ABSTRACT

Title of Thesis:	ASSESSMENT AND LIFE-CYCLE ANALYSIS OF RECYCLED MATERIALS FOR SUSTAINABLE HIGHWAY
	Yating Zhang, Master of Science, 2016
Thesis Directed By:	Associate Professor Dimitrios, G. Goulias, Professor Ahmet, H. Aydilek, Department of Civil and Environmental Engineering

Recycled materials replacing part of virgin materials in highway applications has shown great benefits to the society and environment. Beneficial use of recycled materials can save landfill places, sparse natural resources, and energy consumed in milling and hauling virgin materials. Low price of recycled materials is favorable to cost-saving in pavement projects. Considering the availability of recycled materials in the State of Maryland (MD), four abundant recycled materials, recycled concrete aggregate (RCA), recycled asphalt pavement (RAP), foundry sand (FS), and dredged materials (DM), were studied. A survey was conducted to collect the information of current usage of the four recycled materials in States' Department of Transportation (DOTs). Based on literature review, mechanical and environmental properties, recommendations, and suggested test standards were investigated separately for the four recycled materials in different applications. Constrains in using these materials were further studied in order to provide recommendations for the development of related MD specifications. To measure social and environmental benefits from using recycled materials, life-cycle assessment was carried out with

life-cycle analysis (LCA) program, PaLATE, and green highway rating system, BE^{2}ST-in-Highway^{TM}.

The survey results indicated the wide use of RAP and RCA in hot mix asphalt (HMA) and graded aggregate base (GAB) respectively, while FS and DM are less used in field. Environmental concerns are less, but the possibly low quality and some adverse mechanical characteristics may hinder the widely use of these recycled materials. Technical documents and current specifications provided by State DOTs are good references to the usage of these materials in MD. Literature review showed consistent results with the survey. Studies from experimental research or site tests showed satisfactory performance of these materials in highway applications, when the substitution rate, gradation, temperature, moisture, or usage of additives, etc. meet some requirements. The results from LCA revealed significant cost savings in using recycled materials. Energy and water consumption, gas emission, and hazardous waste generation generally showed reductions to some degree. Use of new recycled technologies can contribute to more sustainable highways.

ASSESSMENT AND LIFE-CYCLE ANALYSIS OF RECYCLED MATERIALS FOR SUSTAINABLE HIGHWAY

by

Yating Zhang

Thesis submitted to the Faculty of the Graduate School of the University of Maryland, College Park, in partial fulfillment of the requirements for the degree of Master of Science
2016

Advisory Committee:
Associate Professor Dimitrios, G. Goulias
Professor Ahmet, H. Aydilek
Professor Sherif, M. Aggour

Acknowledgements

I would like to thank my advisors Dr. Dimitrios, G. Goulias and Dr. Ahmet, H. Aydilek first. Without their instruction and help, I am unable to complete the marvelous work for my master degree. Their vast knowledge and abundant experience guide me to do work easily and efficiently. Their great patience and encouragement helps me to overcome the difficulties I met in doing the research. Considering that I had less knowledge in pavements and pavement materials at first, they encourage me to sit in their classes to learn more knowledge related with my research. I was benefited a lot from these lectures, and had more confidence to face the challenges in the research. My research requires much reading and writing. I really appreciated that advisors assisted me to modify my reports sentence by sentence and gave me many comments to advance them.

Thanks for the support of Maryland State Highway Administration providing the funding for the research. Thanks for their help to conduct surveys. Thanks for their meaningful suggestions. I am also very thankful to Dr. Sherif, M. Aggour for his time to serve as my committee member. Moreover, thanks for Zorana Mijic who provided me the references and leaching data. Thanks for Tigist Getaneh who taught me the materials experiments. Thanks for Jin Lee and Eleanor Bloom from University of Wisconsin-Madison who answered my questions for life cycle analysis.

Finally, none of this would have been possible without the love and support from my family and friends. I would like to thank my parents for being my strong supporter in my whole life. I would like to express my heart-felt gratitude to my dearest friend Jingheng Wang for being one of the most important people in my life.

Table of Contents

List of Figures

List of Tables

Chapter 1: Introduction

Currently, the use of recycled materials in highway applications is limited due to regulatory, environmental, and technical restrictions on their potential use. A lack of information on the performance of recycled materials is also a major obstacle for highway use. The objectives of this research study were to: (i) document the state-of-the-art practice of the use of selected recycled materials, Task 1; (ii) review their known performance for applications pertinent to Maryland conditions, based on past experience, Task 2; (iii) identify potential constraints and performance concerns reported from past studies, Task 3; and (iv) identify potential specification revisions needed for their safe use in alternative applications of highway projects for Maryland conditions, Task 4. The following four recycled materials were included in this synthesis study, as identified in the RFP:

- recycled concrete aggregate (RCA)
- reclaimed asphalt pavement aggregate (RAP)
- dredged materials (DM)
- foundry sand (FS)

To achieve the objectives of this study, the project team examined the state of the art practice on the use of these recycled materials and identified potential areas of concern, either related to material performance, environmental considerations, design and field performance (when applicable). Furthermore, a survey to state DOTs was conducted through the AASHTO subcommittee on recycled materials to complement the findings of the study. Finally, the project team examined pertinent Maryland specifications for using these recycled materials in highway applications and identified areas that the revised specifications will need to address in terms of technical requirements.

Based on feedback from SHA, the research team identified specific applications that are applicable to Maryland-specific conditions (Tables 1.1 to 1.4). These recycled materials and applications were the focus of the study.

Chapter 1 presents the introduction, research objectives and organization of this report. Chapter 2 presents the results of the survey to state DOTs. Chapter 3 includes the synthesis on the state of knowledge of the four recycled materials in highway applications. Chapter 4 identifies the potential constraints on the use of these materials, and identifies the potential specification revisions needed for their safe use in Maryland conditions. Chapter 5 presents the results of life cycle analysis conducted with two programs, PaLATE and BE^{2}ST-in-Highway^{TM}. Finally, Chapter 6 summarizes the results of the research and makes conclusions.

Table 1.1 Use of Recycled Concrete Aggregate (RCA) in Highway Applications

Applications / Byproducts	GAB	Foam Asphalt	Drainage/Fill	HMA	PCC
RCA					

Note. RCA= Recycled Concrete Aggregate; GAB= Granular Aggregate Base
PCC: Portland Cement Concrete, HMA= Hot Mix Asphalt.

Table 1.2 Use of Reclaimed Asphalt Pavement (RAP) Aggregate in Highway Applications

Applications / Byproducts	GAB	Foam Asphalt	Drainage/Fill*	HMA**	PCC
RAP, Stockpiled					

Note. RAP = Recycled Asphalt Pavement
* Select borrow & common borrow, bedding/backfill for pipes, edge drain.
** Shoulder.

Table 1.3 Use of Foundry Sand (FS) in Highway Applications

Applications / Byproducts	Crack Sealant & HMA	Drainage/ Embankment & Base	Flowable Fill/ SCC	PCC
Foundry Sand				

Note. SCC = Self Consolidated Concrete

Table 1.4 Use of Dredged Materials (DM) in Highway Applications

Applications / Byproducts	Fill Materials*	Lightweight Aggregate/ Bricks	PCC/ Cement
Clay/Silt Sediments			

Note. * Select borrow & common borrow.

Chapter 2: Survey on the State of Practice of Recycled Materials in Highway Applications

In order to receive feedback from various Department of Transportation (DOTs on the use of recycled materials in highway applications, the research team developed a survey, included in the appendix, which was distributed through the AASHTO subcommittee on recycled materials to all 50 states with the help of Maryland State Highway Administration. The summary findings are presented herein. The survey indicated the usage level of the four recycled materials by state and identified the details of their source and uses in highway applications. The following 16 state DOTs responded to the survey: Alaska, Alabama, Colorado, Connecticut, Delaware, Florida, Georgia, Montana, North Dakota, Ohio, South Dakota, Texas, Virginia, Washington D.C., Wisconsin and Wyoming. The questionnaire is attached in the appendix. The responses are summarized in Tables 2.1 through 2.4.

2.1 Results

As seen in the results, RAP and RCA have been widely used, while DM and FS have been used less in highway applications. Many states have reported using RAP primarily in HMA and foamed asphalt. RCA has been mainly used in GAB, drainage/fill, and PCC. No record on the use of DM was reported. FS has been used in flowable fill/SCC materials.

Table 2.5 lists the potential sources of the recycled materials. Bridge and highway structures are the main sources. A few states reuse these materials from demolished buildings or pavement. Only Delaware accepts recycled materials from out of state plants or contractors. One potential reason for preventing some states from using recycled materials may be concerns of environmental suitability (Table 2.6). However, only a few states indicated that using recycled materials may elevate concentrations of metal/organic contaminants and cause high/low pH levels. In addition, the generation of HMA plant fumes is a concern in Alaska and may hinder RAP use.

Table 2.7 presents the technical challenges documented when recycled materials were used in highway applications. The major challenge for using RAP is related to the lack of consistent mechanical properties. Such inconsistent properties can negatively affect the durability, low temperature performance and fatigue resistance in pavements. Other challenges, such as the difficulty of finding the optimum binder replacement and testing the equivalent binder grade, also exist in using RAP, as indicated by Montana and Utah DOTs, respectively. Delaware DOT also indicated that the high permeability of RAP may be a problem in GAB application.

Table 2.1 Use of RCA in Highway Applications

Applications / Byproducts	GAB	Foam Asphalt	Drainage/Fill	HMA	PCC
RCA	AL,CO,D.C., DE,GA,ME, ND,OH,SD,UT,VA,WI, WY,	-	AL,DE,OH,WI	-	AL,CO,OH, VA

Note: GAB= Granular Aggregate Base; PCC= Portland Cement Concrete; HMA= Hot Mix Asphalt.

Table 2.2 Use of RAP in Highway Applications

Applications / Byproducts	GAB	Foam Asphalt	Drainage /Fill	HMA
HMA, Plant	AK	AK,ME, VA,WI	-	AK,AL,CO,CT,D.C., DE,GA,ME, MT, ND,OH, SD, UT,VA,WI,WY

Table 2.3 Use of Foundry Sand in Highway Applications

Applications / Byproducts	Crack Sealant	Base	Drainage/ Embankment	Flowable Fill/ SCC	HMA	PCC
Sand Foundry	-	-	-	WI,OH,AL	-	-

Note. SCC = Self Consolidated Concrete.

Table 2.4 Use of Dredged Materials in Highway Applications

Applications / Byproducts	Fill Materials
Clay/Silty Sediments	-

Table 2.5 Source of Recycled Materials

Source	State
Bridge/ highway structures	CT,D.C.,GA,ME,UT,WI,WY,OH,CO,AL,ND,MT,DE,VA
Buildings/other structures	D.C.,GA,DE,VA
Recycling plants within state	AK,D.C.,GA,WI,OH,AL,DE
Out-of-state recycling plants	DE
Pavements	SD,WI
Contractors	DE

Table 2.6 Environmental Concerns

Environmental concerns	State
Metal/Organic contaminants	UT,CO,AL
High/low pH levels	OH,AL,VA
HMA plant fumes	AK

The major challenge surrounding the use of RCA is related to alkali-silica reaction (ASR), which may cause clogging in drains. According to Ohio DOT, RCA is gradually being recognized in GAB application, since ASR problems have primarily been solved. The problem of RCA gradation may be solved by further processing, as suggested by Delaware DOT. For FS, a concern from Alaska DOT is that FS may carry some toxic ingredients during the production progress. Thus, a stockpile requires approval by state engineers before using FS in construction. For DM, Ohio DOT also indicated that permission for using DM is possible depending on the source.

Table 2.7 Technical Challenges

State	Responses
AL	FS FS chemical reactions during processing of iron and steel are of concern. Thus, a stockpile must be approved by the Materials and Testing Engineer before it may be used.
AL, CT, DE, ME, MT, UT	RAP RAP is too permeable to work as a base material in GAB, though spec allows it. Additional virgin asphalt is needed for RAP to avoid dry and stiff mixtures. Poor performance of RAP results in more frequent resurfacing. Inconsistent RAP properties results in decreased pavement durability. Variable quality of RAP. The optimum binder replacement is difficult to find. RAP quality affects cold temperature and fatigue behavior of the pavement.
DE, OH	RCA RCA gradation variability is of concern. RCA associated in past with clogged drains and tufa formation.
OH	DM No ban for using DM, so there is currently a source for using these materials.

Technical reports from several full-depth reclamation (FDR) projects were provided from the Maine DOT, where the existing asphalt pavement, as well as part of the underlying unbound base, were recycled in-place to produce a stabilized base course (Table 2.8). In these projects, the objective was to solve cracking and rutting problems. Some techniques and recommendations for FDR are mentioned, including how to compact each layer in FDR, determine bulk specific gravity, and select additives and optimum binder contents. Suitable testing procedures and better methods for mix design are also suggested. Increasing structural numbers for surface layers were proposed.

Similar reports from Virginia DOT were provided in projects where RAP was used for in-place recycling for the base and/or sub-base. In the I-81 rehabilitation project, three in-place recycling techniques (FDR, cold-in place recycling (CIR), and cold-central plant recycling (CCPR)) were implemented and the field performance has shown the acceptability of all three methods with RAP. Because of concerns related to lower shear strength and excessive permanent deformation, resulting from large strains as RAP content increases, it was suggested using up to 50% RAP content by weight in virgin aggregate base and subbase layers.

Table 2.8 Study Findings

State	Recycled Materials & Application	Study Results
ME	**RAP in HMA & Base**	*Peabody, 2009. "Full Depth Reclamation with Cement."* ❖ Roadway failure is mainly due to insufficient support for the HMA surface. ❖ Transverse and longitudinal cracking in the soil cement section is a concern. ❖ Four percent cement may be too much to make the pavement section flexible in the harsh environment.
		Marquis et. al., 2004. "Potential Benefits of Adding Emulsion to FDR Material." ❖ Use of emulsion has improved the overall pavement performance, reduced the occurrence of load cracks and rutting of the surface layer, and increased the structural capacity of the pavement. ❖ Preliminary investigation of the existing roadway materials is necessary to select the best alternative for base stabilization and avoid problems during construction.

Table 2.8 Study Findings (continued)

State	Recycled Materials & Application	Study Results
ME	RAP in HMA & Base	*Marquis et. al., 2004. "Using Foamed Asphalt as a Stabilizing Agent in FDR of Route 8 in Belgrade, Maine"* ❖ Sections with FDR had the lowest structural numbers compared to sections with asphalt stabilized base. ❖ Sections treated with FDR material and either granular base, asphalt stabilized base or HMA base had similar costs.
		Mallick et al., 2002. "Development of a Rational and Practical Mix Design System for FDR Mixes" ❖ Use of a slotted mold (i.e., a sample extrusion device to remove emulsified asphalt from compactor immediately after compaction) is suggested to squeeze out of water during compaction of FDR mixes. ❖ Use samples in sealed bags to determine bulk specific gravity in the laboratory. ❖ Use density and resilient modulus versus total additive content (i.e., water and asphalt emulsion) criteria to select optimum additive content. ❖ Mix design for FDR samples (RAP and unbound base material) should be compacted to 50 gyrations. Control strip in the field should meet at least 95% density of in-place loose mixes, and be compacted to 50 gyrations. ❖ Increase structural numbers for FDR layers to design binder and surface layers. Use a suitable test procedure, such as the soaked, conditioned strength, tube suction or stripping test, to evaluate moisture susceptibility of designed mixes.
VA	RAP in HMA	*(Diefenderfer et. al., 2014). "I-81 In-Place Pavement Recycling Project"* ❖ Active fillers (e.g. cement) can improve resistance to moisture and improve the early strength of bitumen stabilized asphalt materials. ❖ On higher volume roads, an asphalt concrete overlay is generally placed over in-place recycling HMA layer, but functional treatments (e.g. chip seals) are used on lower volume roadways. ❖ During construction, cold central-plant and cold in-place recycling HMA layers generally meet or exceed 98% of the modified Proctor density requirements based on AASHTO T 180. ITS and M_R laboratory testing indicated that the performance of CCPR and CIR are similar. Dynamic modulus testing indicated that the CCPR material might have a better performance at higher temperatures. ❖ The field performance tests demonstrated that the section of pavement rehabilitated by the three, in-place recycling methods (FDR, CCPR, CIR) continues to perform well after nearly three years of high volume, interstate traffic.

Table 2.8 Study Findings (continued)

State	Recycled Materials & Application	Study Results
VA	RAP in Base	*Hoppe et al. 2015. "Feasibility of RAP Use as Road Base and Sub-base Material"* ❖ RAP in base and subbase is technically viable. There is a trend of using up to 50% RAP content by weight in virgin aggregate, because of the concern on lower shear strengths and excessive permanent deformations as RAP content increases. ❖ RAP for use in base and subbase layers can be characterized by performance-related parameters, such as grading, resilient modulus, shear strength, and permanent deformation and durability (i.e., frost susceptibility and abrasion). ❖ No leaching concerns on un-stabilized RAP used as base or subbase material. Use of chemical stabilization agents may require environmental assessment on a case-by-case basis.

The specifications provided by DOTs are listed in Table 2.9. Though the details of requirements differ in various states, the concerns in requirements are similar. The concerns involve the source, processing, mix design, tests, plants and construction. Furthermore, the recycled material content, gradation, mechanical properties, leaching properties, stockpile management and plant equipment, as well as quality control during construction are all considered. The requirements differ by application, weather conditions and traffic volume (i.e., high versus low volume roadways).

RAP is widely used in HMA and bituminous concrete. Granular base and shoulders are also considered. Most states have a limit on the percentage of RAP, however an increase in RAP is allowed if approved by DOT engineers. For instance, Alaska DOT restricts the use of RAP to 15% in wearing course and 25% in lower course for HMA construction. South Dakota DOT has a restriction of 20% maximum in mainline HMA mix and 40% maximum in shoulders. Wyoming limits usage of RAP to 20% or less. For applications of bituminous concrete, Connecticut sets up a maximum of 10% RAP used with no binder grade modification; however, a contractor is allowed to increase the RAP percentage in 5% increments up to a maximum of 30%, provided the engineer approves a new JMF (job mix formula). States adjust the requirements in different cases. Georgia limits the usage of RAP to 5% of the total mix for interstate projects, 0 to 40% for remaining roadways, 40% for continuous drum plants and 25% for batch plants. In Ohio, the maximum usage of RAP is determined according to the traffic load and layer. In heavy traffic, where a polymer modified surface mixture is used, the maximum percentage of RAP is 10% by dry weight of mix. Wisconsin has a regulation that, in shoulder applications or surfacing, 45% to 55% RAP (by weight) can be included in reprocessed or blended material.

RCA is often used in granular base. Some states (e.g. Ohio) allow only the use of coarse aggregates since fine aggregates may produce undesirable properties. In South Dakota, the requirements for using RCA in subbase, gravel cushion, aggregate base course, gravel surfacing, pit run and granular bridge end backfill are different. The requirements are mainly related to the percent passing, liquid limit, plasticity and LA abrasion loss. Ohio has requirements in water absorption as well.

FS has been used in granular base, drainage, flowable fill, embankment and other applications. The requirements of FS primarily relate to the gradation and proportioning. Ohio adopted a set of standards to ensure that FS is non-toxic before it is used in highway applications. The leached concentrations of selenium, phenol, cyanide and fluoride are required in Ohio. In addition, it is required that the solution of FS be tested for acidity, alkalinity, pH, sulfates, as well several metals. Table 9 provides some of these requirements and recommendations.

No information on the use of DM in highway applications was provided in the surveys. DM from maintaining navigable waterways routes are not used as a recycled material, since the grain size tend to be very fine-grained, uniform in size and generally cannot be processed to meet gradation requirements for typical highway applications. DM from mining operations of waterways is used, since these locations may provide larger size materials, which generally meet the requirements within construction specifications.

Table 2.9 Technical Data and Specifications

<table>
<tr><th>State</th><th>Item</th><th>Details</th></tr>
<tr><td>AK</td><td>RAP in HMA</td><td>◆ Max 15% in wearing course; max 25% in lower courses</td></tr>
<tr><td>AL</td><td>RAP In HMA</td><td>◆ The allowable use of RAP in:
■ ALDOT 327, Plant Mix Bituminous Base: RAP≤ 25%, RAP+RAS≤ 25%
■ ALDOT 327-E, Permeable Asphalt Treated Base: RAP ≤ 10%, RAS not allowed
■ ALDOT 420, Open Grades Friction Course: RAP≤ 10%, RAS not allowed;
■ ALDOT 423, Stone Matrix Asphalt & Superpave
• surface layers: RAP≤ 20% (with no more than 15% containing chert gravel), RAP+RAS≤ 20%
• all other layers: RAP≤ 25%, RAP+RAS≤ 25%
■ allowable to all Superpave ESAL range mixes that require PG 67-22 liquid binder: RAP≥25 %, or RAP+RAS≤35 % (mixes in base and binder layers)
■ unallowable to surface Superpave ESAL mixes that require PG 76-22 liquid binder: RAP≥25 %, or RAP+RAS≥25 %.
◆ Required test for RAP≥25 %: AASHTO T 319, AASHTO T 240, AASHTO T 315, ALDOT 361
◆ Additional requirements on stockpiles when RAP≥25 %:
<table>
<tr><th colspan="2">Additional RAP Stockpile Requirements for RAP Used in a Job Mix Formula with Increased RAP Content</th></tr>
<tr><td>Control Parameter</td><td>Standard Deviation</td></tr>
<tr><td>Asphalt Content</td><td>0.5%</td></tr>
<tr><td>%Passing #200 Sieve</td><td>1.0%</td></tr>
<tr><td>Sieve with 50% RAP Passing</td><td>5.0%</td></tr>
<tr><td colspan="2">*Based on a minimum of 10 tests.</td></tr>
</table>
◆ Mix design
• job-mix formula approved by the Materials and Tests Engineer, checked by the Division Materials Engineer
• new job-mix formula for new source and new materials; no new job-mix formula for changed liquid asphalt binder source or changed anti-stripping agent, but one-point check (the Air Void, VMA, Stability, Flow, and TSR) is required.</td></tr>
</table>

Table 2.9 Technical Data and Specifications (continued)

<table>
<tr><th>State</th><th>Item</th><th>Details</th></tr>
<tr><td rowspan="2">Al</td><td>RAP in HMA</td><td>◆ Processing
• RAP used in 3/8 inch {9.5 mm} Section: 100 % of the RAP passes the 1/2 inch {12.5 mm} sieve
• RAP used in ALDOT 801 and 802 (no gravel in ALDOT 327 PATB, ALDOT 420 and ALDOT 423 mixes): the maximum size for the mix specified
• RAP used in ALDOT 327 PATB and ALDOT 420 mixes: 100 % of the RAP retained on the No. 4 {4.75 mm} sieve

◆ Construction Requirements:
• equipment; wet weather and temperature limitations; preparation of underlying surface; preparation of mixtures; transporting mixture; placing the mixture; compacting; joints.</td></tr>
<tr><td>RCA in PCC</td><td>◆ Processing
■ Wash and eliminate coatings on coarse aggregate for Portland cement concrete and cover aggregate for bituminous treatment.
■ Coating check: Material shall pass the No. 200 {75 µm} sieve and be checked by visual inspection using a petrographic microscope.
■ The amount of deleterious substances shall not exceed these limits:

<table>
<tr><th colspan="3">Maximum Allowable Deleterious Materials in Coarse Aggregates</th></tr>
<tr><th>Type of deleterious materials</th><th>Bitumen Surface Treatment and Concrete Class A, B, and D</th><th>All other uses</th></tr>
<tr><td>Coal and lignite</td><td>0.25%</td><td>0.25%</td></tr>
<tr><td>Clay lumps</td><td>0.25%</td><td>0.25%</td></tr>
<tr><td>Material passing the No.200 sieve</td><td>1.0%</td><td>2.0%</td></tr>
<tr><td>Flat or elongated particles (5:1 ratio)</td><td>10%</td><td>10%</td></tr>
</table>

■ Aggregate that has an adherent coating will not be acceptable.

