (effective diameter) of 100% recycled asphalt pavement (RAP)

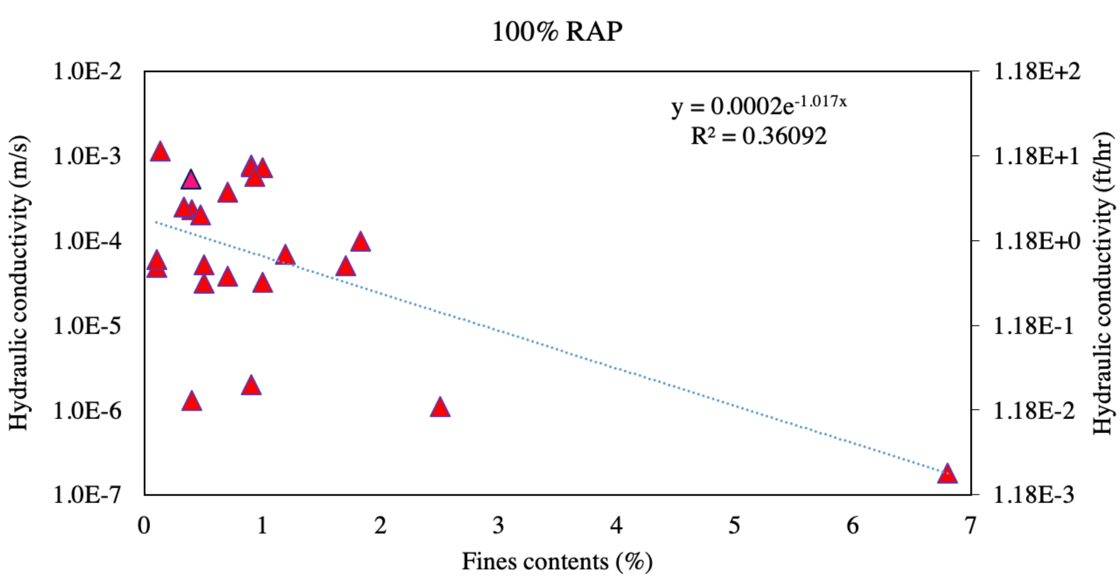

Figure 3.4. Saturated hydraulic conductivity (K_{sat}) vs. fines content of 100% recycled asphalt pavement (RAP)

Figure 3.5 shows the relationship between K_{sat} values of different RAP-VA 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).

Saturated hydraulic conductivity (K_{sat}) values of 100% RAP ranges between 1.8×10^{-7} and 1.1×10^{-3} m/s (2.1×10^{-3} and 13 ft/hr) with a median value of 6.9×10^{-5} m/s (8.1×10^{-1} ft/hr) according to the K_{sat} database.

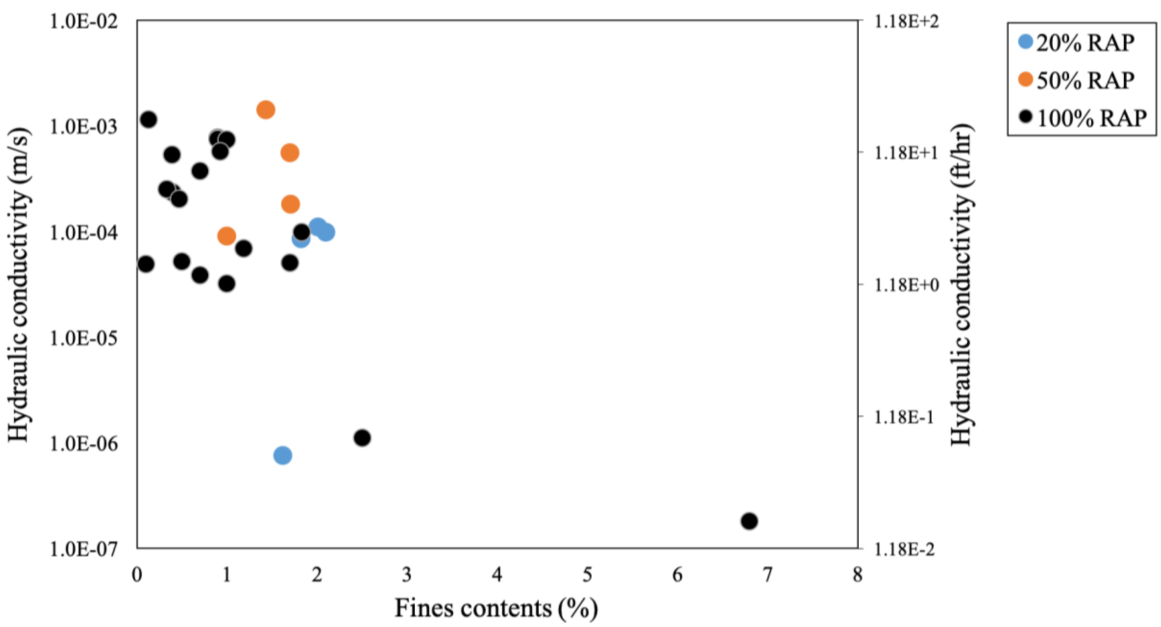

Figure 3.5. Saturated hydraulic conductivity (K_{sat}) vs. fines contents of recycled asphalt pavement (RAP) blends

In Figure 3.6, there are 11 different K_{sat} data for 100% RCA with corresponding fines contents. No trend was observed between K_{sat} values and fines contents of RCA. Saturated hydraulic conductivity (K_{sat}) values of RCA range between 1.05×10^{-6} and 1.2×10^{-3} m/s (1.2×10^{-2} and 14.2 ft/hr) with a median value of 1.7×10^{-5} m/s (2×10^{-1} ft/hr) according to the K_{sat} database.

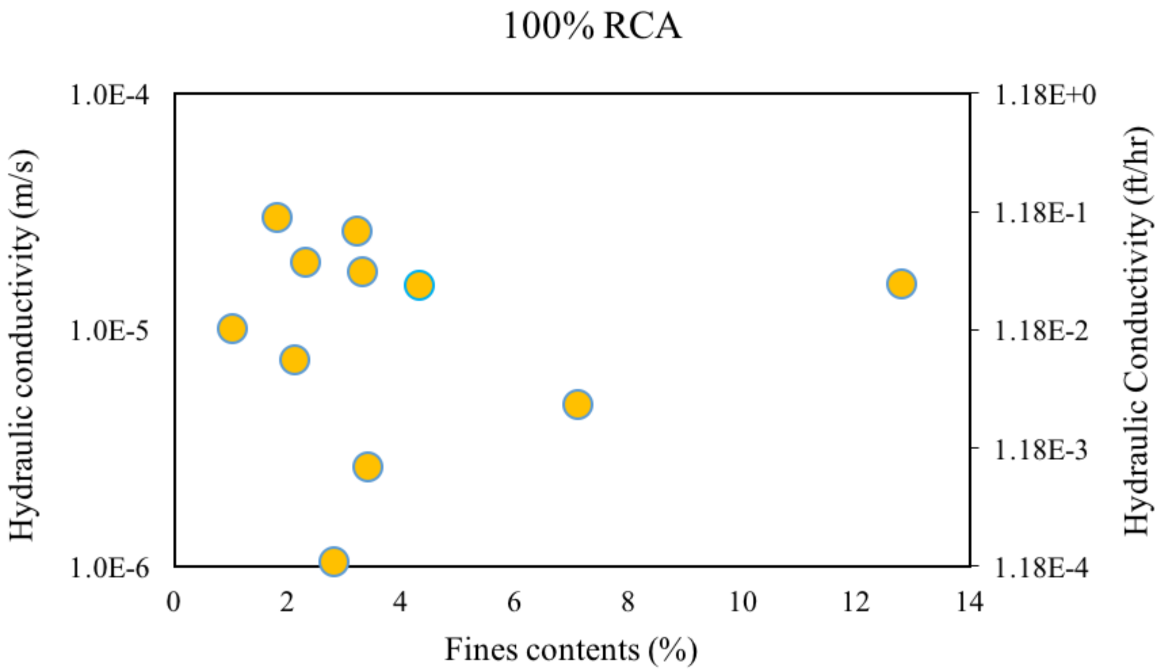

Figure 3.6. Saturated hydraulic conductivity (K_{sat}) vs. fine content of 100% recycled concrete aggregate (RCA)

3.2 CALIFORNIA BEARING RATIO (CBR)

California bearing ratio (CBR) of base layer aggregates is an indication of their mechanical characteristics under vertical loading (i.e., traffic) and is determined as the ratio of the penetration resistance of the base layer aggregate to that of a standard crushed stone. California bearing ratio (CBR) has been used by 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, limerock bearing ratio (LBR), which is a modified version of the 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 CBR database showed that CBR values of 100% RAP range from 18 to 68% with a median value of 28%, while CBR values of 100% RCA are between 58 and 169% with a median value of 146%.

3.2.1 Effect of Index Properties on CBR

Gravel-to-sand (G/S) ratio and fines contents were selected as the index properties to investigate their effects on CBR of RAP and RCA. Figure 3.7 and Figure 3.8 summarize the influence of the G/S ratios and fines contents of RAP on their CBR, respectively. These figures revealed that there was no specific trend between these parameters [this could be because of the low number of data points (5) that could be collected from the literature].

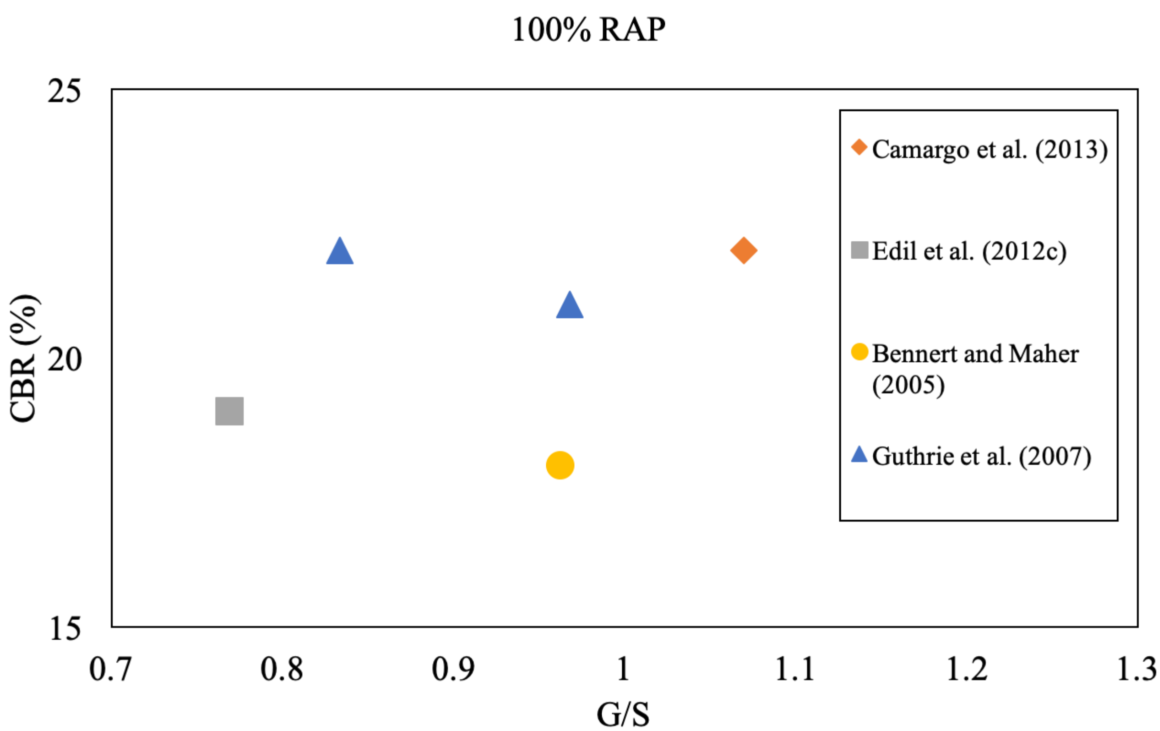

Figure 3.7. California bearing ratio (CBR) vs. gravel-to-sand ratio (G/S) of 100% recycled asphalt pavement (RAP)

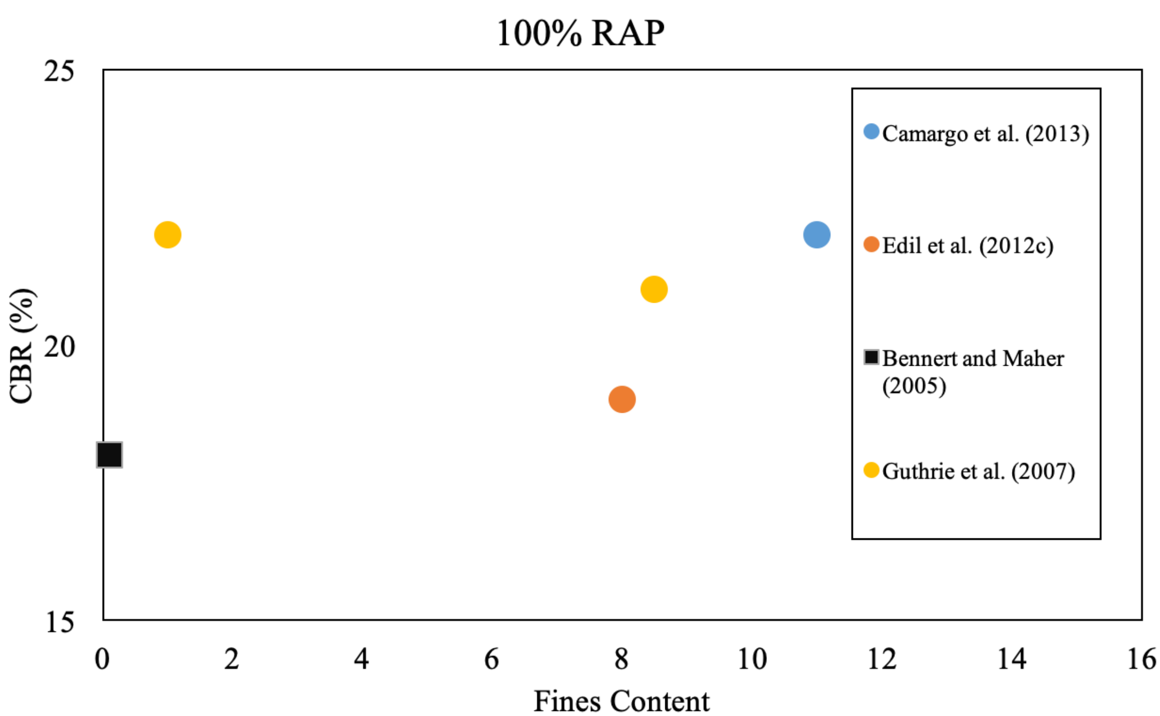

Figure 3.8. California bearing ratio (CBR) vs. fines content of 100% recycled asphalt pavement (RAP)

Figure 3.9 and Figure 3.10 summarize the influence of the G/S ratios and fines contents of RCA on their CBR properties, respectively. These figures revealed that there was no specific trend between these parameters [this could be due to the low number of data (6) that could be collected from the literature].

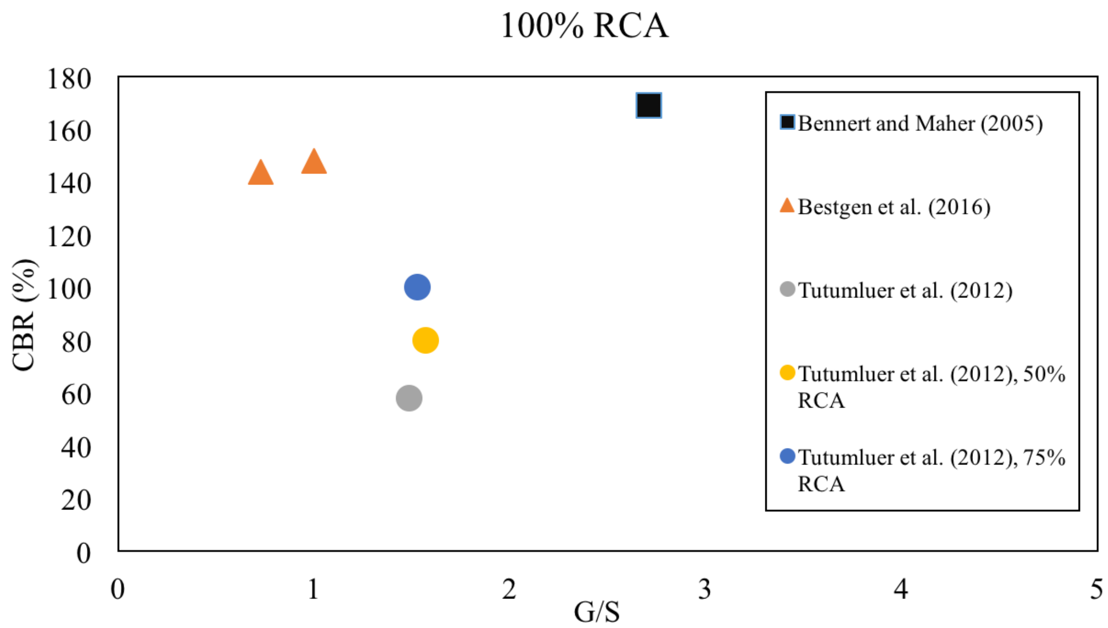

Figure 3.9. California bearing ratio (CBR) vs. gravel-to-sand ratio (G/S) of recycled concrete aggregate (RCA)

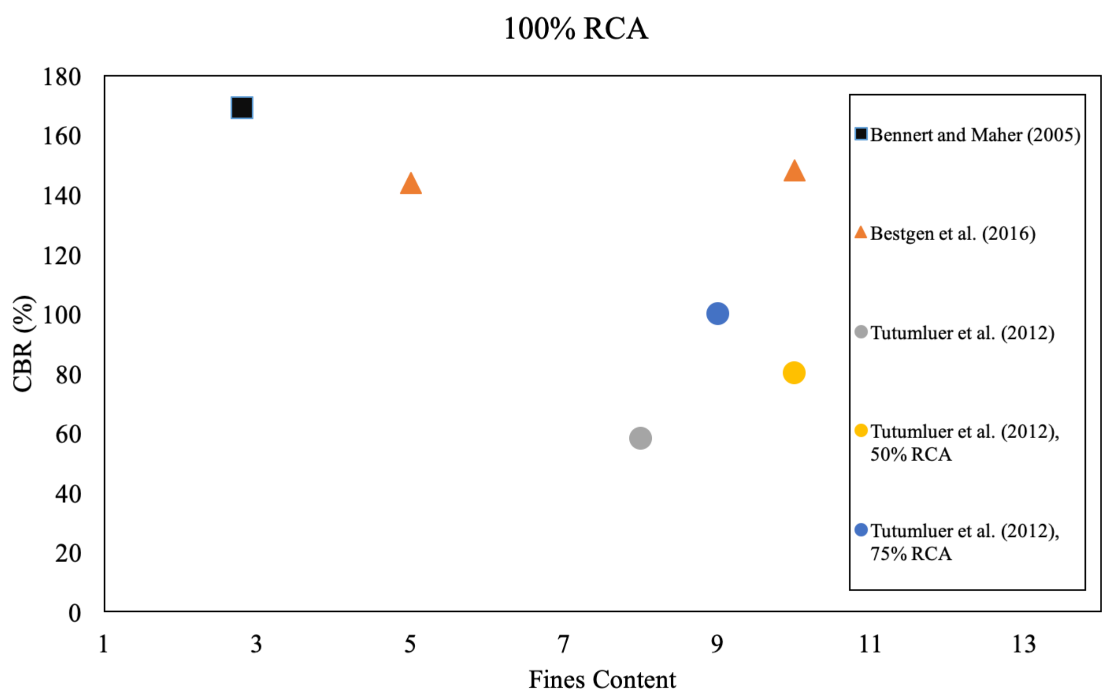

Figure 3.10. California bearing ratio (CBR) vs. fines content of recycled concrete aggregate (RCA)

3.2.2 Effect of RAP/RCA Content on CBR

In general, RAP has lower CBR values than VA. In addition, increasing the RAP content in RAP-VA blends reduces CBR values (Bennert and Maher 2005; Guthrie et al. 2007). Figure 3.11 shows that CBR values of RAP-VA blends tend to decrease with increasing the RAP contents. The asphalt coating around the particles may be the reason for CBR reduction in the presence of RAP in RAP-VA blends since asphalt coating can reduce the bonding and interlocking between particles (Ooi et al. 2010; Taha et al. 1999). In addition, a lower fines content of RAP may leave unfilled voids (i.e., open-graded structure), which may result in lower CBR values (Sayed et al. 1993). Bennert and Maher (2005), Ullah and Tanyu (2019), Cosentino and Bleakley (2013), and Guthrie et al. (2007) conducted CBR tests on RAP-VA blends, and all of them reported a decrease in CBR values with an increase in the RAP content. On the other hand, Cosentino et al. (2003) observed that CBR values of the RAP-VA blends increased first with an increase in the RAP content up to a certain level (~RAP content = 80%) and then started decreasing.

Depending on the physical, chemical, and morphological characteristics of RAP and/or moisture contents used for RAP-VA blends, different trends could be observed in different applications (Thakur and Han 2015). Figure 3.11 reports the type of each material that is blended with RAP. These materials included VA, limerock (LR), base material, dense-graded aggregate base course (DGABC), and fine sand.

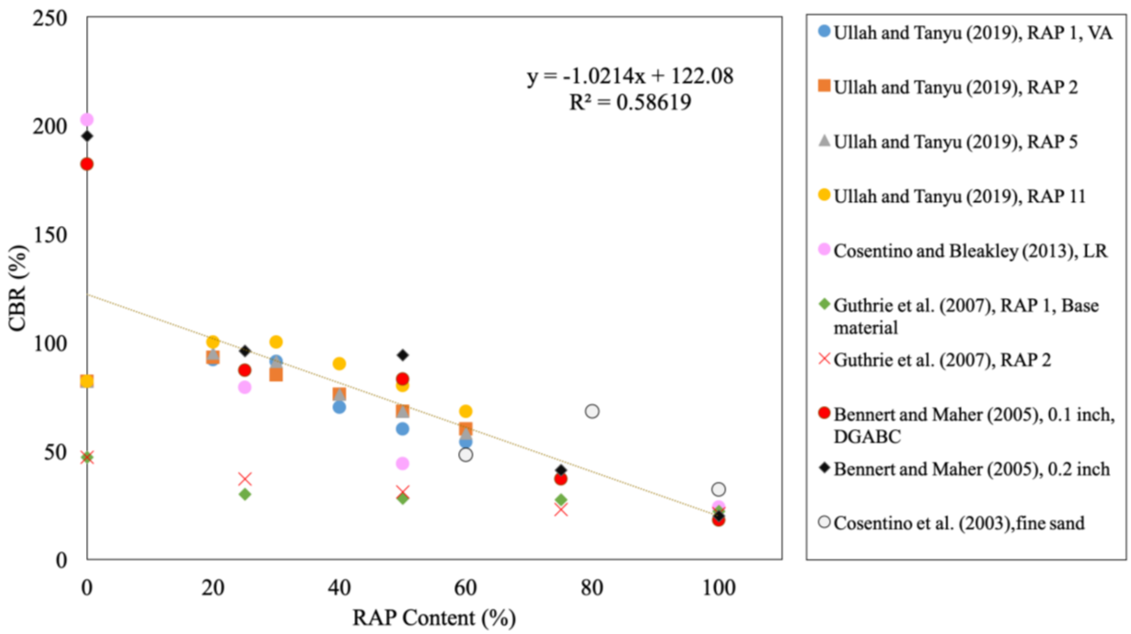

Figure 3.11. California bearing ratio (CBR) vs. recycled asphalt pavement (RAP) (VA = virgin aggregate; LR = limerock; DGABC = dense-graded aggregate base course)

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

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

To investigate CBR properties of RCA-VA blends, Figure 3.12 and Figure 3.13 are presented (Figure 3.13 shows normalized CBR vs. RCA content. Normalized CBR was obtained by dividing CBR of each RCA blend by CBR of 100% RCA determined in the same study). These figures show that there is not a discernible trend between CBR values and RCA contents of the blends.

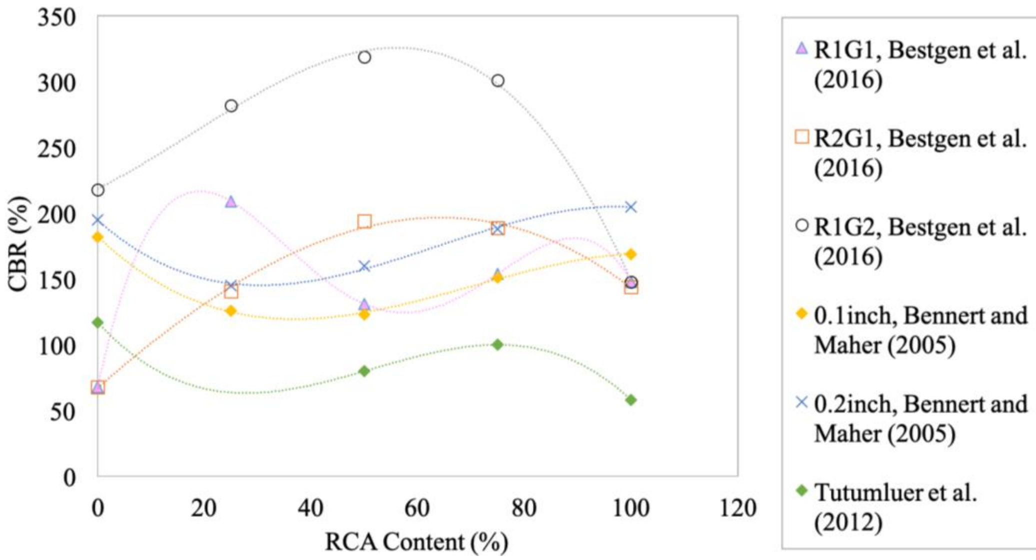

Figure 3.12. California bearing ratio (CBR) vs. recycled concrete aggregate (RCA) content [CBR value corresponding to 0.1 and 0.2 in of penetration was used in Bennert and Maher (2005)]

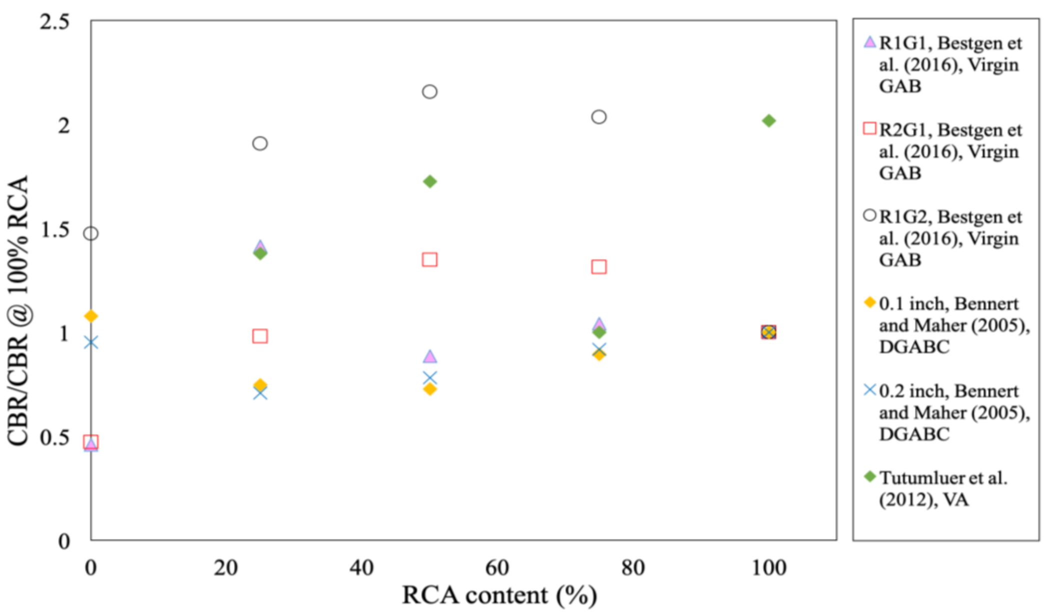

Figure 3.13. Normalized California bearing ratio (CBR) vs. recycled concrete aggregate (RCA) content (virgin GAB = virgin-graded aggregate base; DGABC = dense-graded aggregate base course; VA = virgin aggregate)

3.3 STIFFNESS

Resilient modulus (M_r) is a fundamental material property used to analyze the stiffness of materials under different conditions, such as moisture content, unit weight, and stress level. It is defined as a ratio of applied deviator stress and recoverable strain. M_r properties of RAP and RCA depend on several factors, including moisture content, unit weight, stress history, aggregate type, material composition, gradation, temperature, etc. (Thakur and Han 2015; Bestgen et al. 2016).

Resilient modulus (M_r) plays an important role in pavement design. The AASHTO pavement design guide (AASHTO 1993) and the AASHTOWare PMED use M_r to define subgrade, subbase, and aggregate base stiffness for pavement systems. In general, summary resilient modulus (SM_r), which is considered to be the M_r at specific bulk and octahedral shear stresses, are used in pavement design. For example, for aggregate base layers, SM_r at bulk stress of 208 kPa (30.2 psi) and octahedral shear stress of 48.6 kPa (7 psi) (as recommended by NCHRP 1-28A) are used in pavement design.

Appendix B reports the M_r database for RAP and RCA. Overall, it was observed that RAP and RCA used in aggregate base layers had higher M_r than well-graded VA. For RAP-VA/RCA-VA blends, as the RAP/RCA content increases, M_r tends to increase. More than 400 M_r data investigating the M_r of RAP, RCA, RAP-VA, and RCA-VA blends were collected for the M_r database. The M_r database 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. SM_r values reported in the database for RAP are between 168 and 400 MPa (24,366 and 58,015 psi) with a median value of 262 MPa (37,927 psi). Summary M_r (SM_r) values of RCA range between 123.4 and 370 MPa (17,897 and 53,664 psi) with a median value of 183 MPa (26,541 psi), according to the database.

3.3.1 Effect of Index Properties on M_r

Figure 3.14 presents the relationship between SM_r values of RAP and their gravel-to-sand (G/S) ratios. Figure 3.15 shows that SM_r values of RAP are lower at higher G/S ratios. This can indicate that RAP with higher sand contents would have higher SM_r values in general.

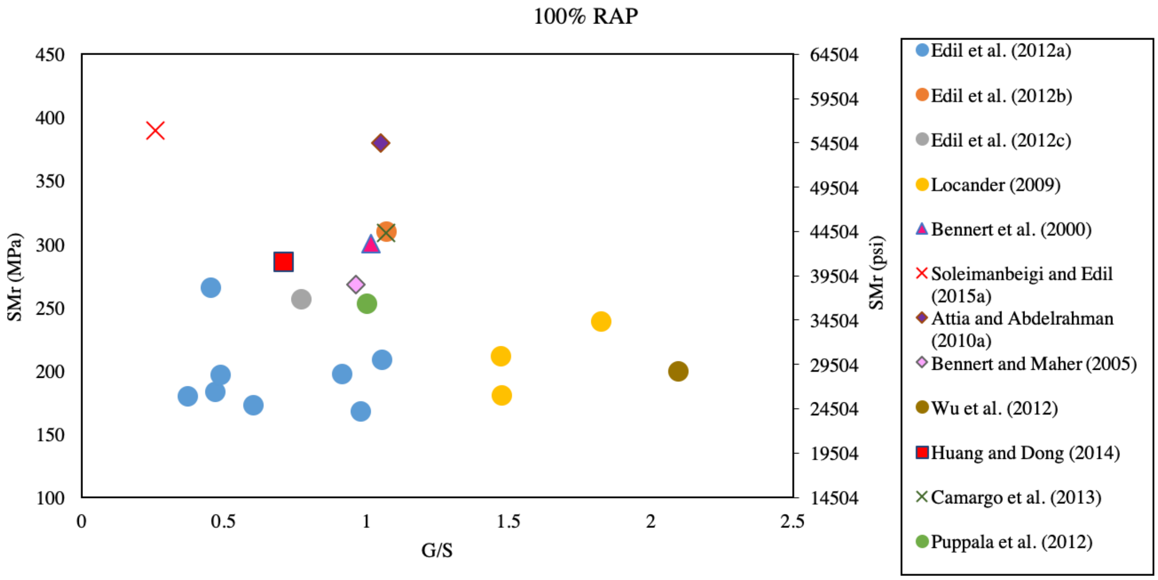

Figure 3.14. Summary resilient modulus (SM_r) vs. gravel-to-sand ratio (G/S) of 100% recycled asphalt pavement (RAP)

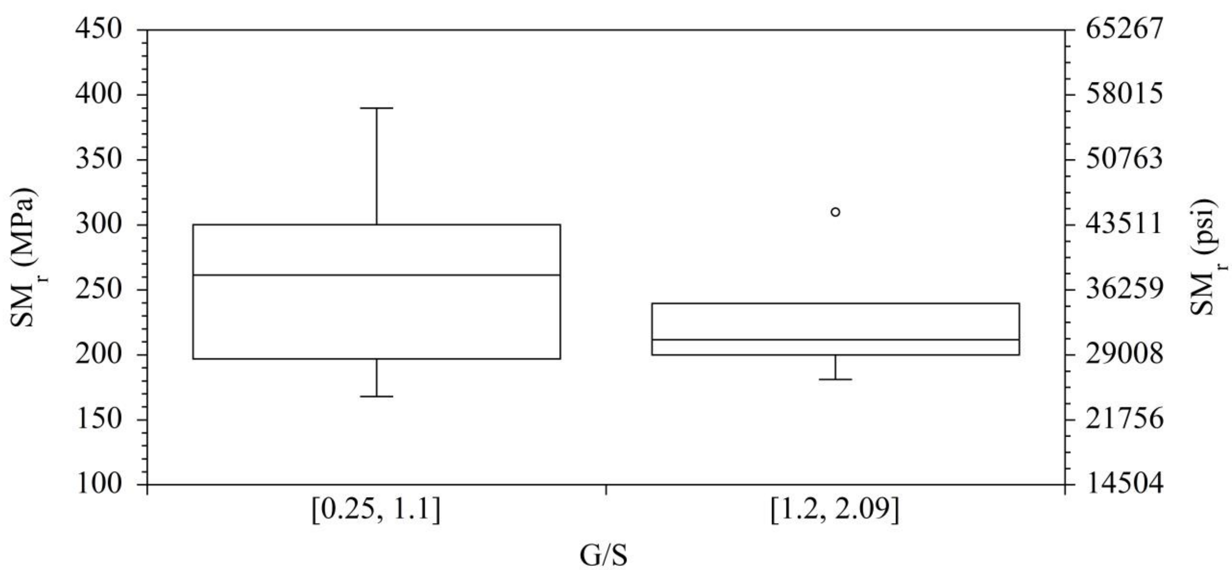

Figure 3.15. Summary resilient modulus (SM_r) vs. gravel-to-sand ratio (G/S) of 100% recycled asphalt pavement (RAP) (box and whisker plot)

Figure 3.16 and Figure 3.17 show that there was no clear trend between SM_r values and fines contents of RAP (within the typical ranges observed in this study).