<table>
<tr><th>Type of Deleterious Materials</th><th>Bitumen Surface Treatment and Specific Concrete Mixtures</th><th>All Other Uses</th></tr>
<tr><td>Flat or elongated particles (3:1 ratio)</td><td>20%</td><td>20%</td></tr>
<tr><td>Other local deleterious substance (Shale ,Mica, Marcasite, etc.)</td><td>2%</td><td>2%</td></tr>
<tr><td>Reactive Silica</td><td>8%</td><td>8%</td></tr>
</table></td></tr>
</table>

Table 2.9 Technical Data and Specifications. (continued)

State	Item	Details
AL	RCA in PCC	■ Three options for designing concrete mixes with limestone aggregates that contain more than 8.0% silica: • Class F fly ash replacing 20% cement by weight; • Ground Granulate Blast Furnace Slag replacing 50% cement by weight (for concrete placed at ambient temperatures of 45 ºF {7 ºC} or above); or • Class C fly ash and microsilica replacing 30% and 5% cement by weight. ■ Restriction of the amount of absorption for gravel aggregates: • gravel for use in bituminous plant mixes and bridge superstructure concrete (except prestressed concrete): absorption ≤2.0% and passing the 3/4 inch {19.0 mm} sieve and retained on the No. 4 {4.75 mm} sieve • require a 15 minute vacuum saturation period prior to the 15-19 hour soaking period ■ The maximum allowable deleterious materials in coarse aggregate used in concrete (minimum 28-Day compressive strength of 3000 psi, ALDOT 501.02) applies only to concrete used for bridge substructures, box culverts, retaining walls and concrete safety barriers.
	FS	◆ The stockpile must be approved by the Materials and Tests Engineer before it may be used.
	DM	◆ **Source** ■ DM from maintaining navigable route of waterways are not used, since the grain size tends to be very fine-grained, uniform in size and generally cannot be processed to meet required gradation. ■ DM from mining operations of waterways are used.
CT	RAP in HMA	◆ **Processing** ■ 100% RAP pass the two in (50 mm) sieve. Additional crushing and sizing may be required if the RAP aggregate exceeds the maximum sieve size for the mix type in CTDOT 828. ■ From pavements previously constructed: • certification for binder substantially free of solvents, tars and other contaminants • label stockpile with a sign reading “ConnDOT RAP” and separate it from all other materials • The request for approval of the RAP material include: o certification for source, stockpile location; and o estimation for quantities to be used. ■ From unknown source: • certification for the component of RAP meeting the specification requirements of CTDOT M.04.01-1a through c and for the binder in the RAP substantially free of solvents, tars and other contaminants • separate stockpiled RAP from all other RAP materials at all times

Table 2.9 Technical Data and Specifications (continued)

State	Item	Details
CT	RAP in HMA	• The request for approval shall include: o a 5-pound (2.5-kg) sample of the RAP incorporated into the recycled mixture & a 5-pound (2.5-kg) sample of the extracted aggregate from the RAP; o viscosity test results; and o a statement that RAP material 100% passing the ½ inch (12.5 mm) sieve and free from contaminants such as joint compound, wood, plastic, and metals. ■ From existing roadway, contractor's RAP stockpile approved by the department, or department stockpile: • for interstate projects, no alluvial gravel or local sand • for shoulder construction, sand or gravel ≤20% • for non-interstate projects, alluvial gravel ≤ 5 % • for mainline or ramps, RAP = 0 ~40% • for continuous mix type plants, RAP ≤40% • for batch type plant, RAP ≤25% ◆ **Applied in bituminous concrete** ■ Comply with requirements in CTDOT M.04.01-1. ■ Limit use of RAP in 10% with no binder grade modification. The JMF should be approved by the Engineer. ■ If greater than 10% of total mix weight (mass), 5% increments up to a maximum of 30% is allowed in the percentage of RAP, provided a new JMF is approved by the Engineer. • JMF shall include: Gradation and asphalt content of the RAP, percentage of RAP to be used, virgin aggregate source(s), total JMF content based on total mixture weight (mass), percentage of bitumen based on total mixture weight (mass), gradation of combined bituminous concrete mixture (including RAP), and grade of virgin added. ◆ **In construction:** ■ Indicate on the ticket the percent of RAP, the moisture content, and the net weight of RAP added to the mixture. ■ Make necessary adjustments to ensure bituminous concrete materials are free from moisture throughout. ■ Do not change the JMF and RAP percentage without prior approval of the engineer in daily construction.
ME	**RAP in HMA**	◆ **Applied in HMA** The percentage for RAP can be reduced up to 10% from the amount list on the JMF but shall not exceed the amount listed in the JMF, or for the specific application, under any circumstance.

Table 2.9 Technical Data and Specifications (continued)

<table>
<tr><th>State</th><th>Item</th><th>Details</th></tr>
<tr><td>ME</td><td>RAP in HMA</td><td>

◆ **Applied in bituminous pavement**
■ 100% of RAP should pass a 2-inch square mesh sieve.
■ It should be free of winter sand, granular fill, construction debris and other materials not generally considered bituminous pavement.

◆ **Full-depth Reclamation (FDR) HMA**
■ It should be rolled with a vibratory pod/tamping foot roller with a minimum 54 inch diameter single drum.
■ The remaining FDR material shall be compacted to a minimum density of 98% of the target density as determined in the control section.

◆ **Plant**
■ It should be capable of automatically compensating for the moisture content of the RAP.
■ The RAP shall be delivered to the mixer at a temperature of no less than 50°F.
■ If a drum type mixing plant is used, the RAP may be heated prior to being mixed with the emulsified asphalt to a temperature not to exceed 195°F.
■ The plant mixed recycled asphalt pavement shall be performed:
• between May 15th and September 15th inclusive in Zone 1 and between May 1st and September 30th inclusive in Zone 2;
• when the atmospheric temperature is 50°F and rising;
• when there is no standing water on the surface;
• during generally dry conditions, or when pulverizing, adding, mixing, and curing can be obtained using proper procedures, or when compaction can be accomplished as determined by the resident; and
• when the surface is not frozen and overnight temperatures are expected to be above 32°F.

◆ **Processing**
■ All material must be no larger than 1 1/2 inch.
■ Material must be stockpiled, but not for longer than 48 hours.
</td></tr>
<tr><td>SD</td><td>RAP in HMA & Base</td><td>

◆ **Applied in asphalt concrete**
RAP shall conform to the following gradation:
<table>
<tr><td>Sieve Size</td><td>Percent Passing</td></tr>
<tr><td>1 1/2 inch</td><td>100</td></tr>
<tr><td>1 inch</td><td>95-100</td></tr>
</table>

◆ **Applied in cold in-place recycling for HMA**
RAP shall conform to the following gradation:
<table>
<tr><td>Sieve Size</td><td>Percent Passing</td></tr>
<tr><td>1 1/4 inch</td><td>100</td></tr>
<tr><td>1 inch</td><td>95-100</td></tr>
</table>
</td></tr>
</table>

Table 2.9 Technical Data and Specifications (continued)

<table>
<tr><th>State</th><th>Item</th><th>Details</th></tr>
<tr><td>SD</td><td>RAP in HMA & Base</td><td>
◆ Applied in granular base
■ requirements for gradation
■ liquid limit, plasticity index, LA abrasion loss

◆ RAP is not typically allowed in Select Borrow.
◆ RAP is allowed in HMA ≤20% (Mainline HMA Mix).
◆ RAP is allowed in shoulders ≤40%.
◆ RCA is not allowed in drainage fabric, edge drains, or other similar drainage systems except in approach drains and transverse drains.

◆ Processing:
■ 100 percent passing a 1 1/4-inch sieve;
■ 75 percent or less of the aggregate passing a No. 4 sieve; and
■ asphalt content: 3% ~6.5%.
■ Department: Assess properties by visual inspection but may test questionable.
■ For the percent passing the 1 1/4-inch sieve, extraction of asphaltic material is not required in the test.
■ For the percent passing the No. 4 sieve and percent of asphalt content, extraction of asphaltic material is required in the test.
</td></tr>
<tr><td>WI</td><td>RAP in Base</td><td>
◆ Contractor can use RAP as 3-inch base, or 1 1/4-inch base without regard to the gradation requirements under WIDOT 305.2.2.1.

◆ Construction
■ For RAP base, stockpile material conforming to WIDOT 306.2 and place material as the plans or special provisions specify. Construct the base conforming to WIDOT 305.3.
■ Excess material becomes the contractor's property.

◆ In asphaltic pavement base
■ 100 percent passing a 1 1/4-inch sieve.
■ For shouldering or surfacing applications, RAP content must equal 45 ~ 55% (by weight).

◆ In open graded base
Furnish crushed concrete conforming to WIDOT 301.2, except for gradation conform to the following:
<table>
<tr><td>Sieve</td><td>1-inch</td><td>3/8-inch</td><td>No. 4</td><td>No. 10</td><td>No. 40</td><td>No. 200</td></tr>
<tr><td>Percent passing (by weight)</td><td>90 - 100</td><td>45 - 65</td><td>15 - 45</td><td>0 - 20</td><td>0 - 10</td><td>0 - 5.0</td></tr>
</table>
</td></tr>
</table>

Table 2.9 Technical Data and Specifications (continued)

<table>
<tr><th>State</th><th>Item</th><th>Details</th></tr>
<tr><td>OH</td><td>RAP in HMA</td><td>

◆ **Processing**

- From verifiable Department, Ohio Turnpike Commission projects:
  - Process and use RAP by one of the following two methods.
- From other sources or the unknown source:
  - Process and blend the RAP into a single uniform stockpile, test according to Level 3 Asphalt Mix Design requirements and obtain District approval for use.
  - Obtain written Laboratory approval for use of unusually large, old RAP stockpiles of unknown content and/or age. Include approved methods in the Quality Control Plan for ongoing processing and testing of piles. Ensure no foreign or deleterious material (OHDOT 703.04, OHDOT 703.05) in RAP.

Method 1-Standard RAP Limits		
Asphalt Mix Applications	Percentage RAP by Dry Weight of Mix, Max.	Total Virgin Asphalt Binder Content, Min
Heavy Traffic Polymer Surface Course	10%	5.2
Medium Traffic Surface Course	20%	5.0
Light Traffic Surface Course	20%	5.2
Intermediate Course	35%	3.0
Base Course 301	50%	2.7
Base Course 302	40%	2.0

Method 2-Extended RAP Limits		
Asphalt Mix Applications	Percentage RAP by Dry Weight of Mix, Max.	Total Virgin Asphalt Binder Content, Min
Heavy Traffic Polymer Surface Course	10%	5.0
Medium Traffic Surface Course	25%	4.8
Light Traffic Surface Course	25%	5.0
Intermediate Course	40%	3.0
Base Course 301	55%	2.5
Base Course 302	45%	1.8

- Determine the final RAP gradation and asphalt binder content on a minimum of four separate stockpile (or roadway for concurrent grinding) samples, all agreeing within a range of 0.4% for asphalt binder content and 5 % passing the No. 4 (4.75 mm) sieve.

</td></tr>
</table>

Table 2.9 Technical Data and Specifications (continued)

<table>
<tr><th>State</th><th>Item</th><th>Details</th></tr>
<tr><td rowspan="2">OH</td><td>RAP In HMA</td><td>◆ Plant
■ Provide enough space for handling at a hot mix facility.
■ Provide a clean, graded base for stockpiles that does not collect water. Test blended RAP and RAS stockpiles to assure uniform gradation and asphalt binder content.
■ Ensure uniform stockpile properties match the JMF submitted RAP and RAS properties, unless the uniform stockpile will be processed into the asphalt plant using plant cold feed in line processing.
■ Record in the JMF submittal both the uniform stockpile and in line processed RAP properties.
■ Give each stockpile a unique identification, distinguishing if RAS piles are from un-used manufactured shingle waste or used roofing tear-off shingles. Provide in the plant lab RAP and RAS properties for each uniform, blended stockpile cross referenced with its identification.
■ Provide the date the stockpile processing was completed and the estimated size in tons. Stockpiles and processing methods are subject to inspection and approval by the DET at any time.</td></tr>
<tr><td>FS</td><td>◆ Mix design
■ Conform to the requirements of OHDOT 703.05 for gradation. Use fine aggregate that is fine enough to stay in suspension within the mixture to ensure proper flow.
■ Meet the requirements of the Division of Surface Water Policy 400.007 “Beneficial Use of Non-Toxic Bottom Ash, Fly Ash and Spent Foundry Sand and Other Exempt Wastes,” and all other regulations.
• The following requirements should be met:
<table><tr><td>Leachate</td><td>Selenium</td><td>Phenol</td><td>Cyanide</td><td>Fluoride</td></tr><tr><td>Maximum content (mg/L)</td><td>1</td><td>10.5</td><td>0.6</td><td>12.0</td></tr></table>
• The solution must be analyzed for the following parameters: acidity, alkalinity, aluminum, arsenic, barium, cadmium, chlorides, chromium, copper, fluoride, iron, lead, manganese, mercury, pH, selenium, specific conductance, sulfates, total dissolved solids, vanadium and zinc.
• At a minimum, annual tests must be performed on the materials.
◆ The applications of nontoxic FS are stabilization/solidification of other waste, soil blending ingredient, landfill, structural fill, pipe bedding, borrow pits and surfacing.</td></tr>
</table>

Table 2.9 Technical Data and Specifications (continued)

State	Item	Details
OH	RCA in PCC	◆ Source ■ RCA source must be from an ODOT project.. Do not use non-ODOT sources. ■ Do not inter-mingle concrete from different ODOT concrete sources. ■ Do not use RCA as a fine aggregate or produce a coarse aggregate material with more than 5% passing the No. 16 sieve, in the concrete. ◆ **Processing coarse RCA** ■ Remove steel, joint sealant, soil and other contaminants. Use necessary crushing, screening, washing and beneficiation methods to remove all fines and impurities and produce coarse aggregate with consistent quality and properties. ■ Meet quality requirements of 703.02-B, except: • percent of wear, Los Angeles test, maximum 50%; • amount passing the No. 200 (75µm) sieve, maximum 1.5%; • chloride content (AASHTO T 260), maximum 0.6 lbs. /yd^3 in new concrete; • specific gravity variability, maximum* 0.100; • absorption variability, maximum* 0.8%; * Stockpile aggregates that have specific gravity and absorption values that fall outside the limits of variability separately. ■ Use only material passing 703.13. Test each coarse aggregate gradation and each different source of RCA by the Department. ■ Meet the gradation requirements of mix design in 1117.04 and 1117.05. ■ Use only coarse RCA with absorption of 7.0% or less. ■ Provide coarse RCA with an asphalt content of 1.0% or less. ■ Stockpile material and do not use until RCA is tested and approved. ODOT will take quality assurance samples of stockpiles to verify the quality and consistency of the RCA. ◆ **Mix design** ■ Proportion the mix so that the nominal maximum aggregate size is 1 inch and the combination of aggregates are workable, finishable and well graded, and within the percent retained on each sieve. ■ When sieve recommendations are not satisfied: ▪ No single sieve requiring a minimum of 8% retained will be below 5% retained and no more than two below sieves will be allowed. ▪ When the percent retained on each of two adjacent sieve sizes is less than 8%, the total percent retained on either of these sieves and the adjacent sieve (that is not below 8%) shall be at least 13%. ▪ A single sieve may retain up to 22%. ■ Use combined RCA and virgin aggregate to obtain a well graded mix.

Table 2.9 Technical Data and Specifications (continued)

State	Item	Details
OH	**RCA in PCC**	■ The cementitious content ≥520 lbs/yd^3. Use fly ash, GGBF slag, and combined pozzolans at the limits defined in 499. ■ Establish maximum water–cementitious (W/C_m) ratio conforming to 499.03 and Supplement 1026. ■ Use a water reducing admixture (705.12) to achieve an acceptable level of consistency, workability and finishability. • Meet the Modulus of Rupture of 600 psi in 7 days and 700 psi in 28 days. Base the strength on the average of three 6"x 6" beam tests results. • Achieve a minimum compressive strength at 28 days of 5500 psi. • Provide concrete with 6 ± 2% air. • Design the mix to mitigate any material-related distresses found during the pavement survey (1117.02). • To mitigate for ASR, use 20% type F fly ash; 30% GGBF slag, or; a combination of both materials up to 50%, not exceeding the maximum content for either material.
OH	**RCA in PCC**	◆ **Construction** ■ Stockpile the RCA in increments of no more than 5,000 tons and test the absorption and specific gravity to make batch adjustments prior to use. Don't use RCA with an absorption exceeding 7%. ■ Maintain moisture above SSD during concrete production by stockpile soaking. Test the moisture content of all aggregates at the beginning of each day's production and retest at least every 1000 yd^3 of concrete. ■ Test gradation daily to maintain gradation within specification limits. ■ Adjust the amount of water added at the mixer, based on the moisture in the aggregate and the moisture the aggregate will absorb. Do not exceed the maximum established water cementitious ratio. ■ Use an approved set-retarding admixture conforming to OHDOT 705.12, when the concrete temperature exceeds 75°F (24°C). ■ Test the air content, slump, unit weight and temperature on the first three loads. If consistent to the engineer's satisfaction, extend testing to every five loads of concrete or as directed by the engineer. ■ Make beams for strength specimens twice a day at the engineer's direction. Perform air, slump, yield and temperature tests when strength specimens are made. ■ Insure that the pavement obtains 600 psi modulus of rupture before subjecting the pavement to traffic. Do not allow moisture runoff from RCA stockpiles to enter streams or groundwater. ■ Establish a slump range approved by the engineer for the mix for each method of placement and control the mixes within the established range. Remove wash water from the mixer prior to batching concrete. ■ If the specific gravity changes by more than 0.02 from the original design, adjust the design weight to conform to the new specific gravity.

Table 2.9 Technical Data and Specifications (continued)

State	Item	Details
UT	RAP in HMA	◆ **Mix design** ■ RAP ≤ 25% of the total weight of the hot mix and asphalt binder ≤ 25% of the total binder. ■ RAP aggregate is required to meet the requirement as follows with exception of Sand Equivalent:

Aggregate Properties Required for HMA

Test Method	Test No.	75 Design Gyrations and Greater	Less Than 75 Design Gyrations
One Fractured Face	AASHTO T 335	95% minimum	85% min (1 inch and ¾ inch) 90% min (½ inch and ⅜ inch)
Two Fractured Face	AASHTO T 335	90% minimum	80% min (1 inch and ¾ inch) 90% min (½ inch and ⅜ inch)
Fine Aggregate Angularity	AASHTO T 304	45 minimum	45 minimum
Flakiness Index	UDOT MOI 933 (Based on ⅜ inch sieve and above)	17% maximum	17% maximum
L.A. Wear	AASHTO T 96	35% maximum	40% maximum
Sand Equivalent	AASHTO T 176 (Pre-wet method)	60 minimum	45 minimum
Plasticity Index	AASHTO T 89 and T 90	0	0
Unit Weight	AASHTO T 19	Minimum 75 lb/ ft^3	minimum 75 lb/ ft^3
Soundness (sodium sulfate)	AASHTO T 104	16% maximum loss with five cycles	16% maximum loss with five cycles
Clay Lumps and Friable Particles	AASHTO T 112	2% maximum	2% maximum
Natural Fines	N/A	0%	10% maximum

◆ **Test (optional)**

■ Do not adjust the asphalt binder grade: RAP ≤15% by weight and RAP asphalt binder content ≤15% of the total asphalt binder content by weight.

■ Adjust asphalt binder grade according to AASHTO M 323: Asphalt binder = 15 ~ 25% of the asphalt binder weight.

■ Select one grade softer than the grade specified. Don't lower than PG XX-34.

■ Provide test reports indicating the PG grade and quantity of the recovered asphalt binder that is consistent throughout the stockpile.

Table 2.9 Technical Data and Specifications (continued)

State	Item	Details
VA	RAP in HMA	

- **In asphalt mixture**
  - Asphalt surface, intermediate and base mixtures containing RAP shall use the PG grade of asphalt cement as indicated in Table II–14A.
  - The final asphalt mixture shall conform to the requirements for the type specified. Do not contact open flame during the production process.
  - Mixture is handled, hauled, and stored if contamination can be minimized. It is stockpiled and used if the variable asphalt contents and asphalt penetration values don't adversely affect the consistency of the mixture.
  - Ensure that the maximum top size introduced into the mix is two inches. Introduce smaller size into the mix if the reclaimed particles are not broken down or uniformly distributed throughout the mixture during heating and mixing.
  - The mixture being produced should conform to the approved job-mix formula and volumetric properties specified in Table II-14.

Recommended Performance Grade of Asphalt Cement			
Mix Type	Percentage of RAP in Mix		
	%RAP<25.0%	25%<%RAP≤30%	25%<RAP≤35%
SM-4.75A,SM-9.0A, SM-9.5A,SM-12.5A	PG 64S-22	PG 64S-22	
SM-4.75D,SM-9.0D, SM-9.5D,SM-12.5D	PG 64S-22	PG 64S-22	
IM-19.0A	PG 64S-22	PG 64S-22	
IM-19.0D	PG 64S-22	PG 64S-22	
BM-25.0A	PG 64S-22		PG 64S-22
SM-25.0D	PG 64S-22		PG 64S-22

- **In asphalt concrete mixture**

Type E (polymer modified, VDOT 211.04) designated mixtures shall not contain more than 15% reclaimed asphalt pavement (RAP) material (by weight) or 3% recycled asphalt shingles (RAS) by weight.

- **In stone matrix asphalt concrete**

Specified Performance Grade of Asphalt and Use of RAP	
Mix type & PG	Allowable RAP Percentage in Mix
SMA-9.5(64H-22), SMA-12.5(64H-22), &SMA-19.0(64H-22)	0 to 20
SMA-9.5(64E-22), SMA-12.5(64E-22), &SMA-19.0(64E-22)	0 to 15

Table 2.9 Technical Data and Specifications (continued)

State	Item	Details
VA	RAP in HMA	◆ RAP is not permitted in thin hot mix asphalt concrete overlay.
WY	RAP in HMA	◆ Limit usage to 20% or less in HMA.

Mix design is a necessary step in achieving desired properties of recycled materials. It is often thoroughly tested in a laboratory in order to gain optimum performance and sometimes a balance of desired properties. Mitigating ASR is an important issue related to the use of RCA. For example, Ohio requires blending RCA with 20% type F fly ash, 30% granulated blast-furnace slag or a combination of both materials, up to 50%. Moreover, a new mix design for recycled materials is encouraged by several states, but the new design needs to be checked by DOTs before implementation.

2.2 Conclusions

The main conclusions of the survey, based on responses from 16 state DOTs, include:

a. RAP has been used by all the states that responded to the survey. RCA has also been used by several states, while FS is less in use and DM is not used in any highway applications. The main sources of recycled materials are bridges and highways, recycling plants in-state, and demolished buildings or structures. Only a small amount of the recycling materials come from old pavements, recycling plants out-of-state or legal contractors.
b. Environmental concerns of using these materials include metal and organic contaminants, low or high pH level and HMA plant fumes. Yet, environmental effects are not the primary obstacle; technical challenges may be considered as a barrier for the wide use of the recycled materials.
c. The requirements in the state specifications include: source, processing, mix design, tests, plant requirements and construction methods. These may include limitations on the percentage of recycled material, gradation, stockpile processing, mechanical tests, leaching tests, plant equipment requirement, and quality control methods.

2.3 Techanical Reports

Diefenderfer, B.K., and Apeagyei, A.K. (2014). *I-81 In-Place Pavement Recycling Project*. Final Report, Virginia Department of Transportation, Richmond, VA.

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*. Report, Virginia Department of Transportation, Charlottesville, Virginia.

Mallick, R., Kandhal, P.S., Brown, E.R., Bradbury, R.L. and Kearney, E.J. (2002). *Development of a Rational and Practical Mix Design System for Full Depth Reclaimed (FDR) Mixes*. Report, Maine Department of Transportation, Augusta, Maine.

Marquis, B., and Peabody, D. (2004). *Potential Benefits of Adding Emulsion to Full Depth Reclamation Material*. Technical Report, Maine Department of Transportation, Augusta, Maine.

Marquis, B., Peabody, D., and Mallick, R. (2002). *Using Foamed Asphalt as a Stabilizing Agent in Full Depth Reclamation of Route 8 in Belgrade, Maine*. Final Report, Maine Department of Transportation, Augusta, Maine.

Peabody, D. (2009). *Full Depth Reclamation with Cement*. Technical Report, Maine Department of Transportation, Augusta, Maine.

2.4 Selective Specifications of States

ALDOT. (2012). *Standard Specifications for Highway Construction, 2012 Edition*. Alabama Department of Transportation, Montgomery, Alabama. Online available: http://www.dot.state.al.us/conweb/doc/Specificatio ns/2012%20DRAFT%20Standard%20Specs.pdf

ALDOT. (2014). *Amended Standard Specifications for Highway Construction*. Special Provision, Alabama Department of Transportation, Montgomery, Alabama. Online available: http://www. dot.state.al.us/conweb/doc/Specifications/2012%20GASP%20Summary/12-0263(3).pdf

CTDOT. (2015). *The Standard Specifications for Roads, Bridges and Incidental Construction, Merged with Supplemental Specifications.* Connecticut Department of Transportation. Online Available: http://www.ct. gov/ dot/lib/dot/documents/dpublications/816/newver/_jan15_form_816.pdf

GADOT. (2013). *Standard Specifications Construction of Transportation Systems*. Georgia Department of Transportation. Online Available: http://www.dot.ga.gov/PartnerSmart/Business/Source/specs/DOT2013.pdf

MEDOT. (2014). *Maine DOT Policies and Procedures for HMA Sampling and Testing*. Maine Department of Transportation. Online Available: http://www.maine.gov/tools/whatsnew/attach.php?id=197258&an=1

MEDOT. (2014). *Department of Transportation Standard Specifications*. Maine Department of Transportation. Online Available: http://www.maine.gov/mdot/contractors/publications/standardspec/docs /2014/StandardSpecification-full.pdf

NDDOT. (2014). *Standard Specifications for Road and Bridge Construction*. North Dakota Department of Transportation, Bismarck, ND. Online Available: https://www.dot.nd.gov/divisions/environmental/docs/ supspecs/2014StandardSpecifications.pdf

ODOT. (2005). *Embankment Construction Using Recycled Materials.* Supplement Specification 871, Ohio Department of Transportation. Online Available: http://www.dot.state.oh.us/Divisions/ConstructionMgt/ Specification%20Files/871_04152005_for_2013.PDF

ODOT. (2012). *Concrete Using Recycled Coarse Aggregate for Concrete Pavement and Incidental Items.* Supplement Specification 117, Ohio Department of Transportation. Online Available: http://www.dot.state.o h.us/Divisions/ConstructionMgt/Specification%20Files/1117_12312012_for_2013.PDF

ODOT. (2013). *Construction and Material Specifications*. Ohio Department of Transportation, Columbus, Ohio. Online Available: http://www.dot.state.oh.us/Divisions/ConstructionMgt/Specification%20Files/2013%20CMS%2011142012%20FINAL.PDF

SDDOT. (2015). *Standard Specifications for Roads and Bridges*. South Dakota Department of Transportation. Online Available: http://www.sddot.com/business/contractors/specs/2015specbook/2015_SD DOT_SpecBook.pdf

UDOT. (2012). *Hot Mix Asphalt (HMA).* Specification 02741, Utah Department of Transportation. Online Available: https://www.udot.utah.gov/main/uconowner.gf?n=7591302386285401

VDOT. (2007). *Road and Bridge Specification*. Virginia Department of Transportation. Online Available: http://www.virginiadot.org/business/resources/const/2007SpecBook.pdf

VDOT. (2011). *Asphalt Concrete Mix Selection Guidelines*. Specification 605, Virginia Department of Transportation. Online Available: http://www.vaasphalt.org/wp-content/uploads/2012/11/VDOT-Mix-Selecti on-Guidelines.pdf

VDOT. (2015). *Special Provision Copied Notes (SPCNs), Special Provision (SPs) and Supplemental Specifications (SSs).* Virginia Department of Transportation. Online Available: http://www.virginiadot.org/ business/resources/const/07RevDiv_II.pdf

WIDOT. (2015). *2015 Standard Specifications*. Wisconsin Department of Transportation. Online Available: http://wisconsindot.gov/Pages/doing-bus/eng-consultants/cnslt-rsrces/rdwy/stndspec.aspx

Chapter 3: Material Characterization

The findings from the literature review are reported next for each recycled material and application included in this study.

3.1 Recycled Concrete Aggregate (RCA)

3.1.1 RCA in GAB

MECHANICAL PROPERTIES

- Characteristics of RCA
  - The average specific gravity (SG) in dry condition of RCA is 2.49, less than that of natural coarse aggregates (NCA) and natural crushed rock base (NCRB) with 2.62 and 2.60, respectively (Table 3.1). Average bulk specific gravity in saturated surface dry condition (SSD) of RCA is 2.31, which is 8.0 % lower than NCA, and equal to 8.6 % was reported in another research (Ravindrarajah and Tam 2005). The average water absorption of RCA was 6.0 %, which is twice as high of natural aggregates (Kolay and Akentuua 2014).

Table 3.1 Specific Gravity and Absorption of Coarse and Fine Aggregates (Kolay and Akentuua 2014)

Particle Size		Properties	NCA	RCA	NCRB
Passing	Retained				
25.0 mm	19.0 mm	Bulk SG (Dry)	2.43	2.13	2.42
		Bulk SG (SSD)	2.50	2.27	2.49
		Apparent SG	2.62	2.49	2.60
		Absorption (%)	3.00	7.00	3.00
19.0 mm	12.5 mm	Bulk SG (Dry)	2.42	2.14	2.42
		Bulk SG (SSD)	2.49	2.28	2.49
		Apparent SG	2.61	2.50	2.60
		Absorption (%)	3.00	7.00	3.00
12.5 mm	9.5 mm	Bulk SG (Dry)	2.42	2.16	2.42
		Bulk SG (SSD)	2.49	2.30	2.49
		Apparent SG	2.61	2.51	2.60
		Absorption (%)	3.00	6.00	3.00
9.5 mm	4.75 mm	Bulk SG (Dry)	2.44	2.16	2.42
		Bulk SG (SSD)	2.51	2.28	2.50
		Apparent SG	2.63	2.47	2.62
		Absorption (%)	3.0	6.00	3.00
4.75 mm	2.36 mm	Bulk SG (Dry)	2.50	2.31	2.49
		Bulk SG (SSD)	2.55	2.39	2.53
		Apparent SG	2.62	2.51	2.61
		Absorption (%)	2.00	3.00	2.00

  - Well-graded aggregates tend to provide better stability. Degradation of particles within an unbound granular layer can result in instability (Chesner et al. 1998).

- Aggregates without fines (minus No. 200 sized materials) have high internal shear strength, but are difficult to handle during construction. Aggregates with high fines content have insufficient internal shear strength because the aggregate particles float within the fines (Chesner et al. 1998).
- Grading characteristics are affected by the jaw opening of the crusher used in crushing the concrete and the strength of the original concrete (Ravindrarajah and Tam 2005).
- Crushing and screening affect stability of RCA granular base materials. When an additional crusher was added to plant operations to increase the quality of crushed particles, California Bearing Ratio (CBR) values increased by 17% and density increased by 1.5 lb/ft^3 (Petraca and Galdiero 1984).
- Sodium sulfate degradation values of RCA are more than those of natural aggregates (Table 3.2), indicating the softness of RCA. Larger RCA particles degrade the most compared with smaller aggregate particles, due to more adhered mortar on the larger-sized recycled aggregates (Kolay and Akentuua 2014). Water absorption increases with increasing magnesium sulfate soundness loss (Cooley and Hornsby 2012). The sodium sulfate test for RCA has been waived by many U.S. highway agencies, as it disintegrates the concrete aggregate during the test (Kou et al. 2002).

Table 3.2 Sodium Sulfate Soundness Degradation (Kolay and Akentuua 2014)

Type of aggregate	Sieve size		NCA	RCA	NCRB
	Passing	Retained	Average % degraded		
Concrete Aggregate	19.0 mm	4.75 mm	13.0	54.0	15.0
HMA Concrete Aggregate	12.5 mm	2.36 mm	15.0	55.0	16.0
Surface Treatment Aggregate	12.5 mm	4.75 mm	17.0	57.0	17.0

- The micro-deval abrasion loss values (16~18%) obtained for both fine and coarse RCA are within the permissible range specified by mane DOTs (<18%), indicating satisfied durability of RCA aggregates for constructional purposes. RCA is less susceptible to micro–deval degradation compared to natural aggregates (Kolay and Akentuua 2014).
- RCA has higher Los Angeles Abrasion loss than limestone aggregates. Water absorption increases with increasing Los Angeles Abrasion loss (Figure 3.1). Abrasion resistance increases with increasing water permeable voids. RCAs have higher water absorptions than limestone (6.8% vs 1.9%) (Cooley and Hornsby 2012).

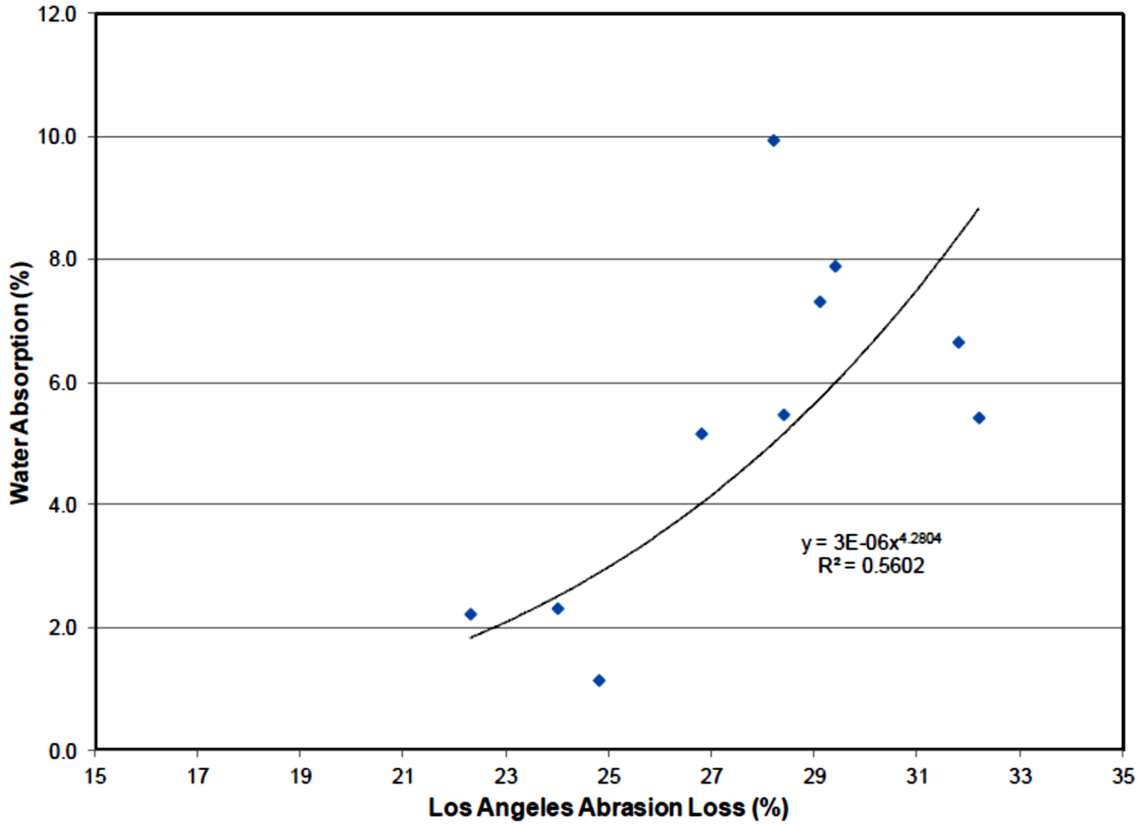


Figure 3.1 Los Angles Abrasion Loss and Water Absorption (Cooley and Hornsby 2012)

- <u>Stiffness and Strength</u>
  - Water absorption results in erratic Proctor compaction test results (determine the optimum moisture and the maximum dry density of coarse aggregate). The reliability and repeatability of Proctor compaction and strength/stiffness test specimens increase by soaking RCA materials overnight at a moisture content equal to the combined (coarse and fine fractions combined volumetrically) water absorption (Cooley and Hornsby 2012).
  - CBR (California Bearing Ratio) values for RCA:
    - ranging from 90.0 % to more than 140.0 % (Senior et al. 1994)
    - ranging from 94.0% -148.0 % from different sources (Gregory and Edil 2009)
    - ranging from 94%-102 %, which is lower than the NCRB range of 142%- 147% (Table 3.3), indicating RCA performs less satisfactorily in carrying traffic loads without excessive deformation or failure (Kolay and Akentuua 2014)
    - significantly higher value than that of NCRB material despite the higher density of the NCRB material, since residual cement in the RCA base material improves density and increases the CBR (Gabr and Cameron 2012)

Table 3.3 CBR Value of RCA Base Material and NCRB Material (Kolay and Akentuua 2014)

Penetration (mm)	RCA Base		NCRB	
	Stress (MPa)	CBR (%)	Stress (MPa)	CBR (%)
2.54	7.005	102.00	9.760	142.00
5.08	9.740	94.00	15.164	147.00
Average CBR (%)	102.00		147.00	

- CBR increases with a rising percent of standard Proctor-based maximum dry density. Average CBR is increased by 24 when the percent standard Proctor density (relative compaction) is increased from 95%- 99% (Figure 3.2), a significant improvement in the structural capacity of a pavement granular layer (Cooley and Hornsby 2012).
- RCA materials fabricated from controlled concrete sources and limestone have higher resilient modulus values (test samples fabricated using both a standard and modified Proctor compactive effort) than RCA materials fabricated from construction debris (Cooley and Hornsby 2012).
- The resilient modulus (standard and modified Proctor compactive efforts) of the materials decreased as the water absorption increased (Figure 3.3; Cooley and Hornsby 2012).
- Resilient moduli (M_R) of RCAs are 2.6 (in optimum moisture content condition) and two time higher (in maximum dry density condition) than that of the NA material. 100% RCA and 100% GAB provide higher M_R values, compared to their different combinations (Figure 3.4) (Aydilek et al. 2015). Low M_R of combined mixtures was the result of poor packing of particles and change in gradation parameters (Kazmee et al. 2012). Stiffness increases with increasing bulk stress due to the continuation of hydration (cementation) reactions in RCA during the freeze-thaw cycles (Aydilek et al. 2015).

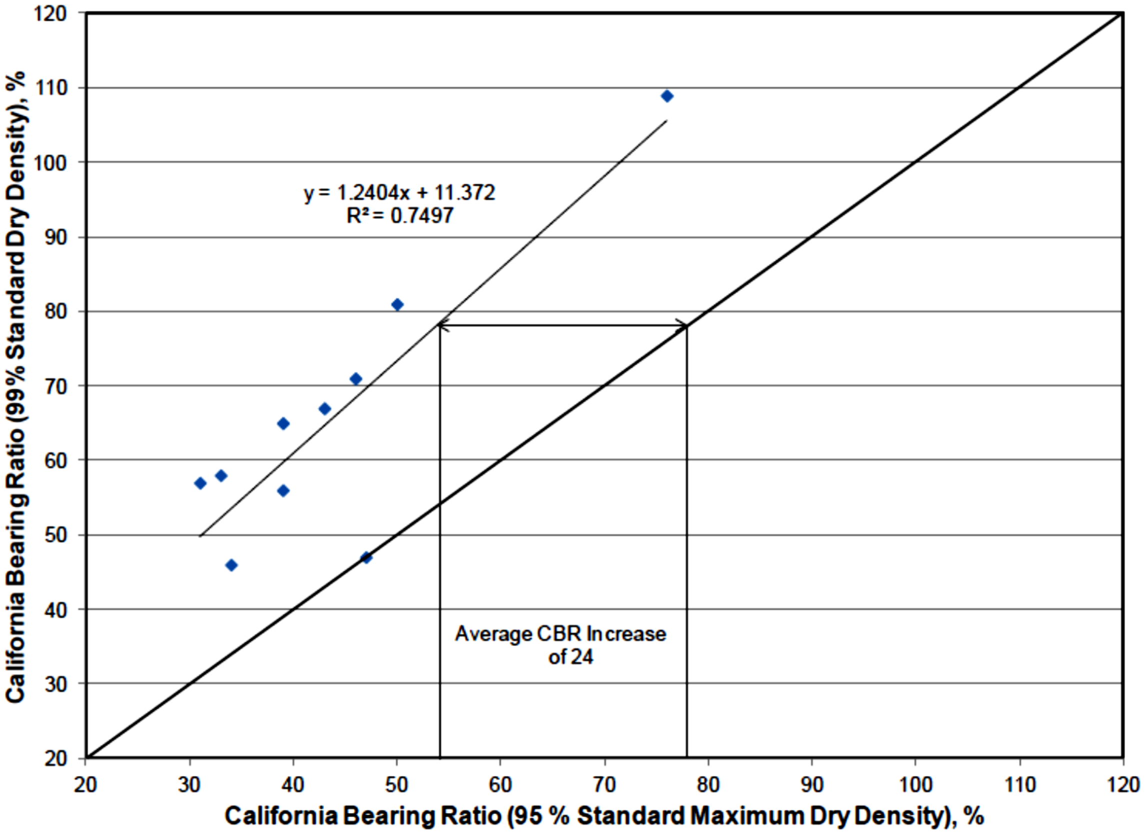


Figure 3.2 CBR at 95 and 99% of Standard Density (Cooley and Hornsby 2012)

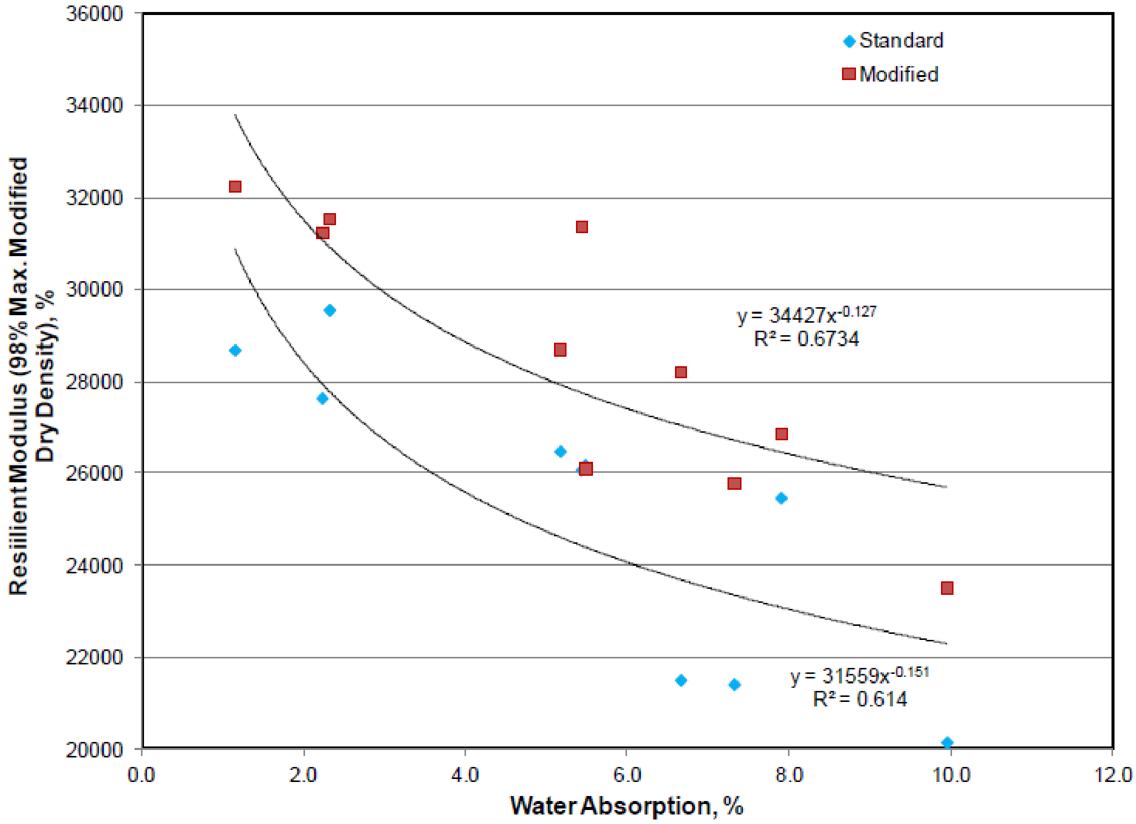


Figure 3.3 Resilience Modulus and Water Absorption (Cooley and Hornsby 2012)

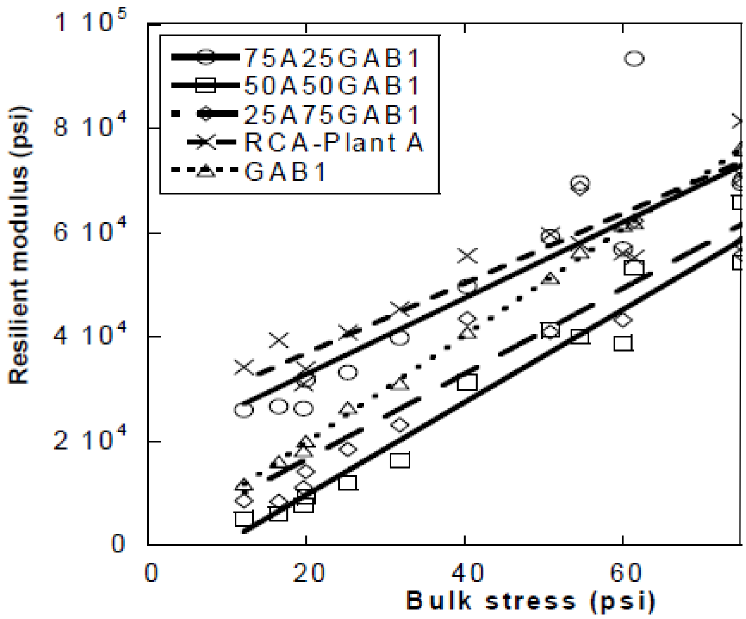


Figure 3.4 M_R for Mixtures with Varying RCA-to-GAB Ratios (Aydilek et al. 2015)

- Plastic fines significantly reduce the load carrying capacity of the granular layer, though plastic fines are highly susceptible to moisture changes. Increases in moisture can cause a significant reduction in shear strength (Cooley and Hornsby 2012).

◆ <u>Permanent Deformation</u>

- Permanent deformation of GAB increases upon mixing with RCA, suggesting low rutting resistance of GAB/RCA blends (Kazmee et al. 2012). Plastic strain in individual GAB and RCA materials, less than that of their mixtures, attributed to poor packing arrangement of particles when these two materials were mixed (Figure 3.5) (Aydilek 2015).
- One hundred percent RCA resulted in less permanent strain under repeated loads compared to conventional aggregates (Figure 3.6) (Bennert et al. 2000).
- Alkali-silica reactivity or alkali-carbonate reactivity (ASR or ACR) cause internal stress within aggregate particles, leading to fracturing and expansion of the concrete; the alkali-silica gel produced in ASR swells in moisture conditions and magnesium produced in ACR combines hydroxyl to form brucite with an increase in volume (Stark 1994, Cooley and Hornsby 2012). The volumetric increase causes fracturing of the aggregate particle leading to increased access of fluid to the interior of the particle. Concrete that has deteriorated because of alkali-aggregate reactivity (AAR) needs raised attention on reuse. Stockpiling of crushed concrete would likely serve to diminish the potential for further AAR deterioration (Cooley and Hornsby 2012).
- For unbound base courses, the degradation of individual aggregate particle will not cause overall expansion of structural material, but will cause particle breakdown leading to reduced shear strength (Cooley and Hornsby 2012).

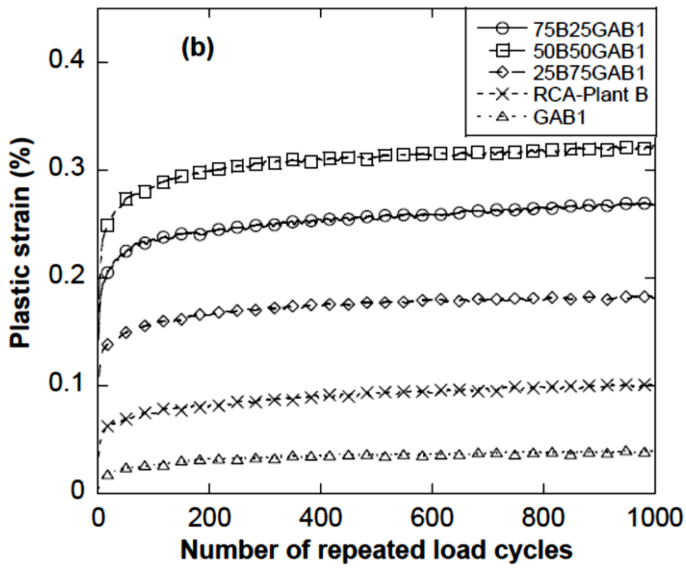


Figure 3.5 Plastic Strain of RCA and their Mixtures with GAB (Aydilek 2015)

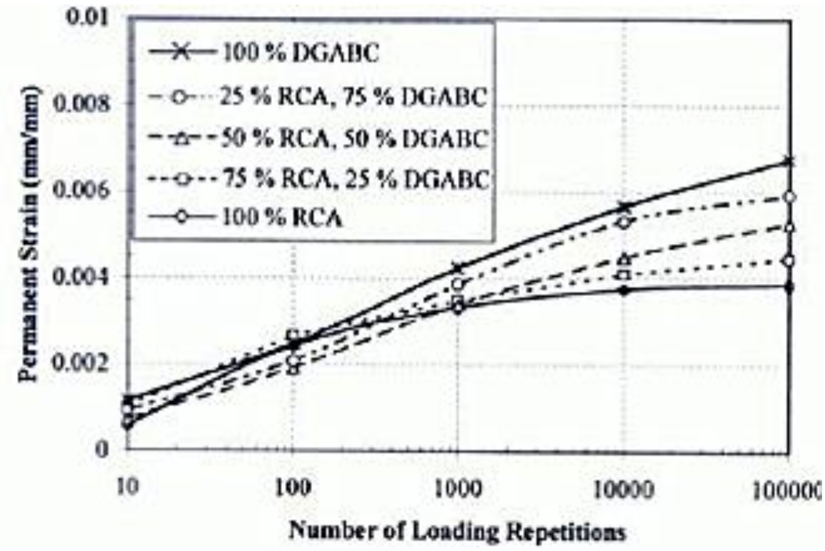


Figure 3.6 Permanent Strain Results for RCA Blended Samples (Bennert et al. 2000)

ENVIRONMENTAL PROPERTIES

- RCA within drainage base layers are likely to precipitate the calcium carbonate that reduces the permittivity of drainage filter fabrics in pavement drainage systems, though permittivity is also reduced by insoluble residue unrelated to RCA. Laboratory tests indicated calcium carbonate precipitate was proportional to the amount of RCA materials passing the No. 4 (4.75mm) sieve. Washing RCA during processing can eliminate formation of calcium carbonate precipitates. (Snyder and Bruinsma 1996).
- Effluent from drainage layers containing RCA materials are alkaline with pH level of 11 to 12. Laboratory leaching results indicated that pH levels reached a peak shortly after water was introduced and decreased over time (Snyder and Bruinsma 1996).
- High chloride contents in RCA may present problems in areas of the country where de-icing salts are used in winter maintenance operations (Chesner et al. 1998).
- Calcium (Ca), Chromium (Cr) and Copper (Cu) concentrations decreased with increased curing time, while Fe showed initial increases followed by slight decreases. Increasing curing time also caused rehydration of cement particles and generally yielded a decrease of pH. The rehydration rate of cement particles in RCA can be improved by allowing the RCA samples to cure for a longer period of time. This may eventually yield encapsulation of particles and contribute to immobilization of metals attached to RCA surface (Aydilek 2015).
- Leached concentrations generally increase with decreased particle size, since a larger surface area in small particles allows for more interaction between aqueous solution and RCA aggregate. Freezing and thawing led to self-cementing, decreased pH and Ca, Cu, Iron (Fe), Cr concentrations (Aydilek 2015). The decreased pH was caused by precipitation of Ca as $CaCO_3$ (Sanchez et al. 2009).
- Leached metal concentrations decrease with increasing L:S (liquid to solid) ratio, since increasing liquid content dilutes leachate (Aydilek 2015). For Ca, decrease is also associated with lower solubility of $CaCO_3$ mineral compared to portlandite and CaO. Carbonation may cause the precipitation of Cu (Gervais et al. 2004).
- The pH-dependent leaching tests showed a cationic leaching pattern for Ca, suggesting decreased pH will lead to more leachate (Figure 3.7). Amphoteric leaching patterns for Cr, Cu, Fe, and Zinc (Zn), implies that

leaching will reach a minimum level at neutral pH, but increase at acidic or basic conditions. Field is normally of neutral pH; the minimal leaching of Cr and Cu is unlikely to cause health issues. Fe concentrations may exceed the SMCL (Secondary Maximum Contaminant Level) which is an optional federal standard for improved taste in drinking water, hence Fe leaching from RCA may not harm environment (Aydilek 2015).