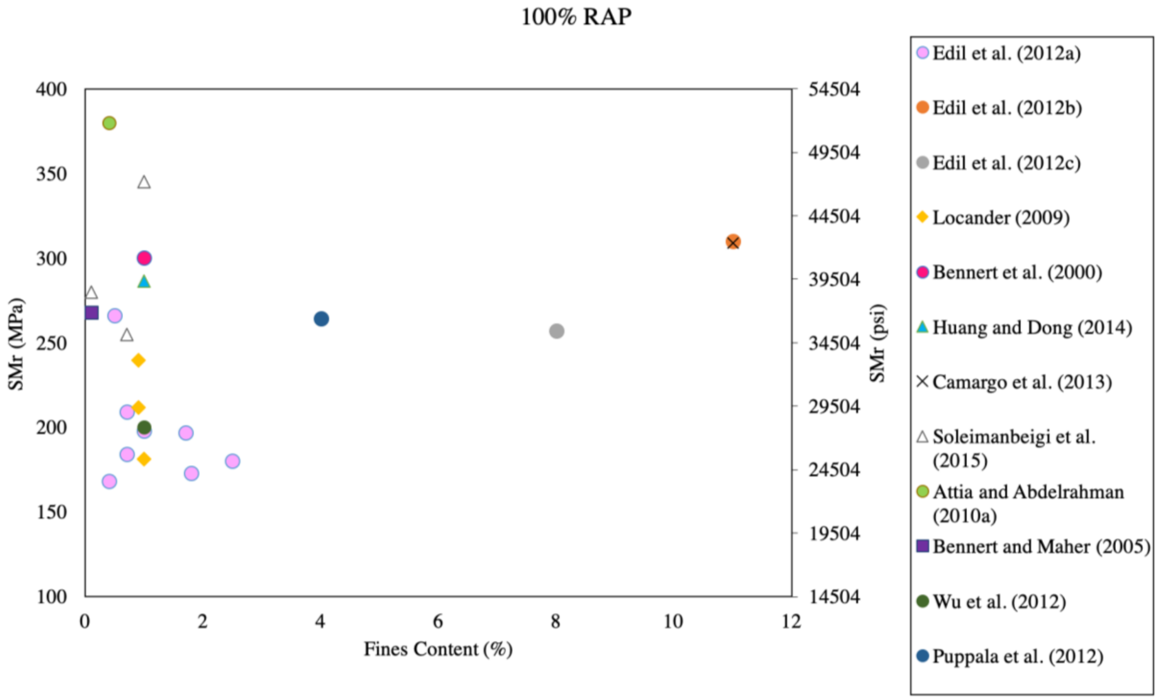

Figure 3.16. Summary resilient modulus (SM_r) vs. fines content of 100% recycled asphalt pavement (RAP)

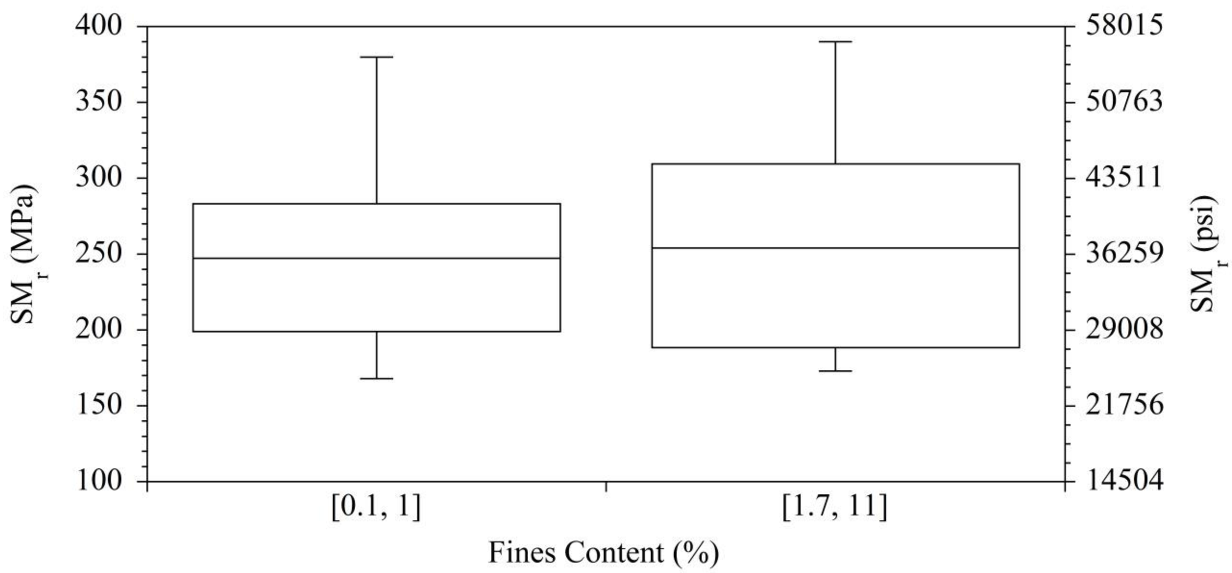

Figure 3.17. Summary resilient modulus (SM_r) vs. fines content of 100% recycled asphalt pavement (RAP) (box and whisker plot)

Figure 3.18 and Figure 3.19 show that the trend between SM_r values and G/S ratios of RCA was not as notable as that observed for RAP. However, it could be concluded that SM_r values of RCA are higher at higher G/S ratios.

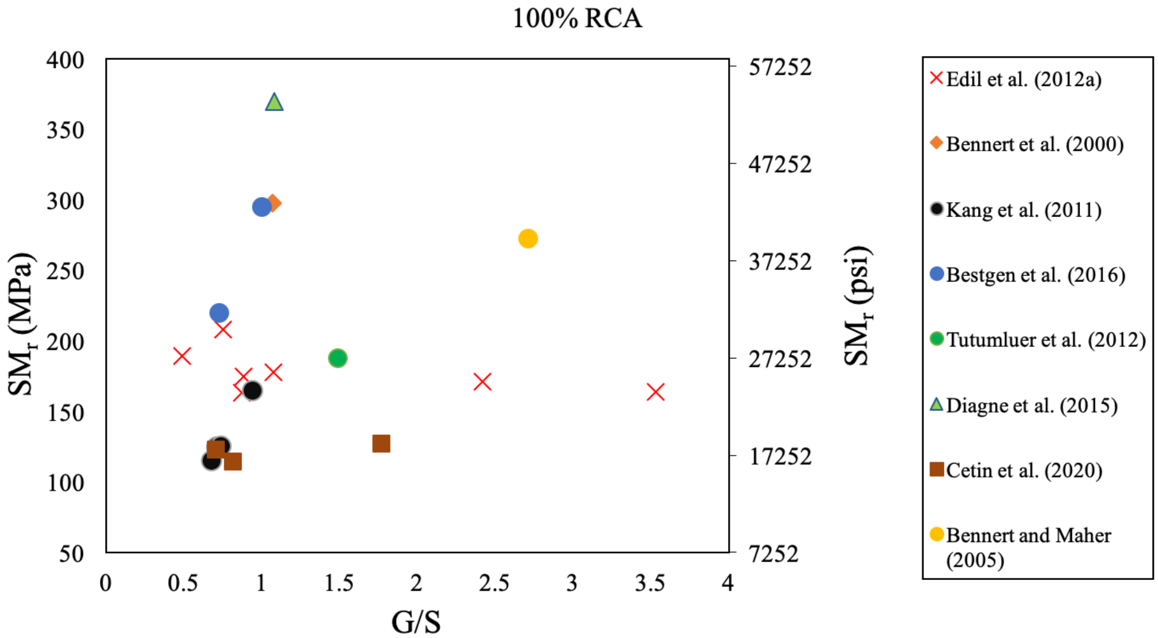

Figure 3.18. Summary resilient modulus (SM_r) vs. gravel-to-sand ratio (G/S) of 100% recycled concrete aggregate (RCA)

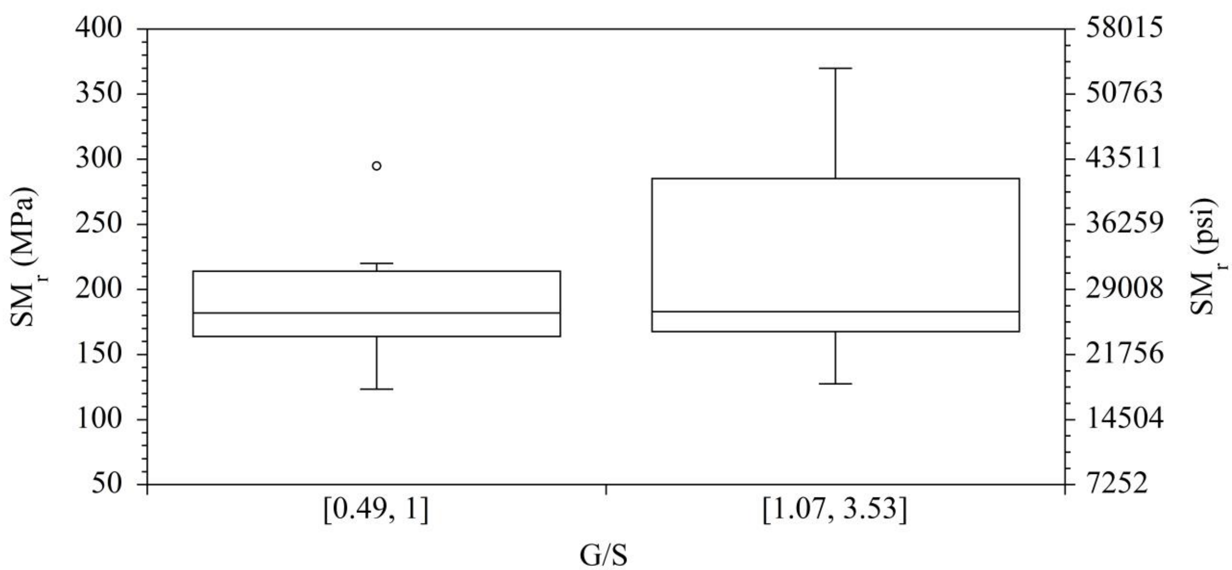

Figure 3.19. Summary resilient modulus (SM_r) vs. gravel-to-sand ratio (G/S) of 100% recycled concrete aggregate (RCA) (box and whisker plot)

According to Figure 3.20 and Figure 3.21, there is a slight decrease in SM_r values of RCA as fines content increases. However, this was not consistent with some other studies, such as Bestgen et al. (2016).

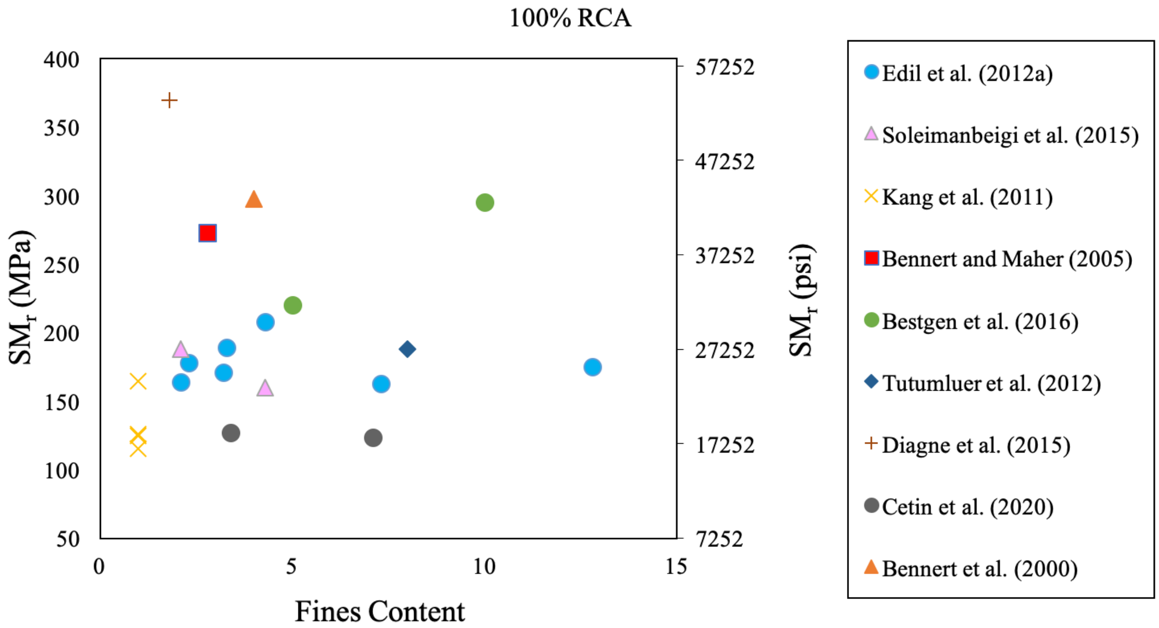

Figure 3.20. Summary resilient modulus (SM_r) vs. fines content of 100% recycled concrete aggregate (RCA)

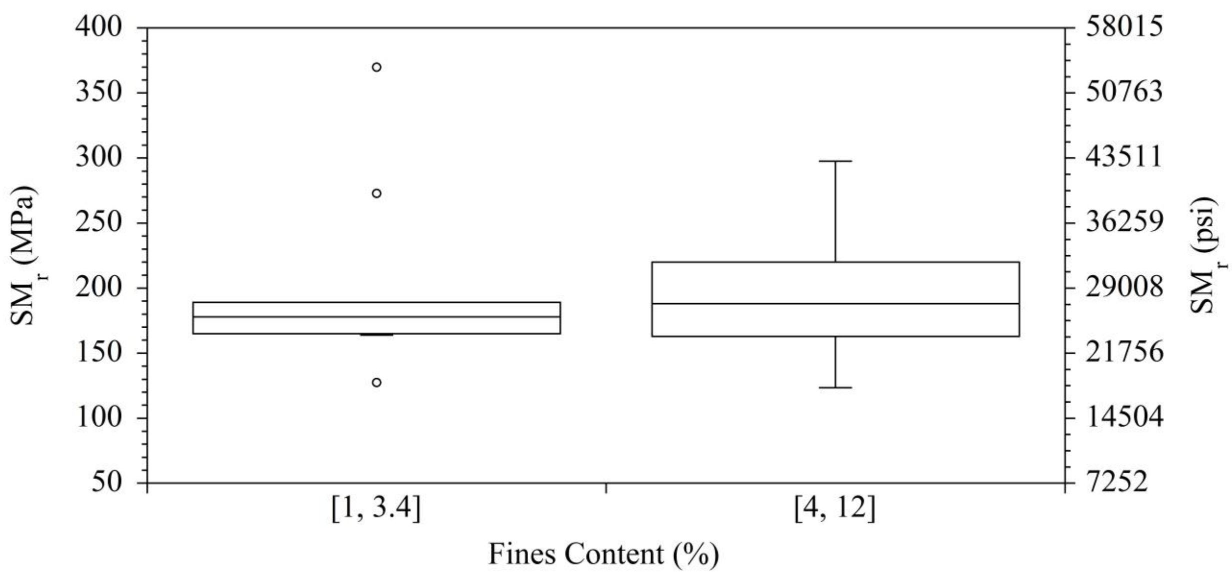

Figure 3.21. Summary resilient modulus (SM_r) vs. fines content of 100% recycled concrete aggregate (RCA) (box and whisker plot)

MDU and OMC values of RAP from different studies were plotted against their SM_r values in Figure 3.22 and Figure 3.23, respectively. To better understand these scatter plots, Figure 3.24 (SM_r vs. MDU) and Figure 3.25 (SM_r vs. OMC) provide summaries in box and whisker plots. However, no significant trend was observed between MDU values of RAP and their SM_r values (Figure 3.24). On the other hand, there was a slight decrease in SM_r values of RAP with an increase in their OMC values (Figure 3.25).

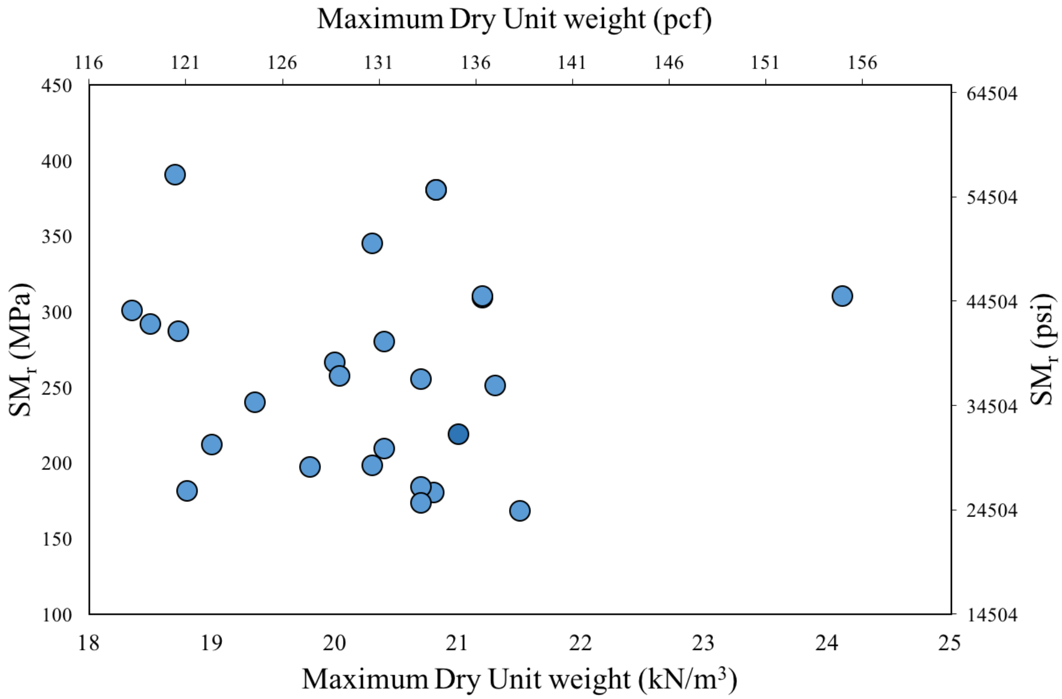

Figure 3.22. Summary resilient modulus (SMr) vs. maximum dry unit weight (MDU) of 100% recycled asphalt pavement (RAP)

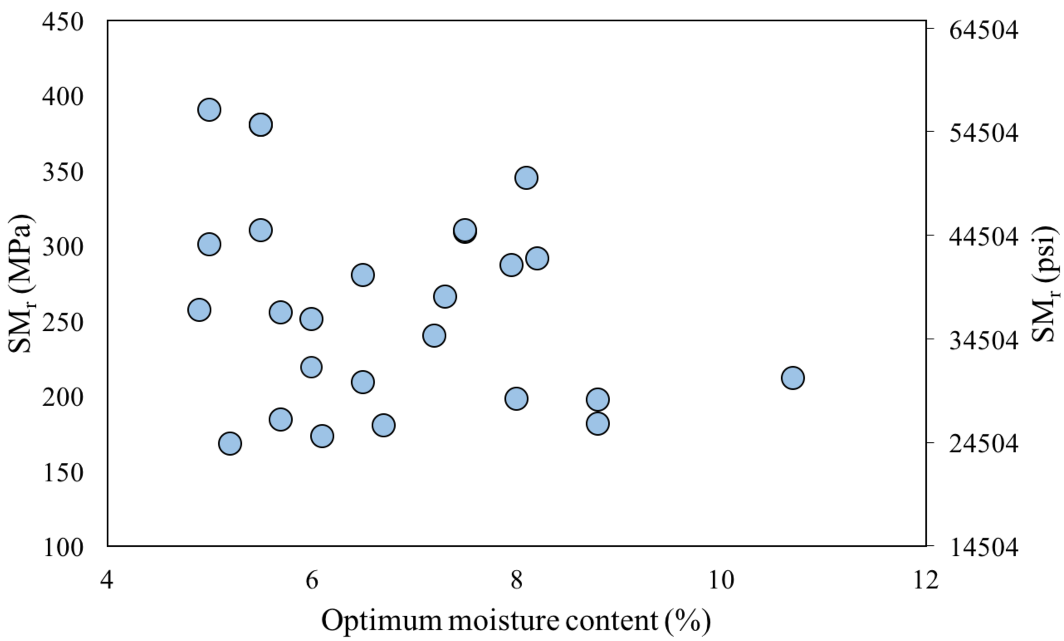

Figure 3.23. Summary resilient modulus (SMr) vs. optimum moisture content (OMC) of 100% recycled asphalt pavement (RAP)

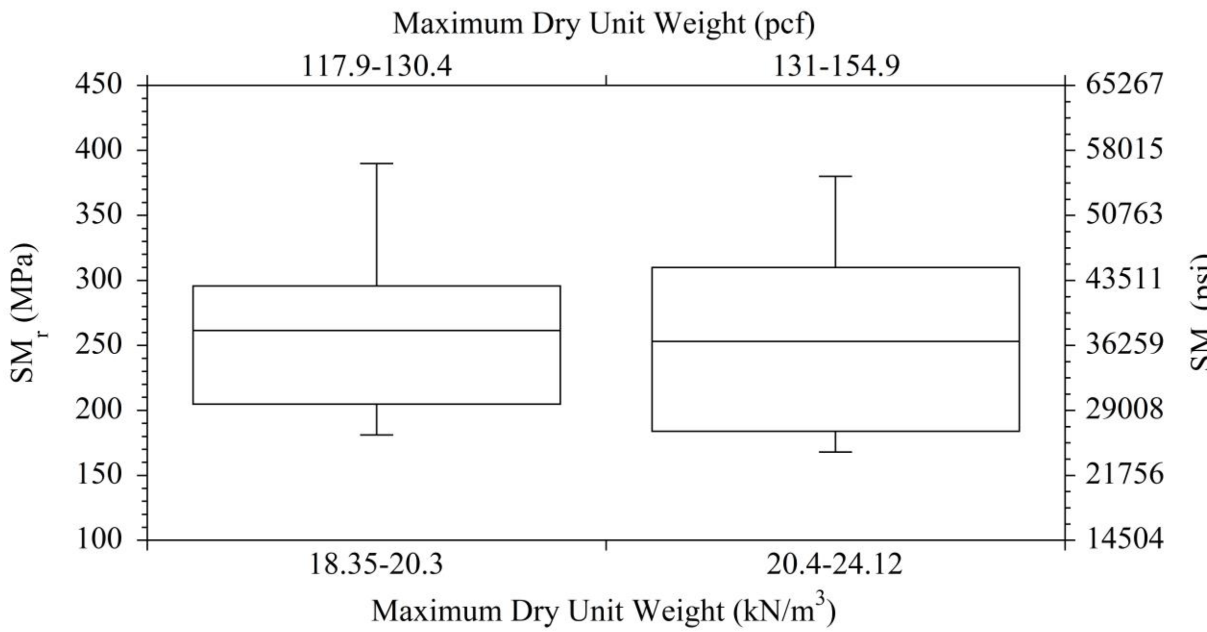

Figure 3.24. Summary resilient modulus (SM_r) vs. maximum dry unit weight (MDU) of 100% recycled asphalt pavement (RAP) (box and whisker plot)

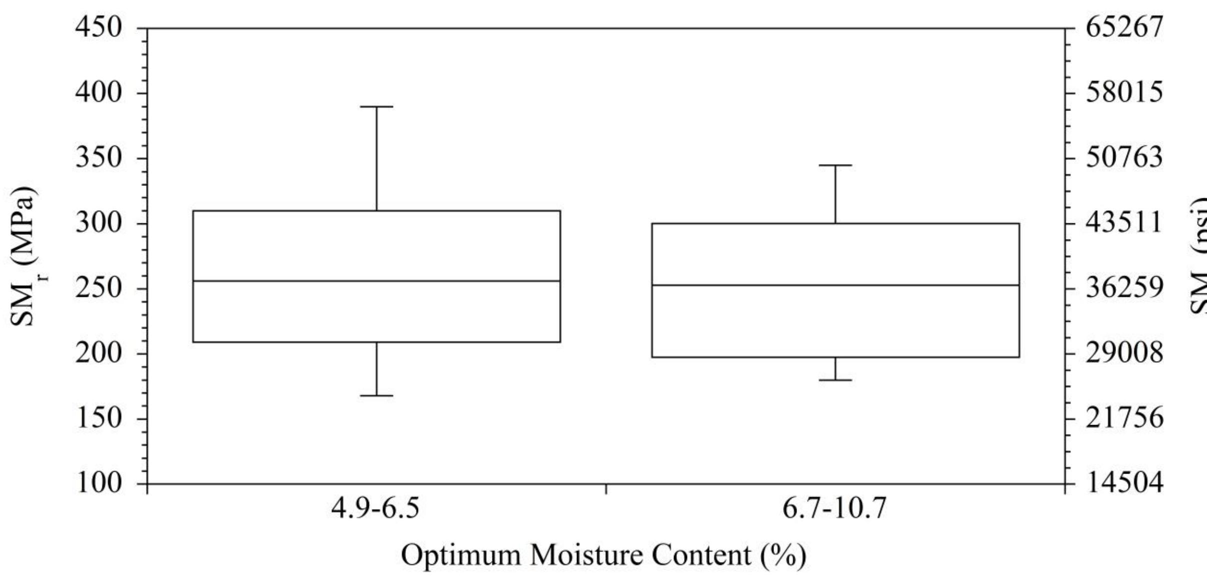

Figure 3.25. Summary resilient modulus (SM_r) vs. optimum moisture content (OMC) of 100% recycled asphalt pavement (RAP) (box and whisker plot)

Diameter at which 30% of the particles are finer (D_{30}) and D_{60} values of RAP from different studies were plotted against their SM_r values in Figure 3.26 and Figure 3.27, respectively. To better understand these scatter plots, Figure 3.28 (D_{30} vs. SM_r) and Figure 3.29 (D_{60} vs. SM_r) provide summaries in box and whisker plots. According to Figure 3.28, RAP with higher D_{30} values tend to have higher SM_r values. A similar trend was also observed in Figure 3.29. It shows that SM_r values of RAP tend to be higher when they have higher D_{60} values.

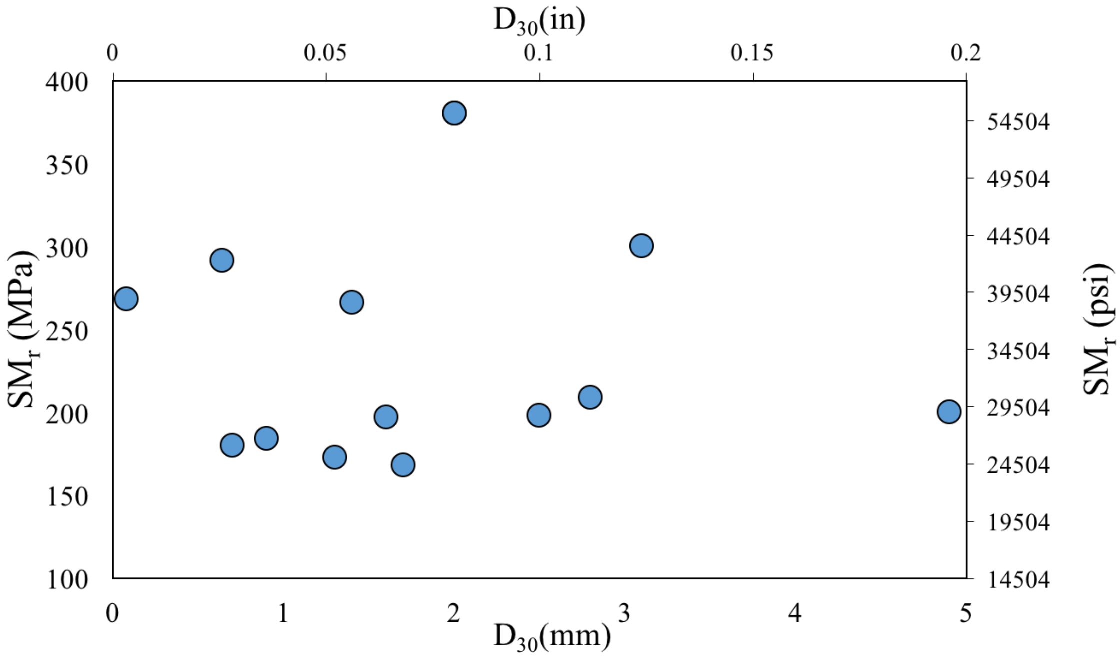

Figure 3.26. Summary resilient modulus (SM_r) vs. D_{30} (diameter at which 30% of the particles are finer) of 100% recycled asphalt pavement (RAP)

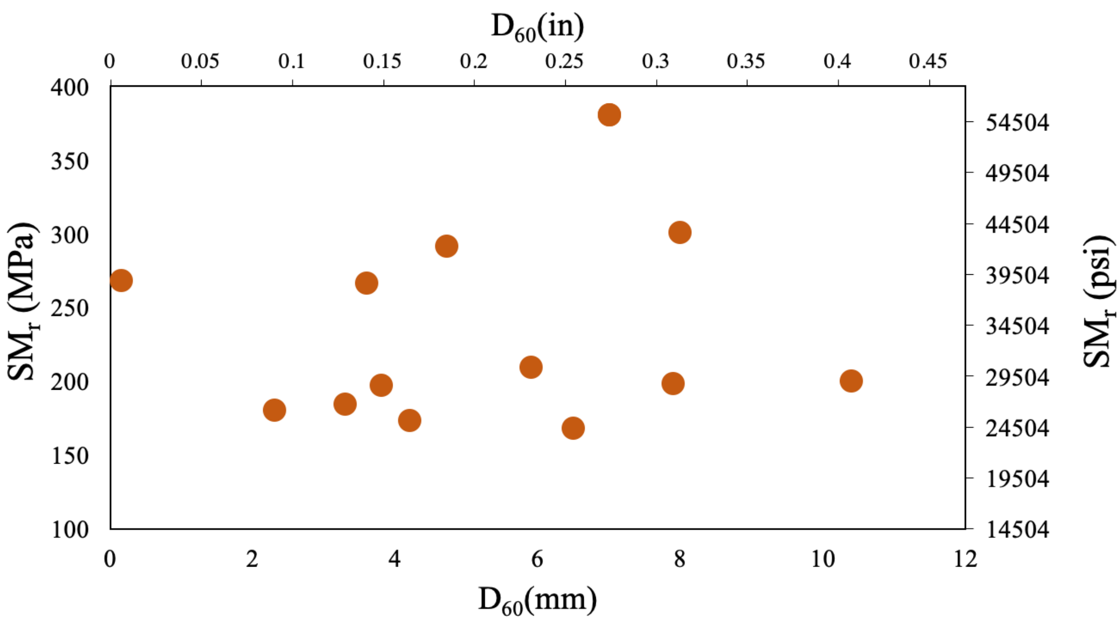

Figure 3.27. Summary resilient modulus (SM_r) vs. D_{60} (diameter at which 60% of the particles are finer) of 100% recycled asphalt pavement (RAP)

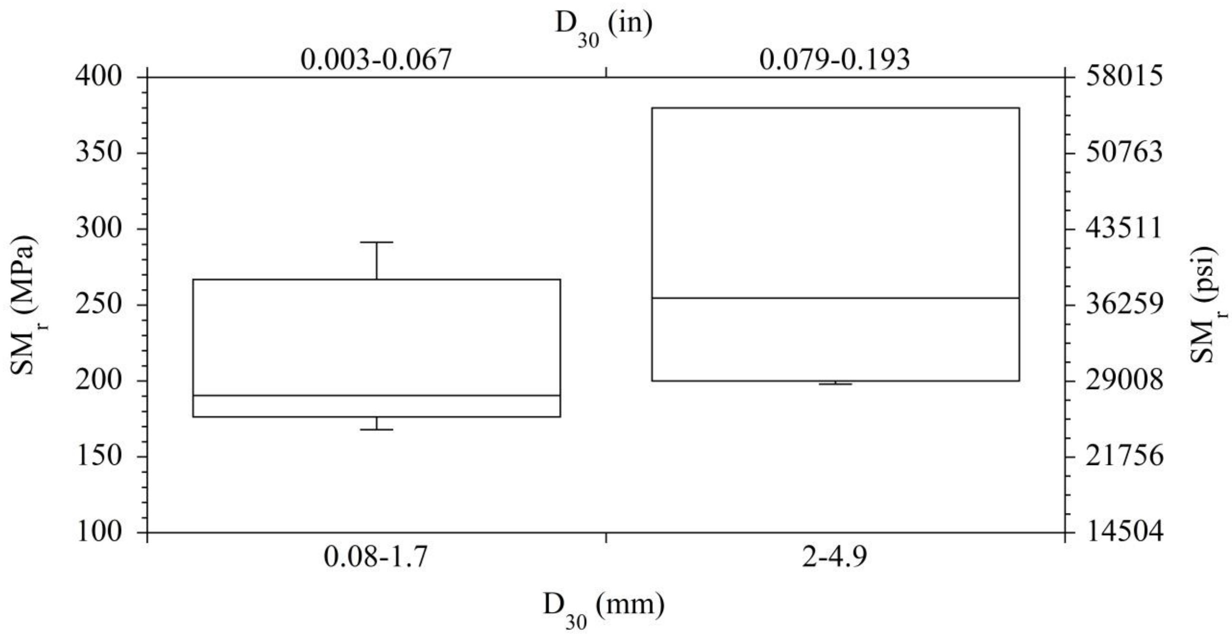

Figure 3.28. Summary resilient modulus (SM_r) vs. D_{30} (diameter at which 30% of the particles are finer) of 100% recycled asphalt pavement (RAP) (box and whisker plot)

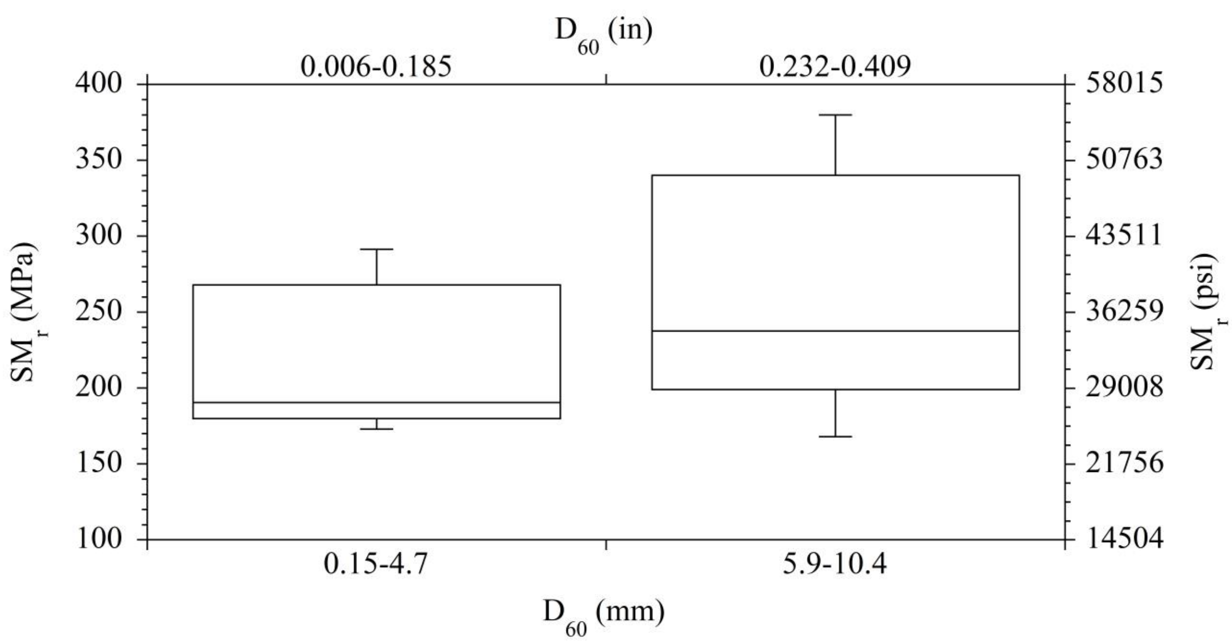

Figure 3.29. Summary resilient modulus (SM_r) vs. D_{60} (diameter at which 60% of the particles are finer) of 100% recycled asphalt pavement (RAP) (box and whisker plot)

Coefficient of curvature (C_c) and C_u of RAP from different studies were plotted against their SM_r values in Figure 3.30 and Figure 3.31, respectively. To better understand these scatter plots, Figure 3.32 (C_c vs. SM_r) and Figure 3.33 (C_u vs. SM_r) provide summaries in box and whisker plots. Based on the data collected, it was observed that SM_r values of RAP were higher when their C_c was lower than 1 and higher than 3 (C_c should be between 1 and 3 for well-graded gravel and sand). It means that poor-graded RAP may have higher SM_r values. Figure 3.33 shows that an increase in C_u values of RAP may yield some increase in their SM_r values.

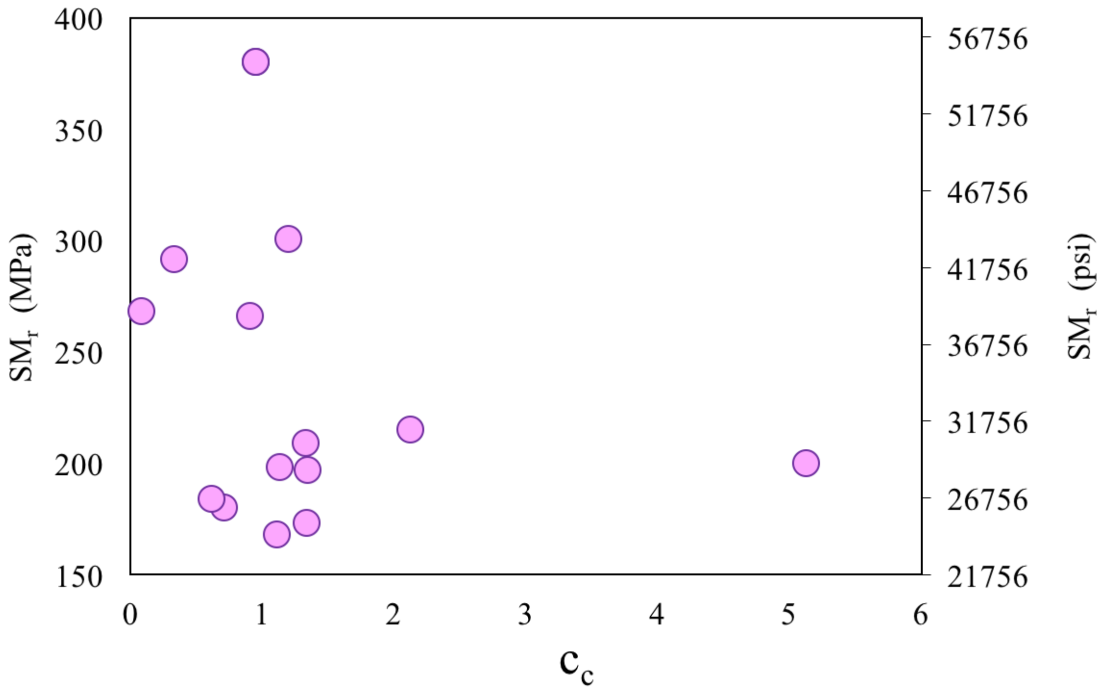

Figure 3.30. Summary resilient modulus (SM_r) vs. coefficient of curvature (C_c) of 100% recycled asphalt pavement (RAP)

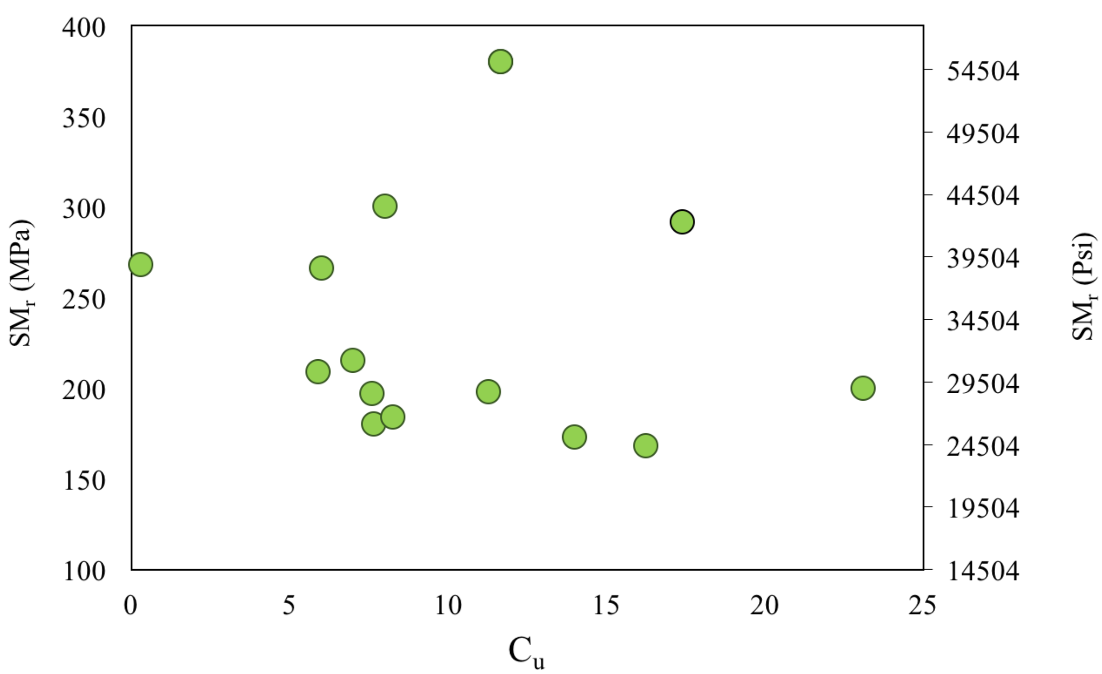

Figure 3.31. Summary resilient modulus (SM_r) vs. coefficient of uniformity (C_u) of 100% recycled asphalt pavement (RAP)

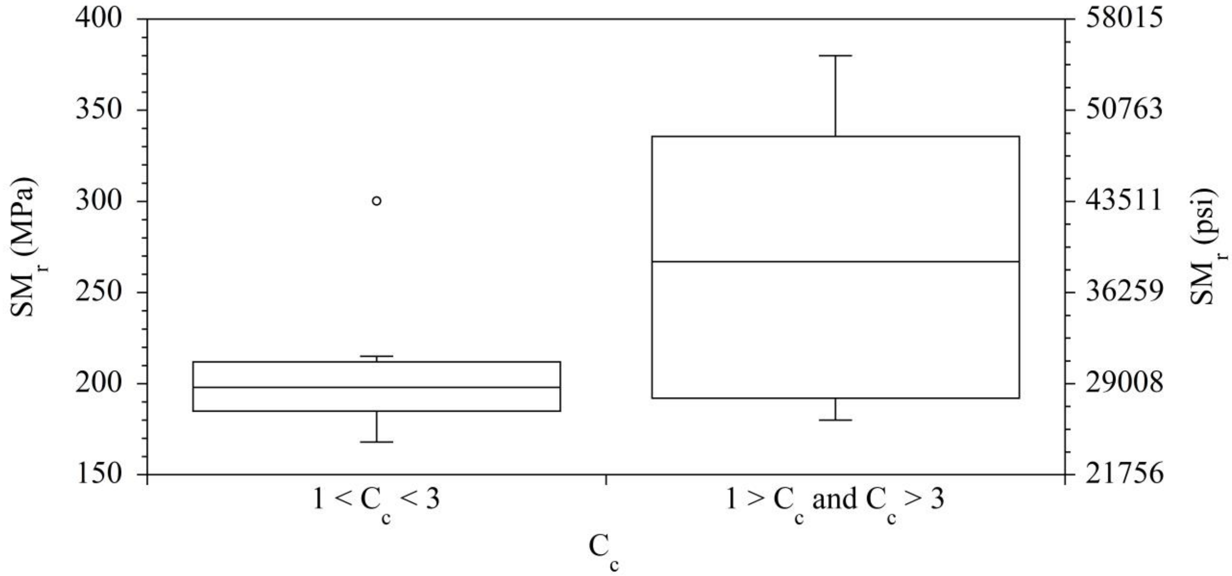

Figure 3.32. Summary resilient modulus (SM_r) vs. coefficient of curvature (C_c) of 100% recycled asphalt pavement (RAP) (box and whisker plot)

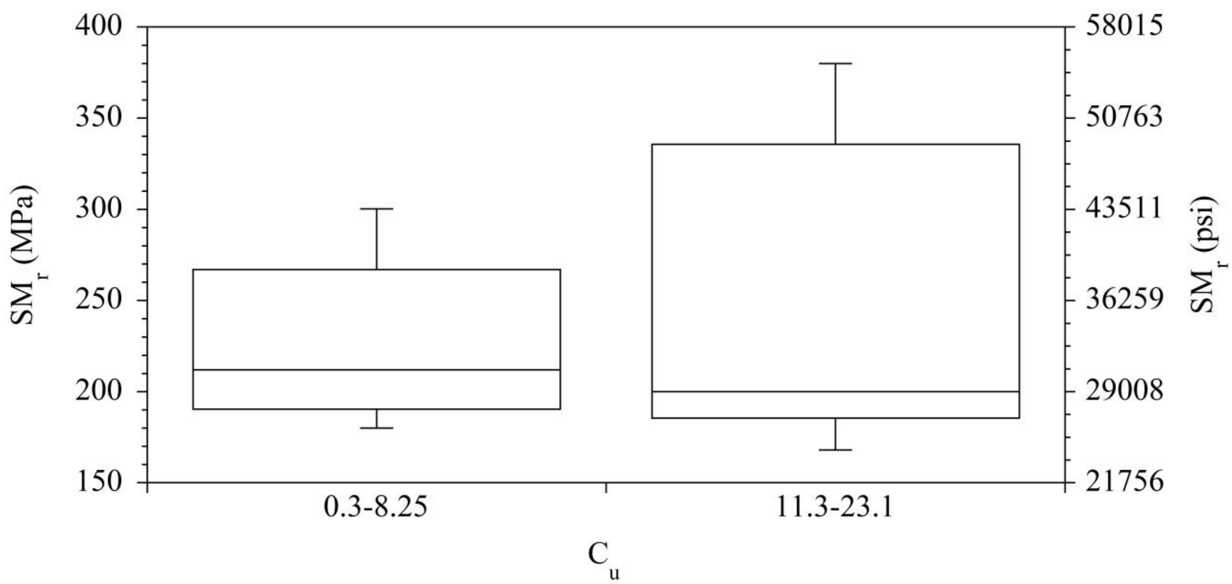

Figure 3.33. Summary resilient modulus (SM_r) vs. coefficient of uniformity (C_u) of 100% recycled asphalt pavement (RAP) (box and whisker plot)

RCA compacted at higher MDU values are likely to have higher SM_r values (Figure 3.34 and Figure 3.35). MDU values of RCA range from 18.9 to 20.9 kN/m^3 (121.4 to 134.4 pcf), while their SM_r values are between 124 and 370 MPa (17,985 and 53,664 psi) (Figure 3.34 and Figure 3.35).

Higher OMC values result in a reduction in SM_r values for RCA (Figure 3.36 and Figure 3.37). OMC values of RCA range from 6.1 to 11.9%, while their SM_r values change between 370 and 124 MPa (53,664 and 17,985 psi) (Figure 3.36 and Figure 3.37).

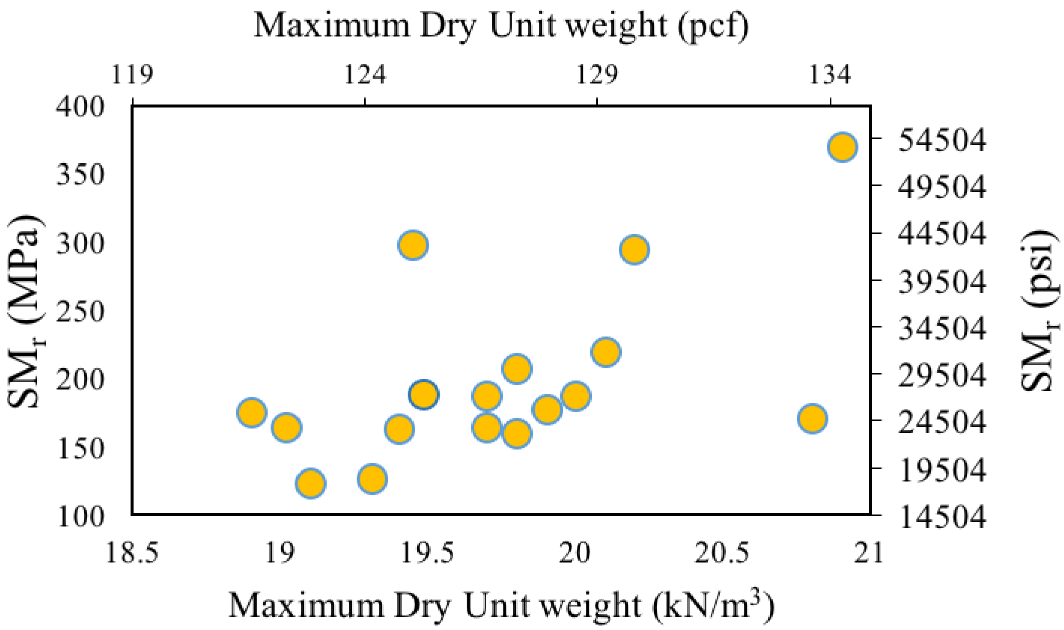

Figure 3.34. Summary resilient modulus (SMr) vs. maximum dry unit weight (MDU) of 100% recycled concrete aggregate (RCA)

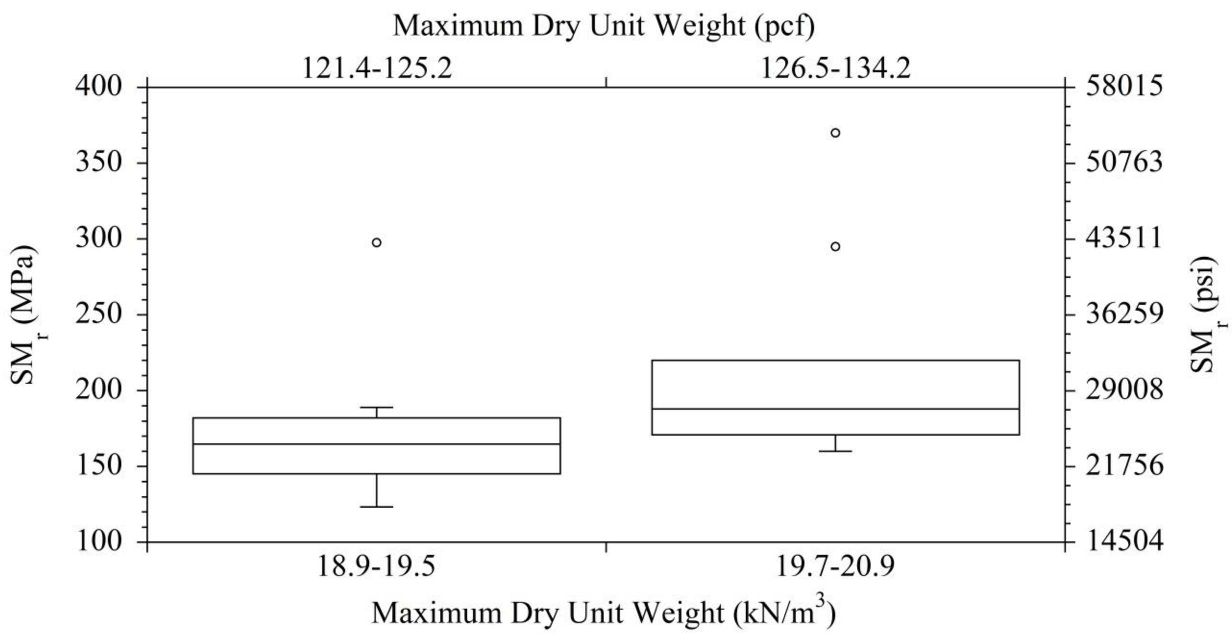

Figure 3.35. Summary resilient modulus (SMr) vs. maximum dry unit weight (MDU) of 100% recycled concrete aggregate (RCA) (box and whisker plot)

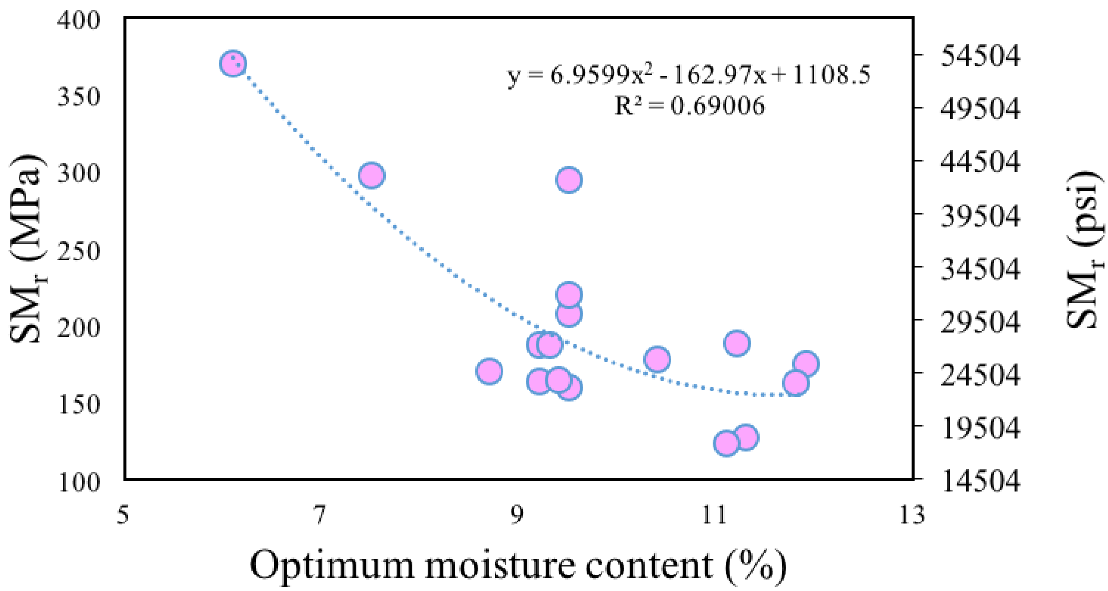

Figure 3.36. Summary resilient modulus (SM_r) vs. optimum moisture content (OMC) of 100% recycled concrete aggregate (RCA)

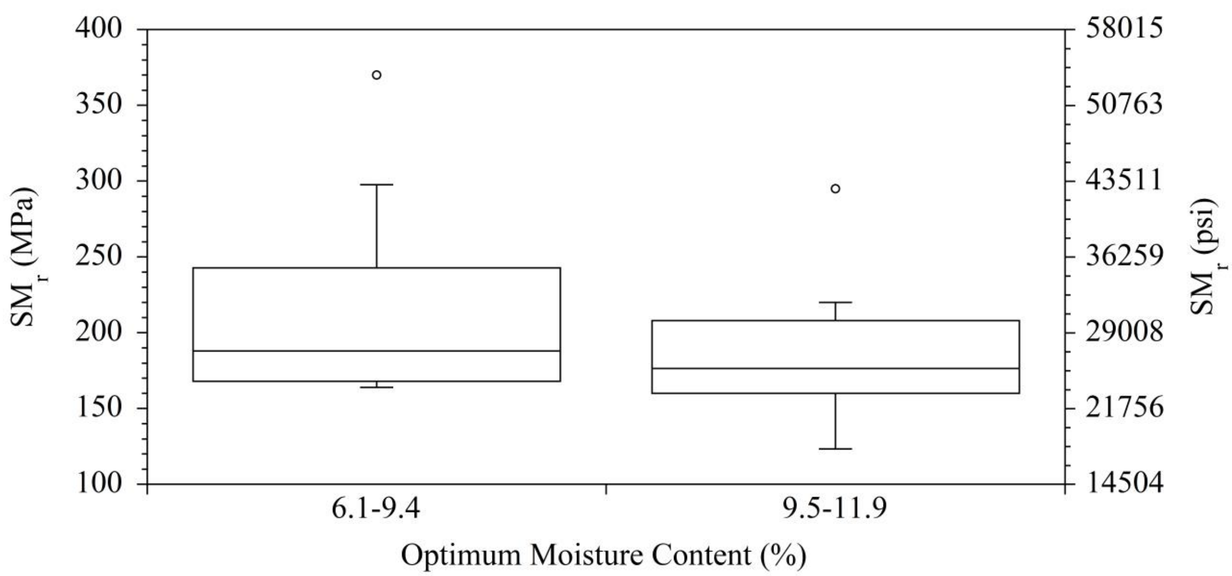

Figure 3.37. Summary resilient modulus (SM_r) vs. optimum moisture content (OMC) of 100% recycled concrete aggregate (RCA) (box and whisker plot)

Figure 3.38, Figure 3.39, Figure 3.40, and Figure 3.41 show that there was no correlation between D_{30} and D_{60} values of RCA and their SM_r values.

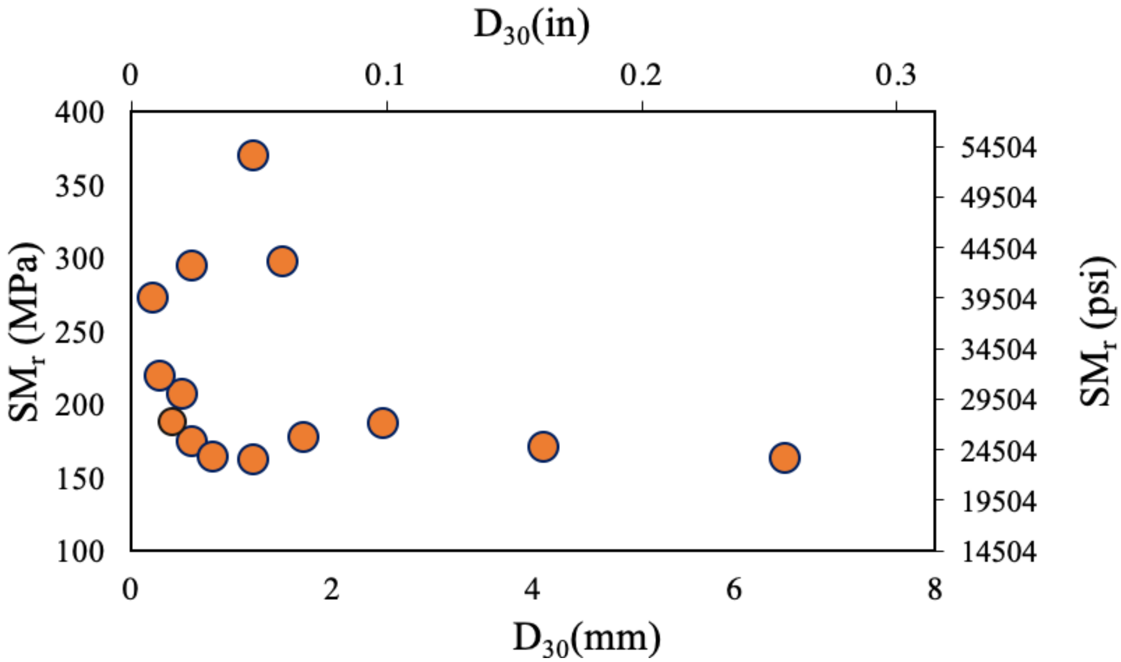

Figure 3.38. Summary resilient modulus (SM_r) vs. D_{30} (diameter at which 30% of the particles are finer) of 100% recycled concrete aggregate (RCA)

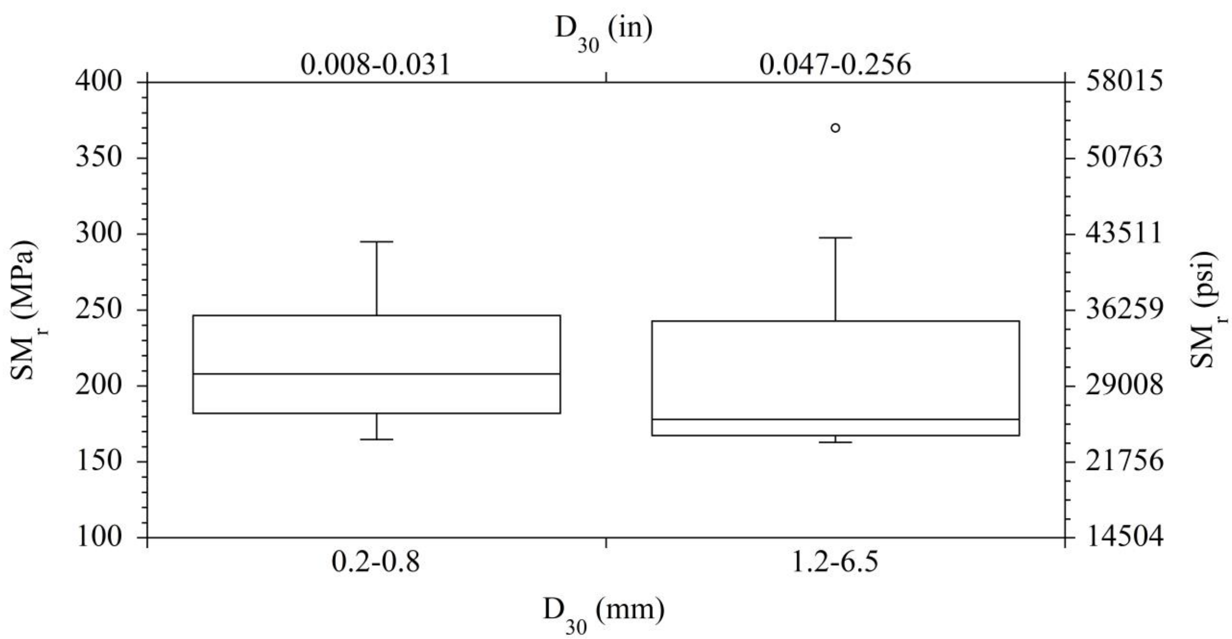

Figure 3.39. Summary resilient modulus (SM_r) vs. D_{30} (diameter at which 30% of the particles are finer) of 100% recycled concrete aggregate (RCA) (box and whisker plot)

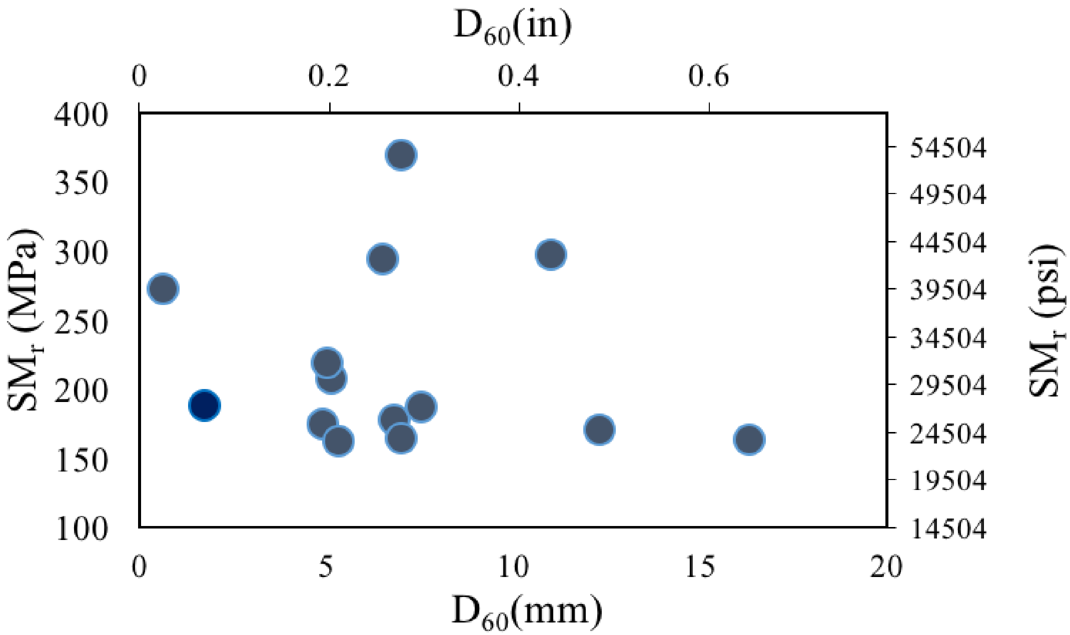

Figure 3.40. Summary resilient modulus (SM_r) vs. D_{60} (diameter at which 60% of the particles are finer) of 100% recycled concrete aggregate (RCA)

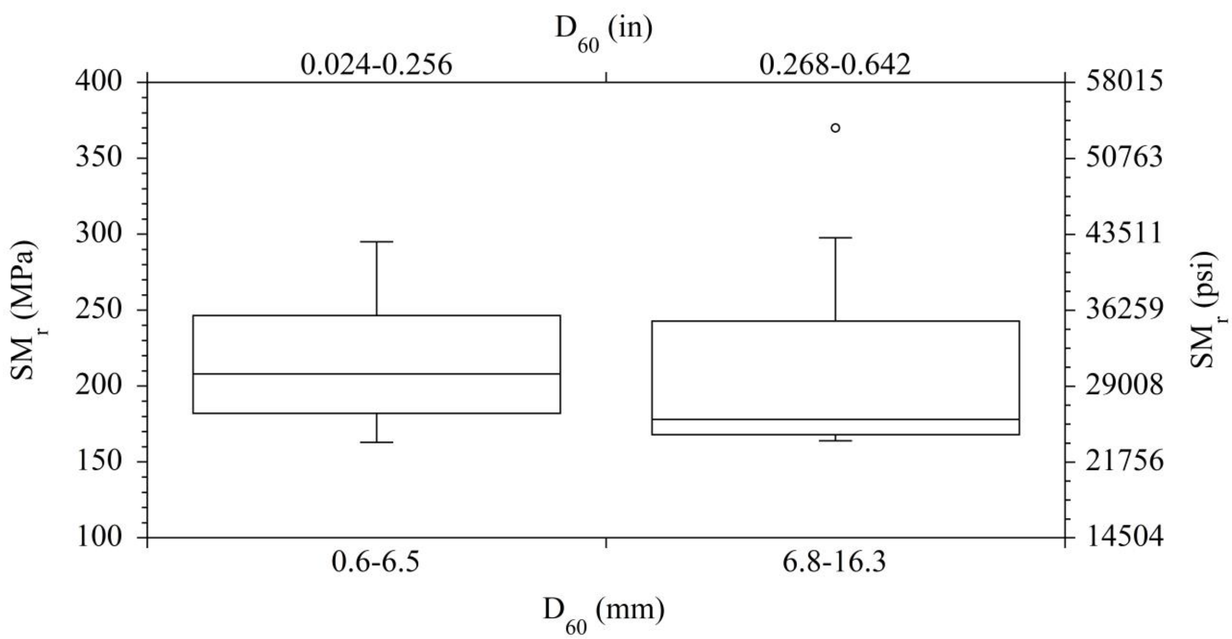

Figure 3.41. Summary resilient modulus (SM_r) vs. D_{60} (diameter at which 60% of the particles are finer) of 100% recycled concrete aggregate (RCA) (box and whisker plot)

According to Figure 3.42 and Figure 3.43, well-graded RCA yield higher SM_r values than poorly graded ones. Summary M_r (SM_r) values of RCA with C_c values between 1 and 3 tend to be higher than those with C_c smaller than 1 or higher than 3. According to Figure 3.44 and Figure 3.45, higher C_u values in RCA could result in higher SM_r values.

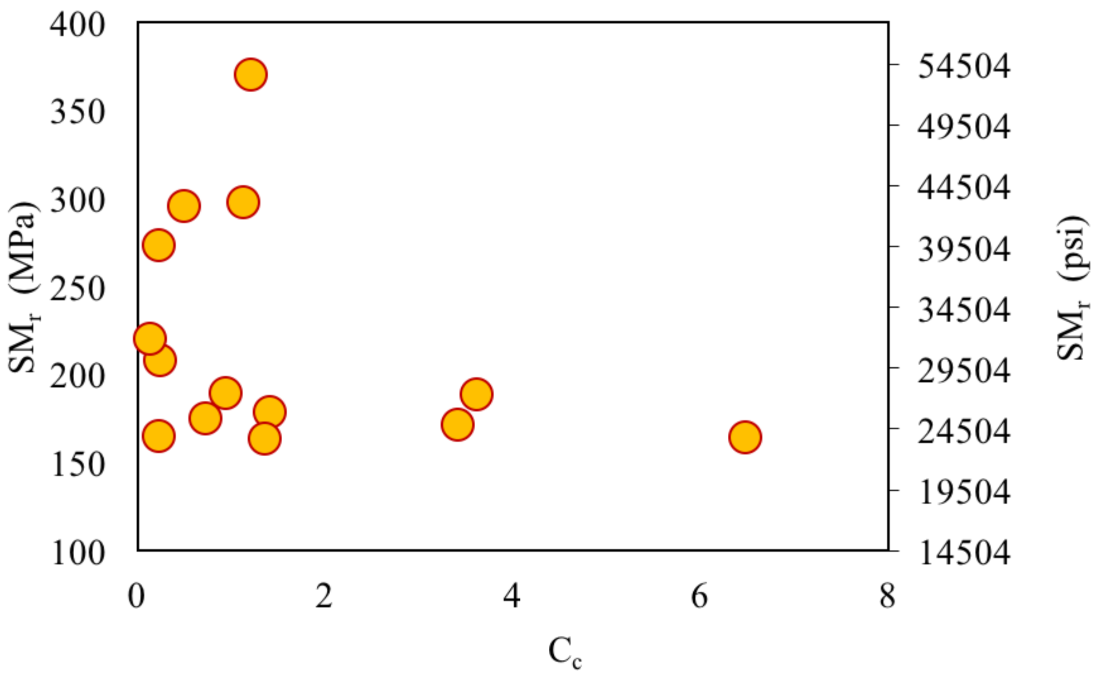

Figure 3.42. Summary resilient modulus (SM_r) vs. coefficient of curvature (C_c) of 100% recycled concrete aggregate (RCA)

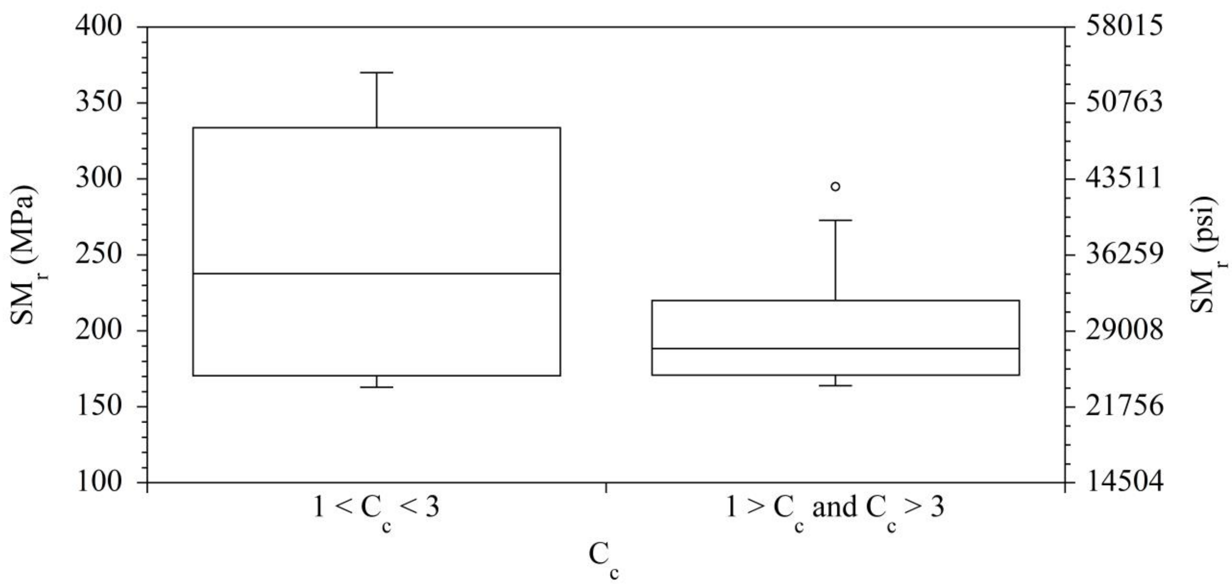

Figure 3.43. Summary resilient modulus (SM_r) vs. coefficient of curvature (C_c) of 100% recycled concrete aggregate (RCA) (box and whisker plot)

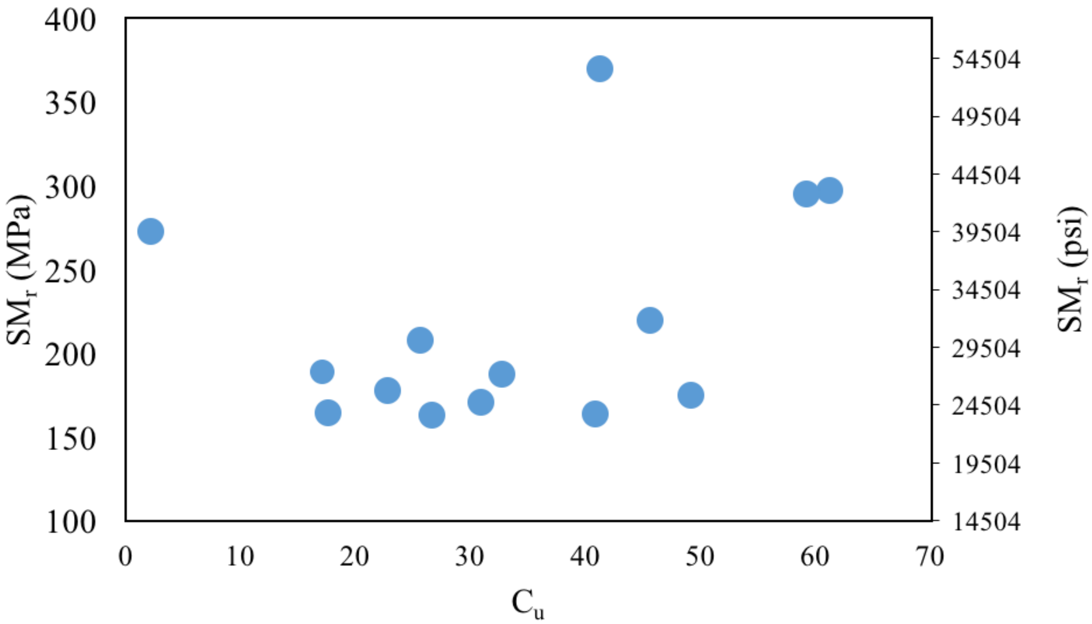

Figure 3.44. Summary resilient modulus (SM_r) vs. coefficient of uniformity (C_u) of 100% recycled concrete aggregate (RCA)

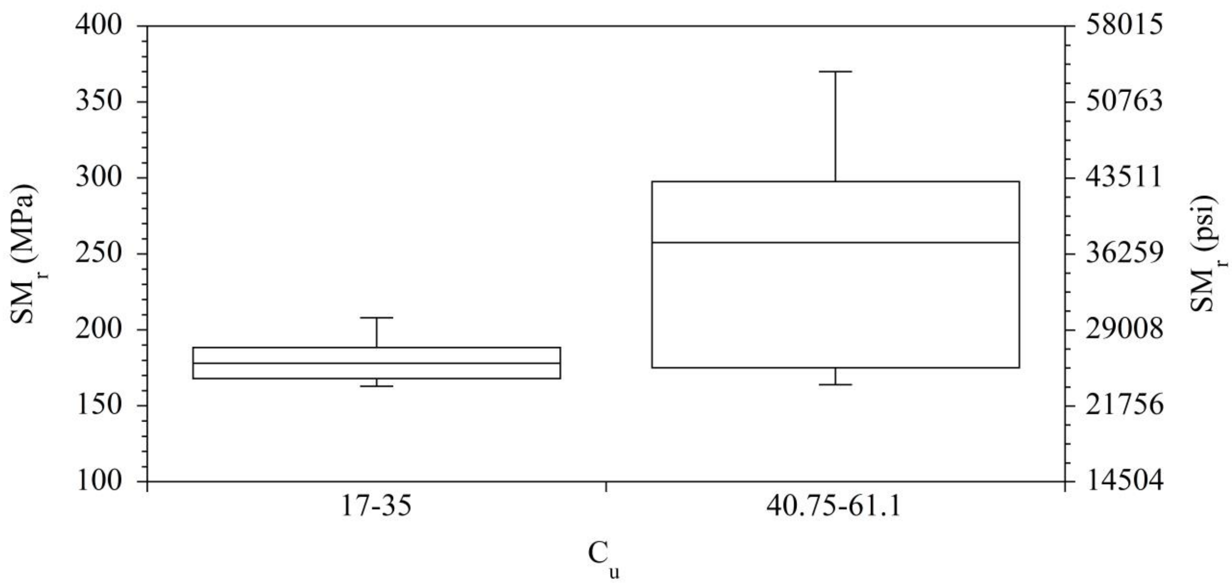

Figure 3.45. Summary resilient modulus (SM_r) vs. coefficient of uniformity (C_u) of 100% recycled concrete aggregate (RCA) (box and whisker plot)

3.3.2 Effect of Temperature on M_r

RAP is sensitive to temperature due to its asphalt binder content, which is a temperature-sensitive material. On the other hand, RCA and VA are not as sensitive to temperature changes as RAP (Wen et al. 2011; Soleimanbeigi et al. 2015). Summary M_r (SM_r) values of RAP-VA blends tend to reduce as temperature increases (Soleimanbeigi et al. 2015). However, Soleimanbeigi and Edil (2015b) claimed that RAP could undergo a thermal preloading process. Thus, RAP 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 the compaction process. The induced elevated temperature increases the 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 the reduction in void space. Therefore, it is important to know the proper thermal conditioning for RAP during compaction when used as an aggregate base layer (Soleimanbeigi and Edil 2015b).