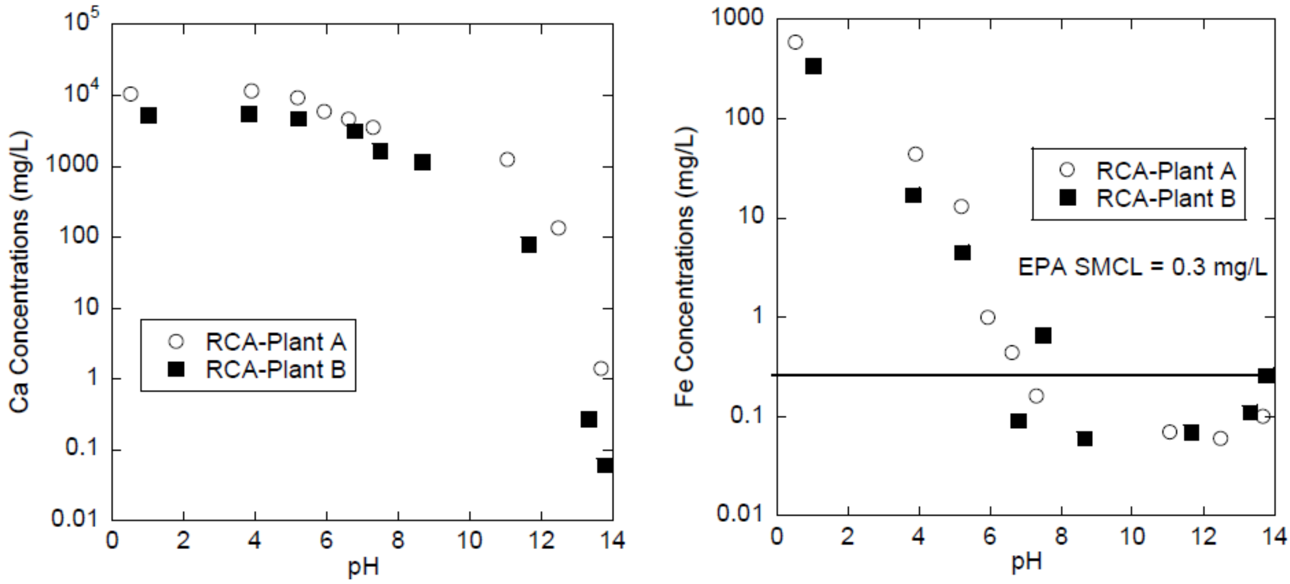


Figure 3.7 Results of pH-Dependent Leaching Test (Aydilek 2015)

DESIGN RECOMMENDATIONS

- Sufficient stability, including shear strength and stiffness, should be ensured in granular base, especially in flexible pavements. Large, angular, cubical and durable aggregates are preferred. More surface texture in angular and cubical particles provide sufficient shear strength to resist lateral displacement (deformation). Thin or elongated aggregates easily segregate and break down. Ensure pavement built with hard durable aggregates can reach its design life (Chesner et al. 1998).
- Good permeability can prevent granular base from frost heave. Layers should be free draining to avoid ice lenses developing. Prevent layer infiltrated by moisture from becoming motivation to loss of stability (Chesner et al. 1998).

FIELD RECOMMENDATIONS

- Preparation process affects RCA properties. Jaw crusher modifies particle distribution and shape. Dry and wet processes help to classify and eliminate harmful substances. Wet process is preferred to remove crushing dust. Picking belts separate large substances (particle size greater than 1.77in. - 45mm) to be crushed into small granulates (Kuo et al. 2001b).
- Use magnetic separators to remove reinforcing steel. Use impact mills to crush rubbles into various sizes. Use air classifiers to remove lightweight debris (i.e., wood and plastic). Remove dust by washing to prevent tufa (porous limestone formed from calcium carbonate) formation (Kuo et al. 2001b).

- Clean up harmful impurities such as lead and asbestos. Buildings or structures should be certified clear of asbestos before recycled to ensure RCA is asbestos free (Kuo et al. 2001a).
- Quality control requires: monitor output quality systematically and rigorously; sample and test material characteristics (including environmental properties) intensively; manage materials selection and storage effectively (Kuo et al. 2001b).

BENEFITS

- Many sources for RCA: Portland cement concrete (PCC) structures such as PCC pavements, sidewalks, curbing, building slabs and runways.
- RCA can be simply and economically recycled by crushing concrete in place with a mobile plant, though it may be better to haul demolished concrete to a central facility for stockpiling and processing before being used in a granular base (Construction & Demolition Recycling Association, 2015).
- RCA has good bearing strength and drainage properties. RCA can gain strength over time due to self-cementation. RCA helps stabilize wet, soft, underlying soils to improve strength (Construction & Demolition Recycling Association, 2015).
- RCA met all requirements for long-term performance of dense-graded aggregate base or subbase in New York projects that took place between 1977 and 1982 (Petraca and Galdiero 1984).
- RCA reduces the water and energy needed for mining virgin aggregate and reduces carbon dioxide emissions. Reusing RCA saves landfill space. RCA reduces the need for transporting natural materials from distant quarries and concrete to disposal sites, saving energy and reducing emissions (Construction & Demolition Recycling Association, 2015).

SUGGESTED SPECIFICATIONS

The following specifications have been suggested for use of RCA in base layers:

Table 3.4 Granular Aggregate Test Procedures (Chesner et al. 1998)

Property	Test Method	Reference
Gradation	Sizes of Aggregate for Road and Bridge Construction	ASTM D448/AASHTO M43
	Sieve Analysis of Fine and Coarse Aggregate	ASTM C136/AASHTO T27
Particle Shape	Flat and Elongated Particles in Coarse Aggregate	ASTM D4791
	Uncompacted Voids Content of Fine Aggregate (As influenced by Particle Shape, Surface Texture, and Grading	AASHTO T304
	Index of Aggregate Particle Shape and Texture	ASTM D3398
Base Stability	California Bearing Ratio	ASTM D1883/AASHTO T193
	Moisture-Density Relations of Soils Using a 5.5 lb (2.5 kg) Rammer and a 12-in. (305mm) Drop	ASTM D698/AASHTO T99
	Moisture-Density Relations of Soils Using a 10-lb (4.54 kg) Rammer and an 18-in. (457 mm) Drop	AASHTO T180
Permeability	Permeability of Granular Soils (Constant Head)	ASTM D2434/AASHTO T215
Plasticity	Determining the Plastic Limit and Plasticity Index of Soils	ASTM D4318/AASHTO T90
	Plastic Fines in Graded Aggregates and Soils by Use of the Sand Equivalent Test	ASTM 2419/AASHTOT176
Abrasion Resistance	Resistance to Degradation of Large-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine	ASTM C535
	Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine	ASTM C131/AASHTO T96
Resilient Modulus	Resilient Modulus of Unbound Granular Base/Subbase Materials and Subgrade Soils - SHRP Protocol P46	AASHTO T307

3.1.2 RCA in Drainage/Fill

MECHANICAL PROPERTIES

- <u>Drainage</u>
  - RCA void percentage increases with increasing particle size. Large void content allows for smaller drain dimensions (Minnesota Pollution Control Agency 2000).
  - LA abrasion is 43.7% for RCA of No.4 gradation, but varies between 32% and 38% when particles smaller than 4 mm are removed by wet sieving, indicating that RCA easily degrades and generates fines (Plesser et al. 2006).
  - The pH of RCA changes little over time, since RCA degrades during the initial period and keeps unchanged particle size afterwards. Acidic environment degrades particles more than an alkaline environment does (Plesser et al. 2006).
  - Water flow has little effect on density of RCA. Bulk density increases and then decreases in acidic environment, but tends to increase in an alkaline environment (Plesser et al. 2006).
  - Water absorption remains constant in an alkaline environment but drops greatly in an acidic environment. (Plesser et al. 2006).
  - RCA mixtures have lower strengths than virgin cement (water-to-cement ratio of 0.5), Figure 3.8, (Nam et al. 2014, Dafalla 2013).

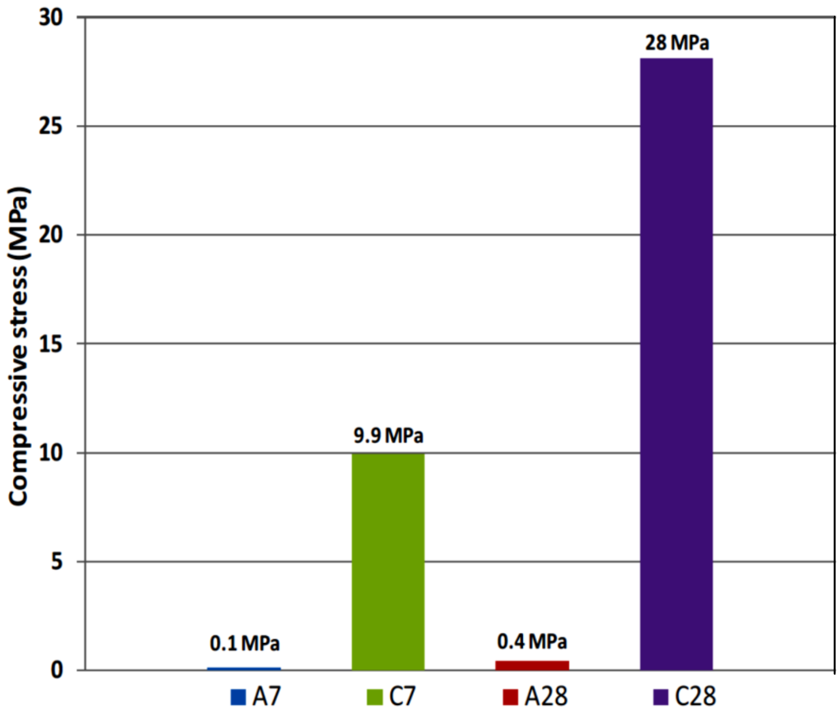


Figure 3.8 Compressive Strength Results for RCA and Virgin Cement (Nam et al. 2014)

Note. A7 and A28: RCA mixtures at 7 days and 28 days, respectively; C7 and C28: virgin cement mixtures at 7 days and 28 days.

  - RCA does not rehydrate under moisture conditions (Nam et al. 2014).

- Aggregate size dominates water flow. Increasing fine content decreases water flow, but No. 4 gradation does not block water flow. A linear relationship exists between flow rate and head diameter (Nam et al. 2014).
- Reducing fine particles can improve permeability, but they also reduce stability of drainage layer. (Nam et al. 2014).

◆ <u>Flowable Fill</u>

- Flowable fill with CCA (crushed concrete aggregate) requires more water to meet given flow value (8 in.), compared to mixtures made with concrete sand, since CCA contains a substantial amount of fine particles (Lim et al. 2003).
- Entraining air into CCA mixtures is not economical, since in order to entrain 23% air into flowable fill mixtures, CCA requires 10 times more air entraining agent than concrete sand (Table 3.5) (Lim et al. 2003).

Table 3.5 Air Entraining Agent Dosage for Flowable Fill Mixtures (Lim et al. 2003)

Mix ID	Unit Weight (lb/ft^3)	Air (%)	Air Entraining Agent (oz/yd^3)	Bleedwater (%)
AE/50/100	111.6	7.5%	10.09	0.75%
AE/50/50	114.8	11.0%	4.94	0.81%
AE/50/0	106.7	23.0%	0.78	0.87%
FA/50/100	116.7	2.0	0	0.61%
FA/50/50	1.28.3	2.0%	0	0.33%
FA/50/0	135.8	3.0%	0	0.70%
FA/100/100	117.9	2.0%	0	0.21%
FA150/100	119.4	2.0%	0	0.11%
FA/200/100	118.5	2.0%	0	0.22%

Note. Mix Type[a] / Cement Content / Aggregate[b]; [a]AE = Air Entrained, FA = Fly Ash;
[b]100, 50 = CCA, 0 = Concrete Sand.

- Air-entrained flowable fill mixtures containing CCA are unable to develop enough penetration resistance. Splitting tensile strengths are consistently low over time and are unaffected by the addition of CCA. Compressive strengths are very low and are also unaffected by the addition of CCA (Lim et al. 2003).
- The addition of fly ash improves long-term strength, as well as cohesion and ductility of mixtures with CCA, because of the pozzolanic reaction between fly ash and calcium hydroxide from CCA. Fly ash/CCA flowable fill mixtures take a longer time to develop penetration resistance than mixtures containing concrete sand, due to increasing water demand of CCA. Splitting tensile strength of the mix is lower than that of concrete sand mix, because of increased water content in the mixtures with CCA. Compressive strengths of the mix are lower than that of concrete sand mix, due to increased

water demand of the mixtures containing CCA (Lim et al. 2003).

- Penetration resistance increases as cement content increases (Figure 3.9; Lim et al. 2003).

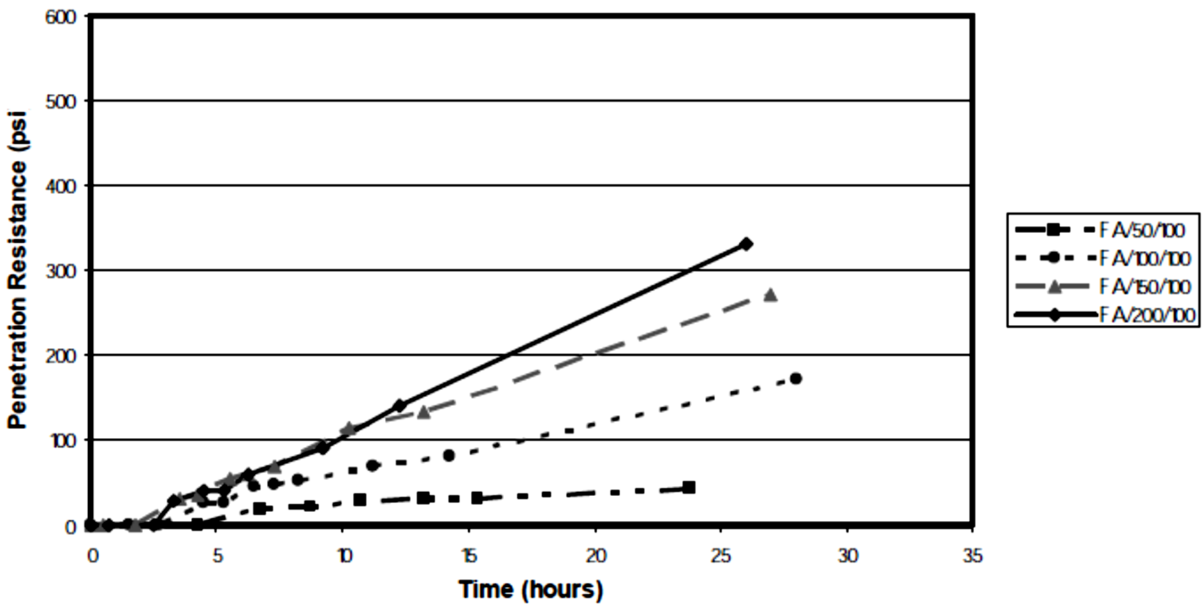


Figure 3.9 Setting Time Graph for Fly Ash Flowable Fill Mixtures (Lim et al. 2003)

Note. Mix Type[a] / Cement Content / Aggregate[b]; [a]AE = Air-Entrained, FA = Fly Ash; [b]100, 50 = CCA, 0 = Concrete Sand.

- Splitting tensile strength increases with a higher cement content. Increased cement content results in a higher splitting tensile strength of CCA mixtures than that of concrete sand mixtures. Since high splitting tensile strength is detrimental to excavation, high cement content is not advisable (Lim et al. 2003).
- Compressive strength also increases with a higher cement content. This results in a higher compressive strength of CCA mixtures than that of concrete sand mixtures. Since flowable fill materials do not require high strength, high cement content is not advisable (Lim et al. 2003).
- For any given cement content, air-entrained or fly ash/CCA flowable fill materials are more ductile and reach ultimate strength with larger deflections. As cement content increases, mixtures containing CCA show a decrease in ductility and increase in strength (Lim et al. 2003).
- Mixtures containing CCA have a similar load-deflection trend with mixtures containing concrete sand (Figure 3.10).

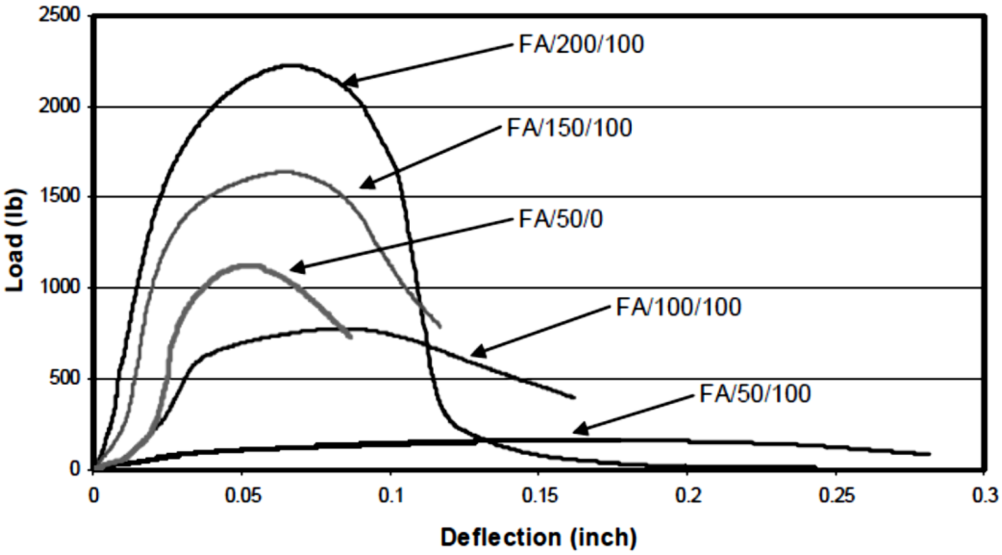


Figure 3.10 Load-Deflection Response of Fly Ash Mixtures at 28 days (Lim et al. 2003)

Note. Mix Type[a] / Cement Content / Aggregate[b]; [a]AE = Air Entrained, FA = Fly Ash;
[b]100, 50 = CCA, 0 = Concrete Sand.

ENVIRONMENTAL PROPERTIES

- RCA mixtures have an initial pH of 12.5 but quickly decrease to pH 12.3 in the first 24 hours, at which point they keep relatively constant at pH 12.1 (Figure 3.11) (Nam et al. 2014). An initial high pH is likely due to already dissolved calcium, sodium and potassium hydroxides. Calcium carbonate then precipitates, leading to decreasing pH (Steffes 1999).

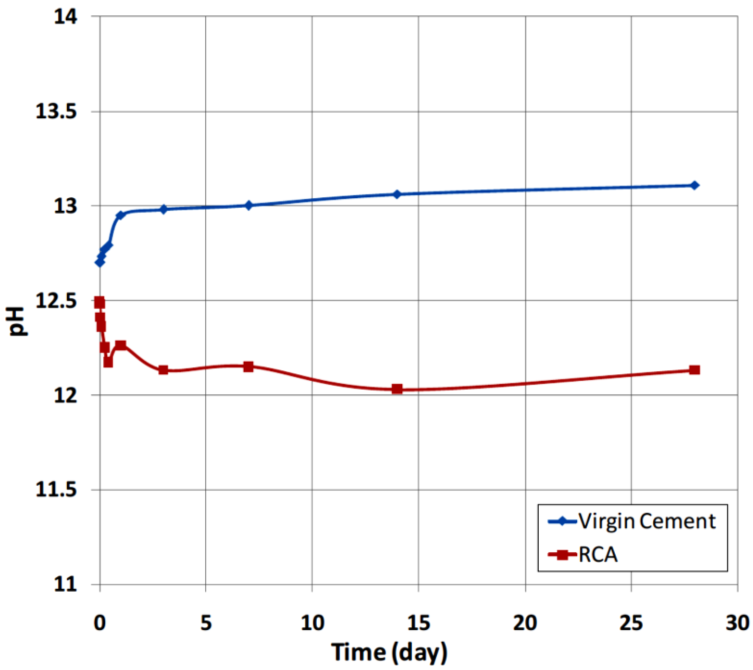


Figure 3.11 pH Testing Results for Virgin Cement and RCA (Nam et al. 2014)

- Mass loss exists in acidic or alkaline environment because of cement dissolution (Nam et al. 2014).
- Concentration of silicon and calcium in drainage water is relatively constant over time at both acidic and alkaline levels (Figure 3.12; Nam et al. 2014).

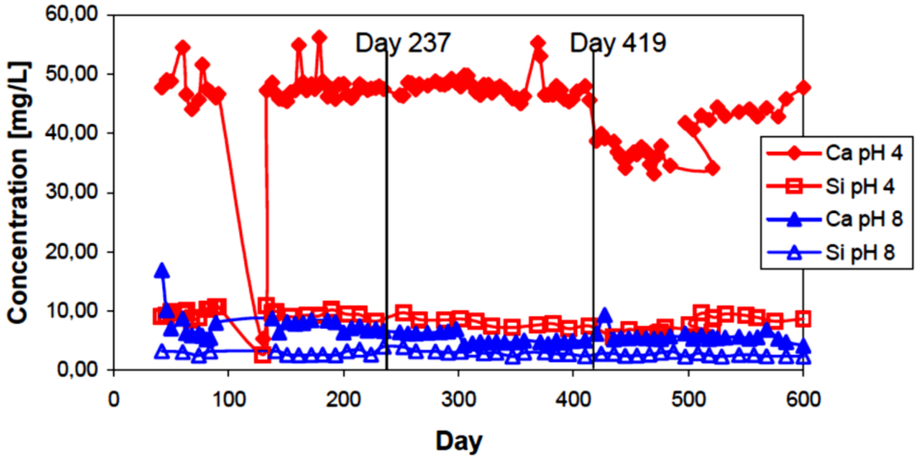


Figure 3.12 Calcium and Silicon Concentration in Drainage Water (Nam et al. 2014)

- RCA precipitates more calcite than limestone, since limestone aggregate and hydrated cement paste included in RCA contribute to more calcium ion. Higher percentage of fines can produce more calcite. Calcium carbonate can be reduced by washing RCA several times or reducing usage of hydrated cement (Figure 3.13) (Nam et al. 2014).

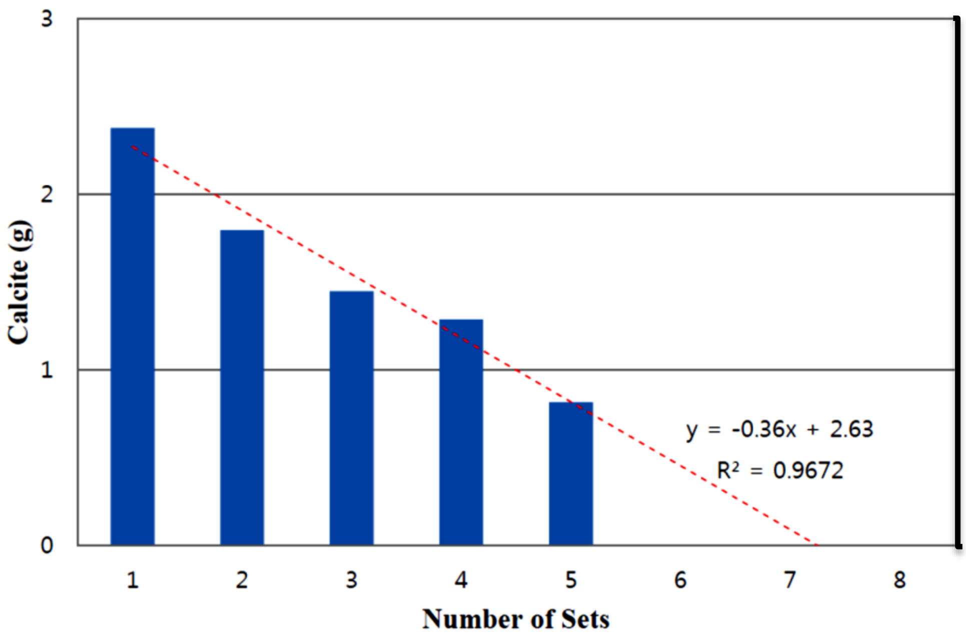


Figure 3.13 Decreasing Trend of Consecutive Calcium Carbonate Precipitation Cycles and Predicted Calcium Carbonate Reduction, 1 set = 6 Calcite Simulation Cycles (Nam et al. 2014)

RECOMMENDATIONS

- Impurities included in RCA should be limited to gain high quality and consistency. (Gonzalez 2002).
- Take care of the un-hydrated cement contained in RCA, which may alter its properties and complicate stockpiling (Snyder and Bruinsma 1996).
- Leaching of calcium hydroxide from RCA may clog filter fabrics when used as a drainage layer or near a water source, since it will react with atmospheric carbon dioxide forming calcium carbonate (Snyder and Bruinsma 1996).
- Material transporting, handling and storage need additional care to avoid segregation of coarse and fine aggregates, which make RCA mixtures difficult to work (Dam et al. 2011).
- Stockpiles should be separated from water courses to avoid contamination with highly alkaline leachate (Gonzalez 2002).