Edil et al. (2012a) and Soleimanbeigi et al. (2015) conducted M_r tests on RAP obtained from Colorado (CO), Texas (TX), and New Jersey (NJ) at 7, 23, 35, and 50°C (44.6, 73.4, 95, and 122°F), and it was observed that SM_r values of all RAP decreased with an increase in temperature (Figure 3.46). Soleimanbeigi and Edil (2015b) showed that SM_r values of RAP could also be affected by the compaction temperature. Summary M_r (SM_r) values of RAP increased when RAP was compacted at higher temperatures and then cooled and tested for M_r, as shown in Figure 3.46. Figure 3.47 shows the normalized SM_r values of RAP at different temperatures [normalized SM_r was obtained by dividing SM_r of RAP at different temperatures by SM_r of RAP at 23°C (73.4°F)]. There was a decreasing SM_r trend with higher temperatures for all the RAP except for the NJ RAP and CO RAP, which had a slightly increasing SM_r trend.

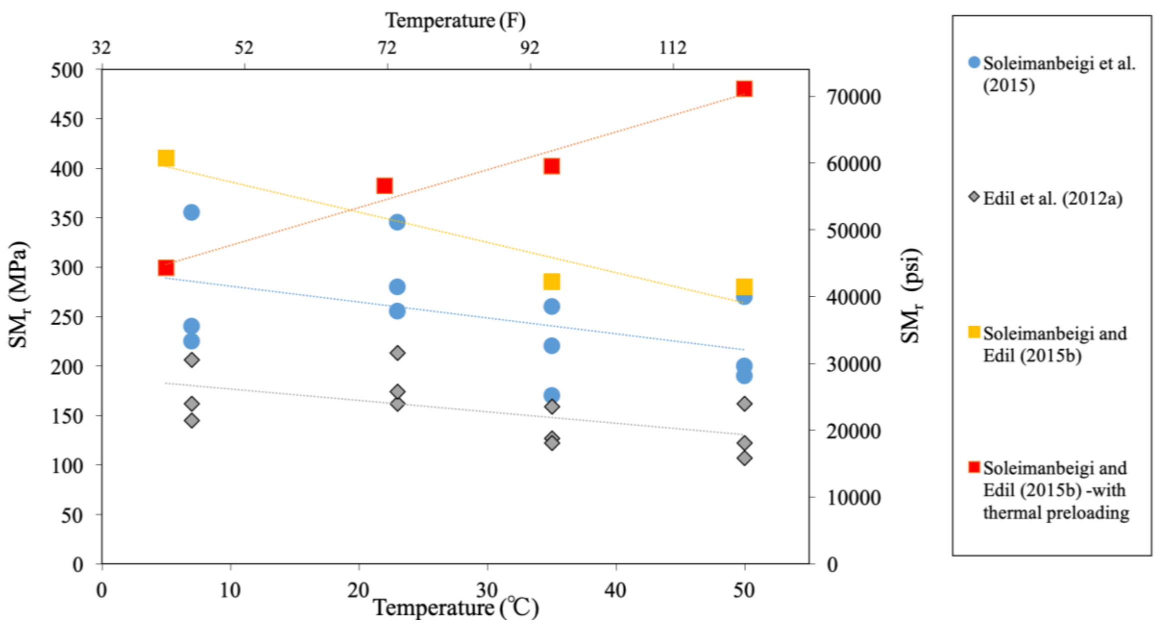

Figure 3.46. Summary resilient modulus (SM_r) vs. testing temperature of 100% recycled asphalt pavement (RAP)

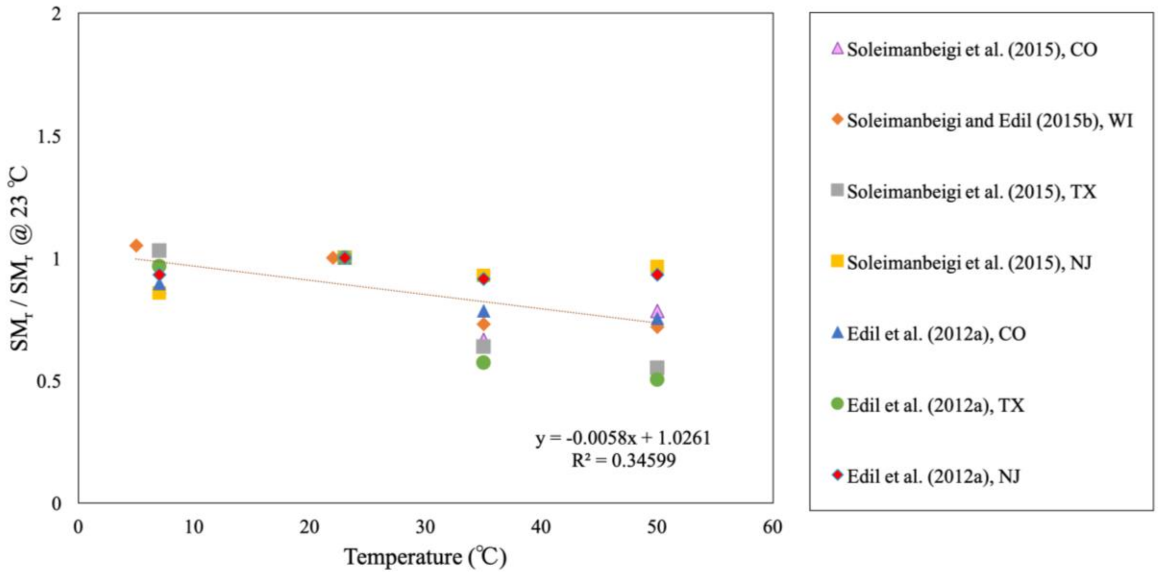

Figure 3.47. Normalized summary resilient modulus (SM_r) vs. testing temperature of 100% recycled asphalt pavement (RAP)

Figure 3.48 shows that SM_r values of RCA are not temperature-dependent. Figure 3.49 shows the normalized SM_r values of RCA at different temperatures [normalized SM_r was obtained by dividing SM_r of RCA at different temperatures by SM_r of RCA at 23°C (73.4°F)]. The low R^2 value (0.12) provided in Figure 3.49 reveals that there was no relationship between SM_r values of RCA and temperature during testing. As summarized in Figure 3.50 with box and whisker plots, SM_r values of RCA are independent of temperature.

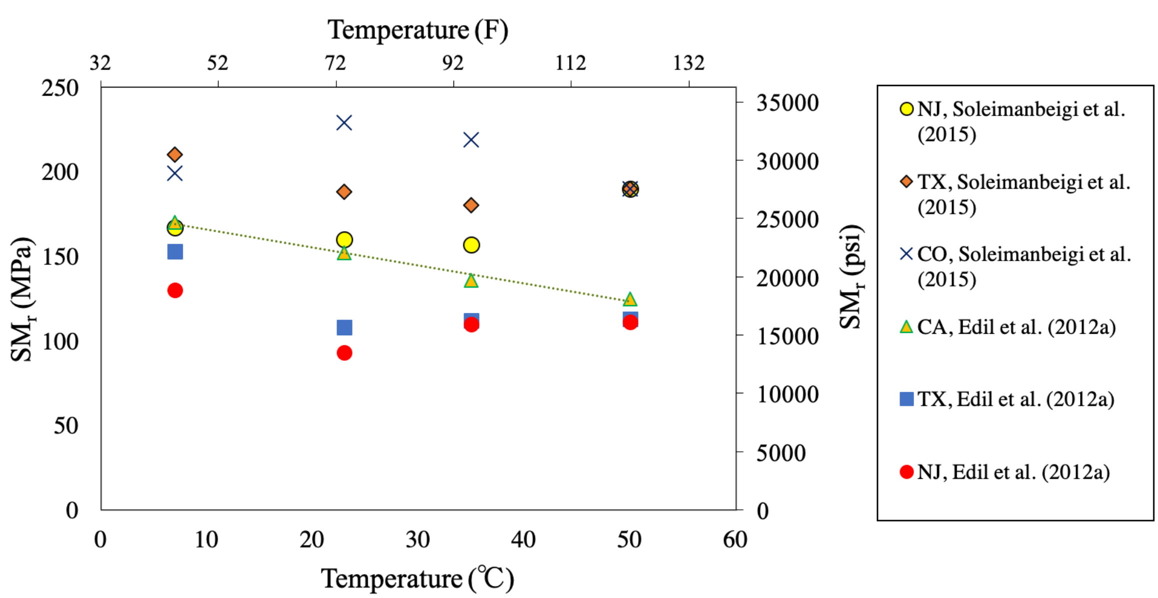

Figure 3.48. Summary resilient modulus (SM_r) vs. testing temperature of 100% recycled concrete aggregate (RCA)

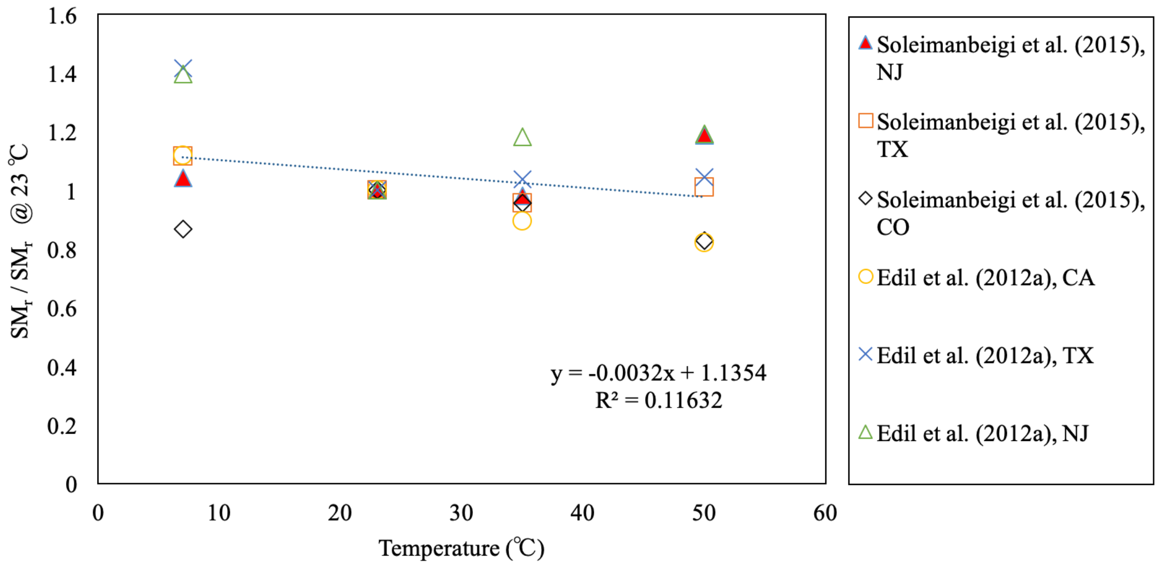

Figure 3.49. Normalized summary resilient modulus (SM_r) vs. testing temperature of 100% recycled concrete aggregate (RCA)

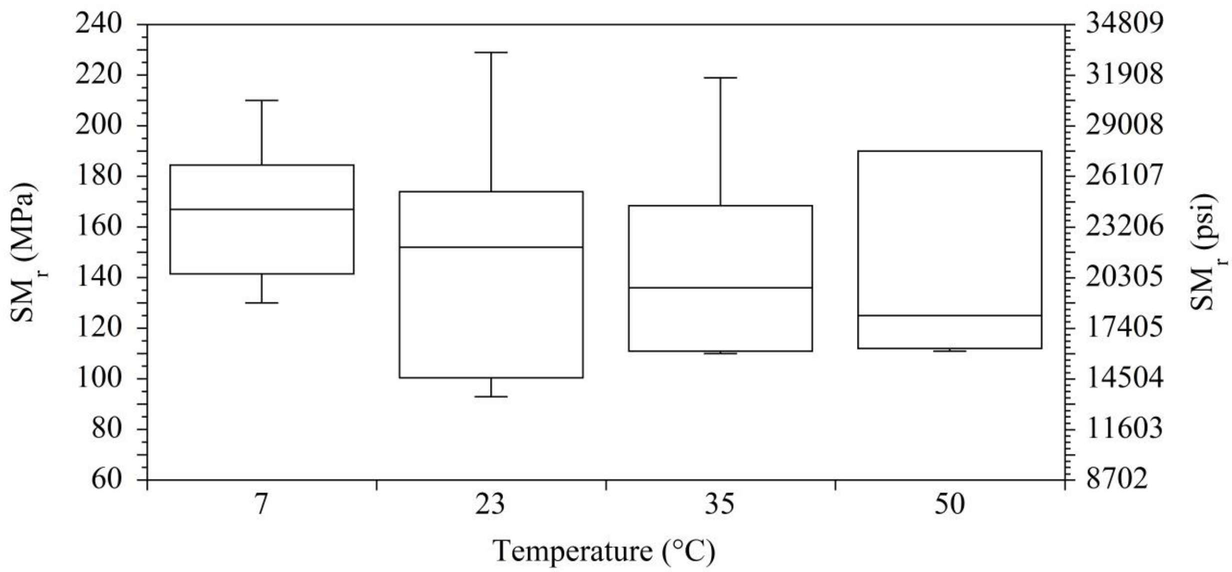

Figure 3.50. Summary resilient modulus (SM_r) vs. testing temperature of 100% recycled concrete aggregate (RCA) (box and whisker plot)

According to Figure 3.51, an increase in the moisture content of RAP causes a decrease in their SM_r values. Attia and Abdelrahman (2010b) conducted research on 100% RAP and 50% RAP-VA blend. In both cases, a reduction in SM_r values was observed as the moisture contents of the specimens increased. In addition, Edil et al. (2012a) tested RAP obtained from Texas (TX) and Ohio (OH) and concluded that as the moisture content increased, SM_r values of these materials decreased.

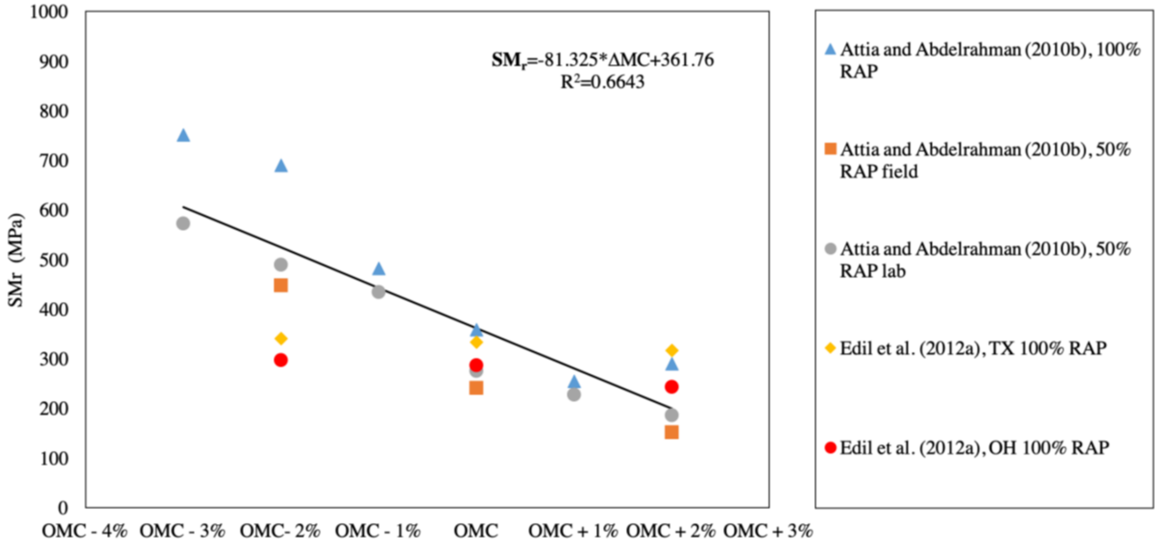

Figure 3.51. Summary resilient modulus (SM_r) vs. optimum moisture content (OMC) of 100% recycled asphalt pavement (RAP) (ΔMC = percent change in OMC)

3.3.3 Effect of RAP/RCA Contents on M_r

The use of a higher amount of RAP/RCA generally increases the M_r of RAP-VA/RCA-VA blends (Bennert et al. 2000) (Figure 3.52, Figure 3.53, and Figure 3.54 for RAP-VA blends, and Figure 3.55, Figure 3.56, Figure 3.57, and Figure 3.58 for RCA-VA blends). However, there were several exceptions in the literature. For instance, Bestgen et al. (2016) did not observe a consistent increase in SM_r values of VA when they were mixed with RCA until RCA contents reached 75% by weight in the mix design.

Ullah and Tanyu (2019) conducted M_r tests on three different RAP (RAP1, RAP2, and RAP3) that were mixed with VA at 20, 30, 40, 50, 60% by weight. Summary M_r (SM_r) value of VA in this study was 141.1 MPa (20,450 psi). This study showed that RAP1 with a higher asphalt binder content (5.6-5.8%) resulted in the highest SM_r values in the RAP-VA blend, while RAP2 with a lower asphalt binder content (4.5-4.7%) had the lowest SM_r value in the blend. Overall, this study claimed that the asphalt binder contents of RAP had a slight effect on SM_r values of the blends.

Attia and Abdelrahman (2010a) tested Minnesota Class 5 aggregate and RAP-VA blends consisting of 50% RAP+50% Class 5, 75% RAP+25% Class 5, and 100% RAP material. They showed that materials with 50% RAP would have an SM_r value of 265 MPa (38,435 psi) while SM_r value of 75% RAP blend was 210 MPa (30,458 psi). 100% RAP had an SM_r value of 400 MPa (58,015 psi), which was higher than any blends.

Alam et al. (2010) collected RAP from millings of the 2001 rehabilitation project on MnROAD Cell 26 constructed in 1994 and subjected to 20,000 vehicles daily (Mulvaney and Worel 2002). These RAP were mixed with VA at 30, 50, 70, and 100% by weight and subjected to a series of M_r tests. This study determined that SM_r values increased from 154 to 270 MPa (22,336 to 39,160 psi) with an increase in the RAP content from 30 to 100%.

Bradshaw et al. (2016) studied two different types of RAP-VA blends. The materials included cold-recycled RAP-VA blends that were prepared off-site and RAP-VA blends that were generated in situ during full-depth reclamation (FDR). Summary M_r (SM_r) values of the cold-recycled RAP-VA blends (14-39% RAP content) are between 120 and 502 MPa (17,404 and 72,809 psi). The possible reason for slightly higher SM_r values of these blends than those of RAP-VA blends in the literature could be the differences in aggregate composition and/or particle morphology. Summary M_r (SM_r) values of the FDR RAP-VA blends (57-71% RAP content) range from 171 to 578 MPa (24,802 and 83,832 psi), which were higher than SM_r values of the cold-recycled RAP-VA blends. This could be due to the higher RAP contents used in the FDR RAP-VA blends.

Kang et al. (2011) tested RAP and VA. RAP was collected from Highway 61 in Minneapolis, MN. VA was collected from a pit south of Jordan, MN. RAP was mixed with VA at 25, 50, and 100% by weight. In this study, SM_r values of the blends increased with an increase in the RAP content. However, generally, the reported SM_r values were smaller than any other study ranging from 90 to 192 MPa (13,053 to 27,847 psi).

Abdelrahman and Noureldin (2014) conducted research on one RAP supplied by MnDOT from a trunk highway. This RAP was blended with Minnesota Class 5 aggregate at 50, 75, and 100% by weight. According to the results, SM_r values changed from 289 MPa (41,916 psi) (50% RAP) to 262 MPa (38,000 psi) (75% RAP). It was reported that SM_r value of 100% RAP was 330 MPa (47,863 psi), which was higher than the RAP-Class 5 blends.

Kim et al. (2005) obtained the reclaimed materials from County Road (CR) 3 in central Minnesota (MN). An in situ blends, mixtures of 25, 50, and 75% RAP and crushed aggregate, 100% RAP, and pure aggregate materials were taken separately during FDR. This was the only study that reported similar SM_r values for different blends. 100% RAP had an SM_r value of 170 MPa (24,656 psi).

Wu et al. (2012) obtained crushed aggregate (basalt) from POE Asphalt Paving, Inc. (Pullman, WA) and RAP from the Fairmount Road construction site in Pullman, WA. To eliminate the effect of gradation, one single gradation was selected, meeting the Washington State DOT (WSDOT) 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 (29,008 psi), whereas 80% RAP blend had an SM_r value of 550 MPa (79,771 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. This study reported an increasing M_r trend with higher RAP contents from 25% [202 MPa (29,225 psi)] to 50% [234 MPa (33,895 psi)] and from 75% [214 MPa (31,009 psi)] to 100% RAP [268 MPa (38,870 psi)]. Summary M_r (SM_r) value of 50% RAP was observed to be higher than that of the blend containing 75% RAP.

Hasan et al. (2018) collected subgrade soils and RAP from the Interstate 40 (I-40) construction site at the milepost of 141 near Albuquerque, New Mexico (NM), and the RAP was supplied from a stockpile. They reported an SM_r value of 175MPa (25,382 psi) for 25% RAP and an SM_r value of 290 MPa (42,061 psi) for 75% RAP.

Bennert et al. (2000) conducted research on 25, 50, 75, and 100% RAP blended with a dense-graded aggregate base course (DGABC) in New Jersey (NJ). This study reported an increasing trend in SM_r values as the RAP content increased. It also showed that the 25% RAP blend had an SM_r value of 187 MPa (27,122 psi), and 100% RAP had an SM_r value of 300 MPa (43,555 psi).

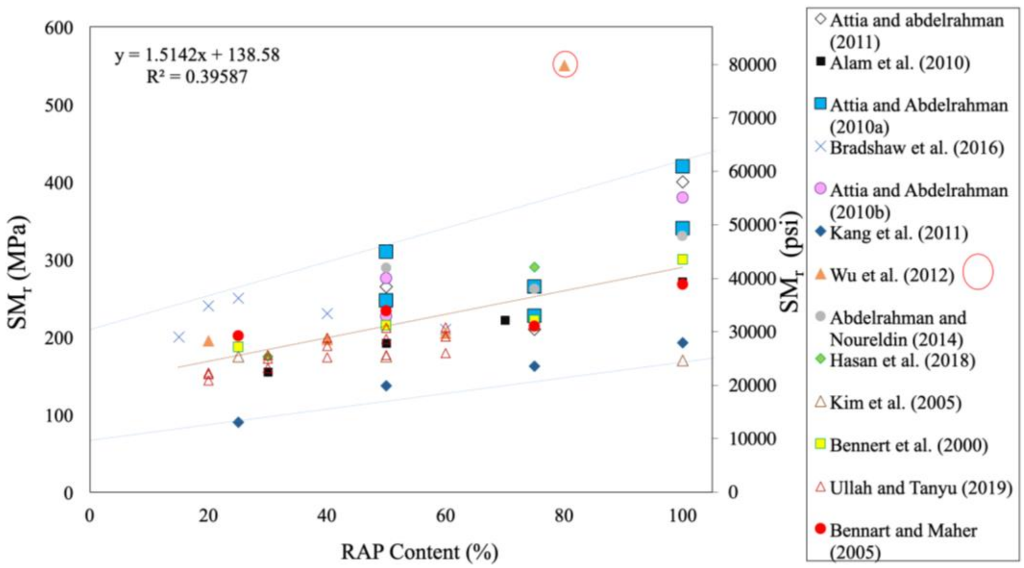

Figure 3.52. Summary resilient modulus (SM_r) vs. recycled asphalt pavement (RAP) content

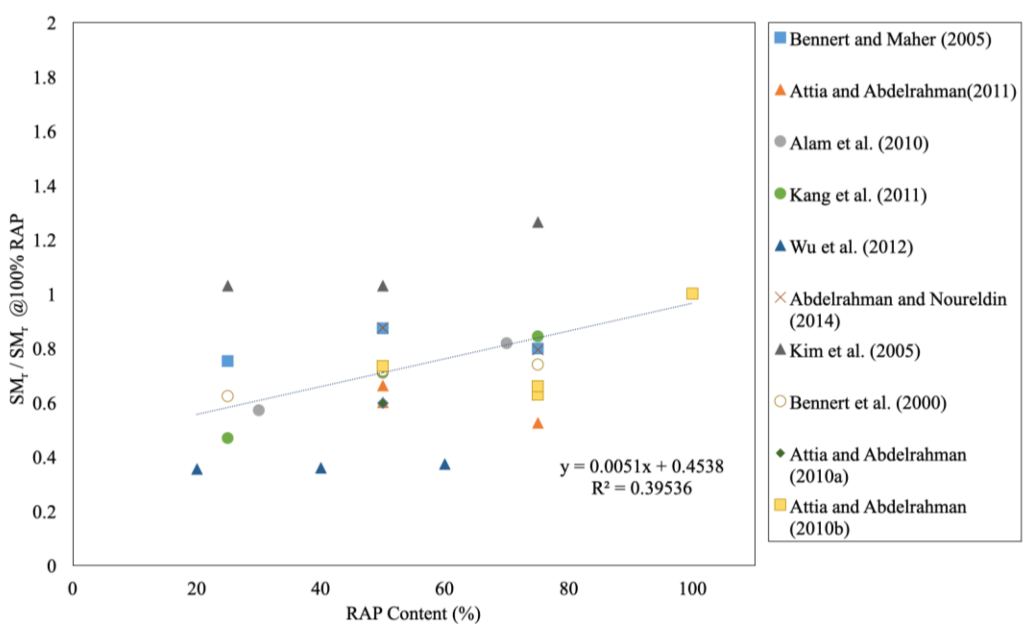

Figure 3.53. Normalized summary resilient modulus (SM_r) vs. recycled asphalt pavement (RAP) content

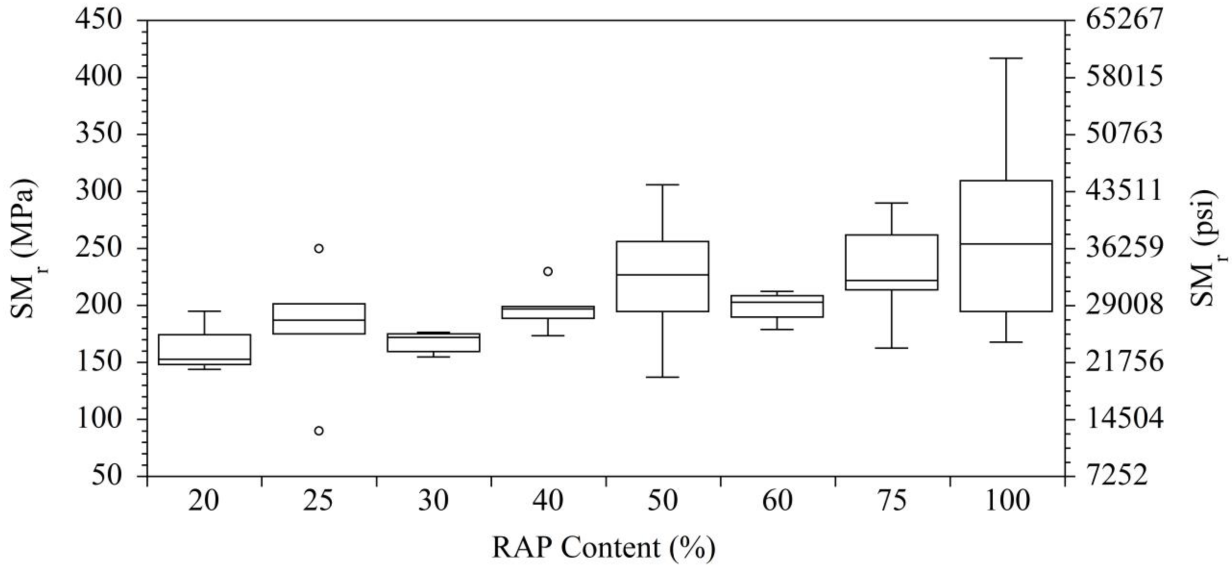

Figure 3.54. Summary resilient modulus (SM_r) vs. recycled asphalt pavement (RAP) content (box and whisker plot)

Figure 3.55 shows that the relationship between the RCA contents and SM_r values is not very clear as the one observed between the RAP contents and SM_r values. According to Bestgen et al. (2016), the presence of higher CaO content in RCA led to higher SM_r values than VA. CaO initiates the cementitious reaction in the aggregate matrix, which can improve the mechanical properties of RCA and blends containing RCA. However, the M_r database contained 18 different RCA-VA blends, and it was observed that each of these blends had different trends.

Bestgen et al. (2016) tested four different VA with two different RCA. RCA was mixed with VA at 25, 50, 75, and 100% by weight. Overall, the blends presented a slight increase in SM_r values when the RCA content was increased from 25 to 50%. Almost all of the mixtures (there were few exceptions) showed an increasing trend in SM_r values as the RCA content increased from 25 to 75%. The overall trend was that 100% RCA had a higher SM_r value than the blends and VA (Figure 3.56). Figure 3.57 shows the normalized SM_r values of each blend of RCA (normalized SM_r was obtained by dividing SM_r of each RCA blend by SM_r of 100% RCA). Figure 3.58 indicates that there was no specific trend between SM_r values of 50 and 75% RCA-VA blends. The number of available data for SM_r values of RCA was lower than that 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 3.58 confirms that 100% RCA has a higher M_r than any RCA blends and VA.

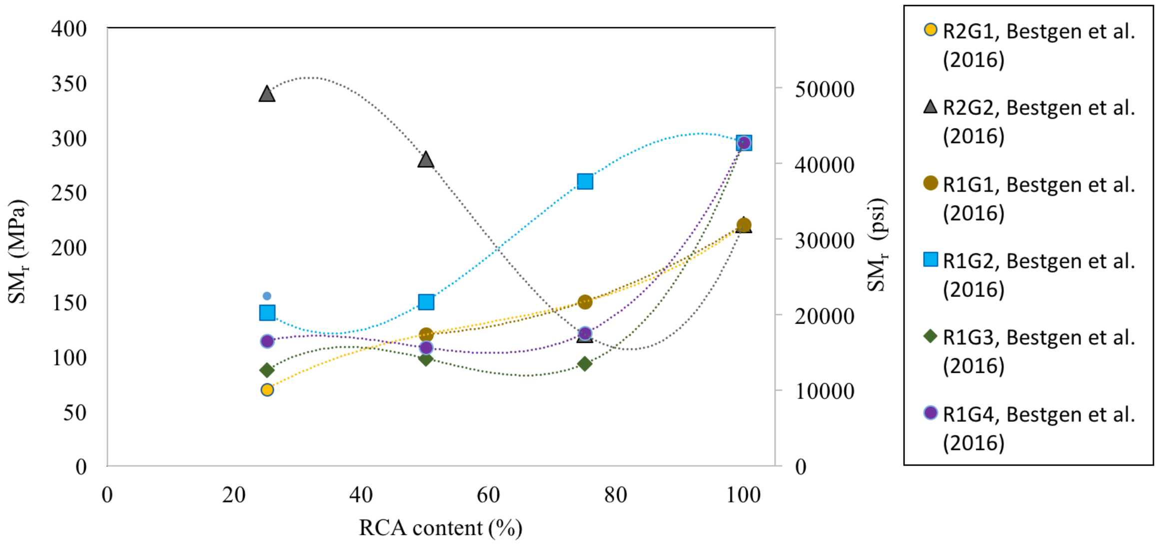

Figure 3.55. Summary resilient modulus (SM$_r$) vs. recycled concrete aggregate (RCA) content

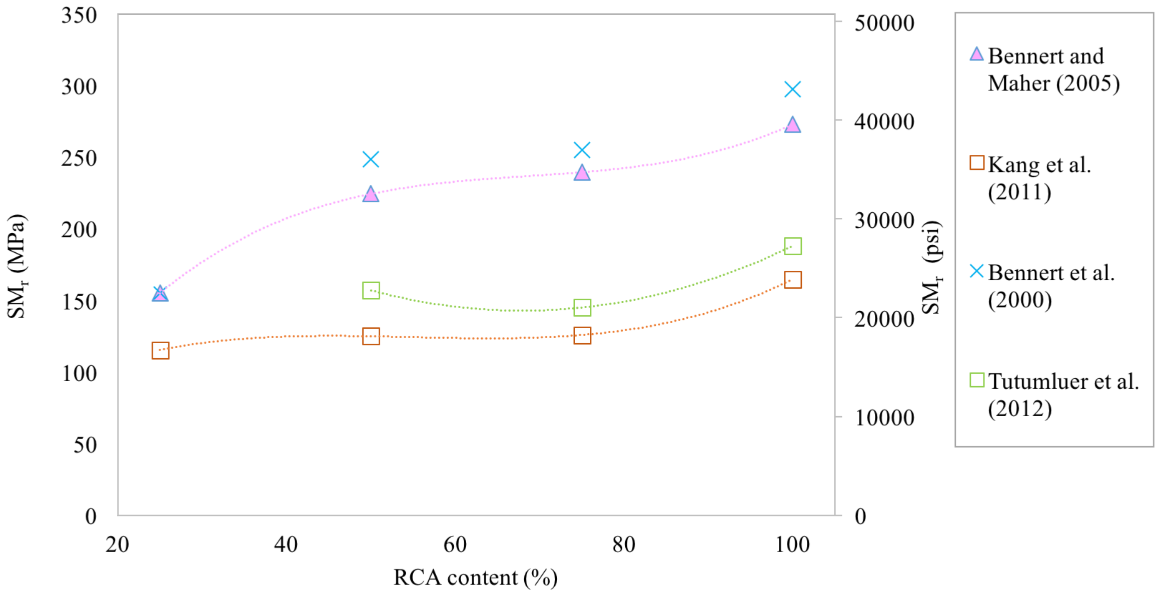

Figure 3.56. Summary resilient modulus (SM$_r$) vs. recycled concrete aggregate (RCA) content

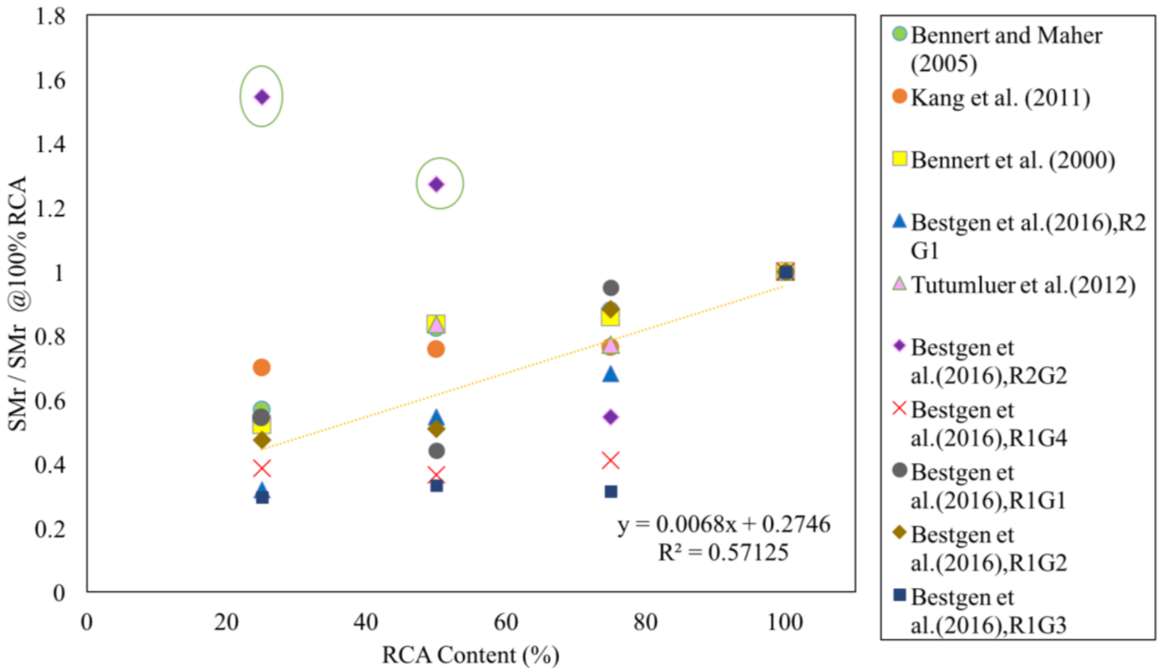

Figure 3.57. Normalized summary resilient modulus (SM_r) vs. recycled concrete aggregate (RCA) content

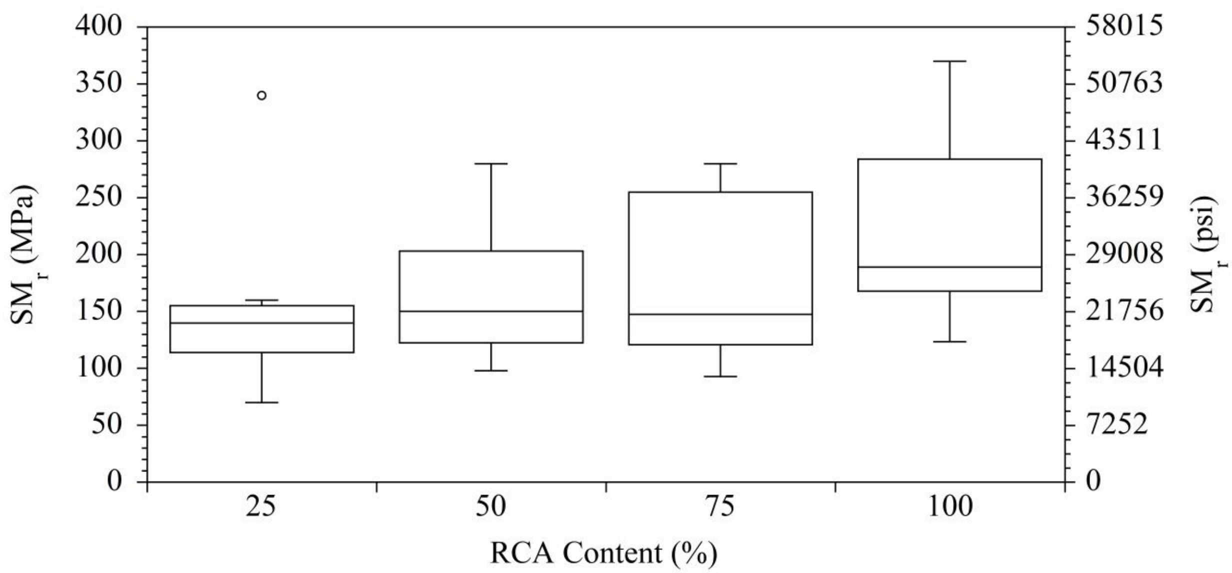

Figure 3.58. Summary resilient modulus (SM_r) vs. recycled concrete aggregate (RCA) content (box and whisker plot)

3.4 PERMANENT DEFORMATION

Permanent deformation failure is attributed to the vertical compressive strains of geomaterials under repeated loading conditions, which lead to failure in 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. According to Thompson and Smith (1990), permanent deformation plays an important role in determining pavement performance.