BENEFITS

- RCA use reduces the need for natural aggregate and landfill disposal (Dam et al. 2011).
- RCA use reduces cost and energy to only demolish and remove old concrete, and to crush and process demolition (Dam et al. 2011).
- Fuel consumption and transportation costs are reduced if RCA is recycled on site (Dam et al. 2011).

SUGGESTED SPECIFICATIONS

Table 3.6 Aggregate Specification Tests on RCA (Mills-Beale and You 2010)

Physical property	Coarse aggregate	Fine aggregate
Gradation	ASTM C136-96a	ASTM C136-96a
Specific gravity	ASTM C 127-88/ AASHTO T-85	ASTM C 128-93/ AASHTO T-84
Absorption	ASTM C 127-89	ASTM C 128-93/ AASHTO T-84
Uncompacted void content	–	AASHTO T 326
Flat and elongated particles	ASTM 4791	–
Fractured faces	ASTM D5821	–
LA abrasion	ASTM C535/AASHTO T96	–

3.1.3 RCA in HMA

MECHANICAL PROPERTIES

- Characteristics of RCA
    - RCA particles consist of original natural aggregate and a partially covered mortar layer. Attached cement is more porous and less dense than original natural aggregate; it has a weak bonding with the natural aggregate, resulting in lower density (low bulk-specific dry and saturated surface-dry density), higher water absorption, increased Los Angeles abrasion loss and higher sulphate content (de Juan and Gutierrez 2009, Tam et al. 2007). .
    - Attached mortar has variable thickness, composition, porosity, and texture, leading to variable RCA properties (Tam et al. 2007). Mortar content can be diminished by increasing the number of crushing processes to improve aggregates quality, though production costs will increase (Pasandin and Perez 2015).
    - Other materials contained in RCA, such as mortar fragments, stones and aggregates without mortar, and other impurities such as gypsum or metals, diminish RCA's heterogeneity.
    - Small cracks produced in the crushing process also degrade the properties of RCA (Lee at al. 2012).

- Marshall Mix Design
    - The Marshall method is used to select the asphalt binder content at a desired density that satisfies stability and flowability requirements. Parameters in design include optimum asphalt contents (OAC), air voids (Va), voids in the mineral aggregate content (VMA), voids filled with binder (VFB), Marshall stability and Marshall flow (Pasandin and Perez 2015).
    - HMA using RCA has a higher OAC than conventional mixtures. OAC increases linearly with RCA content (Figure 3.14), since high absorptivity and porous structure of RCA have more voids and greater surface area to absorb asphalt cement (Bushal et al. 2011, Wong et al. 2007).

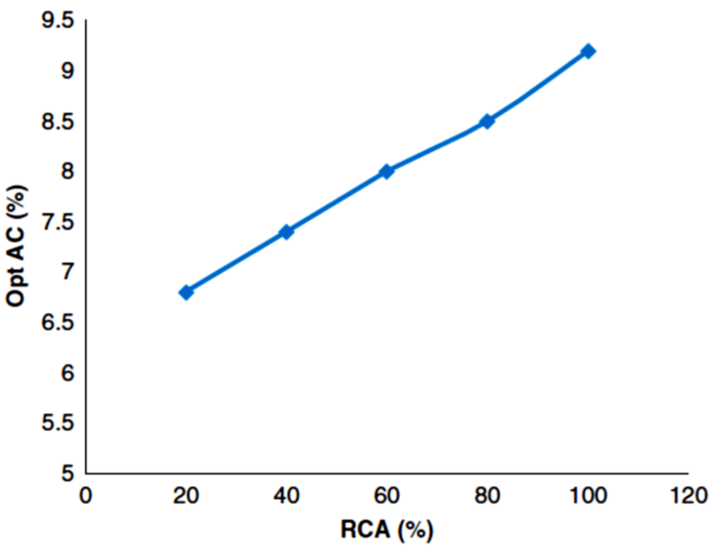


Figure 3.14 OAC at Different RCA Percentages (Bushal et al. 2011)

- Longer curing time allows aggregate to absorb more binder, leading to higher bitumen consumption (Pasandín and Pérez 2014).
- Fine RCA has a high OAC because of greater absorption capacity and a larger specific surface area. Coarse RCA can prevent high OAC, which is economically advisable (Bushal et al. 2011).
- Other materials used (natural aggregates and fillers) influence asphalt consumption (Pasandin and Perez 2015).
- HMA with RCA has higher air voids,Va, (3% to 5% higher) than conventional mixtures, since pores of RCA absorb more asphalt binder, leaving less asphalt binder to fill up voids (Paranavithana and Mohajerani 2006, Pérez et al. 2007).
- Air voids rise with increasing RCA content (Figure 3.15). Fine aggregates have more air voids compared with coarse aggregates, since greater surface area of fine aggregates absorb more asphalt, leaving less asphalt to fill pores (Rafi et al. 2011).

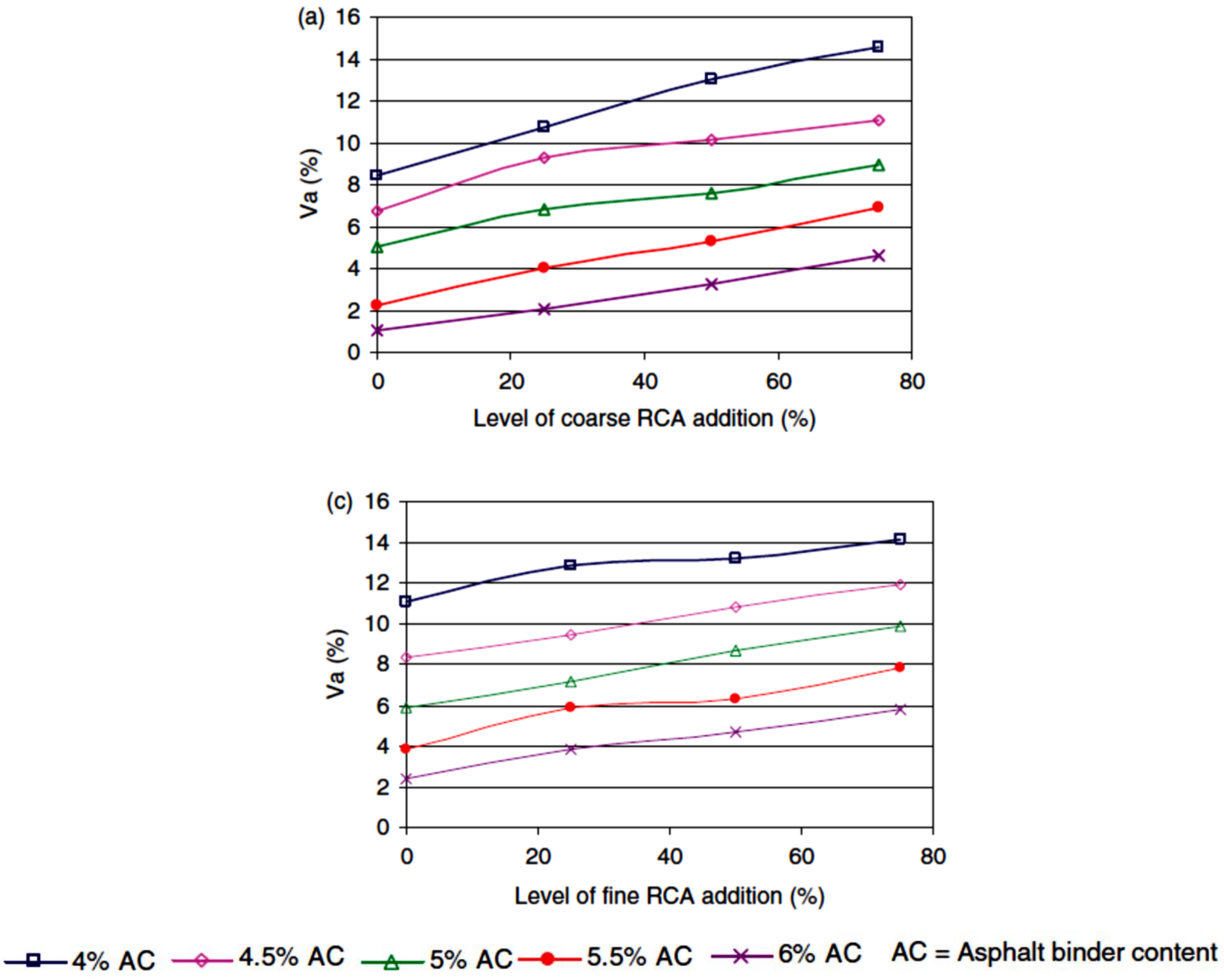


Figure 3.15 Variation in Air Voids of Mix with Trial Asphalt Binder Content (Rafi et al. 2011)

- Longer aging time reduces air voids, since bitumen cannot completely fill RCA pores in a short time (Pasandín and Pérez 2014).
- Because RCA absorbs greater amounts of bitumen, it produces a thinner film around the aggregate (Pasandín and Pérez 2015). Thin asphalt film results in better stiffness, permanent deformation resistance and low resistance to moisture damage (Zulkati et al. 2013).
- Lower voids in mineral aggregate content (VMA) imply lower effective asphalt, making the mixture more prone to moisture and aging damage. VMA of mixes containing RCA is lower than conventional mix due to the higher absorption of RCA. VMA increases with higher binder content after lower VMA value point, Figure 3.16 (Paranavithana and Mohajerani 2006).

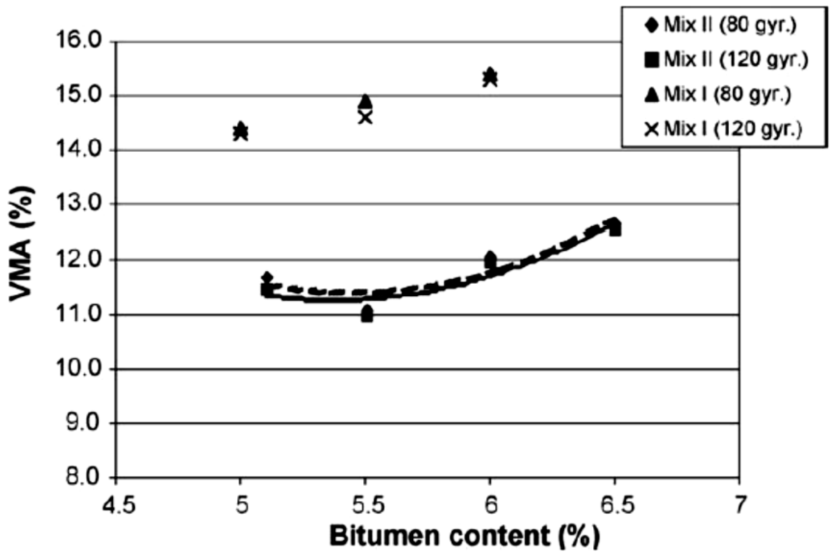


Figure 3.16 Effect of Bitumen Content and Compaction Effort on VMA (Paranavithana and Mohajerani 2006)

Note. Mix II contains RCA as coarse aggregates and Mix I contains natural aggregates.

- VMA reduces with increasing compactive effort since a reduction in air voids is observed as mixture compaction increases (Paranavithana and Mohajerani 2006).
- VMA increases with increasing RCA content (Figure 3.17). Fine aggregates produce a higher VMA than coarse aggregates (Rafi et al. 2011).
- Voids filled with binder (VFB) for mixes with RCA are lower than conventional asphalt mixtures due to the higher absorption of RCA (Paranavithana and Mohajerani 2006).
- The Marshall S/F (stability/flow) ratio is lower as the RCA percentage increases, implying a lower resistance to permanent deformation (Pérez et al. 2012).
- RCA coated with bitumen emulsion has adequate volumetric properties to reach compliance with required traffic categories T1~T4, according to Superpave PG-3 specifications (MOD 2015). Bitumen content influence the mixtures' ability to serve low or heavy traffic level. RCA mixtures require higher bitumen and filler content, allowing better moisture resistance to meet PG-3 specifications for roads with light traffic (T4) (Pérez et al. 2007, Pasandin and Perez 2014).
- RCA coated with slag cement paste may produce a lower Marshall stability (Lee et al. 2012). Heat-treated RCA mixtures will produce lower Marshall stability (Wong et al. 2007).

- The Marshall mix-design method may be insufficient in designing mixtures with RCA, since the compaction approach used in Marshall mix-design may fracture RCA coarse particles, causing lower values of HMA properties (Cho et al. 2011).

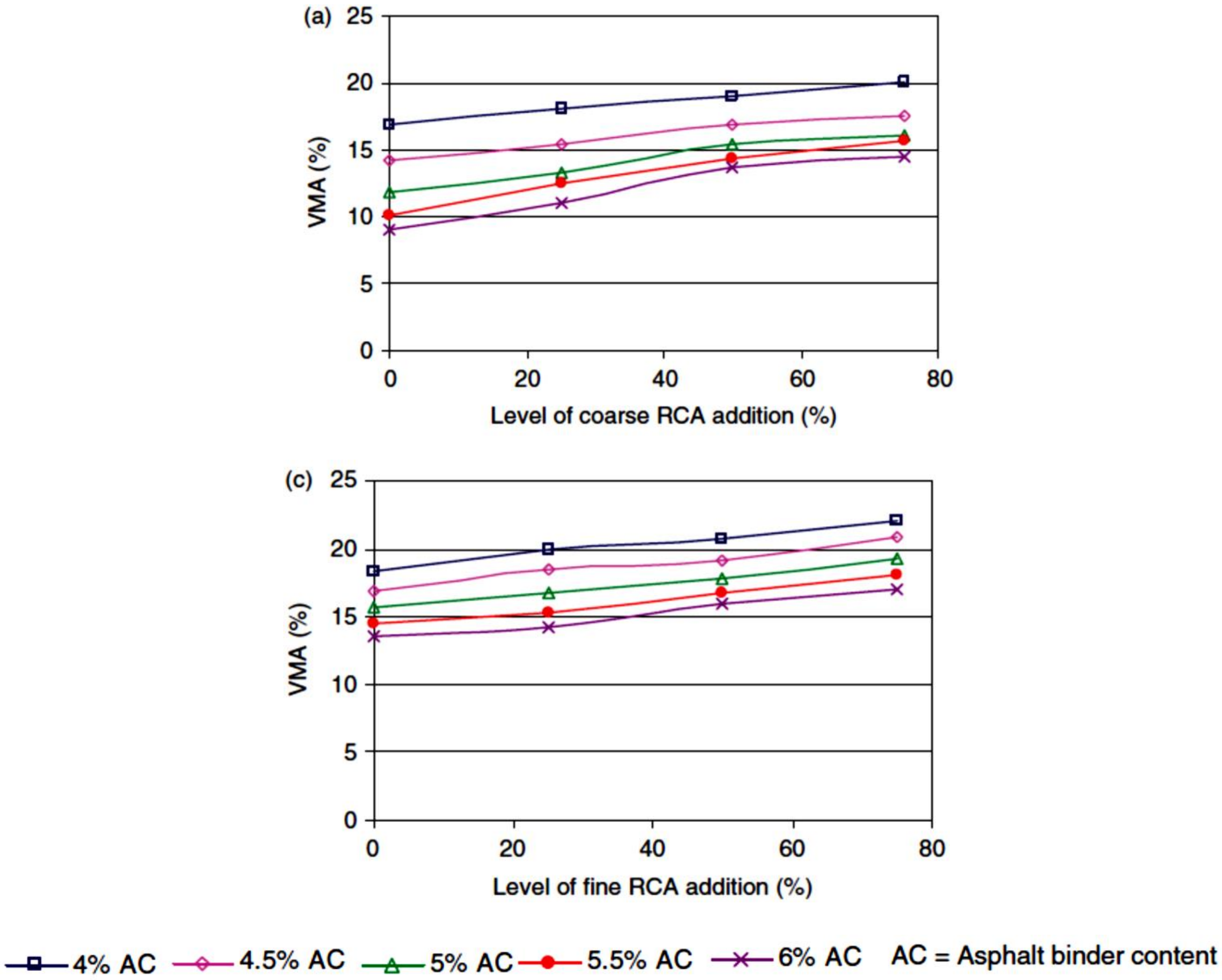


Figure 3.17 VMA of (a) Coarse RCA Mix and (c) Fine RCA Mix (Rafi et al. 2011)

- <u>Stiffness and strength</u>
  - HMA mixes containing RCA as coarse aggregate have lower stiffness compared to conventional mixes, due to the low strength mortar attached to the RCA particles. Stiffness decreases with increasing binder content or increasing RCA content (Figure 3.18) (Paranavithana and Mohajerani 2006, Mills-Bales and You 2010).
  - Resilient modulus (M_R) of HMA with RCA is more temperature dependent than conventional mixtures. M_R increases with decreasing temperature (Figure 3.19), due to viscosity of the asphalt binder (Mills-Beale and You 2010, Arabani et al. 2012b). Temperature showed greater effect on M_R than the percentage of RCA in mixes. Another study indicated that stiffness is dominated by the binder at high temperatures, and by mineral skeleton at low temperatures (Chen et al. 2013).
  - Higher compaction level increases M_R and improves load-spreading capacity (Mills-Beale and You 2010).

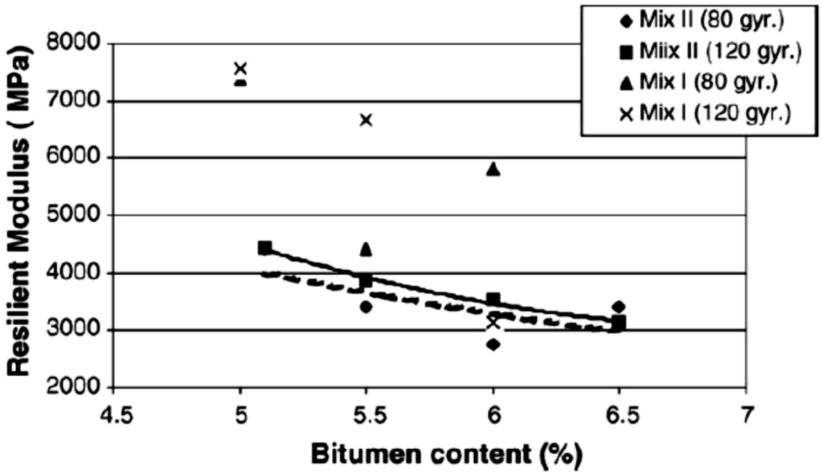


Figure 3.18 Effect of Bitumen and Compaction Effort on M_R (Paranavithana and Mohajerani 2006)

Note. Mix I (conventional HMA, Mix II (HMA with RCA as coarse aggregate).

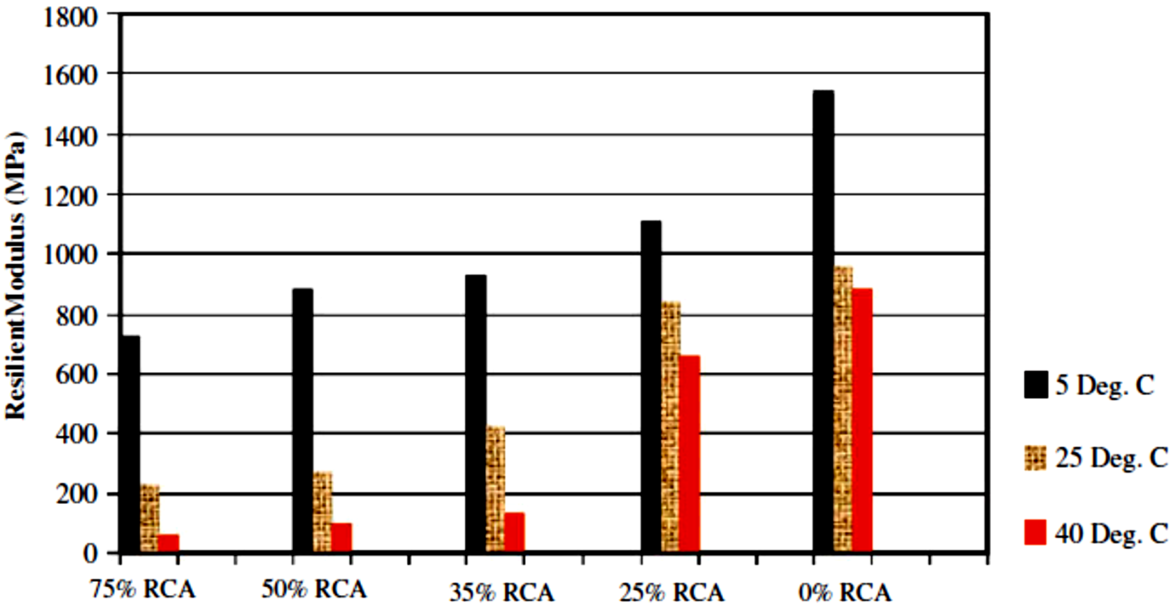


Figure 3.19 M_R Test Result (Mills-Beale and You 2010)

- HMA with RCA has lower dynamic moduli (E*) than conventional HMA mixtures. Increasing RCA percentage decreases dynamic modulus (Figure 3.20) because of the lower stiffness of the attached mortar (Bhusal and Wen 2013, Paranavithana and Mohajeranie 2006, Mills-Beale and You2010).
- However, another study indicated that mixtures with RCA have higher M_R than conventional HMA, since structural integrity is improved by automatic breakdown of friable concrete fillers and fines, generating more (or even finer) fillers that fill voids in HMA (Wong et al. 2007). Yet, some studies indicated that using RCA as filler does not influence M_R (Chen et al. 2013). In still another study, it was concluded that binder and RCA content do not affect M_R, which eventually was attributed to a low RCA content (i.e., between 0% and 30%) used in that research. (Pasandín and Pérez 2014).

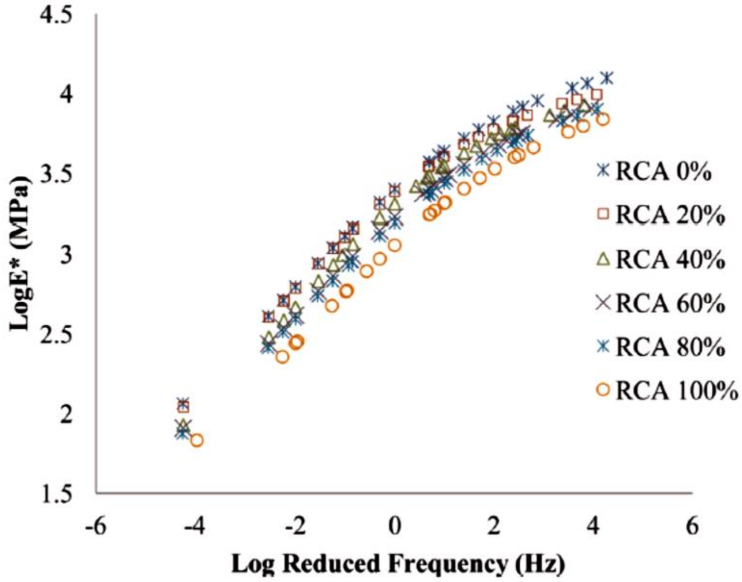


Figure 3.20 Dynamic Modulus Master Curves of RCA Asphalt Mixes (Bhusal and Wen 2013)

- Morphology of cement-treated concrete fillers shows an irregular and porous structure, which leads to lower M_R (Wong et al. 2007).
- The M_R of RCA coated with bitumen emulsion similar to those of conventional mixtures may increase RCA percentages, leading to a reduction in HMA stiffness. Smaller cariations of M_R are observed at different temperatures, thus implying a uniform HMA behavior. HMA stripping is improved because of better chemical affinity between RCA and bitumen (Pasandin and Perez 2014).
- HMA with fine RCA has higher M_R because of the angularity of RCA particles, whereas coarse RCA has lower M_R due to weak attached mortar (Figure 3.21; Arabani et al. 2012a, Arabani et al. 2012b).

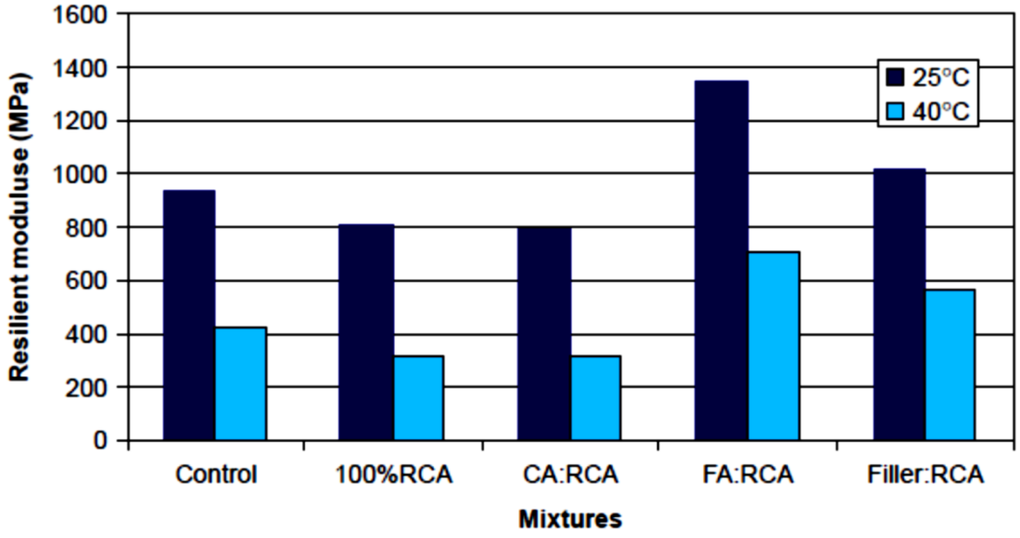


Figure 3.21 Comparison of M_R at 25℃ and 40℃ for HMA with RCA Aggregate (Arabani et al. 2012b)

- HMA with fine RCA exhibits higher fatigue life than HMA with limestone powder. Using fine RCA filler can also reduce low temperature cracking resistance and creep strain. Mixtures with fine RCA filler have higher stiffness at higher temperatures (Chen et al. 2013).
- One hundred percent RCA replacement of virgin material improves fatigue life of asphalt mixtures due to more angularity of RCA, which contributes to high frictional and abrasion resistance (Nejad et al. 2013).
- RCA reduces low temperature performance of HMA, i.e., resisting thermal cracking at low temperatures (Wu et al. 2013, Zhu et al. 2012). RCA and asphalt content affect HMA low-temperature performance (Bushal and Wen 2013).
- Mixes made with cement filler are stiffer than mixes using natural aggregate filler plus lime filler, since lime absorbs moisture and/or chemically reacts with the mortar of RCA (Pérez et al. 2012).