Increasing the number of load cycles leads to an increase in the permanent deformation of pavement foundation materials regardless of the aggregate type. 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-VA blends increased with an increase in the RAP content (Figure 3.59). On the other hand, an increase in the RCA content of RCA-VA blends caused relatively lower permanent deformations (Bennert et al. 2000) (Figure 3.60).

In general, RCA showed the lowest permanent deformation among RCA, RAP, and VA, while RAP showed the highest permanent deformation (Bennert et al. 2000; Edil et al. 2012a). Having the highest permanent deformation in RAP may be due to the progressive breakdown of the asphalt binder (Bennert et al. 2000). Moreover, the 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 values of 100% RAP range 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 RCA (Edil et al. 2012a).

Different permanent deformation trends can be observed between RAP, RCA, and VA due to their different gradation characteristics (e.g., fines contents). Fines content can significantly affect the permanent deformation properties of aggregates. VA can show lower permanent deformation than RCA due to its lower fines content (Bestgen et al. 2016). A relatively higher fines content can lead to a higher permanent deformation of aggregates (Mishra and Tutumluer 2012). Moreover, repetitive load cycles may break the hydrated cement particles and/or residual mortar; thus, reduce the angularity of RCA, which can finally lead to a higher permanent deformation (Bestgen et al. 2016).

Particle sizes of RCA and RAP are important since the presence of larger aggregates in the material matrix can lead to higher stiffness and resistance against deformation (Gray 1962; Kazmee et al. 2016). Moreover, the thicker aggregate base layers can result in lower permanent deformation than the thinner aggregate base layers due to the improved vertical stress distribution (Cetin et al. 2010; Schaertl 2010).

It was also observed that temperature is very crucial for RAP’s permanent deformation properties. An increase in temperature yields an increase in the permanent deformation of RAP because of the temperature-sensitivity of the asphalt binder (Edil et al. 2012a; Soleimanbeigi et al. 2015). On the other hand, the temperature has little to no effect on the permanent deformation properties of RCA and VA (Edil et al. 2012a).

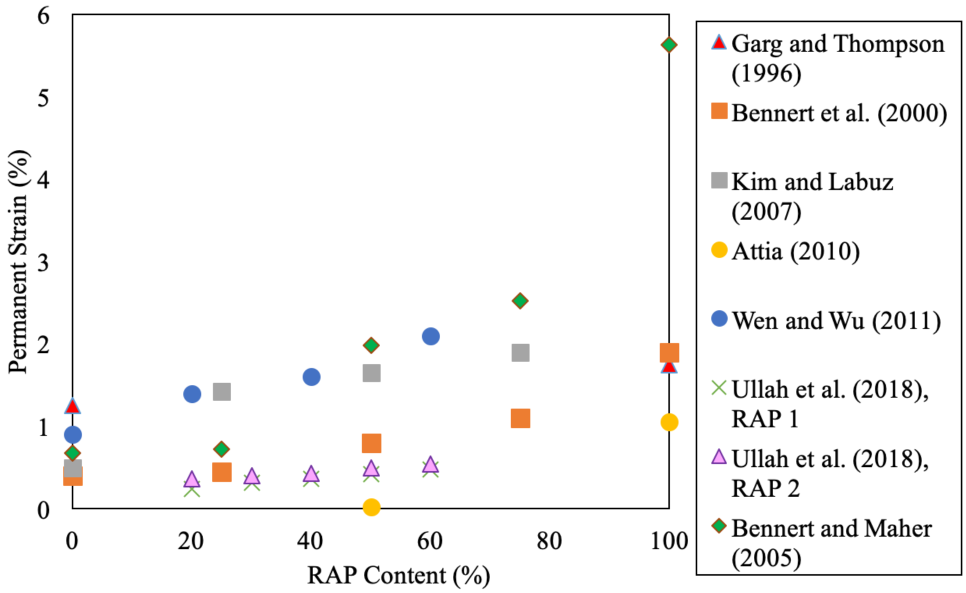

Figure 3.59. Permanent strain vs. recycled asphalt pavement (RAP) content (since there is no established standard to test permanent deformation characteristics of materials, it should be kept in mind that different sources applied different loads or load cycles to the specimens)

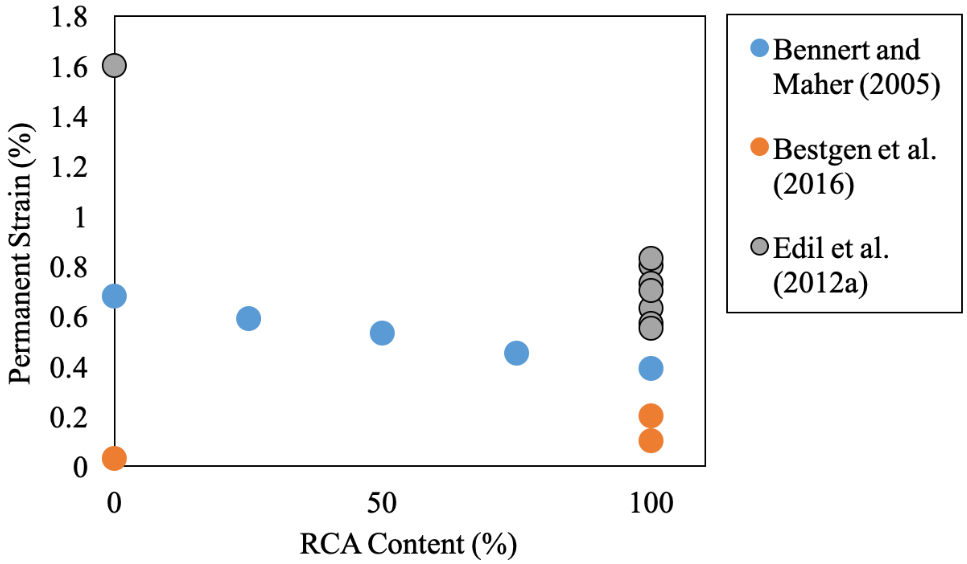

Figure 3.60. Permanent strain vs. recycled concrete aggregate (RCA) content (since there is no established standard to test permanent deformation characteristics of materials, it should be kept in mind that different sources applied different loads or load cycles to the specimens)

3.5 SHEAR STRENGTH

Shear strength is the maximum shear stress that a soil can sustain. Attia (2010) defined 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 (ϕ), 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 the shear strength parameters (ϕ and cohesion) of RAP-VA blends. Results of these studies showed that the ϕ and cohesion values of 100 % RAP varied from 44 to 52° and from 0 to 131 kPa (0 to 19 psi), respectively. The large variation in the cohesion values of RAP may be due to the variation of the asphalt binder age/properties/content of the RAP used by different researchers. No correlations or trends were observed between the ϕ and cohesion values of RAP-VA blends and their RAP contents (Figure 3.61 and Figure 3.62, respectively). There were less available data regarding the shear strength of RCA. The ϕ values of RCA range from 19 to 52.7°, and the cohesion values of RCA range from 24.1 to 191 kPa (3.5 to 27.7 psi) (Figure 3.63 and Figure 3.64, respectively).

The typical ϕ ranges for the granular materials classified as GW, GP, SW, and SP [per the unified soil classification system (USCS) (ASTM D2487)] are 33-40°, 32-44°, 33-43°, and 30-39°, respectively (Swiss Standard SN 670 010b and Koloski et al. 1989). In addition, the typical ϕ ranges are between 35-51° for muddy shale and Stone Mountain granite rocks (Goodman 1980).

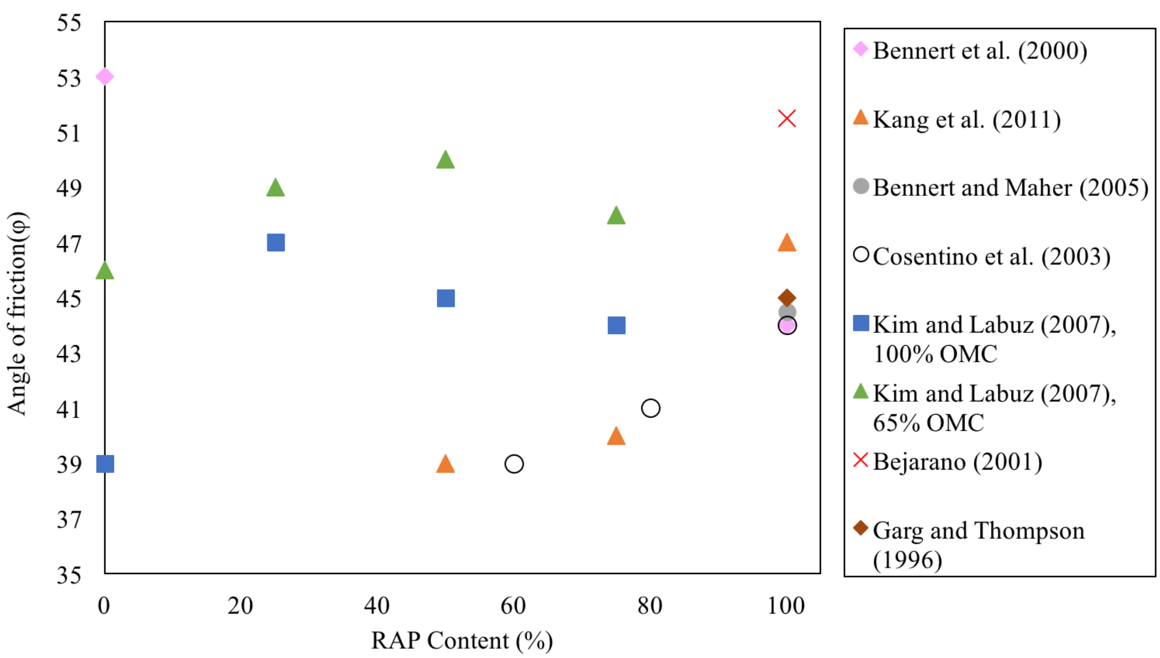

Figure 3.61. Friction angle (ϕ) vs. recycled asphalt pavement (RAP) content

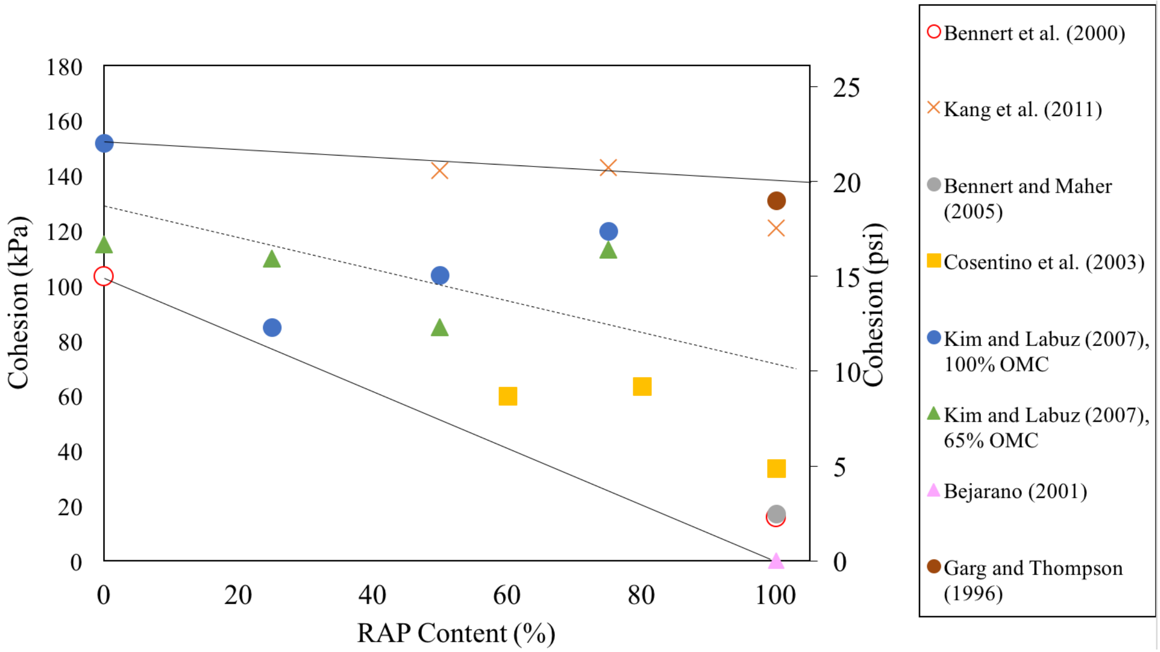

Figure 3.62. Cohesion vs. recycled asphalt pavement (RAP) content

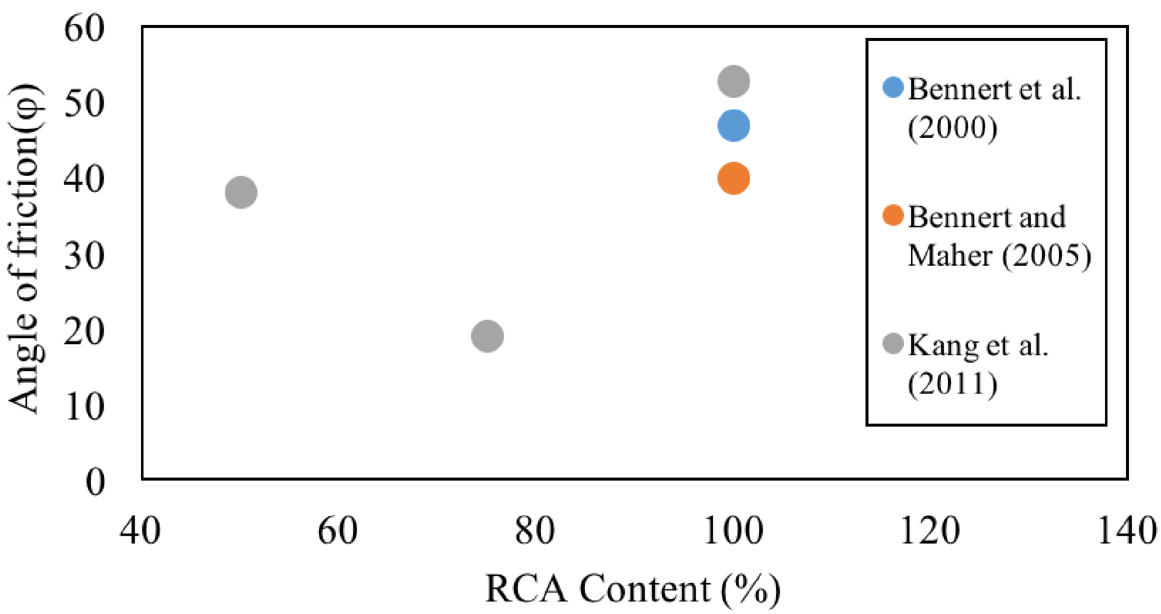

Figure 3.63. Friction angle (ϕ) vs. recycled concrete aggregate (RCA) content

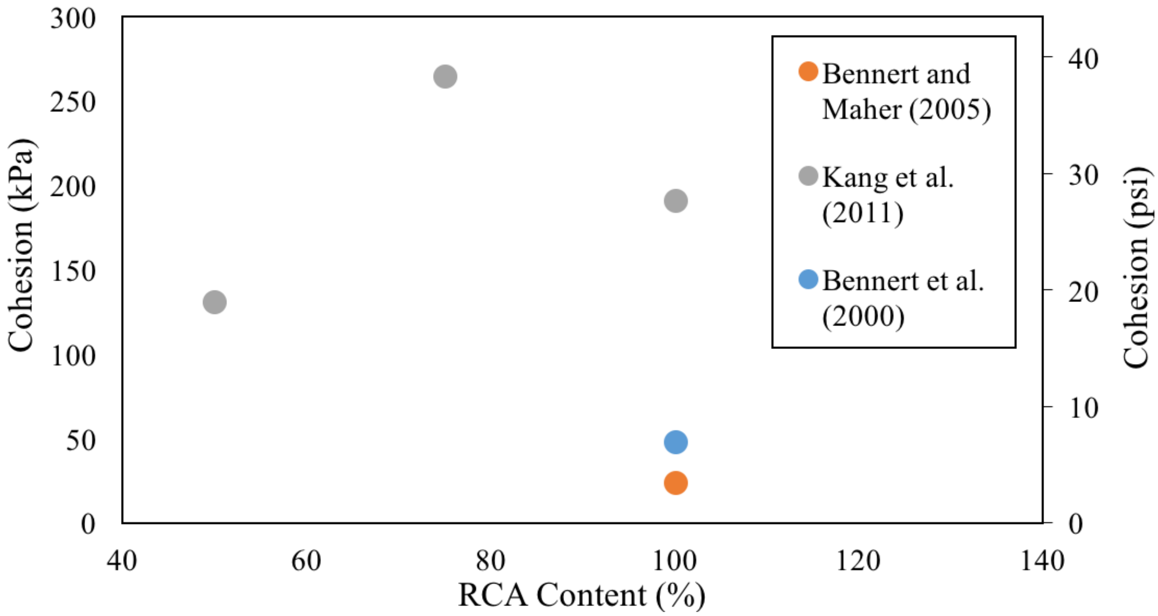

Figure 3.64. Cohesion vs. recycled concrete aggregate (RCA) content

CHAPTER 4: SELECTED PRACTICES FOR STATE DOTS

The materials which do not meet the specifications, which the state DOTs have established, should not be used in pavement construction due to high failure risk (NCHRP-838). As more DOTs understand the importance of RAP and RCA, they tend to develop guidelines for RAP and RCA usage in pavement systems as they can be more economical and environmentally friendly than VA. This chapter 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 one another. California DOT (Caltrans), MnDOT, Missouri DOT (MoDOT), and Wisconsin DOT (WisDOT) allow RAP and RCA to be used as a base course material in pavement systems if they meet the requirements for gradation and quality characteristics. Michigan DOT (MDOT) and Illinois DOT (IDOT) only allow RCA in aggregate base layer applications even though IDOT recently starts considering the use of RAP in such applications as well. More detailed information about DOT specifications is discussed below.

4.1 CALTRANS

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

Aggregate base and subbase applications of the recycled aggregates are discussed in Sections 25 and 26, respectively, of the Caltrans Standard Specifications (Caltrans 2015). Clean broken stone, crushed gravel, natural rough-surfaced gravel, sand, reclaimed processed Portland cement concrete (PCC), and asphalt concrete (AC) can be used as subbase and aggregate base layers. Subbase aggregates must meet the gradation ranges of Class 1, Class 2, or Class 3, as shown in Table 4.1 (Section 25). In addition, the aggregates must have adequate quality characteristics presented in Table 4.2 depending on their class. The aggregates used as aggregate base layers should meet the gradation requirements and quality characteristics of Class 2 or Class 3 aggregate shown in Table 4.3 (Class 2 gradation), Table 4.4 (Class 2 quality characteristics), Table 4.5 (Class 3 gradation), and Table 4.6 (Class 3 quality characteristics) (Section 26).

Contract compliance is a larger range than the Operating Range and is used to adjust for 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 test results indicate the material is still outside the Contract Compliance requirements, Caltrans generally has the right to ask for the removal or a payment deduction.

Table 4.1. 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 4.2. 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 4.3. 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 4.4. 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 4.5. 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 4.6. 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 IDOT

Sections 311 and 351 of the IDOT Standard Specifications for Road and Bridge Construction published in 2016 allow crushed concrete produced from PCC, crushed gravel, and crushed stone for the aggregate base and subbase courses (IDOT 2016). According to Section 1004, twenty different aggregate classes are defined for different applications (Table 4.7). Crushed concrete must meet the requirements for CA6 or CA10 aggregates for aggregate base layer applications (Table 4.8) (IDOT 2016).

Specifications for the quality control of coarse aggregates are established by IDOT in Section 1004 (Table 4.9). Crushed concrete should be evaluated as a Class D aggregate for checking its quality in terms of Illinois Test Procedure (ITP) 96 [Los Angeles (LA) abrasion test] and should be evaluated as a Class C aggregate for Illinois Test Procedure (ITP) 203, which is used for the determination of deleterious particles in coarse aggregates. According to the LA abrasion limit defined for class D aggregate, abrasion loss should be less than 45%. 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). California bearing ratio (CBR) should be 80 for aggregate base layer applications of conventional materials; however, there is no requirement specified for crushed concrete (IDOT 2016).

Per Section 303, IDOT allows the use of RAP in aggregate subgrade improvement applications. RAP should be selected according to “Reclaimed Asphalt Pavement (RAP) for Aggregate Applications” provided by IDOT - Bureau of Materials. RAP may be mechanically blended with CS 01, CS 02, and RR 01 aggregate gradations defined by IDOT (Table 4.10), but the total product must contain less than 40% well-graded RAP (although RR 01 is listed here, it is no longer used for aggregate subgrade improvement). The size of RAP particles must be less than 4 in (100 mm) when mixed with CS 01, CS 02, and RR 01 aggregates. Well-graded RAP with 100% passing 1 1/2-in (37.5-mm) sieve may be used as a capping aggregate on the top 3 in when aggregate gradations CS 01, CS 02, or RR 01 are used in lower lifts.

Table 4.7. Gradation ranges of different aggregates (IDOT 2016)

Grad No.	COARSE AGGREGATE GRADATIONS — Sieve Size and Percent Passing												
	3 in. 75 mm	2 1/2 in. 63 mm	2 in. 50 mm	1 1/2 in. 37.5 mm	1 in. 25 mm	3/4 in. 19 mm	1/2 in. 12.5 mm	3/8 in. 9.5 mm	No. 4 4.75 mm	No. 8 2.36 mm	No. 16 1.18 mm	No. 50 300 µm	No. 200 [1/] 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 4.8. 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 4.9. 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/]	---

Table 4.10. CS01, CS02, and RR01 gradations (Kazmee and Tutumluer 2015)

Gradation Band	Coarse Aggregate Subgrade Gradations							
	Sieve Size and Percent Passing							
	203 mm	152 mm	102 mm	76 mm	51 mm	38 mm	4.76 mm	0.074 mm
	8"	6"	4"	3"	2"	1 ½"	#4	#200
CS 01	100	97 ± 3	90 ± 10		45 ± 25		20 ± 20	
CS 02		100	80 ± 10		25 ± 15			
RR 01				100		53 ± 23		

4.3 MNDOT

RAP and RCA are both allowed in Section 2211 of the MnDOT Standard Specifications for construction published in 2018 to be used as the 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 4.11). In addition, RAP and RCA should meet the quality requirements, which are the same for all aggregate classes (Table 4.12) (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 stockpiled aggregates (McGarrah 2007). Unless RAP content is less than 25% (by volume), the blend of RAP and aggregate is named recycled blend. A small percentage of recycled aggregate (< 25%) can be mixed with aggregate with no change in the class of aggregate and no change in the quality control measurements specified for aggregate (McGarrah 2007). While 3% of bitumen content was allowed in 2005 (MnDOT 2005), it was increased to 3.5% in 2018 (MnDOT 2018).

Almost all concrete pavements in Minnesota are recycled as dense-graded base aggregate material (Gonzalez and Moo-Young 2004). Fine-grained (< No. 4 sieve) RCA particles must be removed to avoid drainage issues. Moreover, open-graded RCA can be mixed with natural aggregates to minimize the problems associated with the leaching of heavy metals (Snyder 1995, as cited in Gonzalez and Moo-Young 2004).

Per Section 3138, depending on the project, the blends of VA and recycled aggregates with less than 25% recycled aggregates used as an aggregate base layer material should meet the gradations specified for different aggregate classes (Table 4.13) (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 4.14. In addition, if 75% or more recycled concrete is used, the mixture should meet the gradation criteria shown in Table 4.15 (MnDOT 2018).

Table 4.11. Quality requirements for virgin aggregates (VA) (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 4.12. 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 4.13. Gradation of base layer 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 4.14. Gradation of base layer 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 4.15. Gradation of base layer 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 MODOT

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 4.16), Type 5 (Table 4.17), and Type 7 aggregates (Table 4.18) (MoDOT 2018). These aggregate types are the aggregates that can be used as aggregate base layers. 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. The plasticity index (PI) of particles passing a No. 40 sieve should not be more than 6 (MoDOT 2018).

Table 4.16. 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 4.17. 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 4.18. 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 WISDOT

Aggregates, breaker run, crushed gravel, crushed stone, pit run, reclaimed asphalt, and crushed concrete can be used for different aggregate base layer applications according to Section 301 of the WisDOT Standard Specifications published in 2018 (Table 4.19). Reclaimed asphalt is only suitable for dense 1 1/4-in (31.5-mm) aggregate base type, while crushed concrete is suitable for dense 3/4-in (19-mm), dense 1 1/4-in (31.5-mm), and dense 3-in (75-mm) 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 4.20), and the 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 4.21. For reclaimed asphalt, gradation is primarily assessed visually, e.g., reclaimed asphalt 100% passing 1 1/4-in (31.5-mm) sieve may be used for 1 1/4-in (31.5-mm) 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 4.19. 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 4.20. 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 4.21. 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 MDOT

Sections 302 and 902 of the MDOT Standard Specifications for Construction published in 2012 allows the use of crushed concrete along with natural aggregate and iron blast furnace slag as base materials if they meet the gradation (Table 4.22) and quality (Table 4.23) 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 in (305 mm) [with Class I, II, IIA, or IIAA aggregates (Table 4.24)]

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 the granular layer between the dense-graded aggregate base and the underdrain (MDOT 2012).

Table 4.22. 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 4.23. 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 4.24. 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 AASHTOWARE PMED

In order to produce reliable and reasonable results, the AASHTOWare 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 AASHTOWare PMED software allows users to enter input parameters 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 laboratory studies, leading to the most reliable pavement distress analyses, while level 3 input provides less reliable pavement distress predictions.

For instance, traffic data in its simplest form can simply be an estimate of vehicle traffic volumes. Since the AASHTOWare PMED analyses rely on traffic data to calculate the loads that a pavement system can be subjected to, the software will need to convert traffic information into a load factor by assuming a typical distribution of vehicle types. However, if the information regarding the actual traffic counts for a project site along with the information about vehicle classes is available, this will allow an additional level of input in the hierarchy. Assumptions will still need to be made about the spectrum of actual loads based on equivalency factors [equivalent single axle loads (ESALs)]. At the top of the hierarchy, in addition to monitoring vehicle counts, vehicle weight data near the site is needed to determine the actual load distribution. 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 reliability of the design assumptions. However, the AASHTOWare PMED process can still function at lower levels of detail.

During the AASHTOWare PMED analyses in this study, the design inputs of the pavement surface and subgrade layers were kept constant to be able to investigate the effect of the properties of RAP and RCA base layers on predicted pavement distresses. All analyses were conducted at a 90% reliability level. Table 5.1 summarizes the general inputs used in the AASHTOWare PMED analyses. For the design simulations, Minnesota MnROAD climate was used. To enable specific selection of the MnROAD climate, six neighboring weather stations [coordinates: (1) 45.5016205, -93.7609675; (2) 45.001088, -93.7521709; (3) 45.5003105, -93.1249918; (4) 45.0003614, -93.1247624; (5) 45.4994913, -94.3722707; (6) 45.0000331, -94.368383] were selected to create a virtual weather station at MnROAD.

Table 5.1. General inputs for AASHTOWare PMED analyses

Input	Value
Design Period	20 years
SM_r of Subgrade, psi (MPa)	15,000 (103)
Subgrade Gradation	A-1-b
Groundwater Depth, ft (m)	10 (3)
Flexible Pavement Input	
Binder Grade	Superpave PG 58-34
Base Poisson's Ratio	0.35
HMA Poisson's Ratio	0.35
Rigid Pavement Input	
PCC Unit Weight, pcf (kN/m^3)	150 (23.6)
PCC Poisson's Ratio	0.15

SM_r = summary resilient modulus; HMA = hot mix asphalt; PCC = Portland cement concrete.

For designing rigid pavement systems, dowel bars were included in the designs. A dowel bar diameter of 1.25 in (32 mm) and a dowel bar spacing of 12 in (305 mm) were used in the designs. Three different traffic volumes (low, medium, and high traffic) were considered for the analyses. Table 5.2 shows the traffic data used in the AASHTOWare PMED analyses along with surface and base layer thicknesses, which were selected per recommendations of Schwartz et al. (2011).

Table 5.2. Traffic inputs for AASHTOWare PMED analyses

Input	Low Traffic	Medium Traffic	High Traffic
AADTT	1,000	7,500	25,000
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 (km/h)	50 (80)	50 (80)	50 (80)
Asphalt Thickness in Flexible Pavement, in (mm)	2 (50)	3 (75)	4 (100)
Base Thickness in Flexible Pavement, in (mm)	8 (200)	10 (250)	12 (305)
PCC Thickness for Rigid Pavement, in (mm)	8 (200)	9 (230)	11 (280)
Base Thickness in Rigid Pavement, in (mm)	4 (100)	6 (150)	8 (200)

AADTT = annual average 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 aggregates, the lowest, highest, and median values of SM_r, gradation, K_{sat}, OMC, and MDU of these materials were collected from the database. Summaries of these inputs are also shown in Appendices C and D for RAP and RCA, respectively. The highest and lowest values were obtained from the database for each property shown in Table 5.3, Table 5.4, Table 5.5, Table 5.6, Table 5.7, Table 5.8, Table 5.9, Table 5.10, Table 5.11, and Table 5.12, while the median values were calculated from all the available data for each material property available in the database.

The lowest SM_r value of RAP was reported to be 24,366 psi (168 MPa) by Edil et al. (2012a); thus, other inputs shown in Table 5.3 were chosen from that paper accordingly. On the other hand, the highest SM_r

value of RAP was 58,015 psi (400 MPa) from Attia and Abdelrahman (2010a), and other inputs were collected from the same paper as well.

Table 5.3. Base inputs investigating SM_r effect of recycled asphalt pavement (RAP)

Data Value	Varied Parameter = SM_r, psi (MPa)	Gravel, %	Sand, %	Fines, %	MDU, pcf (kN/m^3)	OMC, %	K_{sat}, ft/hr (m/s)
Lowest*	24,366 (168)	49.3	50.4	0.4	138 (21.7)	5.2	2.73 (2.3x10^{-4})
Median	37,927 (262)	45	54	1	126 (19.8)	6.1	0.71 (6x10^{-5})
Highest**	58,015 (400)	51	48.6	0.4	134 (21)	5.5	-

SM_r = summary resilient modulus; MDU = maximum dry unit weight; OMC = optimum moisture content; K_{sat} = saturated hydraulic conductivity. *Edil et al. (2012a); **Attia and Abdelrahman (2010a)

The lowest SM_r value of RCA was reported to be 17,898 psi (123 MPa) by Cetin et al. (2020); thus, other inputs shown in Table 5.4 are chosen from that report accordingly. The highest SM_r value of RCA was 53,664 psi (370 MPa) from Diagne et al. (2015), and other inputs were collected from the same paper as well.

Table 5.4. Base inputs investigating SM_r effect of recycled concrete aggregate (RCA)

Data Value	Varied Parameter = SM_r, psi (MPa)	Gravel, %	Sand, %	Fines, %	MDU, pcf (kN/m^3)	OMC, %	K_{sat}, ft/hr (m/s)
Lowest*	17,898 (123)	38.3	54.6	7.1	123 (19.3)	11.1	0.06 (5.1x10^{-6})
Median	26,542 (183)	50.8	45.5	3	127 (20)	10.8	0.2 (1.7x10^{-5})
Highest**	53,664 (370)	47.2	48.6	1.8	134 (21)	6.1	0.35 (3x10^{-5})

SM_r = summary resilient modulus; MDU = maximum dry unit weight; OMC = optimum moisture content; K_{sat} = saturated hydraulic conductivity. *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 5.5 are chosen from that report accordingly. The highest fines content of RAP was 11% from Camargo et al. (2013), and other inputs were collected from the same paper as well.

Table 5.5. Base inputs investigating fines content effect of recycled asphalt pavement (RAP)

Data Value	Varied Parameter = Fines, %	Gravel, %	Sand, %	MDU, pcf (kN/m^3)	OMC, %	K_{sat}, ft/hr (m/s)	SM_r, psi (MPa)
Lowest*	0	3	97	-	-	-	39,349 (271)
Median	1	45	54	126 (19.8)	6.1	0.71 ($6x10^{-5}$)	37,927 (262)
Highest**	11	46	43	136 (21.4)	7.5	-	44,962 (310)

SM_r = summary resilient modulus; MDU = maximum dry unit weight; OMC = optimum moisture content; K_{sat} = saturated hydraulic conductivity. *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 5.6 are chosen from that report accordingly. The highest fines content of RCA was 15% from Chen et al. (2013), and other inputs were collected from the same paper as well.

Table 5.6. Base inputs investigating fines content effect of recycled concrete aggregate (RCA)

Data Value	Varied Parameter = Fines, %	Gravel, %	Sand, %	MDU, pcf (kN/m^3)	OMC, %	K_{sat}, ft/hr (m/s)	SM_r, psi (MPa)
Lowest*	0.1	68.8	31.1	127 (20)	14.4	-	-
Median	3	50.8	45.5	127 (20)	10.8	0.2 ($1.7x10^{-5}$)	26,542 (183)
Highest**	15	41	44	121 (19)	11.9	-	27,412 (189)

SM_r = summary resilient modulus; MDU = maximum dry unit weight; OMC = optimum moisture content; K_{sat} = saturated hydraulic conductivity. *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 5.7 are chosen from that report accordingly. The highest gravel content of RAP was 68.1% from Garg and Thompson (1996), and other inputs were collected from the same paper as well.

Table 5.7. Base inputs investigating gravel content effect of recycled asphalt pavement (RAP)

Data Value	Varied Parameter = Gravel, %	Sand, %	Fines, %	MDU, pcf (kN/m^3)	OMC, %	K_{sat}, ft/hr (m/s)	SM_r, psi (MPa)
Lowest*	3	97	0	-	-	-	39,349 (271)
Median	45	54	1	126 (19.8)	6.1	0.71 ($6x10^{-5}$)	37,927 (262)
Highest**	68.1	28.1	3.8	135 (21.2)	6	-	44,962 (310)

SM_r = summary resilient modulus; MDU = maximum dry unit weight; OMC = optimum moisture content; K_{sat} = saturated hydraulic conductivity. *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 5.8 are chosen from that report accordingly. The highest gravel content of RCA was 94.1% from Mahedi and Cetin (2020), and other inputs were collected from the same paper as well.

Table 5.8. Base inputs investigating gravel content effect of recycled concrete aggregate (RCA)

Data Value	Varied Parameter = Gravel, %	Sand, %	Fines, %	MDU, pcf (kN/m^3)	OMC, %	K_{sat}, ft/hr (m/s)	SM_r, psi (MPa)
Lowest*	31.8	64.9	3.3	125 (19.6)	11.2	-	27,412 (189)
Median	50.8	45.5	3	127 (20)	10.8	0.2 ($1.7x10^{-5}$)	26,542 (183)
Highest**	94.1	4.9	1	118 (18.5)	12.6	-	-

SM_r = summary resilient modulus; MDU = maximum dry unit weight; OMC = optimum moisture content; K_{sat} = saturated hydraulic conductivity. *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 5.9 are chosen from that report accordingly. The highest sand content of RAP was 97% from Alam et al. (2010), and other inputs were collected from the same paper as well.

Table 5.9. Base inputs investigating sand content effect of recycled asphalt pavement (RAP)

Data Value	Varied Parameter = Sand, %	Gravel, %	Fines, %	MDU, pcf (kN/m^3)	OMC, %	K_{sat}, ft/hr (m/s)	SM_r, psi (MPa)
Lowest*	28.1	68.1	3.8	135 (21.2)	6	-	31,702 (219)
Median	54	45	1	126 (19.8)	6.1	0.71 ($6x10^{-5}$)	37,927 (262)
Highest**	97	3	0	-	-	-	39,349 (271)

SM_r = summary resilient modulus; MDU = maximum dry unit weight; OMC = optimum moisture content; K_{sat} = saturated hydraulic conductivity. *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 5.10 are chosen from that report accordingly. The highest sand content of RCA was 64.9% from Edil et al. (2017), and other inputs were collected from the same paper as well.

Table 5.10. Base inputs investigating sand content effect of recycled concrete aggregate (RCA)

Data Value	Varied Parameter = Sand, %	Gravel, %	Fines, %	MDU, pcf (kN/m^3)	OMC, %	K_{sat}, ft/hr (m/s)	SM_r, psi (MPa)
Lowest*	4.9	94.1	1	118 (18.5)	12.6	-	-
Median	45.5	50.8	3	127 (20)	10.8	0.2 ($1.7x10^{-5}$)	26,542 (183)
Highest**	64.9	31.8	3.5	125 (19.6)	11.2	-	27,412 (189)

SM_r = summary resilient modulus; MDU = maximum dry unit weight; OMC = optimum moisture content; K_{sat} = saturated hydraulic conductivity. *Mahedi and Cetin (2020); **Edil et al. (2017)

The lowest D_{60} value of RAP was reported to be 0.09 in (2.3 mm) by Edil et al. (2012a) in a RAP sample from Minnesota; thus, other inputs shown in Table 5.11 are chosen from that report accordingly. The highest D_{60} value of RAP was 0.409 in (10.4 mm) from Wu et al. (2012), and other inputs were collected from the same paper as well.

Table 5.11. Base inputs investigating D_{60} (diameter at which 60% of the particles are finer) effect of recycled asphalt pavement (RAP)

Data Value	Varied Parameter = D_{60}, in (mm)	Gravel, %	Sand, %	Fines, %	MDU, pcf (kN/m^3)	OMC, %	K_{sat}, ft/hr (m/s)	SM_r, psi (MPa)
Lowest*	0.09 (2.3)	26.3	71.2	2.5	134 (21)	6.7	0.013 (1.1×10^{-6})	26,107 (180)
Median	0.19 (4.8)	45	54	1	126 (19.8)	6.05	0.71 (6×10^{-5})	37,927 (262)
Highest**	0.409 (10.4)	67	32	1	-	-	-	29,008 (200)

SM_r = summary resilient modulus; MDU = maximum dry unit weight; OMC = optimum moisture content; K_{sat} = saturated hydraulic conductivity. *Edil et al. (2012a); **Wu et al. (2012)

The lowest D_{60} value of RCA was reported to be 0.067 in (1.7 mm) by Edil et al. (2012a); thus, other inputs shown in Table 5.12 are chosen from that report accordingly. The highest D_{60} value of RCA was 0.642 in (16.3 mm) from Edil et al. (2012a), and other inputs were collected from the same paper as well.

Table 5.12. Base inputs investigating D_{60} (diameter at which 60% of the particles are finer) effect of recycled concrete aggregate (RCA)

Data Value	Varied Parameter = D_{60}, in (mm)	Gravel, %	Sand, %	Fines, %	MDU, pcf (kN/m^3)	OMC, %	K_{sat}, ft/hr (m/s)	SM_r, psi (MPa)
Lowest*	0.067 (1.7)	31.8	64.9	3.3	125 (19.6)	11.2	-	27,412 (189)
Median	0.268 (6.8)	50.8	45.5	3	127 (20)	10.8	0.2 (1.7×10^{-5})	26,542 (183)
Highest*	0.642 (16.3)	76.3	21.6	2.1	127 (20)	9.2	-	23,786 (164)

SM_r = summary resilient modulus; MDU = maximum dry unit weight; OMC = optimum moisture content; K_{sat} = saturated hydraulic conductivity. *Edil et al. (2012a)

CHAPTER 6: PAVEMENT DISTRESSES

The following pavement distresses were analyzed via the AASHTOWare PMED software: 1) IRI and rutting for flexible pavements and 2) IRI and mean joint faulting for rigid pavements.

The threshold values to define the failure criteria for flexible pavements in terms of IRI and total rutting (90% reliability) are summarized in Table 6.1. IRI values greater than 170 in/mile (2.68 m/km) were marked as the IRI failure criterion in this study per suggestions of Elbheiry et al. (2011), and this value was determined as the terminal IRI. The failure criterion for total rutting was determined to be 0.75 in (20 mm) (Ceylan et al. 2015). Table 6.2 represents the threshold values to define the failure criteria for rigid pavements in terms of IRI and mean joint faulting. The failure criteria for IRI and mean joint faulting were determined to be 172 in/mile (2.71 m/km) and 0.12 in (3 mm), respectively. In this chapter, pavement distress analyses were performed using the inputs provided in Chapter 5.

Table 6.1. Pavement distress types and threshold values for flexible pavement

Parameter	Threshold Value	Reliability, %
Terminal IRI, in/mile (m/km)	170 (2.68)	90
Total Rutting, in (mm)	0.75 (20)	90

IRI = international roughness index

Table 6.2. Pavement distress types and threshold values for jointed plain concrete pavement (JPCP)

Parameter	Threshold Value	Reliability, %
Terminal IRI, in/mile (m/km)	172 (2.71)	90
Mean Joint Faulting, in (mm)	0.12 (3)	90

IRI = international roughness index

6.1 IRI FOR FLEXIBLE PAVEMENTS

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 (2.68 m/km) for flexible pavements (Elbheiry et al. 2011). The initial IRI value was determined to be 63 in/mile (1 m/km), which was in accordance with the suggestions provided by Izevbekhai and Akkari (2011) and Ceylan et al. (2015).

6.1.1 Effect of SM_r on IRI

The predicted IRI values using the inputs provided in Table 5.3 and Table 5.4 are shown in Figure 6.1 for RAP and Figure 6.2 for RCA in flexible pavements. Both Figure 6.1 and Figure 6.2 show that higher traffic volume and aggregate base layers with lower SM_r values yielded greater IRI damage in flexible pavements, indicating that stiffness of aggregate base layers had an effect on IRI performance. However, it was concluded that the SM_r values of aggregate base layers did not cause notable differences in terms of IRI performance, and none of the results exceeded the terminal IRI value [170 in/mile (2.68 m/km)]. Thus, acceptable RAP/RCA base layer performance in terms of IRI was obtained using different SM_r values presented in the database.

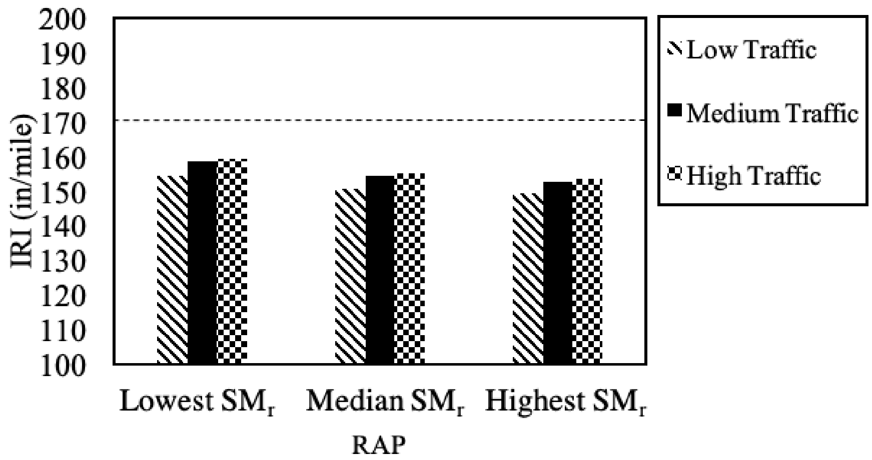

Figure 6.1. International roughness index (IRI) vs. summary resilient modulus (SM_r) of recycled asphalt pavement (RAP)

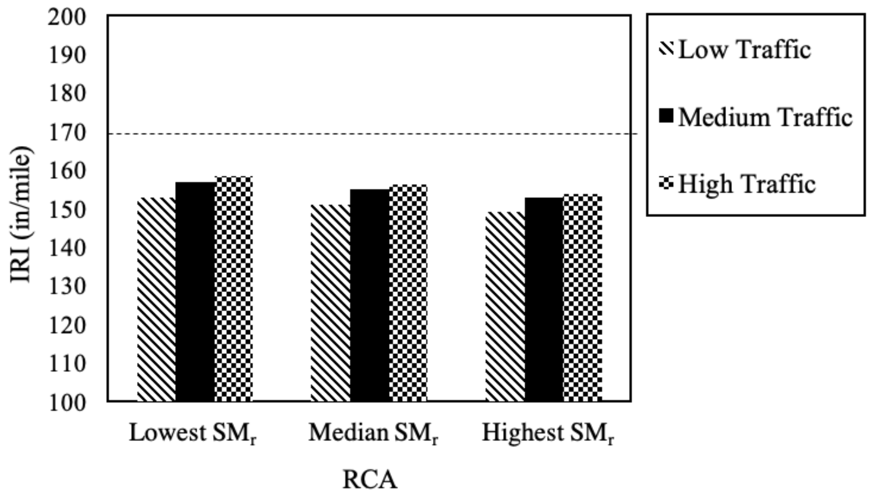

Figure 6.2. International roughness index (IRI) vs. summary resilient modulus (SM_r) of recycled concrete aggregate (RCA)

6.1.2 Effect of Fines Content on IRI

The predicted IRI values using the inputs provided in Table 5.5 and Table 5.6 are shown in Figure 6.3 for RAP and Figure 6.4 for RCA in flexible pavements. Results showed that higher fines content of RAP (ranging between 0 and 11%) and RCA (ranging between 0.1 and 15%) used as base layers yielded higher IRI values in flexible pavements (Figure 6.3 and Figure 6.4). However, none of the results exceeded the terminal IRI value [170 in/mile (2.68 m/km)], indicating that acceptable RAP/RCA base layer performance in terms of IRI was obtained using different fines contents values presented in the database. In addition, higher traffic volume yielded higher IRI values regardless of fines contents of RAP and RCA.

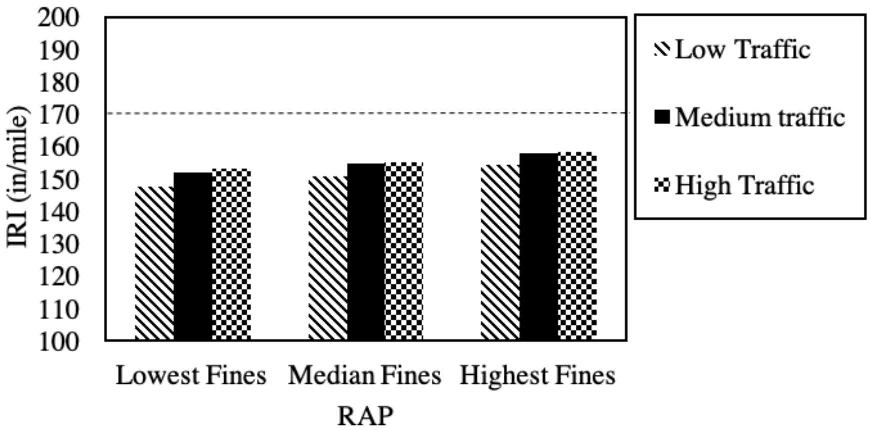

Figure 6.3. International roughness index (IRI) vs. fines content of recycled asphalt pavement (RAP) in flexible pavement

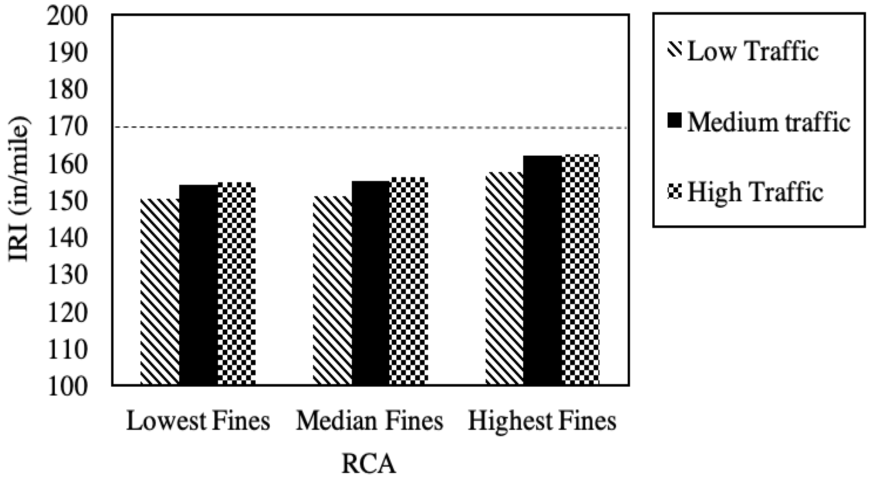

Figure 6.4. International roughness index (IRI) vs. fines content of recycled concrete aggregate (RCA) in flexible pavement

6.1.3 Effect of Gravel Content on IRI

The predicted IRI values using the inputs provided in Table 5.7 and Table 5.8 are shown in Figure 6.5 for RAP and Figure 6.6 for RCA in flexible pavements. Results showed that higher gravel content of RAP (ranging between 3 and 68.1%) and RCA (ranging between 31.8 and 94.1%) used as base layers yielded slightly higher IRI values (almost negligible). As expected, higher traffic volume resulted in higher IRI values. Moreover, none of the results exceeded the terminal IRI value [170 in/mile (2.68 m/km)], indicating that acceptable RAP/RCA base layer performance in terms of IRI was obtained using different gravel contents presented in the database.

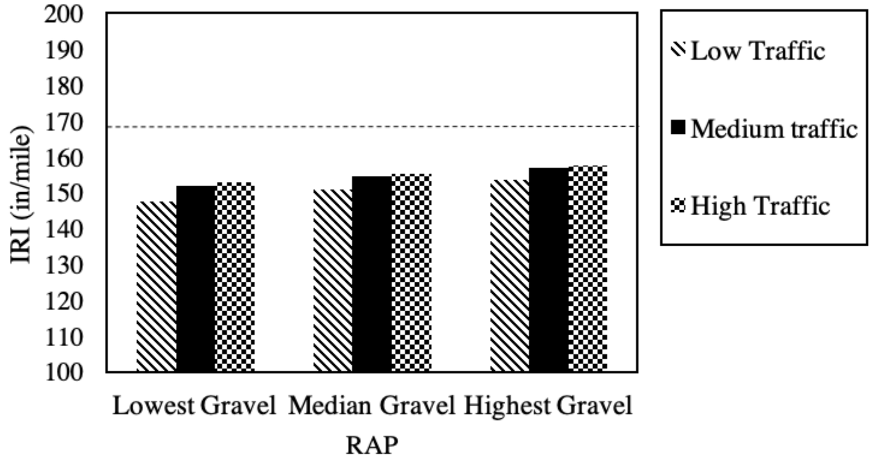

Figure 6.5. International roughness index (IRI) vs. gravel content of recycled asphalt pavement (RAP) in flexible pavement

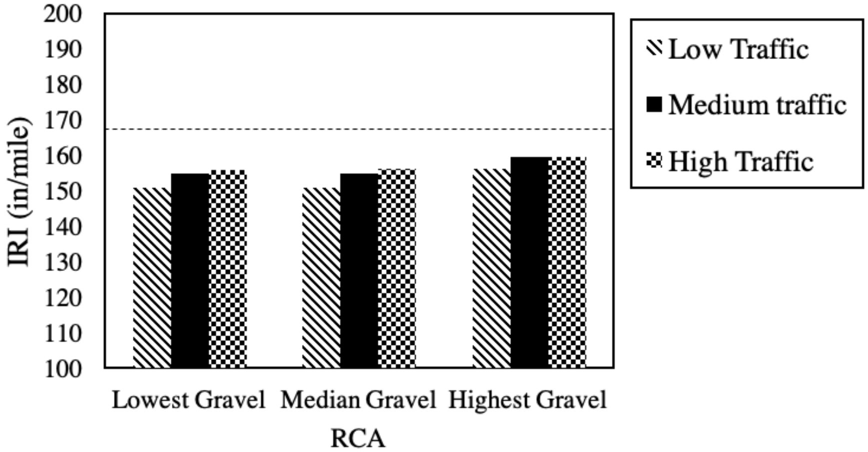

Figure 6.6. International roughness index (IRI) vs. gravel content of recycled concrete aggregate (RCA) in flexible pavement

6.1.4 Effect of Sand Content on IRI

The predicted IRI values using the inputs provided in Table 5.9 and Table 5.10 are shown in Figure 6.7 for RAP and Figure 6.8 for RCA in flexible pavements. Results showed that higher sand content of RAP (ranging between 28.1 and 97%) and RCA (ranging between 4.9 and 64.9%) used as base layers provided slightly higher IRI values. However, these changes were very small and can be negligible. As expected, higher traffic volume resulted in higher IRI values. Moreover, none of the results exceeded the terminal IRI value [170 in/mile (2.68 m/km)], indicating that acceptable RAP/RCA base layer performance in terms of IRI was obtained using different sand contents presented in the database.

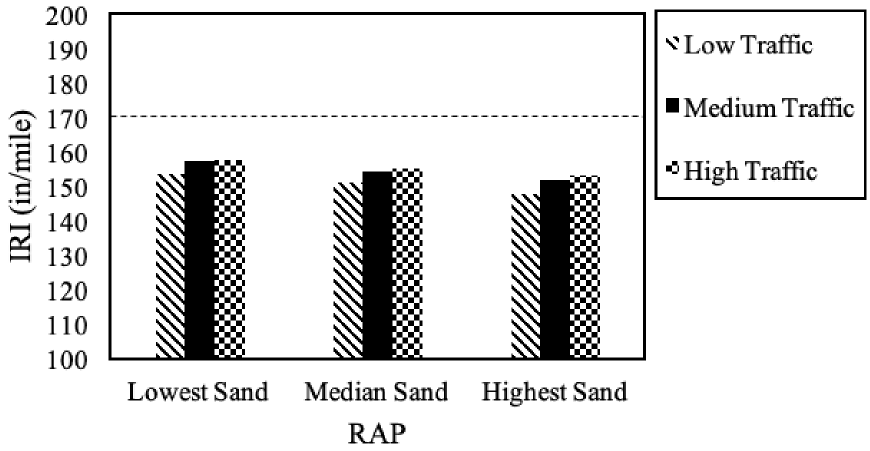

Figure 6.7. International roughness index (IRI) vs. sand content of recycled asphalt pavement (RAP) in flexible pavement

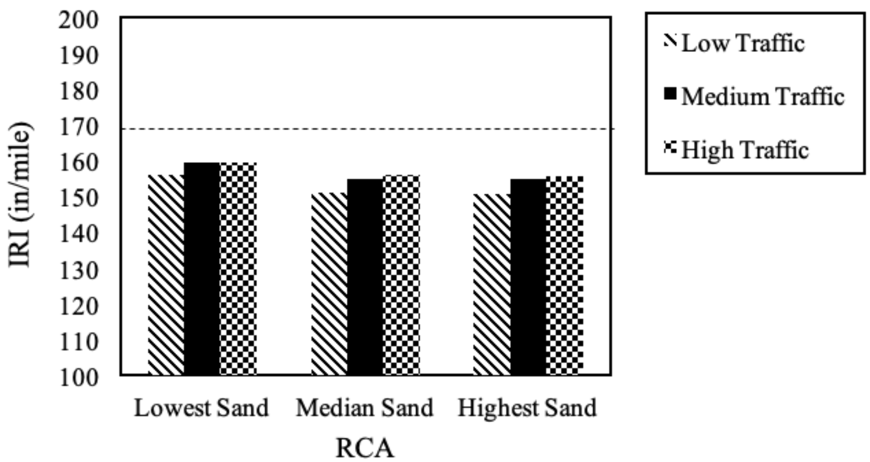

Figure 6.8. International roughness index (IRI) vs. sand content of recycled concrete aggregate (RCA) in flexible pavement

6.1.5 Effect of D_{60} on IRI

The predicted IRI values using the inputs provided in Table 5.11 and Table 5.12 are shown in Figure 6.9 for RAP and Figure 6.10 for RCA in flexible pavements. Based on the results, it was concluded that there was no clear trend between the D_{60} values of RAP and RCA used as base layers and the IRI values. As expected, higher traffic volume resulted in higher IRI values.

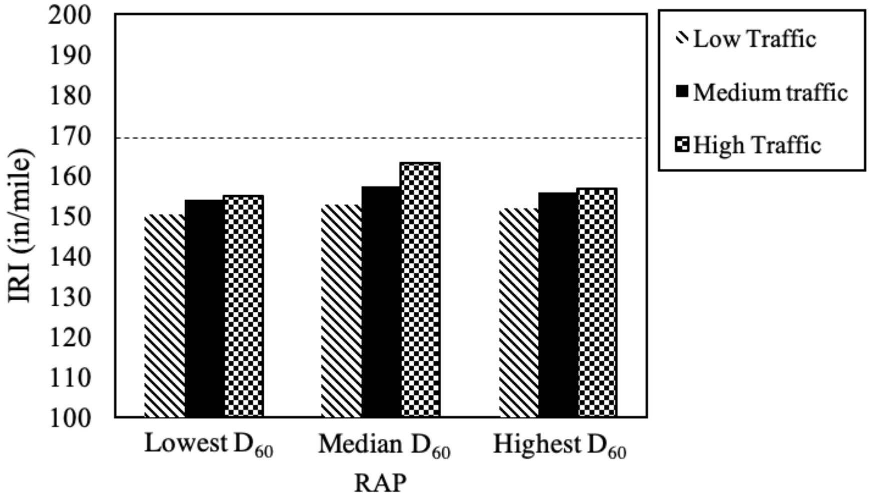

Figure 6.9. International roughness index (IRI) vs. D_{60} (diameter at which 60% of the particles are finer) of recycled asphalt pavement (RAP) in flexible pavement

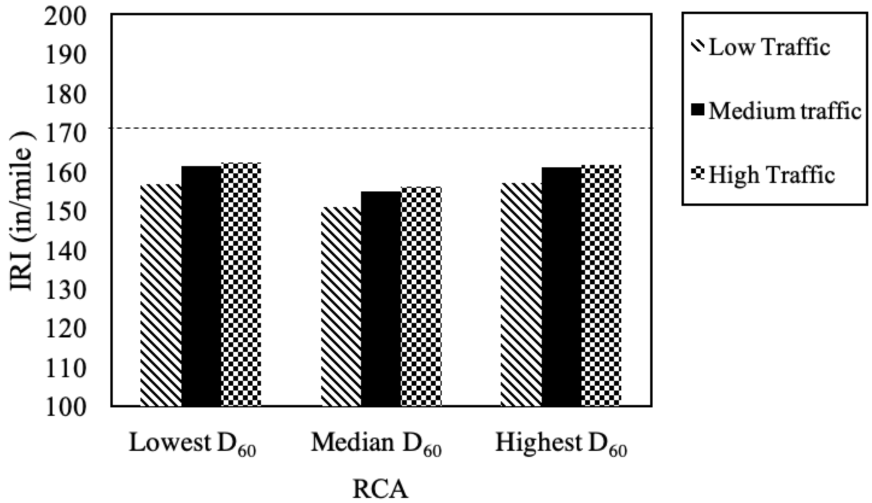

Figure 6.10. International roughness index (IRI) vs. D_{60} (diameter at which 60% of the particles are finer) of recycled concrete aggregate (RCA) in flexible pavement

6.2 IRI FOR RIGID PAVEMENTS

The initial IRI value was determined to be 63 in/mile (1 m/km), which was in accordance with the suggestions provided by Izevbekhai and Akkari (2011) and Ceylan et al. (2015).

6.2.1 Effect of SM_r on IRI

The predicted IRI values using the inputs provided in Table 5.3 and Table 5.4 are shown in Figure 6.11 for RAP and Figure 6.12 for RCA in rigid pavements. Results showed that there was no clear trend between the SM_r values of RAP and RCA used as base layers and the IRI values. However, traffic volume had a significant effect on the IRI of rigid pavements. As expected, higher traffic volume resulted in higher IRI values. For low traffic condition, none of the results exceeded the terminal IRI value [172 in/mile (2.71

m/km)]. Thus, acceptable RAP/RCA base layer performance in terms of IRI was obtained using different SM_r values presented in the database under low traffic. On the other hand, overall, the IRI values under medium and high traffic volumes exceeded the terminal IRI value [172 in/mile (2.71 m/km)] (except for the case when RAP had the lowest SM_r under medium traffic), indicating that SM_r of base layers are influential on rigid pavement distresses under higher traffic loads.

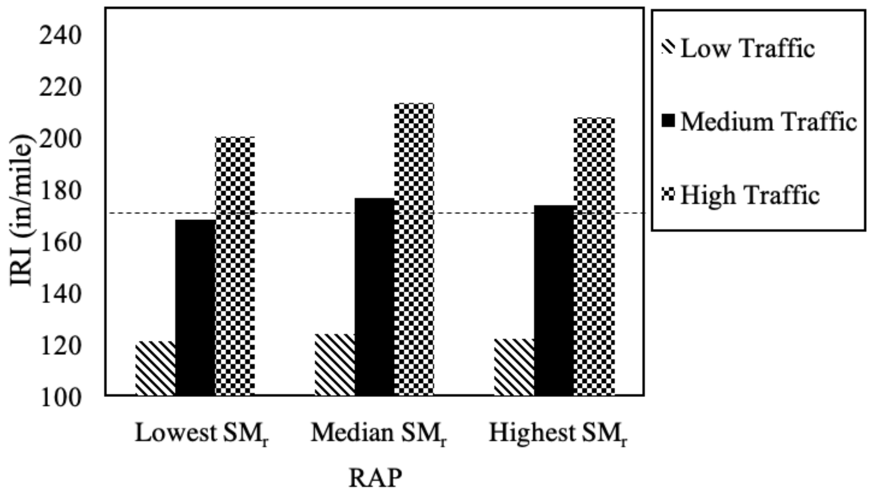

Figure 6.11. International roughness index (IRI) vs. summary resilient modulus (SM_r) of recycled asphalt pavement (RAP) in rigid pavement

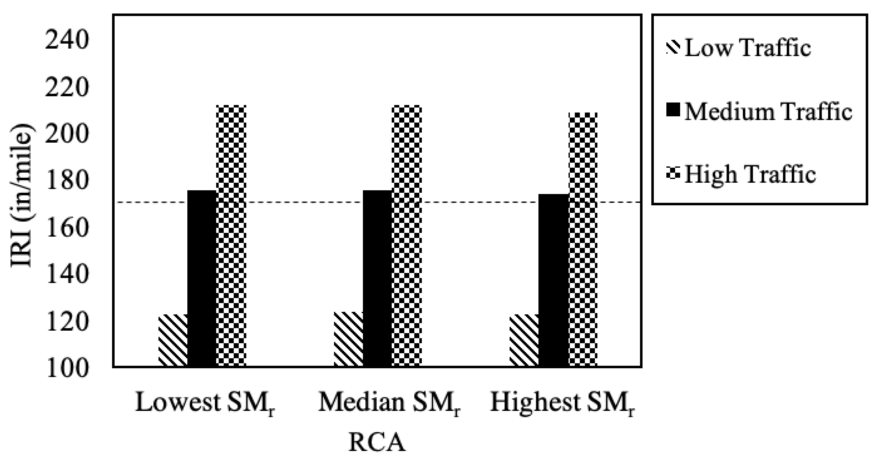

Figure 6.12. International roughness index (IRI) vs. summary resilient modulus (SM_r) of recycled concrete aggregate (RCA) in rigid pavement

6.2.2 Effect of Fines Content on IRI

The predicted IRI values using the inputs provided in Table 5.5 and Table 5.6 are shown in Figure 6.13 for RAP and Figure 6.14 for RCA in rigid pavements. Figure 6.13 shows that an increase in the fines content of RAP (ranging between 0 and 11%) used as base layers caused a slight decrease in the IRI values for rigid pavements. On the other hand, for RCA used as base layers, an increase in the fines content (ranging between 0.1 and 15%) resulted in higher IRI values (Figure 6.14). In addition, higher traffic volume yielded higher IRI values regardless of the fines contents of RAP and RCA. For low traffic condition, none of the results exceeded the terminal IRI value [172 in/mile (2.71 m/km)]. Thus, acceptable RAP/RCA base layer performance in terms of IRI was obtained using different fines content values presented in the database under low traffic. However, overall, the IRI values under medium and high traffic volumes exceeded the terminal IRI value [172 in/mile (2.71 m/km)] (except for the cases when RAP had the highest fines content and RCA had the lowest fines content under medium traffic).

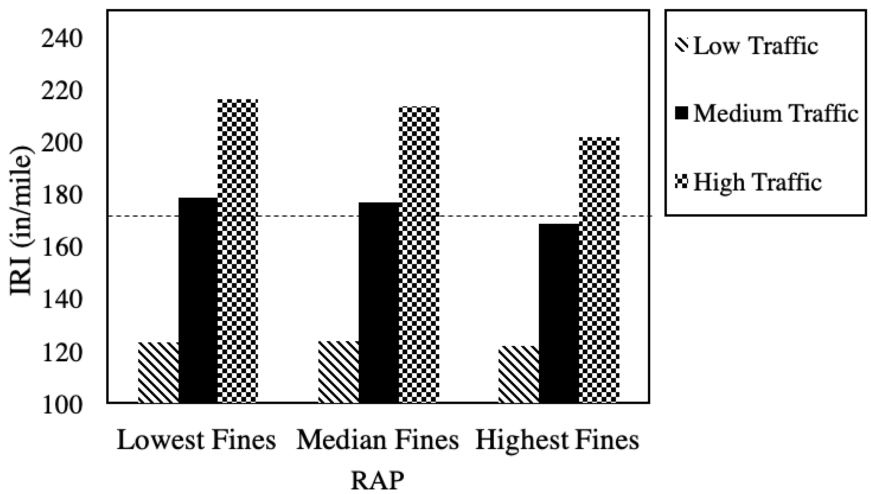

Figure 6.13. International roughness index (IRI) vs. fines content of recycled asphalt pavement (RAP) in rigid pavement

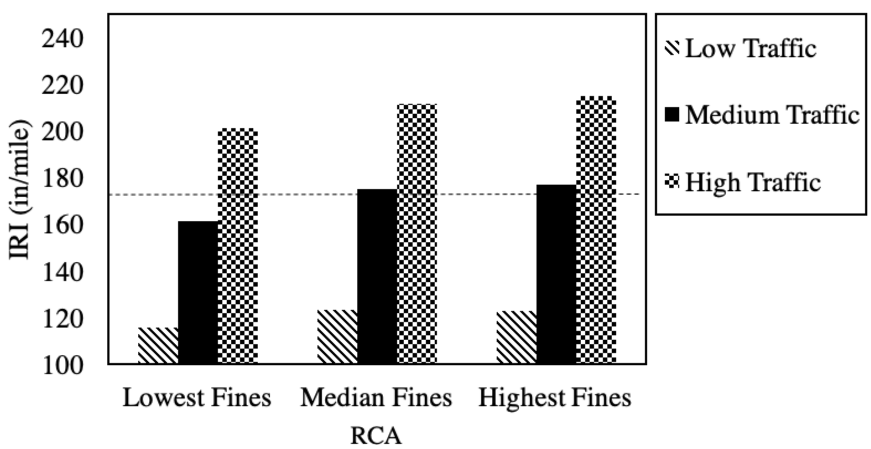

Figure 6.14. International roughness index (IRI) vs. fines content of recycled concrete aggregate (RCA) in rigid pavement

6.2.3 Effect of Gravel Content on IRI

The predicted IRI values using the inputs provided in Table 5.7 and Table 5.8 are shown in Figure 6.15 for RAP and Figure 6.16 for RCA in rigid pavements. Results showed that higher gravel content of RAP (ranging between 3 and 68.1%) and RCA (ranging between 31.8 and 94.1%) used as base layers yielded lower IRI values. In addition, it was observed that terminal IRI value [172 in/mile (2.71 m/km)] was exceeded when the lowest and median gravel contents were used under medium and high traffic volumes. While the IRI values were higher than terminal IRI value [172 in/mile (2.71 m/km)] when the highest gravel contents were used under high traffic volume, the IRI values were lower than terminal IRI value [172 in/mile (2.71 m/km)] when the highest gravel contents were used under medium traffic volume. This suggests determining the gravel content of RAP and RCA before their use as a base layer aggregate for rigid pavement design.

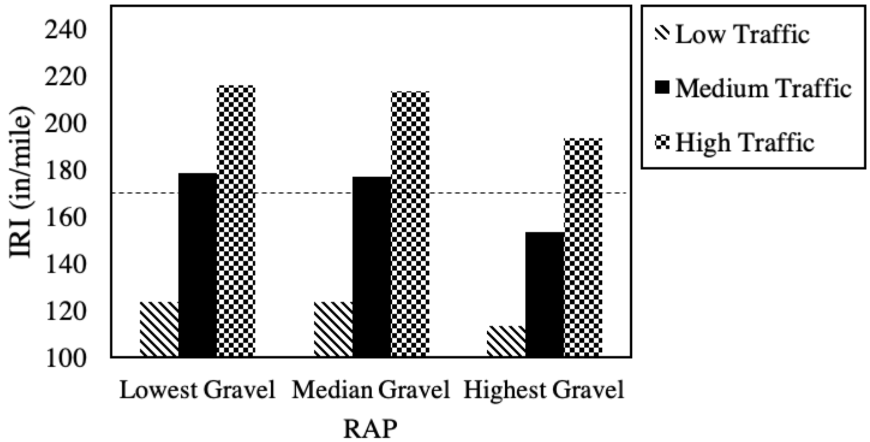

Figure 6.15. International roughness index (IRI) vs. gravel content of recycled asphalt pavement (RAP) in rigid pavement

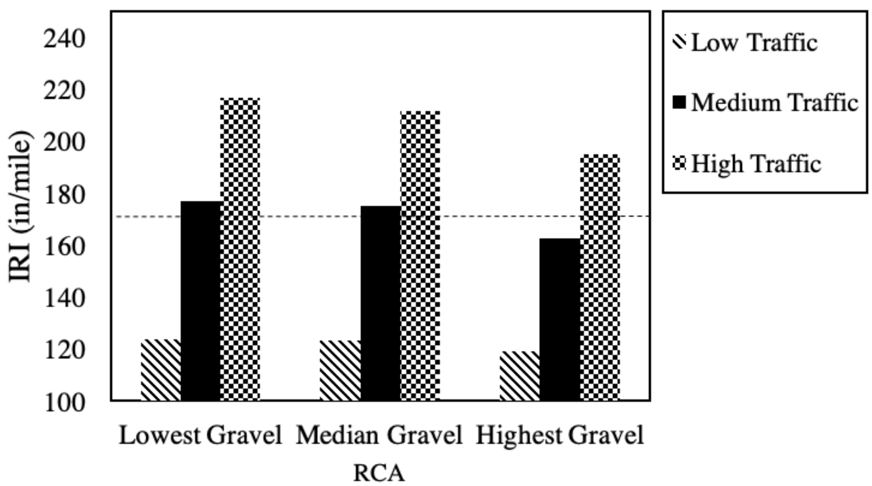

Figure 6.16. International roughness index (IRI) vs. gravel content of recycled concrete aggregate (RCA) in rigid pavement

6.2.4 Effect of Sand Content on IRI

The predicted IRI values using the inputs provided in Table 5.9 and Table 5.10 are shown in Figure 6.17 for RAP and Figure 6.18 for RCA in rigid pavements. Figure 6.17 and Figure 6.18 show that IRI values of rigid pavements with RAP and RCA base layers increased significantly when sand contents change from the lowest (28.1% for RAP and 4.9% for RCA) to median values. On the other hand, no solid trends were observed between the IRI values when changing sand contents from the median to highest values for both RAP and RCA. In addition, higher traffic volume yielded higher IRI values regardless of the sand contents of RAP and RCA. For each low traffic volume condition, none of the results exceeded the terminal IRI value [172 in/mile (2.71 m/km)]. Thus, acceptable RAP/RCA base layer performance in terms of IRI was obtained using different sand content values presented in the database under low traffic. It was also observed that terminal IRI value [172 in/mile (2.71 m/km)] was exceeded when the median and highest sand contents were used under medium and high traffic volumes. These results suggest that sand contents of RAP and RCA used as base layers could be a critical parameter to be checked during designing rigid pavement systems.