◆ Durability

- Moisture damage resistance depends on the content and source of RCA. Moisture resistance decreases with increasing RCA contents (Pasandin and Perez 2015).
- Anti-stripping agents improve moisture resistance of HMA with RCA. Increasing the percentage of anti-stripping agent improves TSR, while increasing the RCA percentage has the opposite effect (Table 3.7) (Bhusal and Wen 2013).

Table 3.7 Moisture Sensitivity Test Results for RCA Mixes (Bhusal and Wen 2013)

	RCA Percentage					
	100 %	80 %	60 %	40 %	20 %	0 %
Anti-Strip, %	TSR	TSR	TSR	TSR	TSR	TSR
0	0.76	0.77	0.80	0.82	0.87	0.88
0.25	0.81	0.83	0.81	0.86	0.89	0.89
0.5	0.93	0.97	0.90	0.93	0.90	0.92
0.75	0.96	0.96	1.00	0.97	0.93	0.97
1	0.99	1.01	1.03	1.00	1.01	1.01

- Asphalt mixture with fine RCA has better moisture resistance compared to limestone powder, since lower specific gravity of fine RCA needs higher volume to meet the required weight. A higher volume of mixture has higher absorption to asphalt binder, resulting in better water resistance of asphalt mixture (Chen et al. 2013).
- RCA coated with liquid silicone resin has higher water absorption and fracture resistance, resulting in greater moisture damage resistance (Zhu et al. 2012).
- RCA coated with 5% bitumen emulsion has higher water resistance, since bitumen emulsion obstructs pores, preventing water entry. Coating treatment also strengthens mortar, preventing further fragmentation that could create new pathways for water. Rutting performances and fatigue resistance are improved, which are similar to conventional mixtures (Pasandin and Perez 2014).

- Coating RCA is difficult during the mixing process, particularly for siliceous particles and quartzite. High absorption capacity of mortar leaves less effective binder to cover aggregates. The rough texture of RCA introduces additional difficulties in coating (Perez et al. 2012).

◆ Permanent Performance

- One study indicated that rutting or permanent deformation increases as RCA content increases (Figure 3.22) (Mills-Beale and You 2010, Bhusal and Wen 2013). However, another study indicated that HMA with RCA performed better than conventional HMA in respect to permanent deformation (Perez et al. 2007).
- Mixtures with RCA in both fractions (coarse and fine) display higher resistance to permanent deformation than natural aggregates, though the use of only fine RCA in HMA reduces resistance to permanent deformation (Zhu et al. 2012, Gul 2008). However, another study indicated that using RCA in both coarse and fine fractions has worse performance against permanent deformation, compared with only coarse or fine fractions (Cho et al. 2011).
- RCA content does not have a significant effect on permanent deformation over time (Pasandín and Pérez 2014).
- One study showed that using RCA as filler improves resistance to permanent deformation (Chen et al. 2013). Another study indicated that RCA as filler has no effect on permanent deformation (Wong et al. 2007).

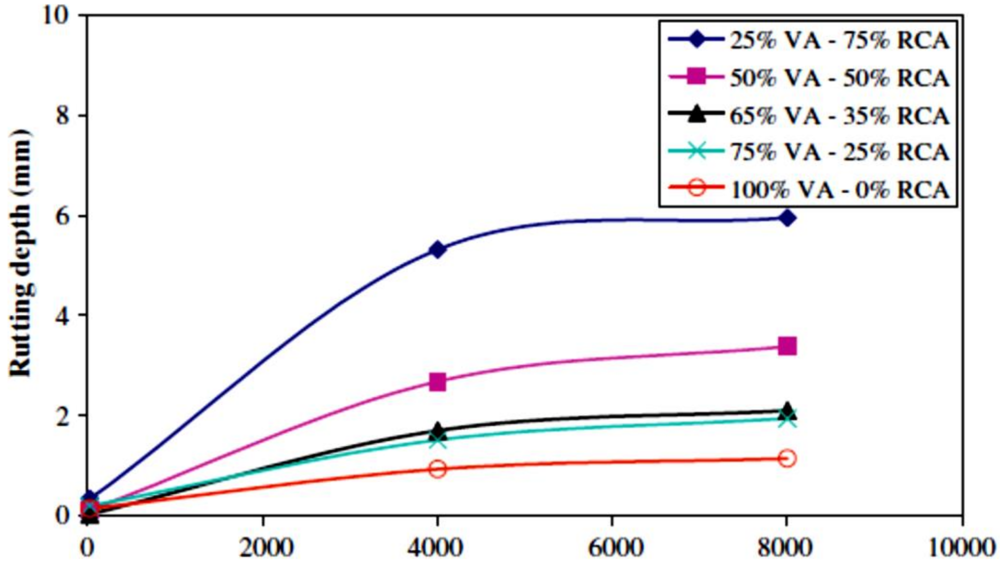


Figure 3.22 Rutting Depth of HMA Over 8000 Cycles Loading (Mills-Beale and You 2010)

ENVIRONMENTAL PROPERTIES

- ◆ Fine RCA diluted in water increases pH (Wong et al. 2007). Increased pH is the result of forming soluble calcium hydroxide produced by a hydration reaction in RCA cement residual.
- ◆ In HMA, leachates are avoided because aggregates are coated with bitumen, which is water-impermeable (Pasandín and Pérez 2014).

RECOMMENDATIONS

- Air void can be reduced by mixture compaction to reduce asphalt binder requirements and improve durability. Coarse RCA aggregates in HMA can prevent a high OAC and thus provide an economic benefit advantage (Mills-Beale and You2010).
- Fine RCA may be stiffer than coarse RCA and can work as filler in HMA (Chen et al. 2013).
- Lower water resistance of HMA with RCA can be improved by pretreating RCA with different sealants (i.e., bitumen emulsion, slag cement paste, liquid silicone resin), calcinating RCA, or heating the mixture in an oven prior to compaction (Pasandin and Perez 2015). An anti-stripping additive is advisable (Bhusal and Wen 2013).
- Marshall-mix design method can lead to underestimating HMA properties, since dynamic loading in Marshall test compaction method may increase friction between RCA and asphalt mixtures as a result of the breakdown of RCA particles (Cho et al. 2011).
- Most studies used national requirements for conventional mixtures. However, new specifications are required for a specific location to account for the use of RCA in specific roads and heavy traffic conditions (Pasandin and Perez 2015).

BENEFITS

- Reduce the need for quarrying and landfill sites, energy consumption, and greenhouse gas emissions in asphalt paving (Pasandin and Perez 2015).
- Density of HMA with RCA is lower, which means a lower mass of mixture is required (Pasandin and Perez 2015).

SUGGESTED SPECIFICATIONS

Table 3.8 Aggregate Specification Tests on RCA (Mills-Beale and You 2010)

Physical property	Coarse aggregate	Fine aggregate
Gradation	ASTM C136-96a	ASTM C136-96a
Specific gravity	ASTM C 127-88/ AASHTO T-85	ASTM C 128-93/ AASHTO T-84
Absorption	ASTM C 127-89	ASTM C 128-93/ AASHTO T-84
Uncompacted void content	–	AASHTO T 326
Flat and elongated particles	ASTM 4791	–
Fractured faces	ASTM D5821	–
LA abrasion	ASTM C535/AASHTO T96	–

Table 3.9 Performance Tests on HMA RCA Mixtures (Mills-Beale and You 2010)

Mix property	Test standard specification	Test conditions
Rutting failure	AASHTO TP63-03	52 °C 8000 cycles
Dynamic modulus (E^*)	AASHTO TP62-03	4, 21.3 and 39.2 °C 25, 10, 5, 1 and 0.1 Hz
Moisture susceptibility	ASTM D 4867/D 4867-M04	25 °C
IDT resilient modulus	ASTM D4123-82	5, 25 and 40 °C 200 load repetitions

3.1.4 RCA in PCC

MECHANICAL PROPERTIES

- RCA Properties (specific gravity, absorption, Los Angeles abrasion, ASR)
  - The specific gravity of RCA ranges from 2.1 to 2.4, due to the permeable mortar around the natural aggregate which typically ranges between 2.4 to 2.9 (Snyder 2006).
  - Absorption capacity of RCA is 3.7% to 8.7%, more than that of natural aggregate (NA) which ranges from 0.8% to 3.7% (Snyder, 2006). Greater absorption capacity of RCA can reduce the water-cement ratio (Garber et al. 2011).
  - Mass loss in Los Angeles abrasion test for RCA is 20-45% compared to 15-30% for NA, which indicates the softness of the RCA aggregate. Low mortar-to-aggregate bond strength also weakens stiffness of RCA aggregates (Amorim et al. 2012). RCA reduces stiffness of PCC mixture (Snyder 2006).
  - RCA promotes alkali-silica reaction (ASR), producing internal pressure and cracking in concrete (Snyder 2006). The crushing process exposes more internal surface, facilitating the chemical reactivity. RCA experiencing ASR during its primary service life has significant potential for expansion (Ideker et al. 2011).

- Fresh Concrete Properties (slump, permeability, air content)
  - RCA replacement for coarse aggregate decreases workability of fresh concrete, since more friction in RCA aggregates is caused by angular shape, rougher surface and reduced water-cement ratio (Amorim et al. 2012, Garber et al. 2011).
  - Higher rapid slump loss occurs from the high absorption capacity of RCA, which can be balanced by wet treatment and density separation of RCA fines (Snyder 2006, Weimann and Muller 2004).
  - Permeability of RCA PCC is about five times that of conventional PCC, which can be mitigated by reducing the water-cement ratio by 0.05 to 0.1, or by substituting fly ash or slag cement for part of the cement.
  - High porosity and permeability increase carbonation of RCA PCC. In turn, carbonation depth prompts water absorption.
  - The air content of concrete mixtures with coarse RCA are slightly higher and more variable than those with only NA, since adhered mortar causes increase in air content and greater porosity to RCAs. It has been suggested that adhered mortar should be removed as much as possible prior to using RCA in concrete (Snyder 2006).

- Hardened Concrete Properties (strength, rupture, shrinkage, thermal expansion, creep)
  - Compressive strength of concrete incorporating coarse RCA is about the same, if not slightly lower, than with only NA, since i) RCA has better interfacial transition zone with the new cement paste and ii) the possible presence of unhydrated cement on the RCA (Snyder 2006, Wen et al. 2014, Amorim et al. 2012). Fly ash added to RCA PCC improves long-term strength, despite having the similar average 28-days ultimate strength (Figure 3.23; Wen et al. 2014).
  - Coarse RCA reduces the modulus of rupture (MOR) of a concrete mixture by up to 8% because of the increased air content and weaker bond strengths in RCA (Snyder 2006).

- RCA reduces the coefficient of thermal expansion (CTE) of concrete, indicating less expansion and contraction with temperature change (Smith et. al., 2009).
- Coarse RCA increases drying shrinkage since it holds excess water in the pores and a higher paste content (Snyder 2006).
- Shrinkage in PCC with fine RCA could be 20% to 50% higher than a coarse RCA and fine NA aggregate. Using both coarse and fine RCA increases shrinkage by 70% to 100%, since coarse RCA results in excess water in the pores of the RCA and more paste content (Snyder 2006).
- Shrinkage over time follows a parabolic trend similar to those proposed in ACI 209, and is correlated with the cement paste content in the RCA aggregate (Figure 3.24). After calculating the cement paste content, the shrinkage in RCA PPC can be modeled and predicted using a similar approach to what is proposed in ACI 209, Figure 3.25 (Kim et. al., 2014).

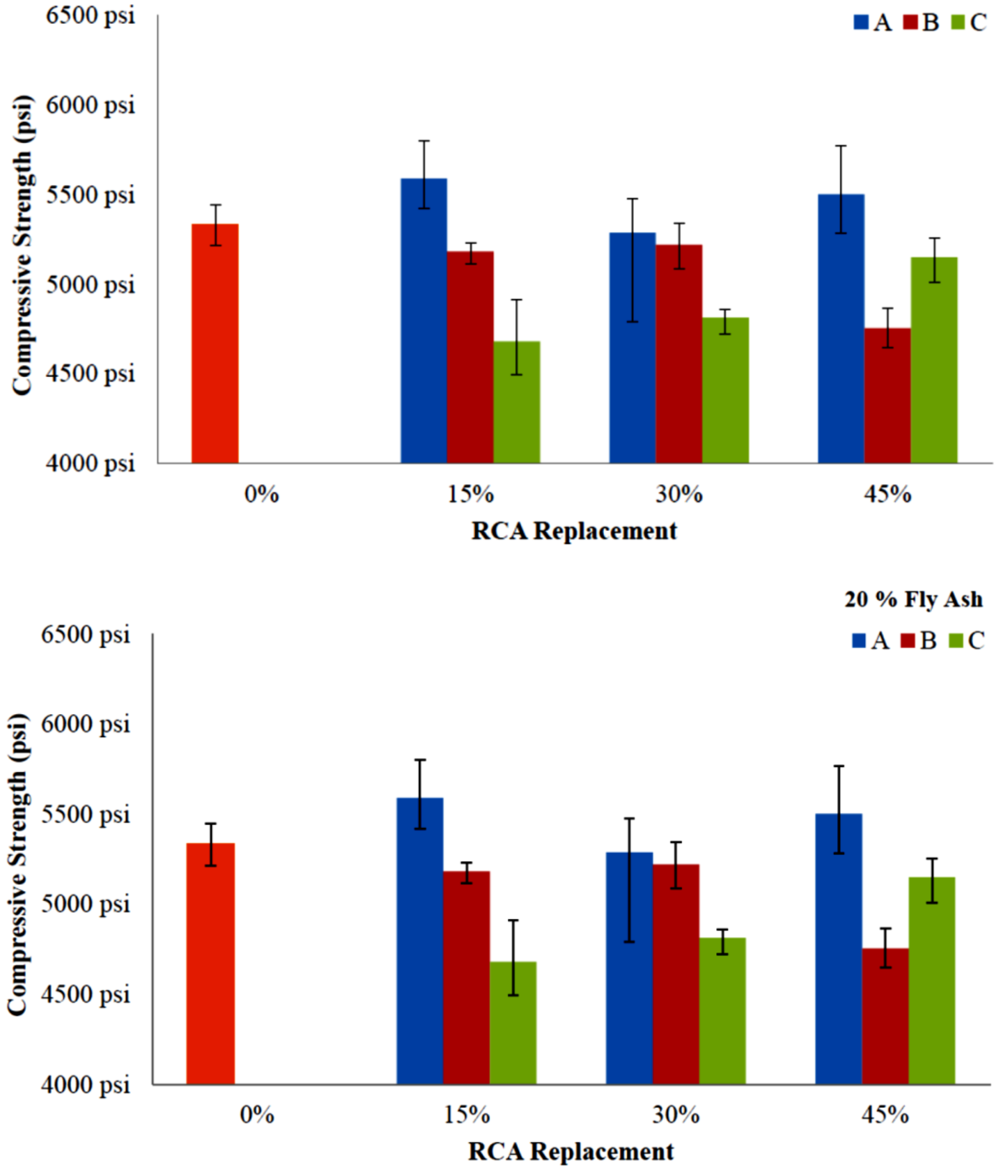


Figure 3.23 Average 28-Day Compressive Strength vs. % RCA Substitution: (top) 0% Fly Ash; (bottom) 20% Fly Ash (Wen et al. 2014)

- RCA PCC with more entrained air is better at resisting degradation and cracking when undergoing shrinkage and expansion associated with freezing and thawing, since more volume is required by freezing water's expansion (Portland Cement Association 2002).
- Carbonation in RCA concrete exacerbates concrete shrinkage (Molin et al. 2004).
- A higher level of shrinkage can result in higher PCC pavement moisture warping stresses; this needs to be addressed in the design by using shorter panel lengths to compensate for the higher stresses (Molin et al. 2004).
- The concrete strength of the original mixture used in RCA influences creep (i.e., accumulated permanent strain). The accumulated permanent deformation showed slightly lower total deformation for the medium strength RCA mix, and more deformation for the high strength RCA mixes, while the low strength RCA showed significant deformation followed by an early failure (Molin et al. 2004).

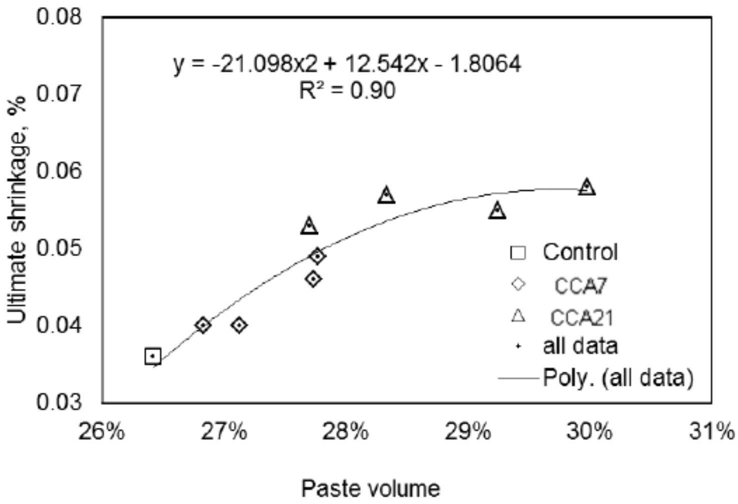


Figure 3.24 Ultimate Shrinkage Versus Cement Paste Volume (Kim et. al. 2014)

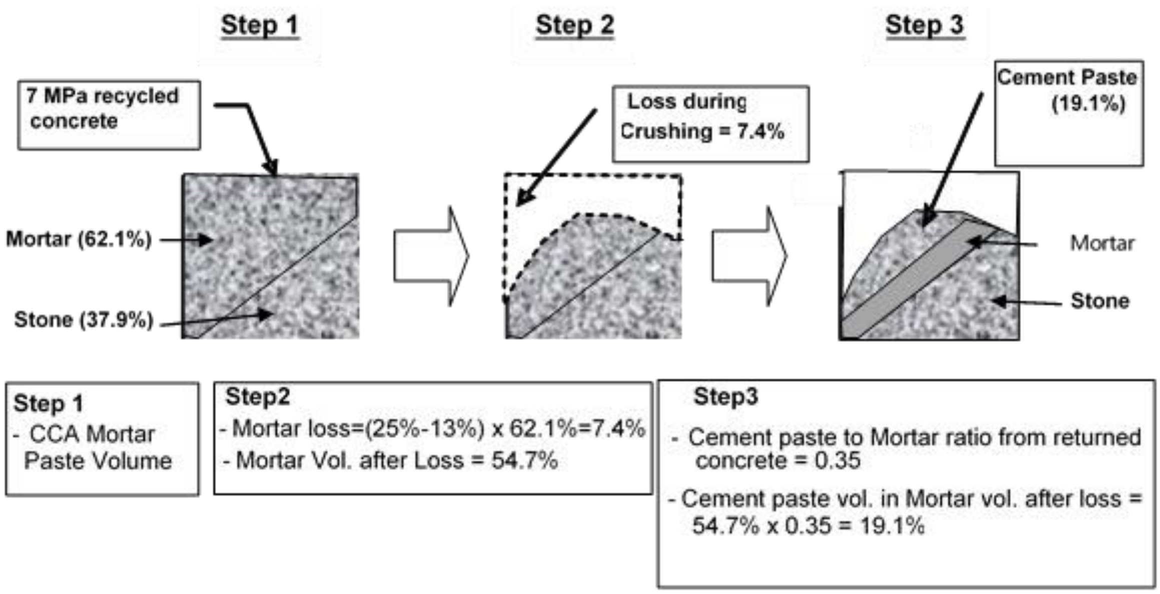


Figure 3.25 Cement Paste Volume Calculations (Kim et. al. 2014)

ENVIRONMENTAL PROPERTIES

- The pH of RCA typically ranges from 11.3 to 12.1 (Table 3.10). Concentrations of Cu and Zn are not related to the content of RCA. Levels of As, Cr, Pb, and Se exceeded USEPA MCL (maximum contaminant level) in some states (Edil et al. 2012).
- Leachate pH is strongly related to a material's pH long-term. A pH dependent leaching of Cu and Zn had similar leaching trends, with maximum leached concentrations at pH ≈ 2.0 and minimum leached concentrations at alkaline or near-neutral pH (7.5–13.0) (Figure 3.26). As pH decreases, leaching concentration for both elements increase, with Cu starting at pH≈6.5 and Zn at pH≈7.5 (Edil et al. 2012).
- Stockpiled RCA had a lower leachate pH and material pH. Concentrations of As, Cr, Pb, and Se may exceed the maximum contaminant levels (MCLs) in the USEPA drinking water standard at some point. Levels of Cr and Pb usually exceed the MCL at first flush with sporadic exceedances occurring afterwards, while As and Se, which mainly come from the cement mortar, exceed the MCLs consistently throughout the whole period (Edil et al. 2012).

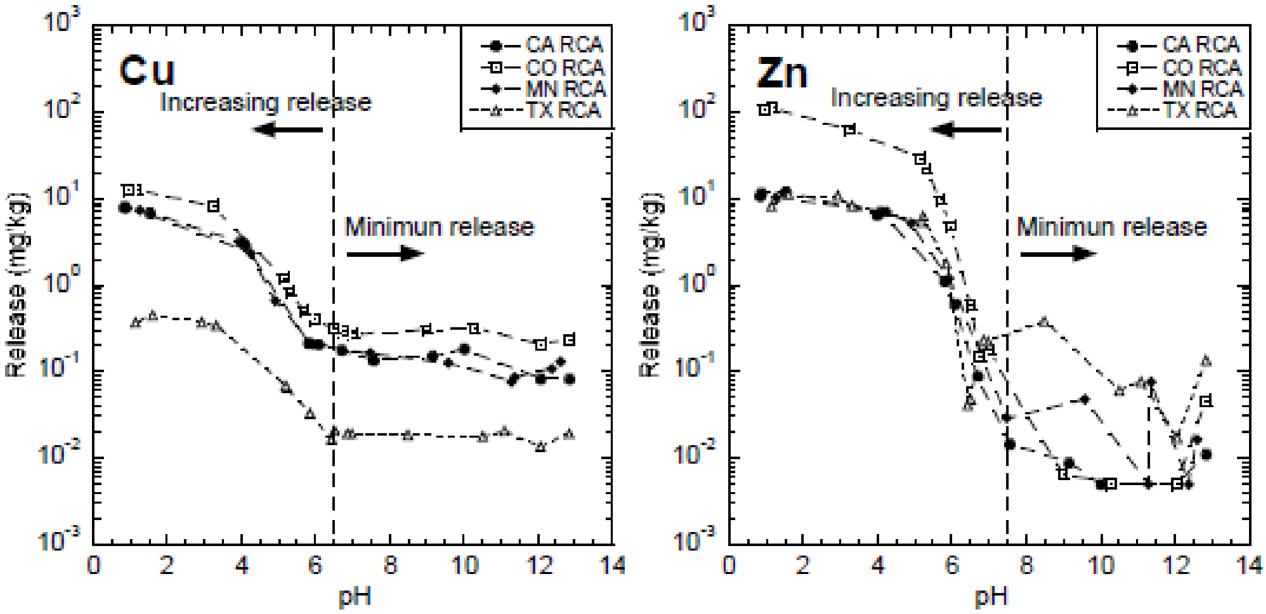


Figure 3.26 pH-Dependent Leaching of Cu and Zn from Unfractionated RCAs (Edil et al. 2012)

Methods: 1. ASTM D1557; 2. AASHTO T85; 3. ASTM D5856; 4. SC144 DR sulfur and carbon analyzer (LECO Inc., St. Joseph, MO, USA); 5. USEPA 3050(B); 6. Batch tests (de-ionized water only) with liquid to solid ratio = 1:10, contact time = 72 h, in a 30 revolution per min tumbler. 7. " –" = data unavailable.

- Water passing through a RCA layer can become highly alkaline, causing metal culvert and rodent guard corrosion, as well as vegetation kill near some drainage system outlets. Unbound layers have low permeability; thus, the alkalinity increase in passing water is ignored (Cooley et al. 2007).
- RCA originated from previously D-cracked (cracking of concrete pavements caused by the freeze-thaw deterioration of the aggregate within concrete) or ASR concretes is more likely to have D-cracking or ASR experience (Cooley et al. 2007).
- Mitigating measures to control alkali-silica reactivity (ASR) include: incorporation of fly ash, ground, granulated blast-furnace slag, or silica fume into the mix design; use of a blended cement; or use of a low-

alkali Portland cement (Springenschmid and Sodeikat 1998).

- For high quality RCA, little difference is present between RCA and conventional concrete in chloride ion penetration effect. The negative effect is significant in the case of low grade RCA (Otsuki et al. 2001, Shayan and Xu 2003). Chloride ion permeability is controlled by increasing the curing period or incorporating proper types and amounts of supplementary cementitious materials (Volz et al. 2014).

Table 3.10 Properties of RCA (Edil et al. 2012)

Location (State)	CA	CO	MI	MN	TX	Class V	WR (F)	WR (SP)	WA
Physical properties									
Optimum Water Content[1] (%)	10.9	11.9	8.7	11.2	9.2	8.0	10.8	9.9	-
Max Dry Unit Weight[1] (kN/m^3)	19.8	18.9	20.8	19.5	19.7	20.7	19.4	19.9	16.2
Specific Gravity[2]	2.6	2.6	2.7	2.7	2.6	2.7	2.7	2.6	2.8
Absorption[2] (%)	5.0	5.8	5.4	4.9	5.5	2.2	4.2	4.2	1.8
Void Ratio[2]	0.30	0.36	0.28	0.29	0.30	0.29	0.36	0.30	0.68
USCS	SP	SM	GP	SP	GP-GM	SW-SM	GP	SP	GP
Hydraulic Properties									
Hydraulic Conductivity[3] ($\times10^{-5}$ m/s)	1.9	1.6	2.6	1.8	0.8	0.05	120	71	58
Carbon Content[4]									
Total Carbon (%)	1.9	1.9	4.6	1.6	3.2	2.6	6.8	7.4	11.7
Total Organic Carbon (%)	1.4	0.3	0.2	0.4	0.4	0.1	0.3	0.5	1.0
Total Inorganic Carbon (%)	0.5	1.5	4.4	1.2	2.8	2.5	6.5	6.9	10.7
Total element concentrations[5]									
Major elements									
Ca (%)	6.9	8.5	7.6	16.3	4.9	0.3	>20	>20	>20
Fe (%)	2.3	1.1	1.0	1.6	0.8	0.6	0.8	0.7	0.3
Al (%)	1.7	1.5	1.0	0.8	1.0	0.4	0.5	0.4	0.1
Mg (%)	1.0	0.3	2.0	1.1	0.3	0.2	8.5	8.7	>10
Na (%)	0.4	0.5	0.6	0.4	0.3	0.02	0.8	0.4	0.5
K (%)	0.14	0.14	0.20	0.16	0.13	0.08	0.15	0.11	0.08
Trace elements									
As (mg/kg)	2.5	6.5	2.2	2.4	2.2	0.5	10.9	11.2	6.3
Ba (mg/kg)	165.2	88.8	40.8	69.4	58.0	31.7	20.4	22.8	3.7
Cd (mg/kg)	0.2	0.2	0.1	0.1	0.1	0.7	0.6	0.4	0.3
Co (mg/kg)	4.0	1.8	1.2	2.8	1.2	2.2	2.9	2.3	0.4
Cr (mg/kg)	20.2	7.5	6.2	11.5	8.9	18.7	6.7	6.3	2.5
Cu (mg/kg)	16.5	10	9.1	13.6	6.1	8.2	13.8	10.7	2.6
Mo (mg/kg)	0.1	1.0	0.1	1.0	0.1	0.1	1.1	0.5	0.3
Ni (mg/kg)	21.0	4.7	3.0	8.1	2.7	7.2	5.1	4.6	1.3
Pb (mg/kg)	9.1	8.7	2.1	2.6	5.0	2.5	3.6	3.2	3.7
Sb (mg/kg)	0.9	0.4	0.5	0.2	0.7	0.9	2.8	2.2	1.7
Se (mg/kg)	0.8	0.9	1.0	0.9	1.3	2.6	16.7	17.4	16.3
Zn (mg/kg)	32.4	58.8	34.9	30.4	20.4	11.7	26.8	18.9	17.6
Material pH[6]									
Bulk Specimens	12.1	12.1	12.6	11.3	12.0	9.1	12.3	11.8	10.0
Gravel-sized (75-4.75 mm)	12.1	12.1	12.5	11.6	12.1	-	12.6	11.9	-
Sand-sized (4.75-0.075 mm)	11.9	11.9	12.5	11.2	11.7	-	12.4	11.5	-
Fines (< 0.075 mm)	11.9	11.8	12.1	10.9	11.1	-	12.1	10.9	-

DESIGN RECOMMENDATIONS

- It is suggested to sieve and wash RCA to remove fine material (< No. 4) before usage (Cooley et al. 2007).
- Adding WRA (water-reducing admixture) and fly ash, or blending RCA with conventional aggregates, can minimize the effects of RCA on fresh concrete workability (Cooley et al. 2007).
- RCA stockpiles should be maintained at a moisture content representative of a saturated surface-dry condition. Otherwise, high level of water absorption of RCA could make the proper compaction of gravel cushion and aggregate base course layers variable (Cooley et al. 2007).
- Design recommendations were suggested for specific pavement applications, i.e., continuously reinforced concrete pavement (CRCP) or jointed reinforced concrete pavement (JPCP), as well as subbase type, concrete slabs, panel joints and the presence of reinforcement (Table 3.11; ACPA 2008).

Table 3.11 Summary of RCA PCC Structural Design Considerations (ACPA 2008)

Design Element	Recommendation
Pavement Type	JPCP panel length $\leq$ 15
	JRCP and CRCP interlock needs to be improved with larger top size aggregate or blend of new/RCA coarse aggregate
Slab Thickness	Same as conventional PCC if adequate strength achieved
	Two-course construction: overall slab thickness may need to be increased depending on materials and mix proportions
Joint Spacing	Select to minimize mid-panel cracking
Load Transfer	Same as conventional PCC
Joint Sealant Reservoir Design	Select for shrinkage and thermal movement
Subbase Type	Select for structural requirement
	Consider free-draining subbase for RCA produced from D-cracked and ASR-damaged concrete
Reinforcement	May require higher amounts of longitudinal steel in JRCP and CRCP

FIELD RECOMMENDATIONS

- A concrete mix should have enough water supply to ensure the workability of the concrete due to the high absorption capacity of RCA (Cooley et al. 2007).
- European studies encourage the recycling of old concrete pavement with good strength, durability and condition, instead of existing pavements distressed for D-cracking or ASR (Hall et al. 2007).
- Each RCA source should be tested for ASR following the crushing process and mitigated as necessary (Cooley et al. 2007).
- Jointed RCA PCC with dowels as load transfer through aggregate interlock need further consideration. The thermal coefficient of expansion is different for RCA PCC with conventional PCC, thus requiring slab length adjustments (Kuennen 2007).

Table 3.12 Environmental Impact to Substitute RCA for NA (Evangelista et. al. 2008)

CML Impact Parameter	RCA 30% Fine RCA	RCA 50% Fine RCA	EDIP Impact Parameter	RCA 30% Fine RCA	RCA 50% Fine RCA
Abiotic depletion	–6.4	–19.1	Acidification	–7.3	–21.8
Acidification	–6.9	–21.0	Acute aquatic toxicity	–4.7	–15.1
Aquatic toxicity (fresh water)	–6.9	–20.6	Acute ground ecotoxicity	–7.5	–22.3
Aquatic toxicity sediments (fresh water)	–7.2	–21.7	Aerial human toxicity	–7.5	–22.3
Destruction of the ozone layer	–7.6	–23.0	Aquatic human toxicity	–6.9	–21.5
Eutrophication	–6.7	–20.3	Chronic aquatic toxicity	–4.7	–15.1
Global warming	–6.8	–20.4	Chronic ground ecotoxicity	–7.2	–23.2
Ground ecotoxicity	–6.7	–20.1	Destruction of the ozone layer	–6.4	–19.1
Human toxicity	–6.8	–20.7	Global warming	–6.9	–21.0
Production of photo-oxidant agents	–6.6	–19.7	Ground human toxicity	–7.6	–22.5
			Nuclear waste	–3.3	–10.6
			Nutrients enrichment	–7.4	–22.2
			Overall waste	–8.3	–27.1
			Production of photo-oxidant agents	–6.9	–20.7
			Sludge slag and ashes	–7.0	–22.7
			Toxic waste	–5.2	–16.8
Average Decrease in Environmental Impact	–6.9	–20.7	Average Decrease in Environmental Impact	–6.6	–20.3

BENEFITS

- RCA byproducts vary in price from $1 to more than $16 per ton and result in savings of as much as $4 per ton for PCC paving. Some estimates of savings from recycling PCC are as high as $5 million on a single project (NCHRP 435).
- Both CML and EDIP (two methods used in EcoConcrete software to qualify and quantify the overall environmental impact) indicated a reduction of 6.5% in environmental impact when using 30% RCA replacement for natural fine aggregate, and about 20% when using 50% RCA replacement in PCC (Table 3.12; Evangelista et. al. 2008).
- Processing natural aggregates has heavier environmental load than recycling the concrete portion, especially in the CO_2 emissions, Table 3.13. The singular systems process with the most environmental impact is transport (Estevez et al. 2008).

Table 3.13 Emissions to Air from Extraction Processes of Primary and Secondary RCA (Estevez et al. 2008)

Emissions to Air, g/ton	Inventory of Concrete Recycling, g/ton		Inventories of SimaPró 4.0			
			Gravel I, g/ton		Sand I, g/ton	
	Transport	Crushing	Transport	Electricity	Transport	Electricity
CO_2	1,704	1,261	4,920	2,820	4,920	1,950
NOx	26.36	19.50	94.20	5.36	94.20	5.49
SO_2	1.62	1.20	12.00	0.42	12.00	11.20
Dust	0.17	0.126	0.49	0.037	0.49	0.873

SUGGESTED SPECIFICATIONS

The following specifications were suggested applicable to RCA for use in concrete:

Table 3.14 AASHTO Test Methods for Evaluation of RCA and RCA PCC (ACPA 2008)

AASHTO	Title
T2	Standard Method of Testing for Sampling of Aggregates
T11	Standard Method of Test for Materials Finer Than 75 um (No. 200) Sieve in Mineral Aggregates by Washing
T19	Standard Method of Test for Bulk Density ("Unit Weight") and Voids in Aggregate
T27	Standard Method of Test for Sieve Analysis of Fine and Coarse Aggregates
T85	Standard Method of Test for Specific Gravity and Absorption of Coarse Aggregate
T87	Standard Method of Test for Dry Preparation of Disturbed Soil and Soil-Aggregate Samples for Test
T88	Standard Method of Test for Particle Size Analysis of Soils
T89	Standard Method of Test for Determining the Liquid Limit of Soils
T90	Standard Method of Test for Determining the Plastic Limit and Plasticity Index of Soils
T96	Standard Method of Test for Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine
T99	Standard Method of Test for Moisture-Density Relationships of Soils Using a 2.5 kg (5.5 lb) Rammer and a 305 mm (12 in.) Drop
T103	Standard Method of Test for Soundness of Aggregates by Freezing and Thawing
T104	Standard Method of Test for Soundness of Aggregate by Use of Sodium Sulfate or Magnesium Sulfate
T112	Standard Method of Test for Clay Lumps and Friable Particles in Aggregate
T113	Standard Method of Test for Lightweight Pieces in Aggregate
T161	Standard Method of Test for Resistance of Concrete to Rapid Freezing and Thawing
T176	Standard Method of Test for Plastic Fines in Graded Aggregates and Soils by Use of the Sand Equivalent Test
T180	Standard Method of Test for Moisture-Density Relations of Soils Using a 4.54 kb (10 lb) Rammer and a 457 mm (18 in.) Drop
T193	Standard Method of Test for the California Bearing Ratio
T196	Standard Method of Test for Air Content of Freshly Mixed Concrete by the Volumetric Method
T234	Standard Method of Test for Strength Parameter of Soils by Triaxial Compression
T260	Standard Method of Test for Sampling and Testing for Chloride Ion in Concrete and Concrete Raw Materials
T277	Standard Method of Test for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration
T299	Standard Method of Testing for Rapid Identification of Alkali-Silica Reaction Product in Concrete
T303	Standard Method of Test for Accelerated Detection of Potentially Deleterious Expansion of Mortar Bars Due to Alkali-Silica Reaction
T307	Standard Method of Test for Determining the Resilient Modulus of Soils and Aggregate Materials
T327	Standard Method of Test for Resistance of Coarse Aggregate to Degradation by Abrasion in the Micro Deval Apparatus

Table 3.15 ASTM Test Methods for Evaluating RCA and RCA PCC Applications (ACPA 2008)

ASTM	Title
ASTM C33	Standard Specification for Concrete Aggregates
ASTM C88	Standard Test Method for Soundness of Aggregates by Use of Sodium Sulfate or Magnesium Sulfate
ASTM C125	Standard Terminology Relating to Concrete and Concrete Aggregates
ASTM C131	Standard Test Method for Resistance to Degradation of Small Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine
ASTM C173	Standard Test Method for Air Content of Freshly Mixed Concrete by the Volumetric Method
ASTM C227	Standard Test Method for Potential Alkali Reactivity of Cement Aggregate Combinations (Mortar Bar Method)
ASTM C289	Standard Test Method for Potential Alkali Silica Reactivity of Aggregates (Chemical Method)
ASTM C395	Standard Guide for Petrographic Examination of Aggregates for Concrete
ASTM C342	Standard Test Method for Potential Volume Change of Cement Aggregate Combinations (withdrawn 2001)
ASTM C441	Standard Test Method for Effectiveness of Pozzolans or Ground Blast Furnace Slag in Prevent Excessive Expansion of Concrete Due to the Alkali Silica Reaction
ASTM C586	Standard Test Method for Potential Alkali Reactivity of Carbonate Rocks for Concrete Aggregates (Rock Cylinder Method)
ASTM C618	Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use in Concrete
ASTM C666	Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing
ASTM C856	Standard Practice for Petrographic Examination of Hardened Concrete
ASTM C1202	Standard Test Method for Electrical Indication of Concretes Ability to Resist Chloride Ion Penetration
ASTM C1293	Standard Test Method for Determination of Length Change of Concrete Due to Alkali Silica Reaction
ASTM C1567	Standard Test Method for Determining the Potential Alkali Silica Reactivity of Combinations of Cementitious Materials and Aggregate (Accelerated Mortar Bar Method)
ASTM D2940	Standard Specification for Graded Aggregate Material for Bases or Subbases for Highways or Airports
ASTM D5101	Standard Test Method for Measuring the Soil-Geotextile System of Clogging Potential by the Gradient Ratio
ASTM D6928	Standard Test Method for Resistance of Coarse Aggregate to Degradation by Abrasion In the Micro Deval Apparatus

3.2 Reclaimed Asphalt Pavement Aggregate (RAP)

3.2.1 RAP in GAB

MECHANICAL PROPERTIES

- Density and Permeability
  - Maximum dry density of compacted RAP varies between 115 and 130 pcf, depending on the RAP origin (Yuan et al. 2011). Specific gravity of RAP varies between 2.27 to 2.45, lower than natural aggregates due to its lighter weight. (Ganne 2009).
  - Increasing RAP content decreases maximum dry density (MDD) for RAP-base blends, because of reduced specific gravity caused by asphalt coating on RAP aggregates (Guthrie et al. 2007, Ganne 2009).
  - Compacted density of mixture decreases with increasing RAP content, as asphalt coating inhibits compaction (McGarrah 2007).
  - Permeability of blended granular material containing RAP is higher than that of virgin aggregates; it increases with higher RAP content due to lower air voids (Mokwa and Peebles, 2005). However, conflicting results indicated that 100% RAP has a permeability of 16.9 ft/day in a constant head test and 13.9 ft/day in a falling head test, lower than that of natural aggregates. The permeability decreases as RAP content increases, since asphalt forms compaction and bond between RAP particles (Figure 3.27; Bennett and Maher 2005, Wu 2011).

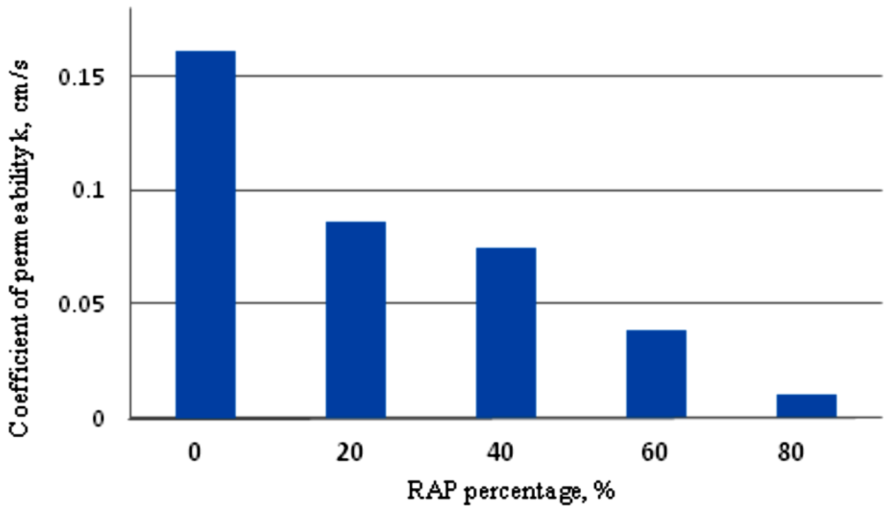


Figure 3.27 Trend of Hydraulic Conductivity with Increase of RAP Percentage (Wu 2011)

  - Permeability of a granular material is directly related to the percentage of fines (particles passing the #200 sieve) present in the material (Yuan et al. 2011). As gradation changes from the coarser end of the gradation band to finer, permeability decreases (Bennett and Maher 2005).
  - Permeability increased after freezing-thawing, due to gradation change of RAP as a result of disintegration.
  - Fines migrate with water flow, resulting in a loss of support for larger aggregates, diminishing overall stability of the aggregate layer and loss of support for the pavement structure (Bennett and Maher 2005).

- There are no durability concerns regarding the use of RAP in granular base, since quality of RAP aggregates usually exceeds the requirements for granular aggregates. However, the thin film of asphalt on the aggregates has some effect on the performance of RAP, as aggregate in unbound pavement layers (Yuan et al. 2011).
- Durability of RAP is mostly affected by aggregates used in the original HMA mix. RAP from pavements that have exhibited stripping have low strength (Saeed 2008).
- Increasing RAP contents decreases maximum dry density (MDD) for RAP-base blends because of reduced specific gravity caused by asphalt coating on RAP aggregates (Figure 3.28; Guthrie et al. 2007, Ganne 2009).
- Optimum moisture content (OMC) varies between 5.3% and 7.1% for RAP-base blends, comparable to that of conventional GAB ranging from 5% and 8% (Ganne 2009). Increasing RAP content decreases OMC RAP blends (Figure 3.28), due to reduced water absorption as a result of asphalt coating of RAP aggregates (Guthrie et al. 2007, Ganne 2009).

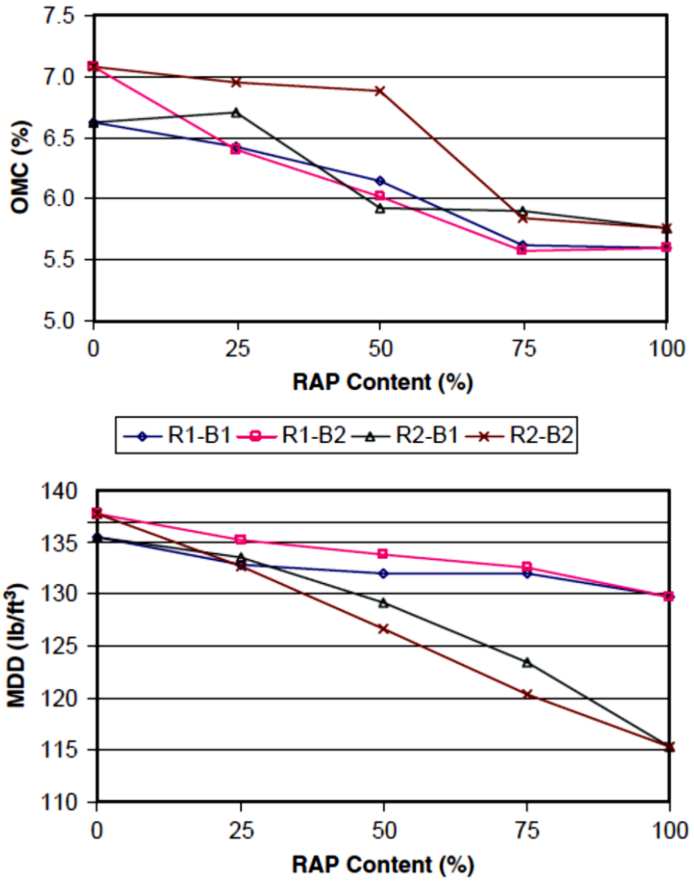


Figure 3.28 Compaction Characteristics: (top) OMC and (bottom) MDD (Guthrie et al. 2007)

Note. R1, R2 represent RAP, with R1 finer than R2. B1, B2 represent virgin base material, where B1 is coarser than B2.

- <u>Stiffness</u>
  - M_R of RAP is higher than virgin aggregate base materials. M_R increases linearly with increasing bulk stress and RAP content. One hundred percent RAP achieves the largest M_R over all of RAP blended with natural aggregates (Bennett and Maher 2005). Bulk stress (θ) model $M_R=K_1*\theta^{K2}$ is used to predict

M_R of different blends (Table 3.16). M_R and θ in units of megapascal and kilopascal at a bulk stress of 345 kPa, respectively (Thakur 2011).

Table 3.16 Bulk Stress (θ) Model Parameters for Prediction of M_R of RAP Aggregate Blends (Thakur 2011)

Reference	M_R test method	RAP content (%) in blends of RAP-aggregate	Model Parameters		R^2
			K_1	K_2	
Modified from Clary et al. (1997)	AASHTO T 294 - 94	0	4.64	0.66	0.88
		10	4.39	0.65	0.97
		30	5.67	0.65	0.97
		50	7.84	0.6	0.97
		100	16.07	0.51	0.93
Bennert et al. (2000)	AASHTO TP 46 - 94	0	9.55	0.5	NA
		25	17.35	0.45	
		50	13.49	0.52	
		75	19.49	0.46	
		100	43.1	0.36	
Modified from Cosentino et al. (2003)	LTTP Protocol P46	60	7.67	0.59	0.85
		80	10.78	0.6	0.95
		100	9.6	0.64	0.98
Modified from Abdelrahman et al. (2010)	LTTP Protocol P46	0	4.79	0.63	0.96
		30	4.59	0.66	0.99
		50	9.2	0.57	0.97
		70	19.09	0.46	0.94
		100	27.39	0.43	0.85

- As gradation becomes finer, M_R decreases. However, this trend is influenced by the percent of coarse particles, density and angularity. Coarser gradation is unstable under cyclic loading; therefore, specimens are unsuitable to be tested under M_R test procedure (Bennett and Maher 2005).
- Higher compactive effort (i.e., compact to 95% of maximum dry density) improves M_R (Bennett and Maher 2005).
- M_R decreases with increasing moisture content. RAP percentage has little effect on sensitivity of M_R to moisture content (Wu 2011).
- M_R decreases with increase of temperature due to reduction of asphalt stiffness. Mixtures with higher RAP content are more sensitive to temperature changes (Figure 3.29; Wu 2011).
- M_R increases with an increase of confining pressure. There is a higher M_R for mixtures containing a higher RAP content, which may be associated with lower air void (Wu 2011).

- Geogrid and geocell improve M_R of RAP layers by providing lateral confinement whereas geotextile provides a tensioned membrane effect (Thakur 2011).
- Rejuvenators (i.e., waste vegetable oil, waste vegetable grease, organic oil, distilled tall oil, aromatic extract, waste engine oil) prevent premature fatigue and low temperature cracking failures in RAP, since rejuvenators cause RAP asphalt binder to effectively blend with virgin materials, reducing stiffness and providing the required binder performance for another service period (Shen et al. 2007).