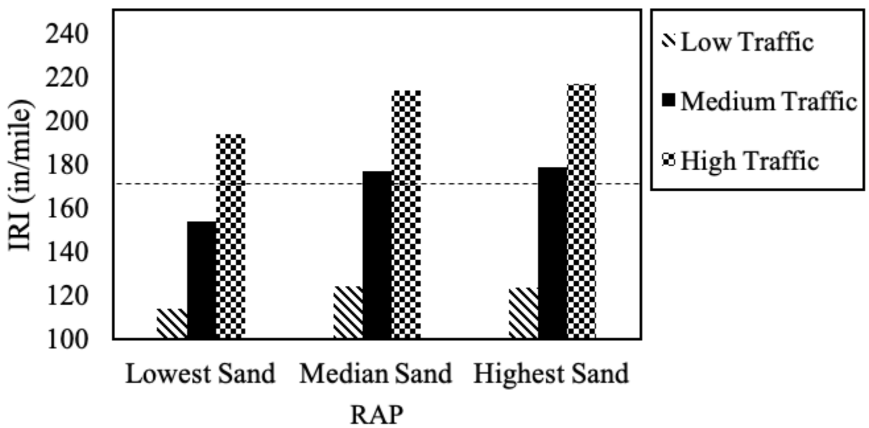

Figure 6.17. International roughness index (IRI) vs. sand content of recycled asphalt pavement (RAP) in rigid pavement

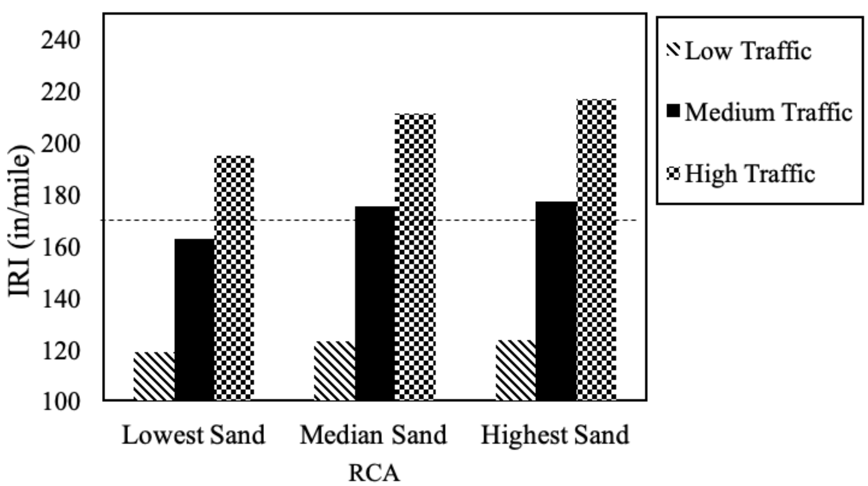

Figure 6.18. International roughness index (IRI) vs. sand content of recycled concrete aggregate (RCA) in rigid pavement

6.2.5 Effect of D_{60} on IRI

The predicted IRI values using the inputs provided in Table 5.11 and Table 5.12 are shown in Figure 6.19 for RAP and Figure 6.20 for RCA in rigid pavements. Results for both RAP and RCA showed that an increase in the D_{60} values from the lowest value [0.09 in (2.3 mm) for RAP and 0.067 in (1.7 mm) for RCA] to the median value [0.19 in (4.8 mm) for RAP and 0.268 in (6.8 mm) for RCA] did not seem to affect the IRI performance of rigid pavements, while it was improved significantly when the D_{60} values were the highest value presented in the database. The results suggest using the D_{60} values higher than the lowest values recorded in the database, which is 0.09 in (2.3 mm) for RAP and 0.067 in (1.7 mm) for RCA. In addition, higher traffic volume yielded higher IRI values regardless of the D_{60} values of RAP and RCA. It was observed that terminal IRI value [172 in/mile (2.71 m/km)] was exceeded when the lowest and median D_{60} values were used under medium and high traffic volumes. In addition, while the IRI values were lower than terminal IRI value [172 in/mile (2.71 m/km)] when the highest D_{60} values were used under medium traffic volume, the IRI values were higher than terminal IRI value [172 in/mile (2.71 m/km)] when the highest D_{60} values were used under high traffic volume.

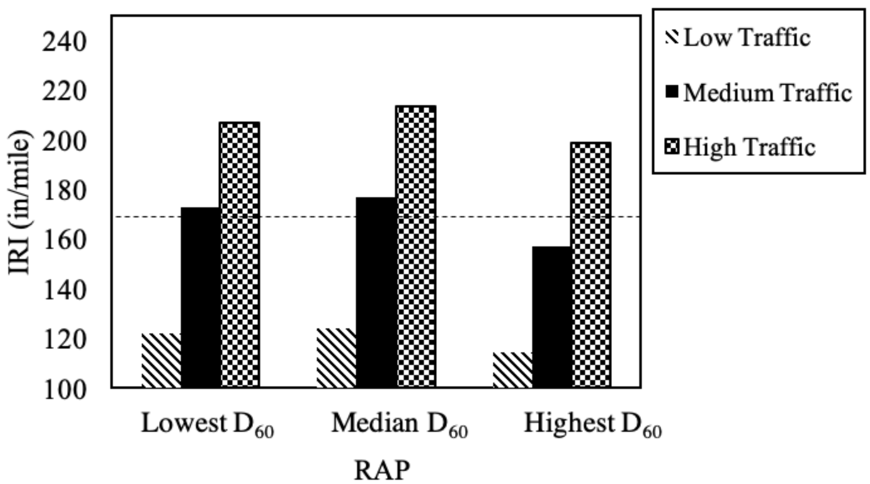

Figure 6.19. International roughness index (IRI) vs. D_{60} (diameter at which 60% of the particles are finer) of recycled asphalt pavement (RAP) in rigid pavement

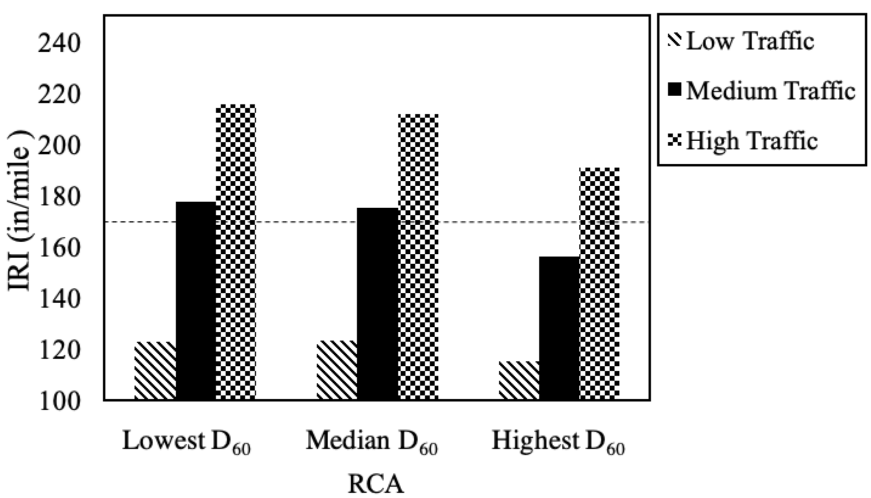

Figure 6.20. International roughness index (IRI) vs. D_{60} (diameter at which 60% of the particles are finer) of recycled concrete aggregate (RCA) in rigid pavement

6.3 TOTAL RUTTING FOR FLEXIBLE PAVEMENTS

6.3.1 Effect of SM_r on Total Rutting

The predicted total rutting values using the inputs provided in Table 5.3 and Table 5.4 are shown in Figure 6.21 for RAP and Figure 6.22 for RCA in flexible pavements. Results showed that the SM_r values of RAP and RCA used as base layers had an effect on the total rutting of flexible pavements. It was observed that an increase in SM_r values of both RAP [ranging between 24,366 and 58,015 psi (168 and 400 MPa)] and RCA [ranging between 17,898 and 53,664 psi (123 and 370 MPa)] caused a similar rate of decrease in total rutting. Since none of the results exceeded the threshold total rutting value [0.75 in (20 mm)], it was concluded that acceptable RAP/RCA base layer performance in terms of total rutting was obtained using different SM_r values presented in the database.

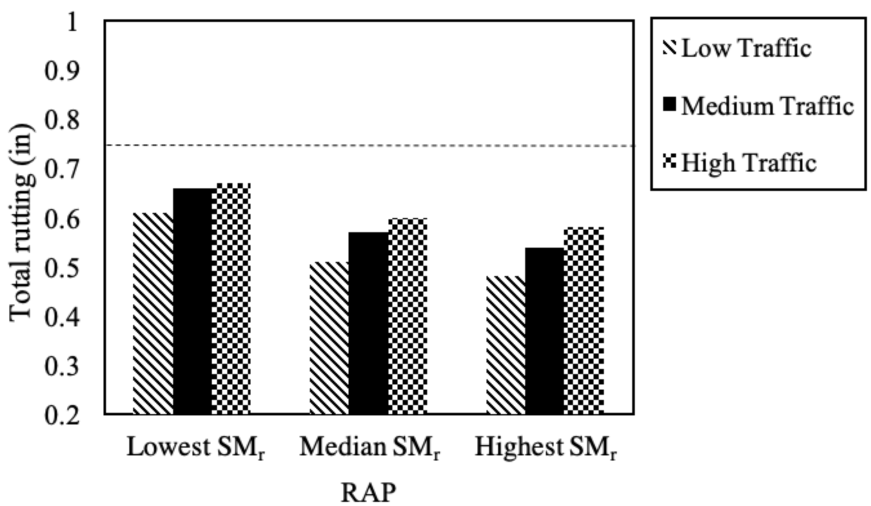

Figure 6.21. Total rutting vs. summary resilient modulus (SM_r) of recycled asphalt pavement (RAP)

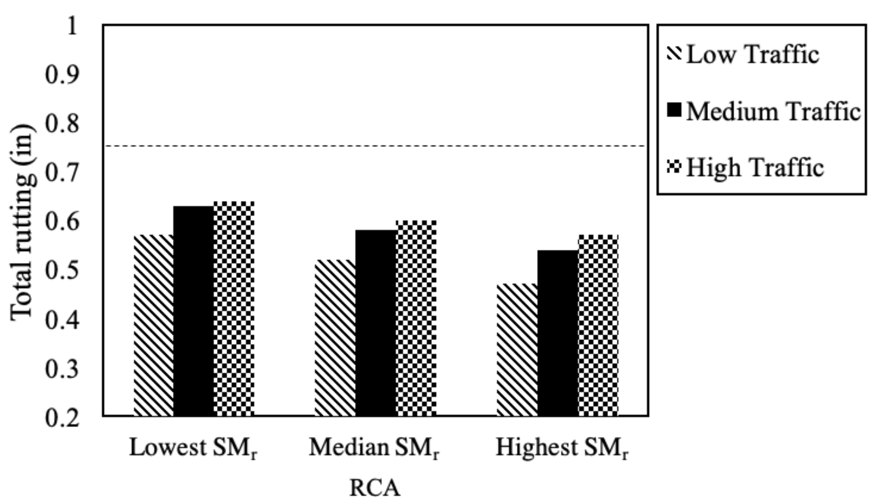

Figure 6.22. Total rutting vs. summary resilient modulus (SM_r) of recycled concrete aggregate (RCA)

6.3.2 Effect of Fines Content on Total Rutting

The predicted total rutting values using the inputs provided in Table 5.5 and Table 5.6 are shown in Figure 6.23 for RAP and Figure 6.24 for RCA in flexible pavements. Results showed that higher fines content of RAP (ranging between 0 and 11%) and RCA (ranging between 0.1 and 15%) used as base layers yielded higher total rutting in flexible pavements. This indicates that extra attention should be paid to the fines contents of these materials even though none of the cases exceeded the threshold total rutting value [0.75 in (20 mm)].

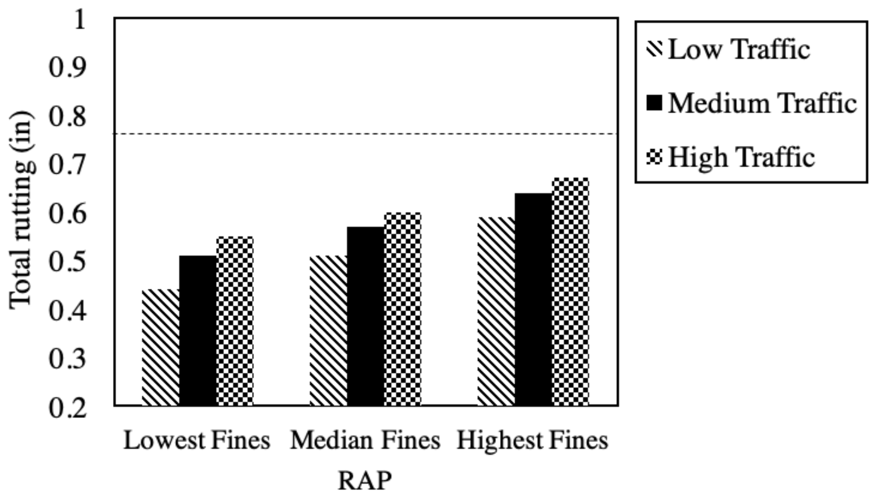

Figure 6.23. Total rutting vs. fines content of recycled asphalt pavement (RAP)

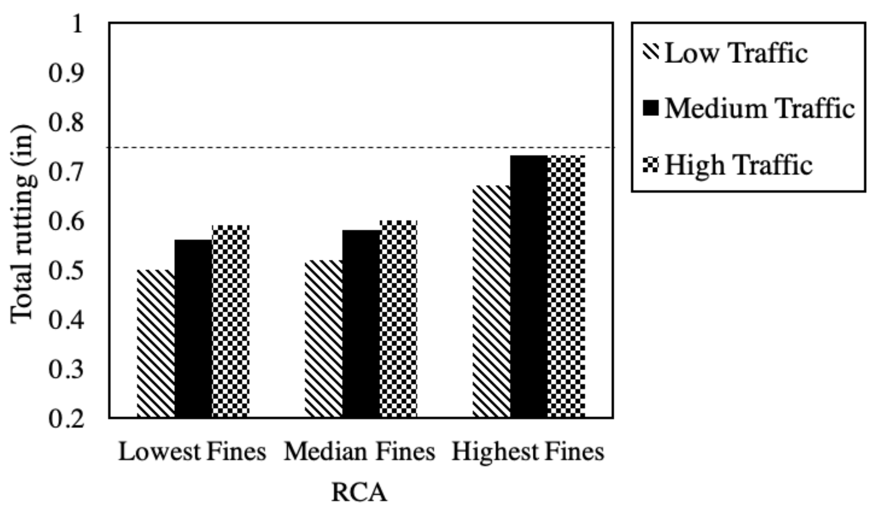

Figure 6.24. Total rutting vs. fines content of recycled concrete aggregate (RCA)

6.3.3 Effect of Gravel Content on Total Rutting

The predicted total rutting values using the inputs provided in Table 5.7 and Table 5.8 are shown in Figure 6.25 for RAP and Figure 6.26 for RCA in flexible pavements. Results showed that higher gravel content of RAP (ranging between 3 and 68.1%) and RCA (ranging between 31.8 and 94.1%) used as base layers yielded higher total rutting. This indicates that extra attention should be paid to the gravel contents of these materials even though none of the cases exceeded the threshold total rutting value [0.75 in (20 mm)].

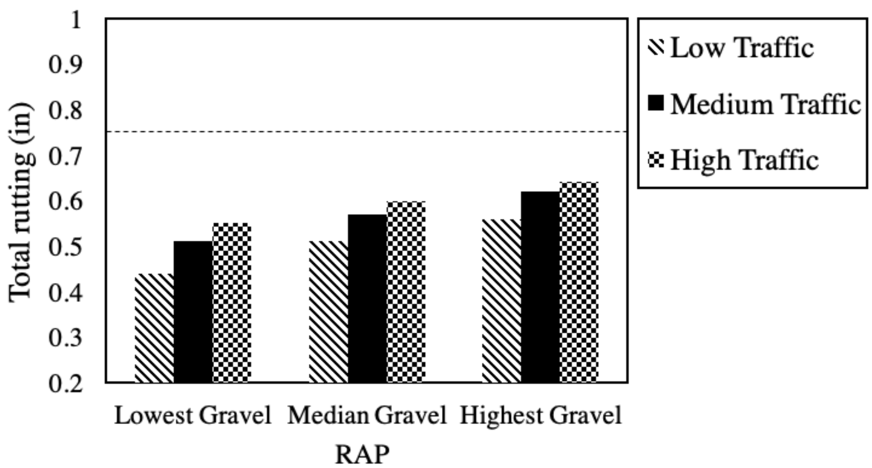

Figure 6.25. Total rutting vs. gravel content of recycled asphalt pavement (RAP)

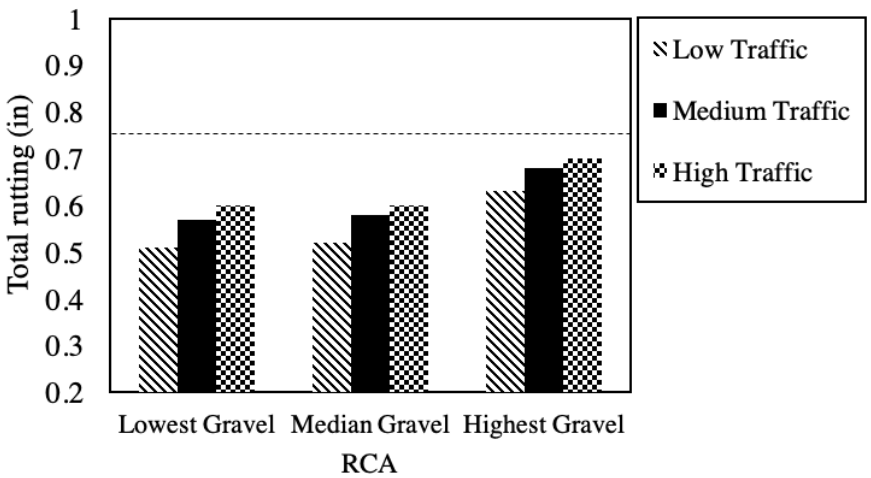

Figure 6.26. Total rutting vs. gravel content of recycled concrete aggregate (RCA)

6.3.4 Effect of Sand Content on Total Rutting

The predicted total rutting values using the inputs provided in Table 5.9 and Table 5.10 are shown in Figure 6.27 for RAP and Figure 6.28 for RCA in flexible pavements. Unlike the trends observed between the fines/gravel contents of RAP/RCA and total rutting, results showed that higher sand content of RAP (ranging between 28.1 and 97%) and RCA (ranging between 4.9 and 64.9%) used as base layers yielded lower total rutting. Moreover, none of the results exceeded the threshold total rutting value [0.75 in (20 mm)], indicating that acceptable RAP/RCA base layer performance in terms of total rutting was obtained using different sand contents presented in the database.

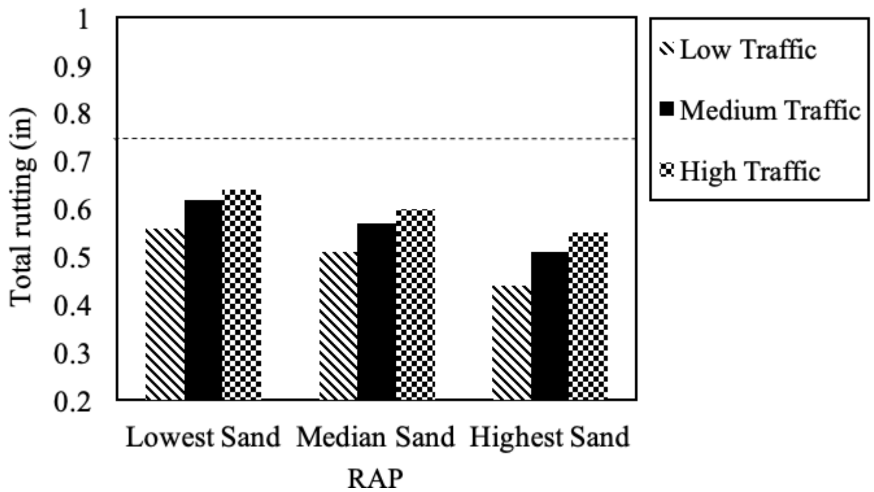

Figure 6.27. Total rutting vs. sand content of recycled asphalt pavement (RAP)

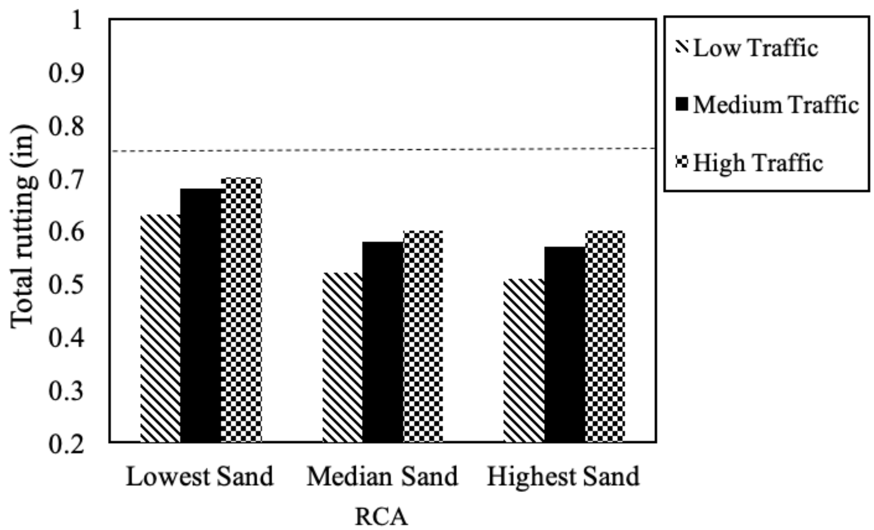

Figure 6.28. Total rutting vs. sand content of recycled concrete aggregate (RCA)

6.3.5 Effect of D_{60} on Total Rutting

The predicted total rutting values using the inputs provided in Table 5.11 and Table 5.12 are shown in Figure 6.29 for RAP and Figure 6.30 for RCA in flexible pavements. Based on the results, it was concluded that there was no clear trend between the D_{60} values of RAP and RCA used as base layers and the total rutting values. Moreover, none of the results exceeded the threshold total rutting value [0.75 in (20 mm)], indicating that acceptable RAP/RCA base layer performance in terms of total rutting was obtained using different D_{60} values presented in the database. As expected, higher traffic volume resulted in higher total rutting values.

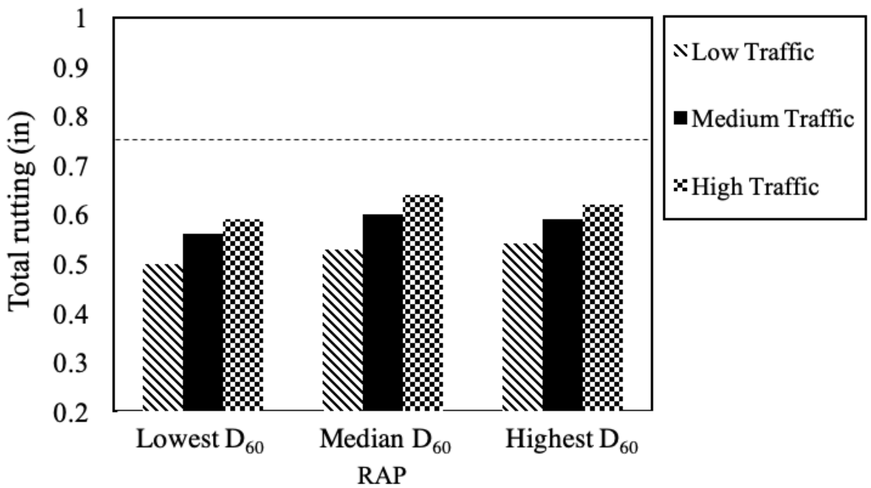

Figure 6.29. Total rutting vs. D_{60} (diameter at which 60% of the particles are finer) of recycled asphalt pavement (RAP)

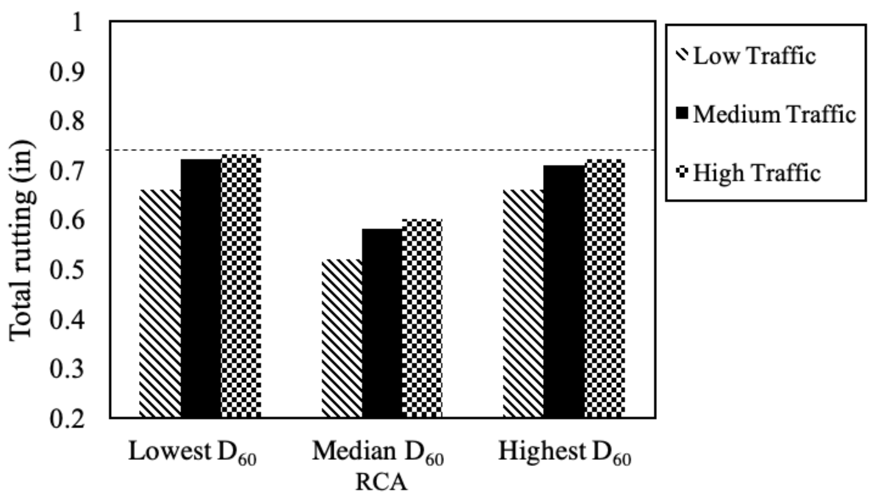

Figure 6.30. Total rutting vs. D_{60} (diameter at which 60% of the particles are finer) of recycled concrete aggregate (RCA)

6.4 MEAN JOINT FAULTING FOR RIGID PAVEMENTS

Transverse joint faulting is one of the main types of distresses in rigid pavements affecting their 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 excess moisture in a pavement system with an erodible base or fine-grained subgrade layer, repeated vehicle loadings may cause the mixture of water and fine 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 build-up beneath the approach corner and loss of support due to a void under the leave corner can result in significant faulting at the joint (especially for rigid pavements without dowels). As mentioned above, it is clear that the properties of base materials may have a great effect on the joint faulting of rigid pavements. Therefore, sensitivity analyses were conducted to determine whether the values available in the database can provide results that are under the threshold limits for joint faulting [0.12 in (3 mm)] for rigid pavement design analyses.

6.4.1 Effect of SM_r on Mean Joint Faulting

The predicted mean joint faulting values using the inputs provided in Table 5.3 and Table 5.4 are shown in Figure 6.31 for RAP and Figure 6.32 for RCA in rigid pavements. Results showed that there was no consistent trend between the SM_r values of RAP used as base layers and mean joint faulting (Figure 6.31). For RCA used as base layers, it was concluded that SM_r had minimal effect on mean joint faulting. As expected, higher traffic volume resulted in higher mean joint faulting values. For low traffic condition, none of the results exceeded the threshold value set for mean joint faulting [0.12 in (3 mm)]. Thus, acceptable RAP/RCA base layer performance in terms of mean joint faulting was obtained using different SM_r values presented in the database under low traffic. On the other hand, mean joint faulting under medium and high traffic volumes exceeded the threshold value [0.12 in (3 mm)].

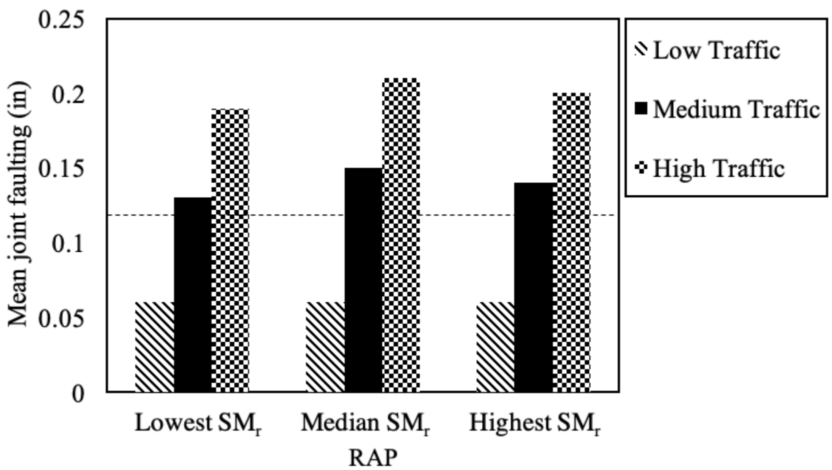

Figure 6.31. Mean joint faulting vs. summary resilient modulus (SM_r) of recycled asphalt pavement (RAP)

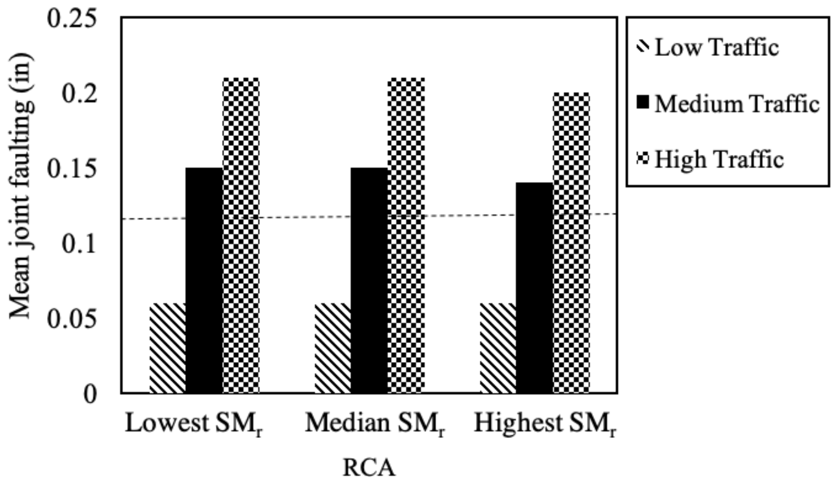

Figure 6.32. Mean joint faulting vs. summary resilient modulus (SM_r) of recycled concrete aggregate (RCA)

6.4.2 Effect of Fines Content on Mean Joint Faulting

The predicted mean joint faulting values using the inputs provided in Table 5.5 and Table 5.6 are shown in Figure 6.33 for RAP and Figure 6.34 for RCA in rigid pavements. Figure 6.33 shows that RAP with the highest fines content (11%) resulted in a slight decrease in mean joint faulting under medium and high traffic volumes. On the other hand, Figure 6.34 shows that joint faulting distresses increased slightly when the fines content value of RCA increased from the lowest (0.1%) to medium (3%). As expected, higher traffic volume resulted in higher mean joint faulting values. For low traffic condition, none of the results exceeded the threshold value set for mean joint faulting [0.12 in (3 mm)]. Thus, acceptable RAP/RCA base layer performance in terms of mean joint faulting was obtained using different fines content values presented in the database under low traffic. However, overall, mean joint faulting under medium and high traffic volumes exceeded the threshold value [0.12 in (3 mm)] (except for the case when RCA had the lowest fines content under medium traffic).

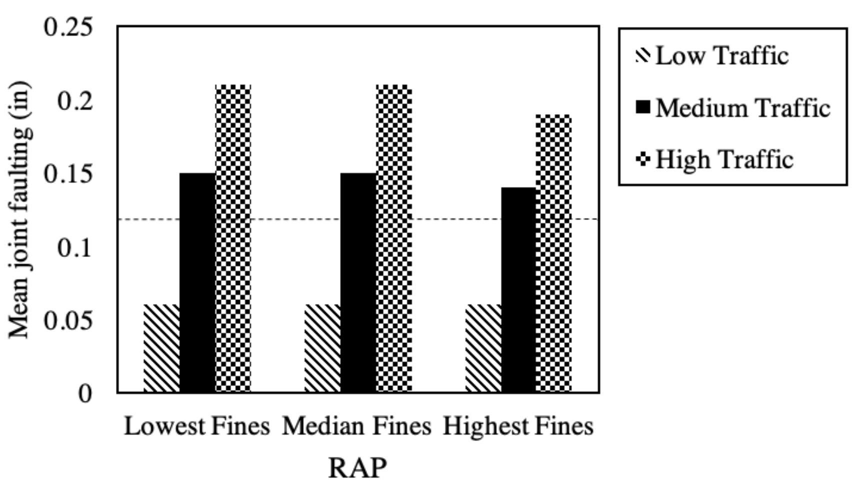

Figure 6.33. Mean joint faulting vs. fines content of recycled asphalt pavement (RAP)

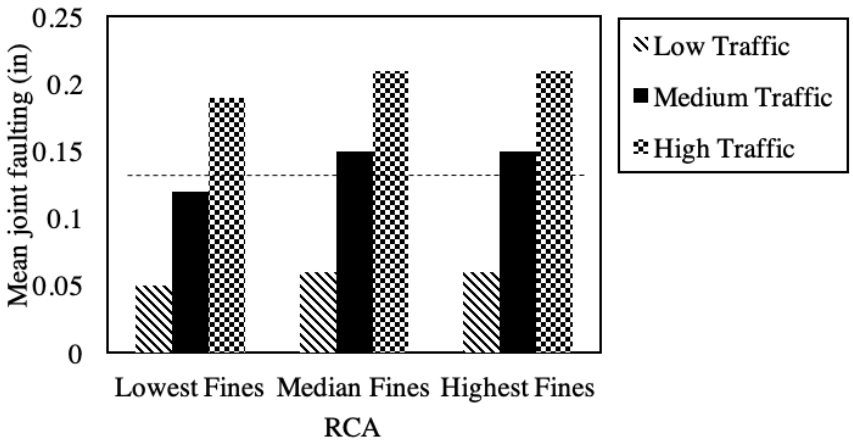

Figure 6.34. Mean joint faulting vs. fines content of recycled concrete aggregate (RCA)

6.4.3 Effect of Gravel Content on Mean Joint Faulting

The predicted mean joint faulting values using the inputs provided in Table 5.7 and Table 5.8 are shown in Figure 6.35 for RAP and Figure 6.36 for RCA in rigid pavements. Results showed that higher gravel contents of RAP (ranging between 3 and 68.1%) and RCA (ranging between 31.8 and 94.1%) used as base layers yielded slightly lower mean joint faulting values under all traffic conditions. As expected, higher traffic volume resulted in higher mean joint faulting values. For low traffic condition, none of the results exceeded the threshold value set for mean joint faulting [0.12 in (3 mm)]. Thus, acceptable RAP/RCA base layer performance in terms of mean joint faulting was obtained using different gravel content values presented in the database under low traffic. However, overall, mean joint faulting under medium and high traffic volumes exceeded the threshold value [0.12 in (3 mm)] (except for the case when RAP had the highest gravel content under medium traffic).

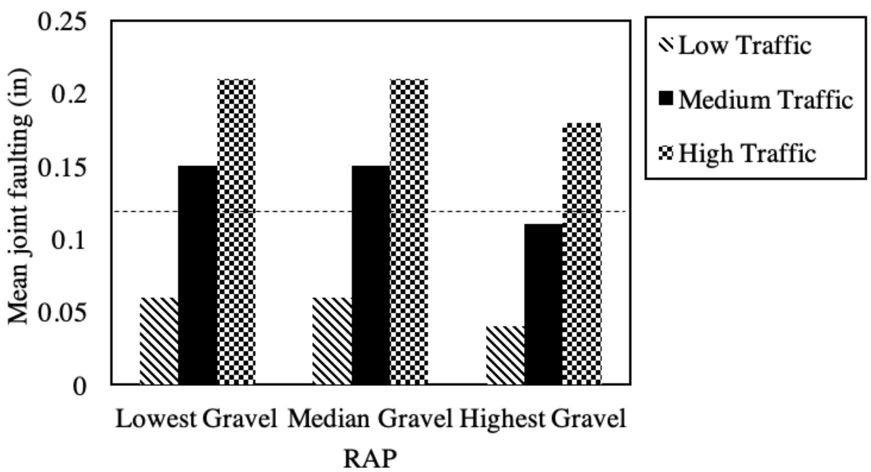

Figure 6.35. Mean joint faulting vs. gravel content of recycled asphalt pavement (RAP)

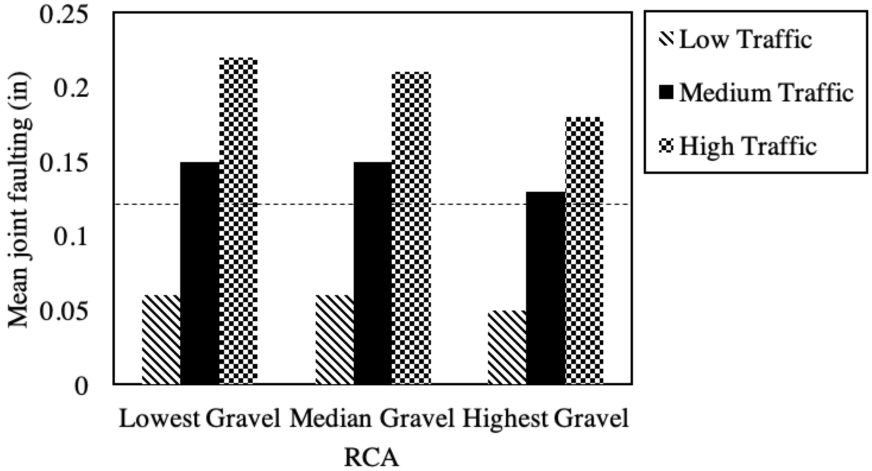

Figure 6.36. Mean joint faulting vs. gravel content of recycled concrete aggregate (RCA)

6.4.4 Effect of Sand Content on Mean Joint Faulting

The predicted mean joint faulting values using the inputs provided in Table 5.9 and Table 5.10 are shown in Figure 6.37 for RAP and Figure 6.38 for RCA in rigid pavements. For both RAP and RCA used as base layers, it was observed that an increase in sand contents (ranging between 28.1 and 97.0% for RAP and 4.90 and 64.9% for RCA) of these materials caused a consistent increase in mean joint faulting under all traffic conditions. As expected, higher traffic volume resulted in higher mean joint faulting values. For low traffic condition, none of the results exceeded the threshold value set for mean joint faulting [0.12 in (3 mm)]. Thus, acceptable RAP/RCA base layer performance in terms of mean joint faulting was obtained using different sand content values presented in the database under low traffic. However, overall, mean joint faulting under medium and high traffic volumes exceeded the threshold value [0.12 in (3 mm)] (except for the case when RAP had the lowest sand content under medium traffic).

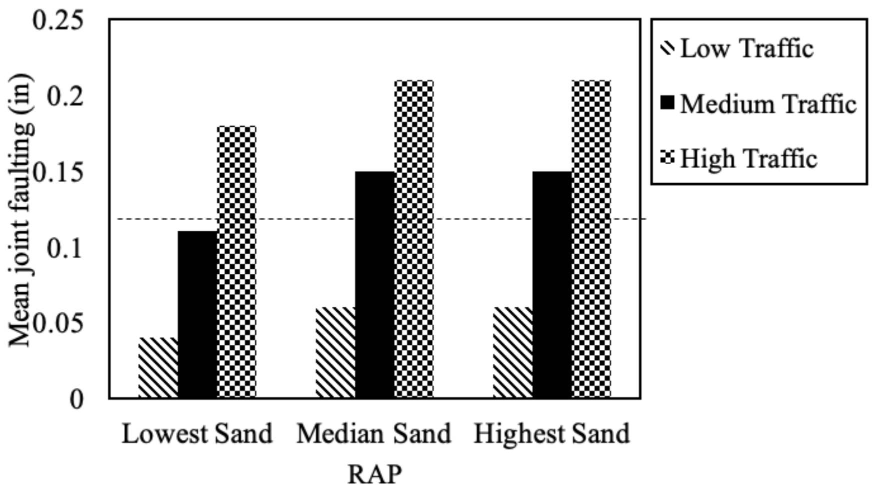

Figure 6.37. Mean joint faulting vs. sand content of recycled asphalt pavement (RAP)

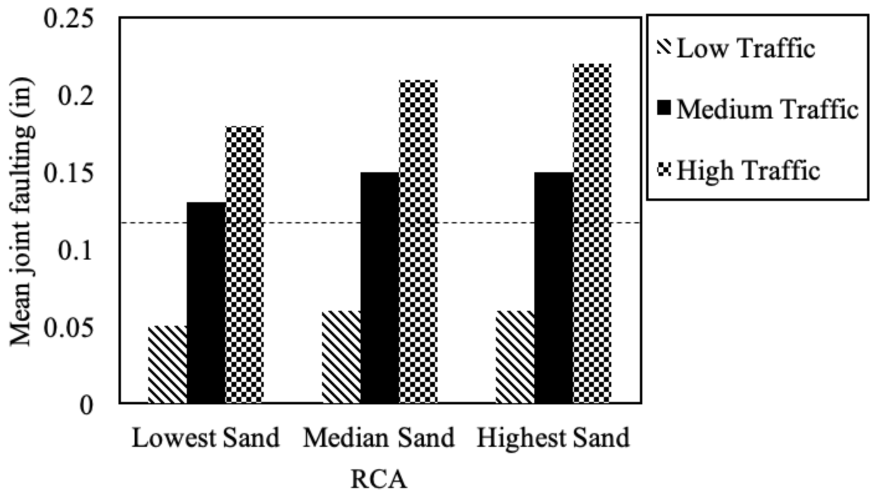

Figure 6.38. Mean joint faulting vs. sand content of recycled concrete aggregate (RCA)

6.4.5 Effect of D_{60} on Mean Joint Faulting

The predicted mean joint faulting values using the inputs provided in Table 5.11 and Table 5.12 are shown in Figure 6.39 for RAP and Figure 6.40 for RCA in rigid pavements. Both Figure 6.39 and Figure 6.40 show that mean joint faulting decreased slightly when the highest D_{60} values from the database were used for both RAP [0.409 in (10.4 mm)] and RCA [0.642 in (16.3 mm)] used as base layers. As expected, higher traffic volume resulted in higher mean joint faulting values. For low traffic condition, none of the results exceeded the threshold value set for mean joint faulting [0.12 in (3 mm)]. Thus, acceptable RAP/RCA base layer performance in terms of mean joint faulting was obtained using different D_{60} values presented in the database under low traffic. However, overall, mean joint faulting under medium and high traffic volumes exceeded the threshold value [0.12 in (3 mm)] (except for the case when RCA had the highest D_{60} value under medium traffic).

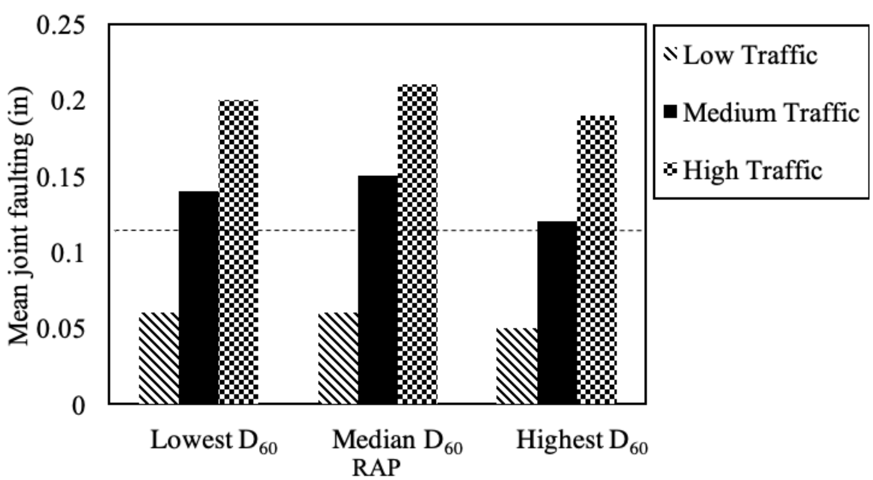

Figure 6.39. Mean joint faulting vs. D_{60} (diameter at which 60% of the particles are finer) of recycled asphalt pavement (RAP)

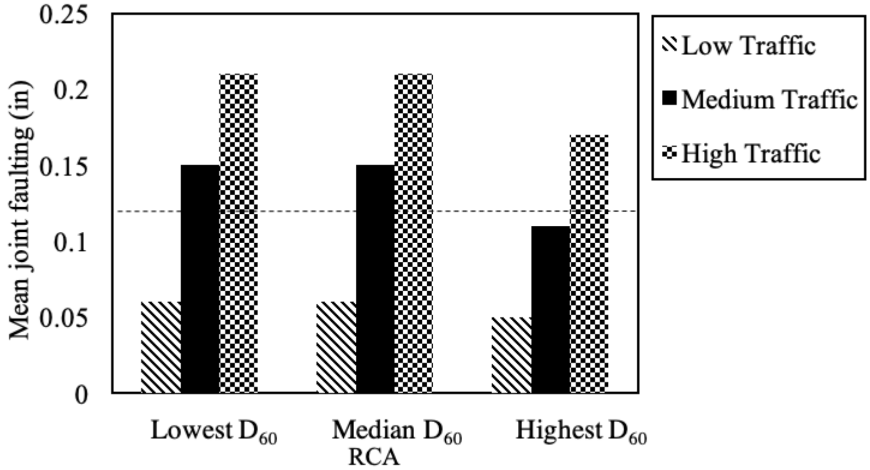

Figure 6.40. Mean joint faulting vs. D_{60} (diameter at which 60% of the particles are finer) of recycled concrete aggregate (RCA)

CHAPTER 7: CONCLUSIONS AND RECOMMENDATIONS

In this project, an extensive literature review was conducted on recycled asphalt pavement (RAP) and recycled concrete aggregate (RCA) used in aggregate base/subbase layers to create a database for inputs that can be used in pavement design efforts (database is summarized in Table 7.1). In addition, data for RAP-virgin aggregate (VA) and RCA-VA blends were collected from the literature as well. The relationships between summary resilient modulus (SM_r), California Bearing Ratio (CBR), saturated hydraulic conductivity (K_{sat}), permanent deformation vs. index properties of these materials were investigated. The AASHTOWare Pavement Mechanistic-Empirical (ME) Design (PMED) software was used to conduct sensitivity analyses to determine how the material properties of 100% RAP and RCA affect pavement performance predictions for both flexible and rigid pavement systems. Based on the analyses of the dataset and the results of the AASHTOWare PMED analyses, the following conclusions are drawn:

- Gravel contents of RAP range from 3 to 68% with a median of 45%, while the gravel contents of RCA are between 32 and 94% with a median of 51%. Thus, it was concluded that RCA tends to be slightly coarser than RAP.
- Sand contents of RAP are between 28 and 97% with a median of 54%. In addition, sand contents of RCA range from 4.9 to 65% with a median of 46%.
- Fines contents of most RAP and RCA are below 12%. Fines contents of RAP range from 0 to 11% with a median of 1%. For RCA, fines contents are between 0.1 to 13% with a median of 2.8%.
- Specific gravity (G_s) values of RAP are between 2.19 and 2.87 with a median of 2.4. For RCA, these values are between 2.12 and 2.7 with a median of 2.39.
- Maximum dry unit weight (MDU) values of RAP range from 17.2 to 22.8 kN/m^3 (110 to 146 pcf) with a median of 19.6 kN/m^3 (126 pcf). MDU values of RCA are between 18.3 and 21.7 kN/m^3 (118 and 139 pcf) with a median of 19.7 kN/m^3 (127 pcf).
- Optimum moisture content (OMC) values of RAP are between 4 and 10.7%. These values range from 6.1 to 14.8% for RCA.
- Summary M_r (SM_r) values of RAP range from 168 to 400 MPa (24,366 to 58,015 psi) with a median of 262 MPa (37,927 psi). Summary M_r (SM_r) values of RCA are between 123 and 370 MPa (17,897 and 53,664 psi) with a median value of 183 MPa (26,542 psi).
- Permanent deformation values of RAP range from 1.05 to 5.63%. For RCA, these values are between 0.1 and 0.83%. RCA tends to show lower permanent deformation than RAP and VA, while RAP are prone to showing the highest permanent deformation.
- California bearing ratio (CBR) values of RAP are between 18 and 68 with a median of 28, while CBR values of RCA range from 58 to 169 with a median of 146.
- For RAP, the angle of friction (ϕ) values are between 44 and 52°, and cohesion values are between 0 and 131 kPa. For RCA, ϕ values range from 19 to 52.7°, and cohesion values range from 24 to 191 kPa.
- For RAP and RCA, there was no specific trend between CBR values and gravel-to-sand (G/S) ratios. In addition, no significant relationship could be found between CBR and fines content.

- Saturated hydraulic conductivity (K_{sat}) values of RAP are between 1.8x10^{-7} and 1.14x10^{-3} m/s (2.12x10^{-3} and 13.46 ft/hr) with a median of 6.89x10^{-5} m/s (8.14x10^{-1} ft/hr). For RCA, K_{sat} values from 1.05x10^{-6} to 1.2x10^{-3} m/s (1.24x10^{-2} to 14.17 ft/hr) with a median of 1.7x10^{-5} m/s (2x10^{-1} ft/hr).
- It was observed that higher D_{10} (effective diameter) values result in higher K_{sat} values, and higher fines contents yielded smaller K_{sat} values for RAP and RCA.
- Summary M_r (SM_r) values of RAP and RCA increased with a higher G/S ratio, while fines contents of the materials had no effect on their SM_r values.
- There was an increasing trend in SM_r values of RAP with higher D_{30} (diameter at which 30% of the particles are finer) and D_{60} (diameter at which 60% of the particles are finer) values.
- RAP with higher values of C_c (coefficient of curvature) and C_u (coefficient of uniformity) yielded higher SM_r values. In addition, RCA with higher C_u values were prone to exhibiting higher SM_r values.
- As the temperature increased, SM_r values of RAP decreased, except for when thermal preloading is applied. On the other hand, SM_r values of RCA were independent of both compaction and testing temperature conditions.
- Summary M_r (SM_r) values of RAP decreased as their OMC values increased.
- Higher permanent deformation values of VA were observed with the addition of RAP to VA, which needs to be considered when designing a pavement system with RAP.
- As RCA contents increased in RCA-VA blends, OMC values increased as well, while MDU values of the blends decreased.
- According to the results of AASHTOWare PMED analyses, SM_r values of base layer aggregate had the greatest influence on the pavement performance among other material inputs.
- Performance prediction models revealed that greater pavement distresses would be expected with an increase in the annual average daily truck traffic (AADTT).
- There was an increasing trend in total rutting with higher fines contents of RAP and RCA.
- There was a decreasing trend in total rutting with higher SMr values for both RAP and RCA.
- As the sand contents of RAP and RCA increased, the total rutting of pavement models decreased.
- While fines content increased, the international roughness index (IRI) increased in both RAP and RCA in flexible pavements.
- No trend was observed between D_{60} values with total rutting for both RAP and RCA. In addition, no trend was observed between D_{60} values and IRI in RAP and RCA in flexible and rigid pavements.
- Mean joint faulting and IRI controlled the rigid pavement design located in high traffic and some cases medium traffic volume as they always fail in all cases for both RAP and RCA.
- In rigid pavements, all cases with RAP and RCA as base course materials in low traffic volume satisfied the minimum required IRI and mean joint faulting distresses.
- All cases in flexible pavement met the failure criteria in terms of IRI and total rutting. However, in some cases, such as high fines content, low sand content, and high gravel content for RCA under medium and high traffic volumes, they came close to the threshold values defined.
- Overall, it can be suggested that flexible pavements with 20 years of design life can provide adequate performance under any traffic conditions. However, it is recommended to determine the gradation and Mr of the base course materials as they are prone to having major effects on total rutting and IRI.

Table 7.1. Summary of database for recycled asphalt pavement (RAP) and recycled concrete aggregate (RCA)

Properties	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} (3.9×10^{-3}) [*30*]	5×10^{-1} (1.96×10^{-2}) [*30*]	1 (3.93×10^{-2}) [*30*]	10^{-1} (3.9×10^{-3}) [*19*]	2.3×10^{-1} (9×10^{-3}) [*19*]	4.3×10^{-1} (1.7×10^{-2}) [*19*]
D_{30}, mm (in)	8×10^{-2} (3.1×10^{-3}) [*27*]	1.5 (6×10^{-2}) [*27*]	4.9 (1.9×10^{-1}) [*27*]	2×10^{-1} (7.9×10^{-3}) [*17*]	1.2 (4.72×10^{-2}) [*17*]	6.5 (2.56×10^{-1}) [*17*]
D_{60}, mm (in)	1.5×10^{-1} (5.9×10^{-3}) [*27*]	4.82 (1.89×10^{-1}) [*27*]	10.4 (4.09×10^{-1}) [*27*]	6×10^{-1} (2.36×10^{-2}) [*17*]	6.8 (2.67×10^{-1}) [*17*]	16.3 (6.42×10^{-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*]	2.4 [*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 (24,366) [*32*]	262 (37,927) [*32*]	400 (58,015) [*32*]	123.4 (17,897) [*18*]	183 (26,541) [*18*]	370 (53,664) [*18*]
CBR (%)	18 [*12*]	28 [*12*]	68 [*12*]	58 [*4*]	146 [*4*]	169 [*4*]
K_{sat}, m/s (ft/hr)	1.8×10^{-7} (2.12×10^{-3}) [*23*]	6.89×10^{-5} (8.14×10^{-1}) [*23*]	1.14×10^{-3} (1.35×10) [*23*]	1.05×10^{-6} (1.24×10^{-2}) [*12*]	1.7×10^{-5} (2.01×10^{-1}) [*12*]	1.2×10^{-3} (1.42×10) [*12*]

Italic numbers provided in square brackets represent the corresponding sample size. D_{10} = diameter at which 10% of the particles are finer – effective diameter; D_{30} = diameter at which 30% of the particles are finer; D_{60} = diameter at which 60% of the particles are finer; C_u = coefficient of uniformity; C_c = coefficient of curvature; G_s = specific gravity; MDU = maximum dry unit weight; OMC = optimum moisture content; SM_r = summary resilient modulus; CBR = California bearing ratio; K_{sat} = saturated hydraulic conductivity.

REFERENCES

AASHTO (1993). Guide for Design of Pavement Structures, 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

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, 04016067.

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.

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, Engineering Research Journal 132, C1-C13.

FDOT (2018). Standard specifications for road and bridge construction. Florida Department of Transportation, Tallahassee, FL.

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

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. State of the Practice National Review, Federal Highway Administration (FHWA), U.S. Department of Transportation, Washington, D.C.

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.

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.

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, 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. and Tutumluer, E. (2015). Evaluation of aggregate subgrade materials used as pavement subgrade/granular subbase (No. FHWA-ICT-15-013). Illinois Department of Transportation, Bureau of Materials & Physical Research, Springfield, IL.

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.

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.

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.

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.

MDOT (2012). Standard Specifications for Construction. Michigan Department of Transportation, Lansing, MI.

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 (2005). Standard Specifications for Construction (2005 Edition). Minnesota Department of Transportation, St. Paul, MN.

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, 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.

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. W., Li, R., Kim, S., Ceylan, H., & Gopalakrishnan, K. (2011). Sensitivity Evaluation of MEPDG Performance Prediction (No. NCHRP Project 1-47). Transportation Research Board of the National Academies, Washington, D.C.

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 (private communication, August 14, 2020)

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, Report No. TNW2011-15. U.S. Department of Transportation, University Transportation Centers Program.

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.

APPENDIX A
GRADATION

Source	Location	Material	Gravel (%)	Sand (%)	Fines (%)	Classification USCS	Classification AASHTO	D_{10} (mm)	D_{30} (mm)	D_{60} (mm)	C_u	C_c	G_s
Edil et al. (2012a)	MN	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
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

Locander (2009) - cont'd	CO	RAP	75	24.1	0.9	GW-GP							2.29
Mokwa and Peebles (2005)	MT	CBC#1 unmixed	52.5	41.6	6.0	GW-GP	A-1-a (6A)						2.67
		CBC#1 20%RAP	55.0	42.4	1.8	GW-GP	A-1-a(5A)						2.67
		CBC#1 50%RAP	49.3	49.0	1.7	SW-SP	A-1-a(5A)						2.59
		CBC#2 unmixed	55.8	41.6	2.6	GW-GP	A-1-a (6A)						2.7
		CBC#2 20%RAP	54.4	43.6	2.1	GW-GP	A-1-a (6A)						2.66
		CBC#2 50%RAP	53.7	42.4	1.7	GW-GP	A-1-a(5A)						2.59
		CBC#3 unmixed	55.5	39.4	5.2	GW-GP	A-1-a(5A)						2.68
		CBC#3 20%RAP	52.3	45.7	2.0	GW-GP	A-1-a(5A)						2.66
		CBC#3 50%RAP	58.5	40.1	1.4	GW-GP	A-1-a(5A)						2.59
		Pitrun unmixed	41.8	40.7	1.1	SP	Spec. Borrow	0.4	1.6	17	42.5	0.37	2.72
		Pitrun 20%RAP	57.7	38.2	1.6	GP	Spec. Borrow	0.4	2	15	37.5	0.66	2.63
		Pitrun 50%RAP	53.1	38.0	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							
		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

Ullah and Tanyu (2019) - cont'd	VA	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		
		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
		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	
		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	4.5	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
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
Camargo et al. (2013)	WI	RPM	46	43	11	GW-GM							
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	
		50% RAP TH 10 +50% Class 5 lab	41.5	56.9	1.7	SP	A-1-b	0.32	0.95	5	15.6	0.56	
		75% RAP TH 10+25% Class 5 lab	46.3	52.7	1.0	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	0.13	0.89	5.08			2.47
		RAP 2	45	54	1	SW	A-1-a	0.51	1.65	4.83			2.47

Guthrie et al. (2007) -	UT	Base1	55	35.5	9.5	GWGM	A-1-a	0.08	1.02	9.65			2.64
		Base2	44	46.5	9.5	SP-SM	A-1-a	0.08	1.27	4.83			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						
		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	NJ	RAP	49	50.9	0.1	SW		0.52	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	
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		0.08	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
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

Chen et al. (2013) - cont'd	WI Stockpile	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	
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	2.51
		APAC Melbourne Milled	41.9	57.6	0.5	SW	A-1-a	0.5	2	5	9.6	1.9	2.52
		Whitehurst Gainesville Milled	54	45.6	0.4	SP	A-1-a	0.4	1.5	4.8	11.2	0.8	2.58
		APAC Jacksonville Crushed	26.6	66.6	6.8	SP	A-1-b	0.1	0.3	3	26.2	0.4	2.60
		75% milled mel and LR	43	56	1			0.39	2	5			
		50% milled Melbourne+ 50% LR LimeRock	45	53	2			0.3	1.8	6			
		25% milled Melbourne+ 75% LR 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			

Kim and Labuz (2007) - cont'd	MN	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	1	SW		0.8	2.5	8			2.34
		CO RAP	31	68.3	0.7	SP		0.4	0.9	3.1			2.23

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
RESILIENT MODULUS (M_R)

Source	Location	Type of Material	Method	SM_r (MPa)
Edil et al. (2012a)	MN	Aggregate class 5	Power function	152
			NCHRP Model	144
		Agg at 0 F-T cycle	Power function	191
		Agg at 5 F-T cycle		186
		Agg at 10 F-T cycle		177
		Agg at 20 F-T cycle		153
		Blend	Power function	182
			NCHRP Model	191
		RAP	Power function	180
			NCHRP Model	174
		RAP at 0 F-T cycle	Power function	238
		RAP at 5 F-T cycle		220
		RAP at 10 F-T cycle		200
		RAP at 20 F-T cycle		180
		RCA	Power function	189
			NCHRP Model	190
	MI	RCA	Power function	171
			NCHRP Model	171
		RCA at 0 F-T cycle	Power function	199
		RCA at 5 F-T cycle		191
		RCA at 10 F-T cycle		257
		RCA at 20 F-T cycle		268
		RPM	Power function	168
			NCHRP Model	161
	CO	RCA	Power function	175
			NCHRP Model	162
		RAP	Power function	184
			NCHRP Model	177
	CA	RCA	Power function	178
			NCHRP Model	166
		RCA at 0 F-T cycle	Power function	262
		RCA at 5 F-T cycle		227
		RCA at 10 F-T cycle		282
Edil et al. (2012a)	CA	RAP	Power function	173
			NCHRP Model	166
		RAP at 0 F-T cycle	Power function	256
		RAP at 5F-T cycle		249
		RAP at 10 F-T cycle	NCHRP Model	223

Edil et al. (2012a) - cont'd	CA	RAP at 20 F-T cycle	NCHRP Model	203
	TX	RCA	Power function	164
			NCHRP Model	151
		RCA at 0 F-T cycle	Power Model	258
		RCA at 5 F-T cycle		211
		RCA at 10 F-T cycle		236
		RCA at 20 F-T cycle		289
		RAP	Power function	198
			NCHRP Model	188
		RAP at 2% dry	Power function	341
		RAP at OMC		334
		RAP at 2% wet		317
		RAP at 0 F-T cycle	Power function	334
		RAP at 5		287
		RAP at 10		272
		RAP at 20		254
	OH	RCA	Power function	163
			NCHRP Model	158
		RCA at 2% dry	Power function	239
		RCA at OMC		222
		RCA at 2% wet		148
		RAP	Power function	197
			NCHRP Model	192
		RAP 2% dry	Power function	297
		RAP at OMC		287
		RAP at 2% wet		243
	NJ	RCA	Power function	208
			NCHRP Model	203
		RAP	Power function	209
			NCHRP Model	207
		RPM	Power function	264
			NCHRP Model	264
	WI	RAP	Power function	266
			NCHRP Model	274
Tutumluer et al. (2015)	IL	Blend	GeoGauge composite surface modulus	90
				137
			LWD	74.7
				80.4
				66.3

Tutumluer et al. (2015) - cont'd	IL	Blend	LWD	79.6
		RAP	GeoGauge composite surface modulus	115
				169
			LWD	99.9
				97.6
				64
				97.9
Locander (2009)	CO	Rap	MR AASHTO	239.6
			T274-82	211.8
				181.1
Bennert et al. (2000)	NJ	DGABC	AASHTO bulk stress model	139.2
		25% RAP		187.1
		50% RAP		215.1
		75% RAP		222
		100%RAP		300.3
		25% RCA		155.1
		50% RCA		248.4
		75% RCA		255.0
		100%RCA		297.6
Huang and Dong (2014)	TN	RAP	Universal model power law	286.5
		Limestone		185.1
Huang and Dong (2014)	TN	gravel		153.4
Ullah and Tanyu (2019)	VA	VA		141.1
		20%RAP1 Plagioclase and Pyroxene with high binder content		144.2
		30%RAP1		176.5
		40%RAP1		199.1
		50%RAP1		211.9
		60%RAP1		212.3
		20%RAP2 Plagioclase and Pyroxene with low binder		152.1
		30%RAP2		159.7
		40%RAP2		173.6
		50%RAP2		176.5

Ullah and Tanyu (2019) - cont'd	VA	60%RAP2		179.1
		20%RAP5 Plagioclase and Pyroxene with medium binder		153.9
		30%RAP5		172.1
		40%RAP5		188.8
		50%RAP5		197.6
		60%RAP5		200.7
Kim et al. (2005)	MN	100% CR 3 at OMC		170
		100% CR 3 at 65% OMC		225
		25% RAP at OMC		175
		25% RAP at 65% OMC		235
		50% RAP at OMC		175
		50% RAP at 65% OMC		225
		75% RAP at OMC		215
		75% RAP at 65% OMC		260
Edil et al.(2017)	MN	Natural aggregate at 15 cm depth 7 F-T cycle	FWD Modulus	127
		Natural aggregate at 30 cm depth 7 F-T cycle		125
		RCA at 15 cm depth 7 F-T cycle		160
		RCA at 30 cm depth 7 F-T cycle		160
		RCA blend at 15 cm depth 7 F-T cycle		160
		RCA blend at 30 cm depth 7 F-T cycle		150
		RAP at 15 cm depth 7 F-T cycle		208
		RAP at 30 cm depth 7 F-T cycle		200
Hasan et al.(2018)	NM	30% RAP with 6.3 MC		160
		30% RAP with 7.1 MC		170
		30% RAP with 5.7 MC		175
		30% RAP with 7.6 MC		155
		75% RAP at 7.1		290
Abdelrahman and Noureldin (2014)	MN	Class 5		137
		100% RAP		330
		75% RAP		262
		50% RAP		289
Wu et al.(2012)	WA	0% RAP	high cyclic stress (cyclic stress/sigma 3 = 7)	177
		20% RAP		195
		40% RAP		197
		60% RAP		205

Wu et al.(2012) - cont'd	WA	80% RAP		550
Puppala et al. (2012)	TX	RAP		251
Soleimanbeigi and Edil (2015b)	WI	RAP at 5 C without thermal preloading		410
		RAP at 22 without thermal preloading		390
		RAP at 35 without thermal preloading		285
		RAP at 50 without thermal preloading		280
		RAP at 5C with thermal preloading		305
		RAP at 35 with thermal preloading		410
		RAP at 50 with thermal preloading		490
Edil et al. (2012c)	MN	RPM	NCHRP 1-28A	257
Edil et al.(2012b)	WI	RPM	Moosazedh and Witczak	310
Soleimanbeigi et al.(2015)	NJ	RCA	RCA at 7 °C	167
			RCA at 23 °C	160
			RCA at 35 °C	157
			RCA at 50 °C	190
	TX	RCA	RCA at 7 °C	210
			RCA at 23 °C	188
			RCA at 35 °C	180
			RCA at 50 °C	190
	CO	RCA	RCA at 7 °C	199
			RCA at 23 °C	229
			RCA at 35 °C	219
			RCA at 50 °C	190
	CO	RAP	RAP at 7 °C	225
			RAP at 23 °C	255
			RAP at 35 °C	170
			RAP at 50 °C	200
	TX	RAP	RAP at 7 °C	355

Soleimanbeigi et al.(2015) - cont'd	TX	RAP	RAP at 23 °C	345
			RAP at 35 °C	220
			RAP at 50 °C	190
	NJ	RAP	RAP at 7 °C	240
			RAP at 23 °C	280
			RAP at 35 °C	260
			RAP at 50 °C	270
Camargo et al. (2013)	WI	RPM	Moosazedh and Witczak	309
Kang et al. (2011)	MN	25%RAP		90.2
		50% RAP		137.1
		75%RAP		162.6
		100%RAP		192.8
		25%RCM		224.2
		50%RCM		313.9
		75%RCM		260.8
		100%RCM		119.6
Attia and Abdelrahman (2010a)	MN	RAP Trunk highway 10	OMC	380
			OMC + 1%	239
			OMC - 1%	482
			OMC - 3%	750
		RAP TH 19-MM 104 field 50-50	OMC	227
			OMC + 2%	182
			OMC - 2%	460
		50% RAP TH 10 +50% Class 5 lab	OMC	275.8
			OMC + 1%	224
			OMC - 1%	441.2
			OMC - 3%	572
Bradshaw et al. (2016)	RI	RAP1, 23%		220
		RAP2, 14%		200
		RAP3, 23%		210
		RAP 3R, 25%		260
		RAP4, 26%		186
Bradshaw et al. (2016)	RI	Rap 5, 19%		240
		RAP 6, 39%		230

Attia and Abdelrahman (2010b)	MN	50% RAP sample 1 Ref case, 50% Class 5	Witczak model	247.8
			MEPDG model	247
		50% RAP sample 2, 50% Class 5	Witczak model	302.7
			MEPDG model	310.3
		75% RAP sample 1 Ref case, 25% Class 5	Witczak model	217.9
			MEPDG model	227.5
		75% RAP sample 2, 25% Class 5	Witczak model	259.9
			MEPDG model	265
		100% RAP sample 1, ref case	Witczak model	334.4
			MEPDG model	340
		100% RAP sample 2	Witczak model	414.4
			MEPDG model	420
Alam et al. (2010)	MN	Class 6		135.6
		RAP 30%		154.9
		RAP 50%		192.1
		RAP 70%		221.9
		RAP 100%		271.3
Attia and Abdelrahman (2011)	MN	RAP TH 10 (100%)		400
		50% RAP TH10 50% Class 5		265
		75% RAP TH10 25% Class 5		210
		RAP TH 19 - MM 104 (50% RAP 50% field agg)		240
Bennert and Maher (2005)	NJ	100% RAP		268
		75% RAP		213.8
		50% RAP		233.7
		25% RAP		201.5
		100% RCA		272.9
		75% RCA		239.5
		50% RCA		224.4
		25% RCA		155.1
Bestgen et al. (2016)	Eastern USA	G1	power model	210
		G2		114
		G3		91
		G4		123
		RCA 1		295
		RCA 2		220
		25R175G1		160
		50R150G1		130
		75R125G1		280

Bestgen et al. (2016) - cont'd	Eastern USA	25R175G2	power model	140
		50R150G2		150
		75R125G2		260
		25R275G1		70
		50R250G1		120
		75R225G1		150
		25R275G2		340
		50R250G2		280
		75R225G2		120
		25R175G3		87
		50R150G3		98
		75R125G3		93
		25R175G4		114
		50R150G4		108
		75R125G4		121
Tutumluer et al. (2012)	IL	RCA	power	188
		75% RCA		145
		50% RCA		157
Diagne et al. (2015)	WI	RCA 0 F-T cycle		370
		RCA 5 F-T cycle		297
		RCA 10 F-T cycle		288
Cetin et al. (2020)	MN	Coarse RCA	MEPDG model	127.4
				122.6
		Fine RCA		123.4
				121.5
		RCA+RAP		114.9
				112.4
Cosentino et al. (2003)	FL	100% RAP modified	mixed with processed organic soil	291.4
		80%- fine sand		261.6
		60%		176.5
Kim and Labuz (2007)	MN	25% RAP from CR 3	100% OMC = 8.7%	175
		50% RAP from CR 3	100% OMC = 8%	190
		75% RAP from CR 3	OMC = 7.2%	230
Bejarano (2001)	CA	RAP	95% maximum wet density	310
			100% maximum wet density	450
Garg and Thompson (1996)	IL	RAP		218.6

APPENDIX C
DATABASE FOR RAP

Source	Location	Gravel (%)	Sand (%)	Fines (%)	D_{10} (mm)	D_{30} (mm)	D_{60} (mm)	SM_r (MPa)	CBR	Unit Weight (kN/m^3)	OMC (%)	K_{sat} (m/s)
Edil et al. (2012a)	MN	26.3	71.2	2.5	0.3	0.7	2.3	180		20.8	6.7	1.10E-06
	MI	49.3	50.4	0.4	0.4	1.7	6.5	168		21.5	5.2	2.31E-04
	CO	31.7	67.7	0.7	0.4	0.9	3.3	184		20.7	5.7	3.82E-05
	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	3.18E-05
	OH	32.1	66.2	1.7	0.5	1.6	3.8	197		19.8	8.8	5.03E-05
	NJ	50.9	48.4	0.7	1	2.8	5.9	209		20.4	6.5	3.69E-04
	WI	30.9	68.5	0.5	0.6	1.4	3.6	266		20	7.3	5.19E-05
Edil et al. (2012b)	WI	46	43	11				310		21.2	7.5	
Edil et al. (2012c)	MN	40	52	8				257	19	20.0	4.9	
Locander (2009)	CO	64	35.1	0.9				239.6		19.4	7.2	7.00E-04
		59	40.1	0.9				211.8		19	10.7	7.40E-04
		59	40	1				181.1		18.8	8.8	7.30E-04
Bennert et al. (2000)	NJ	60	59	1	1	3.1	8	300.3		18.4	5	
Huang and Dong (2014)	TN	41	58	1				286.5		18.7	8.0	
Puppala et al. (2012)	TX	48	48	4				251		21.3	6	
Soleimanbeigi and Edil (2015)	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	

Attia and Abdelrahman (2010a)	MN	51	48.6	0.4	0.6	2	7	380		20.8	5.5	
Attia and Abdelrahman (2010b)	MN	51	48.6	0.4	0.6	2	7	380		20.8	5.5	
Bennert and Maher (2005)	NJ	49	50.9	0.1	0.516	0.08	0.15	268	18			4.87E-05
Wu et al. (2012)	WA	67	32	1	0.45	4.9	10.4	200				
Guthrie et al. (2007)	UT	45	46.5	8.5	0.13	0.89	5.08		21	20.3	5.6	
	UT	45	54	1	0.51	1.65	4.83		22	18.2	5.8	
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				
Cosentino et al. (2003)	FL	40	59.1	0.9	0.27	0.65	4.7	291.4	32	18.5	8.2	2.00E-06
Bejarano (2001)	CA	54	45	1				310		24.1	5.5	
Garg and Thompson (1996)	IL	68.1	28.1	3.8				218.6		21.0	6	
Mijic et al. (2019)	MD	46.3	51.8	1.8						19.6	5.7	9.83E-05
	MD	37.8	61.3	0.9						18.5	6.8	5.66E-04
	MD	45.7	54.1	0.1						17.2	6.3	1.14E-03
	MD	40.7	59	0.3						18.7	6.8	2.51E-04
	MD	44	54.8	1.2						19.2	7.5	6.89E-05
	MD	45.3	54.2	0.5						19.1	6.4	2.01E-04
	MD	47.6	52	0.4						18.5	8.2	5.27E-04
Ullah and Tanyu (2019)	VA	46	53	1	0.5	2	5.1					
	VA	39	60	1	0.5	1.5	4.5					
	VA	26	73	1	0.32	1.1	3					
	VA	42	57	1	0.5	1.7	5					
Ullah et al. (2018)	VA	45	53.5	1.5						19	5.5	
	VA	40	57.8	2.2						19.5	5.5	

Edil et al. (2017)	MN	26.3	71.2	2.5								
Ba et al. 2012	TX	54	45.0	1.0	0.8	2.5	8					
	CO	31	68.3	0.7	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	19.2	5	
	APAC Melbourne Milled	41.9	57.6	0.5	0.5	2	5		60	19.0	6.2	3.10E-05
	Whitehurst Gainesville Milled	54	45.6	0.4	0.4	1.5	4.8		60	19.1	4	1.30E-06
	APAC Jacksonville Crushed	26.6	66.6	6.8	0.1	0.3	3		68	19.6	4.5	1.80E-07
Kang et al. (2011)	MN							193		20.8	4	2.22E-05
Abdelrahman and Noureldin (2014)	MN							330		20.8	5.5	
Attia and Abdelrahman (2011)	MN							400				

APPENDIX D
DATABASE FOR RCA

Source	Location	Gravel (%)	Sand (%)	Fine (%)	D_{10} (mm)	D_{30} (mm)	D_{60} (mm)	SM_r (MPa)	CBR	Unit Weight (kN/m^3)	OMC (%)	K_{sat} (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		19.5	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	48	51	1	0.4	0.8	7	164.8		19.0	9.4	1.02E-05
Bennert and Maher (2005)	NJ	71	26.2	2.8	0.29	0.2	0.6	272.9	169			1.05E-06
Bestgen et al. (2016)	Eastern USA	45	45	10	0.11	0.6	6.5	295	148	20.2	9.5	
	Eastern USA	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	3.00E-05
Cetin et al. (2020)	MN	61.7	34.9	3.4				127.4		19.31	11.3	2.67E-06
	MN	38.3	54.6	7.1				123.4		19.1	11.1	4.85E-06

Mahedi and Cetin (2020)	TX	93.4	5.8	0.8						19	10.9	
	TX	68.8	31.1	0.1						19.7	14.4	
	IA	48.8	51.1	0.1						19	14.8	
	IA	82	17.8	0.2						18.4	14.3	
	MN	94.1	4.9	1						18.3	12.6	
Natarajan et al. (2019)	MN									19.5	11.2	
	MN	55	43	2	0.4	1.9	8			21.4	12	
	MN	37	61	2	0.35	0.8	4			21	11.7	
	MN	52	46	2	0.32	1.4	8			21.7	13.5	
Chen et al. (2013)	CA	50	47	3						19.8	10.9	1.90E-05
	CO	41	44	15						18.9	11.9	1.60E-05
	MI	69	28	3						20.8	8.7	2.60E-05
	MN	32	64	4						19.5	11.2	1.80E-05
	TX	76	21	3						19.7	9.2	8.00E-06
	WI	48	50	2						19.4	10.8	1.20E-03
	WI	65	32	3						19.9	9.9	7.10E-04
Edil et al. (2017)	MN	31.8	64.9	3.3								