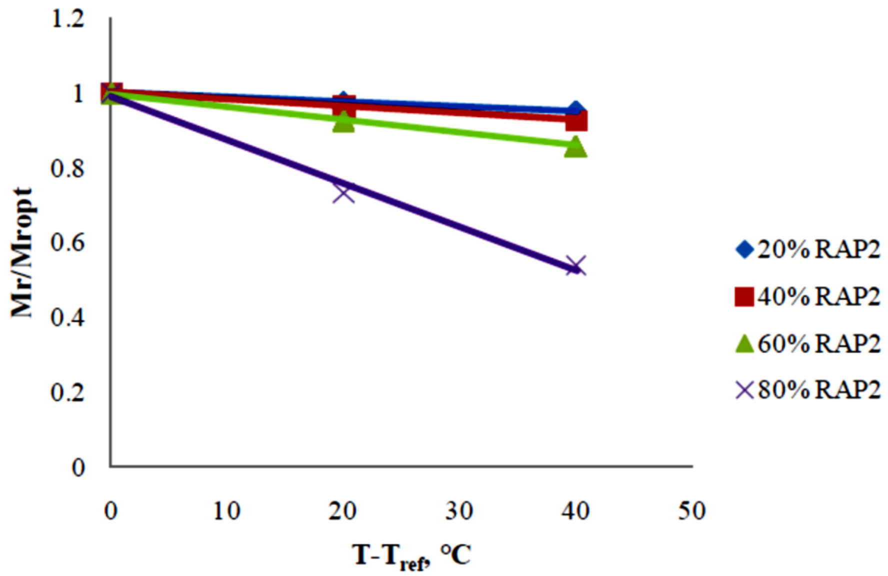


Figure 3.29 Effect of Temperature and RAP Content on M_R (Wu 2011)

- <u>Strength</u>
  - CBR of RAP is lower than that of natural aggregates. As gradation changes from the coarser end of gradation band to the finer end, CBR decreases (Bennett and Maher 2005).
  - CBR of blends decreases with increasing RAP content (Bennert and Maher 2005, Guthrie et al. 2007, Cosentino et al. 2012). However, Cosentino et al. (2003) showed that CBR of blends increase with an elevated RAP content up to a certain level, and then they start decreasing.
  - CBR of 100% RAP ranges from 11 to 33% (Thakur 2011). Variation is caused by the type of RAP, the aggregate, and moisture content used for blends. CBR improves by adding fine sand (i.e. material passing the #40 sieve size) instead of increasing density by doubling compaction.
  - Virgin aggregate samples have lower unconfined compressive strength (UCS) than blends containing 25% or 50% RAP (Guthrie et al. 2007).
  - UCS decreases with increasing RAP content (Guthrie et al. 2007, Ganne 2009). Conversely, Taha et al. (2002), Yuan et al. (2010) and Hoyos et al. (2011) reported that UCS increases with an increasing RAP content.
  - Blends containing coarse RAP aggregates have higher UCS than those containing fine RAP aggregates (Ganne 2009).
  - Friction angle and cohesion of 100% RAP varied from 44° to 45° and from 17 to 131 kPa, respectively. Blends with a higher friction angle show lower cohesion and vice versa. Cohesion obtained from asphalt binder helps particles stick each other when forced together (Thakur 2011).
  - One hundred percent of RAP shows the highest friction angle. Friction angle decreases with the

increase of fine sand percentage in RAP-soil mixtures, since fine sands reduce grain-to-grain contact, causing larger particles to float within a soil matrix. Cohesion increases with a higher fine sand percentage in RAP-soil mixtures, due to capillary pressures caused by attraction of pore water menisci on fine sand particles (Cosentino et al. 2003).

- Coarse friction of aggregates provide shear strength. As gradation changes from the coarser end of the gradation band to finer end, CBR decreases (Bennett and Maher 2005).

◆ Permanent Deformation

- One hundred percent RAP cannot produce a high-quality base courses due to its high deformation and creep (Dong et al. 2014). Higher deformation is caused by a gradual breakdown of material, or by material becoming more susceptible to compaction from additional cyclic loading (Bennett and Maher 2005).
- Permanent deformation increases with increasing RAP contents. Permanent strain (ε_p) increases with the number of loading cycles. The Rate of increase in permanent strain decreases with the increase of loading cycles. Relation ε_p (%) = A* N^B is proposed to predict permanent strain of RAP-aggregate blends (Table 3.17; Thakur 2011).

Table 3.17 Permanent Strain Model Parameters (Thakur 2011)

Reference	RAP content (%)	Model parameters		R^2
		A	B	
Garg and Thompson (1996)	100	0.39	0.22	1
Attia (2010)	50	0.02	0.32	0.98
	100	0.01	0.44	0.93
Bennert et al. (2000)	50	0.05	0.34	0.9
	100	0.1	0.41	0.96
Kim and Labuz (2007)	50	0.23	0.28	0.99

- High angular and coarse aggregates provide resistant to deformation. As gradation becomes finer, permanent strain increases. Greater compactive effort creates denser material with less permanent deformation (Bennett and Maher 2005).
- There is higher permanent deformation at higher RAP percentage in dry conditions, while RAP percentage has little effect on permanent deformation in moist conditions. Increasing moisture content increases permanent deformation (Wu et al. 2011).
- Creep deformations increase with increasing applied vertical stress and RAP content. The rate of increase in creep deformation decreases with time (Cosentino et al. 2003).
- The permanent deformation model of unreinforced and geocell-reinforced RAP bases can be calculated by $PD = K \cdot h_{soil} \cdot \varepsilon_v \cdot \left(\frac{\varepsilon_0}{\varepsilon_r}\right) \cdot e^{-\left(\frac{\rho}{N}\right)^{\beta}}$, where N = number of axle load applications; h_{soil} = thickness of a layer; ε= average vertical resilient strain in a layer. Parameters were obtained according to the water content of 5.6% (Table 3.18; Thakur 2011).

Table 3.18 Model Calibration Parameters for Permanent Deformations of RAP Bases (Thakur 2011)

Types of RAP base	Model parameters	
	Tseng and Lytton (1989)	MEPDG
Unreinforced RAP	$K\left(\frac{\varepsilon_0}{\varepsilon_r}\right) = 92$ $\rho = 3764$ $\beta = 0.2$	$K = 4.2$ $\left(\frac{\varepsilon_0}{\varepsilon_r}\right) = 21.8$ $\rho = 3764$ $\beta = 0.2$
Geocell-reinforced RAP	$K\left(\frac{\varepsilon_0}{\varepsilon_r}\right) = 70$ $\rho = 3764$ $\beta = 0.4$	$K = 2.7$ $\left(\frac{\varepsilon_0}{\varepsilon_r}\right) = 21.8$ $\rho = 3764$ $\beta = 0.2$

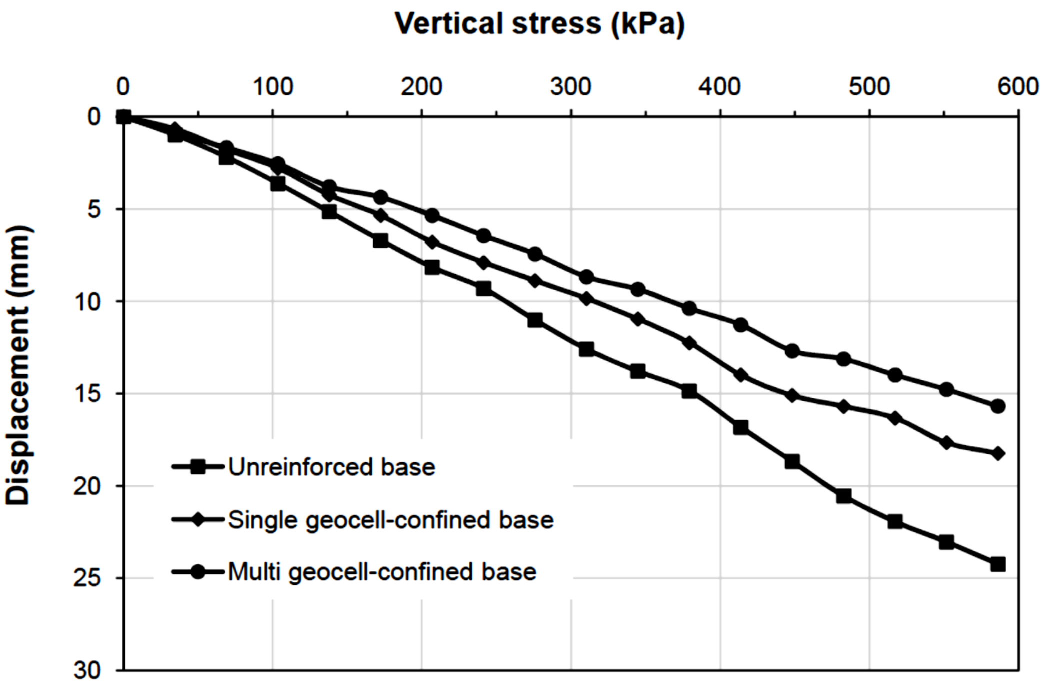


Figure 3.30 Vertical Stress-Displacement Curves for Unreinforced and Geocell-Reinforced RAP Bases (Thakur et al. 2012)

- Geocell reinforcement reduces immediate deformations of RAP blends or bases by 18%- 73% as compared with unreinforced RAP base. Geocell-confined base has 81%- 86% lower creep deformation than unreinforced base. RAP crept more at higher vertical stress and lower degree of confinement and vice versa (Thakur et al. 2013).

- The vertical stress- displacement ratio of single geocell-confined and multi geocell-confined bases is 1.2 and 1.6 times of an unreinforced base (unreinforced RAP sample extruded from a Proctor compaction mold), respectively (Figure 3.30; Thakur 2011, Thakur et al. 2012).

ENVIRONMENTAL PROPERTIES

- RAP does not pose any threat to the environment (Cosentino et al. 2003, Legret et al. 2005). Most leaching concentrations are below the detection limit of equipment used. With four different testing protocols to evaluate, none of the results are near the EPA Standards (Table 3.19; Cosentino et al. 2003).

Table 3.19 Environmental Testing Summary for RAP (Cosentino et al. 2003)

	Silver	Cadmium	Chromium	Lead	Selenium
Field Surface Runoff Ave > BDL (μg/l)	1.79	2.78	< 5	131.31	2.29
% BDL	62	94	100	86	50
Field Leachate Water Ave > BDL (μg/l)	1.65	< 1	< 5	7.76	11.46
% BDL	45	100	100	93	9
Lab Column Leaching with DDW Ave > BDL (μg/l)	< 1	< 1	< 5	< 5	1.22
% BDL	100	100	100	100	67
Lab Column Leaching with SAR Ave > BDL (μg/l)	< 1	6.33	< 5	< 5	< 1
% BDL	100	33	100	100	100
Detection Limit (μg/l)	1	1	5	5	1
EPA Standard (μg/l)	5000	1000	5000	5000	1000

Note. BDL = below detection limit; DDW= distilled-deionized water; SAR= synthetic acid rain.

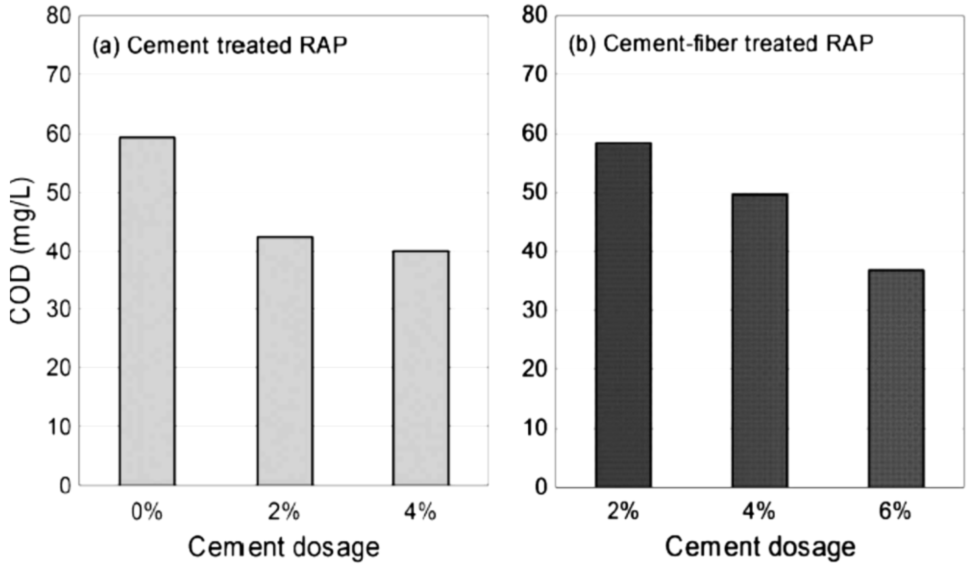


Figure 3.31 COD Test Results on Treated RAP (Hoyos et al. 2011)

- RAP has higher concentrations of total hydrocarbons and some PAHs, compared to new conventional asphalt. There are higher pollutant concentrations at initial leaching stages, but they decrease rapidly and eventually are less than detection limits (Legret et al. 2005).
- RAP in small grain size has a higher leaching pH, due to increasing particle areas. Slowing water percolation induces more extracted Zn and Cu, which diffuse more easily at low flow rates. Hydrocarbon concentrations decreases with slower flow because of degradation over time (Legret et al. 2005).
- Chemical oxygen demand (COD) measures the oxygen equivalent of organic matter content in water that is susceptible to oxidation by a strong chemical oxidant. COD concentrations are lower than the EPA benchmark of 120 mg/L (Figure 3.31; Hoyos et al. 2011).

DESIGN RECOMMENDATIONS

- The percent by total weight allowed for RAP blended with dense graded aggregate base course should be limited to 50%. At percentages greater than 50%, permeability and CBR greatly reduce despite occasional increases in resilient modulus and accumulated permanent deformation (Bennett and Maher 2005).
- Re-blending or fractionating 100% RAP is not recommended as a method to produce base or subbase material.
- Un-stabilized RAP material must be blended with a minimum of 75% approved base course aggregate material, and meet the LBR (limerock bearing ratio) strength requirement. The asphalt binder content of total blend should not exceed 1.5% by weight (Cosentino et al. 2012).

FIELD RECOMMENDATIONS

- The site should be of sufficient size to conduct a comprehensive field testing program over 12 months. Field testing includes density, temperature, CBR, dynamic cone penetrometer and falling weight deflectometer data (FWD) (Cosentino et al. 2012).
- FWD testing should be conducted to measure if effects of cyclic loads are consistent with rutting, by applying repetitive FWD loads (9 kips) at specified site and recording data of rut depth versus loading cycle. Record deflections following each sequence of four, 9 K load applications. Use creep pressure to determine rut depths. Creep loading requires a constant pressure equivalent to 9 kips on FWD loading plate (110 in^2) or about 80 psi (Cosentino et al. 2012).
- Record temperature profiles along with ambient temperatures.
- Evaluate field compaction methods (i.e., padfoot, vibratory steel wheel and pneumatic rubber tired) alone or in combination. Determine compaction process by adjusting field density results to lab density data. Consider pneumatic rollers and compaction trains of pneumatic and steel drum, based on results from gyratory compaction (Cosentino et al. 2012).

BENEFITS

- Using RAP materials in road construction reduces both the depletion rate of natural resources and the amount of construction debris disposed in urban landfills (Hoyos et al. 2011).
- RAP base materials yield considerable savings in overall costs of pavement construction projects. Using between 20%- 50% RAP can result in a cost savings of between 14%- 34% per tonnage (TFHRC 2010).
- RAP is used in new bituminous materials by either a hot-mix or cold-mix recycling process. However, a

large quantity of RAP materials remains unused, which can be reduced by using RAP as base and subbase aggregate materials (Thakur and Han 2015).

SUGGESTED SPECIFICATIONS

Table 3.20 Granular Aggregate Test Procedures (Cosentino et al. 2012 and Chesner et al. 1998)

Test	Standard
Unconfined Compression Test	AASHTO T208, ASTM D2166
Gyratory Compaction	ASTM 6925
Marshall Compression Test	AASHTO T245
Vibratory Compaction	ASTM D4253
California Bearing Ratio Test	ASTM D1883, AASHTO T193
Indirect Tensile Splitting Test	ASTM D3967
Permeability	ASTM D2434, AASHTO T215
Abrasion Resistance	ASTM C535, ASTM C131, AASHTO T96
Resilient Modulus	AASHTO T307
Base Stability	ASTM D698, AASHTO T99, AASHTO T180

Table 3.21 Base Course Gradation for RAP (Cosentino et al. 2012 and McGarrah 2007)

Sieve Size / Percent Passing	NJDOT	FDOT
2 in.	100	100
1-1/2 in.	85-100	95-100
3/4 in.	55-90	65-90
3/8 in.	-	45-75
#4	23-60	35-60
#10	-	25-45
#50	3-25	5-25
#200	0-10	0-10

3.2.2 RAP in FASB

MECHANICAL PROPERTIES

- Moisture and Density
  - Optimum moisture content (OMC) varies between 5.3%- 7.1% for cement or fly ash-treated RAP-base blends (Ganne 2009). Increasing RAP content decreases OMC for cement or fly ash-treated RAP blends, due to reduced water absorption because of asphalt coating of RAP aggregates (Guthrie et al. 2007, Ganne 2009).
  - Increasing RAP contents decreases the maximum dry density (MDD) for cement treated mixes/RAP-base blends, because of reduced specific gravity caused by asphalt coating on RAP aggregates (Guthrie et al. 2007, Ganne 2009).

- Stiffness (Resilient Modulus)
  - Resilient modulus (M_R) ranges between 100 ksi and 800 ksi. (The range reflects a variety of aggregates, binders, mixing and curing conditions, and compaction).
  - Cement addition increased M_R and dependency on bulk stress (Jenkins et al. 2007).
  - Temperature sensitive; there was a 30%- 44% stiffness reduction as testing temperature increased from 50ºF to 104ºF (Nataatmadja 2002).
  - Influential factors to M_R:
    - More by loading rate and less by stress (Fu et al. 2009).
    - Higher influence by confining pressure than deviatoric stress (Fu and Harvey 2007).
    - Higher influence by loading rate and temperature than confining pressure and deviatoric stress (Khosravifar et al. 2012).
  - M_R increases with an increasing percent of cement (Figure 3.32). Cement appears to be an effective stabilizer for RAP in achieving high strength and stiffness. Cement-fiber-stabilized RAP mixtures have higher M_R than cement-stabilized RAP specimens (Potturi 2006).

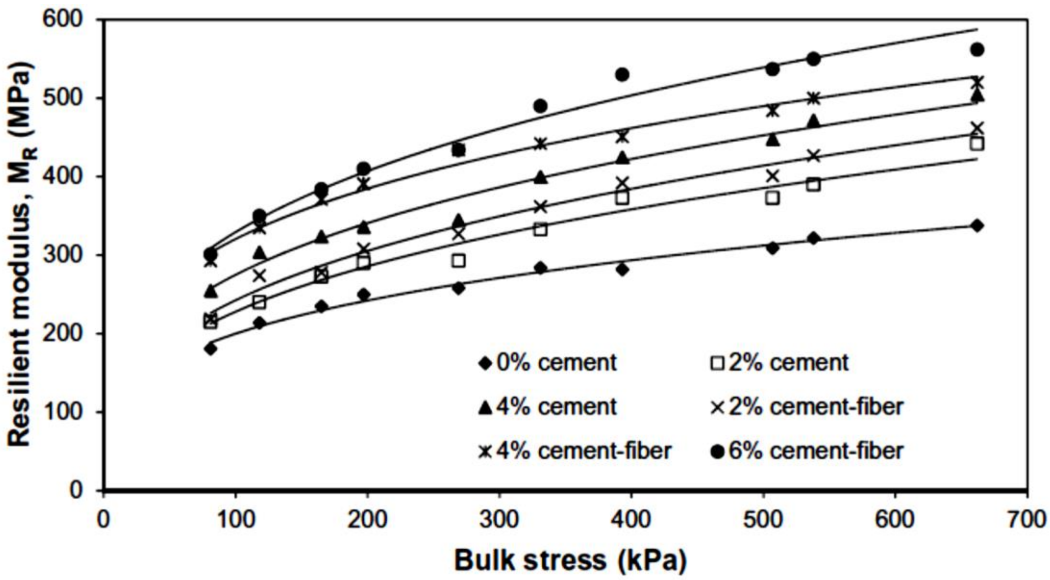


Figure 3.32 M_R of Cement-Stabilized and Cement-Fiber-Stabilized RAP Specimens (Potturi 2006)

- M_R increases with increasing fly ash content and curing period (Li et al. 2007, Wen et al. 2010).
- M_R of chemically stabilized RAP also increases with higher stabilizing agent content (Thakur and Han 2015).

◆ <u>Strength</u>

- CBR increases linearly with fly ash content (Figure 3.33). The relation, CBR%=A*stabilizing agent%+B, is proposed to predict CBR of chemical-stabilized RAP specimens, based on CBR test results (Cosentino et al. 2012).

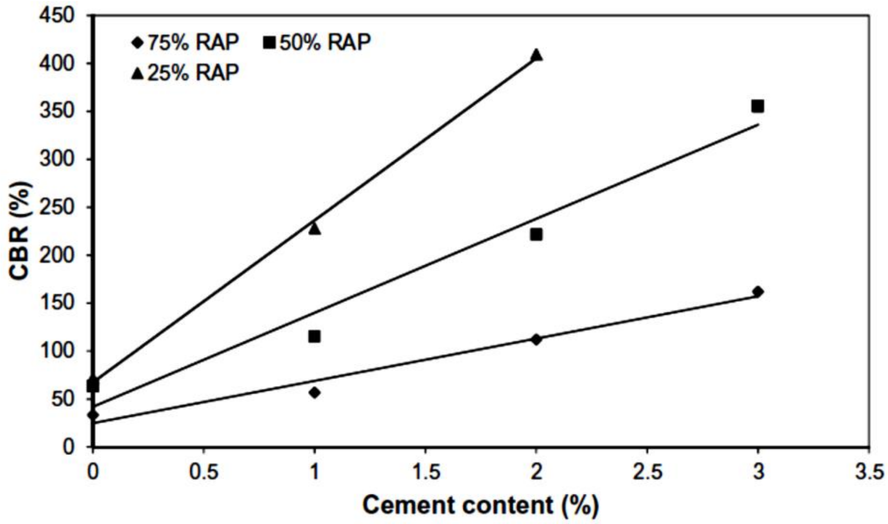


Figure 3.33 Effect of Cement Content on CBR of RAP Specimen (Cosentino et al. 2012)

- The UCS of 4%-6% cement-treated RAP are similar to those reported for recycled concrete and crushed limestone for similar cement dosages (Figure 3.34; Lim and Zollinger 2003). UCS of treated RAP increases as cement dosage increases. Inclusion of fibers has little effect on the UCS of cement-fiber-treated RAP (Hoyos et al. 2011).

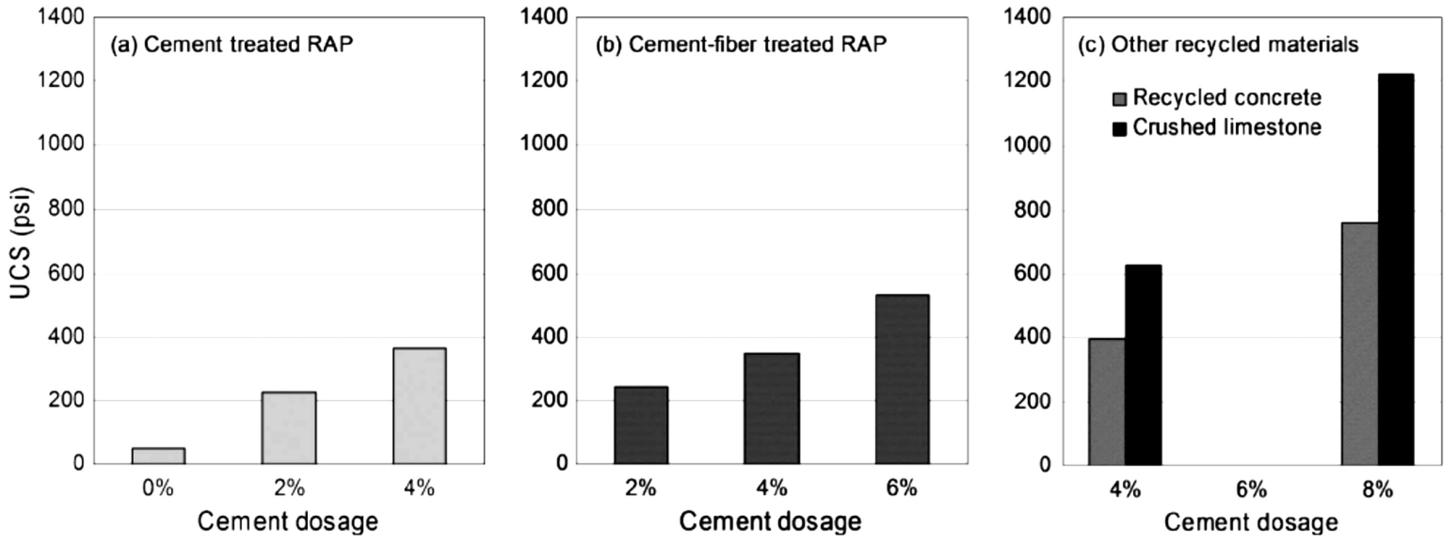


Figure 3.34 UCS of Treated RAP and Other Reclaimed Materials (Hoyos et al. 2011)

- UCS increases by increasing stabilizing agent (i.e., cement, fly ash) content and curing period, and decreases with RAP content (Wen et al. 2010, Taha et al. 2002).
- Strength is reduced with increasing RAP content under both dry and soaked conditions due to reduction in inter-lock between aggregates (He et al. 2006). Other studies indicated that an increase in RAP percentage improved soaked indirect tensile strength (ITS) for mixtures containing GAB material, and a decrease in ITS for RAP mixtures containing RCA (Schwartz et al. 2013).
- The addition of Portland cement increased dry and soaked ITS by providing stiff, brittle cementitious bonds (Ruckel et al. 1983). Cement promotes early strength gain (Fu et al. 2008). An increase of 40% in unsoaked ITS and over 300% in soaked ITS is reported when adding 1% cement (Table 3.22; Schwartz et. al 2013). The tensile strength ratio (TSR) significantly improved as well.
- Soaking for 24 hours can obtain consistent ITS value, which is more effective than soaking for 72 hours or vacuum saturation (Khosravifar 2012).
- Stockpiling significantly reduces strength of soaked and dry ITS by an average of 27% and 16%, respectively (Figure 3.35; Khosravifar et al. 2012).
- Foamed asphalt content should be limited to 3%, as excessive foamed asphalt acts as a lubricant between aggregates, leading to shear failure (Wirtgen 2010).

Table 3.22 Effect of Cement on ITS Results of Mix G with 100% RAP-2 (Schwartz at al. 2013)

Mix G	0% cement	1% cement
Foamed asphalt (%)	2.3	2.3
Unsoaked ITS (psi)	44	62
Soaked ITS (psi)	15	61
TSR (%)	34	98

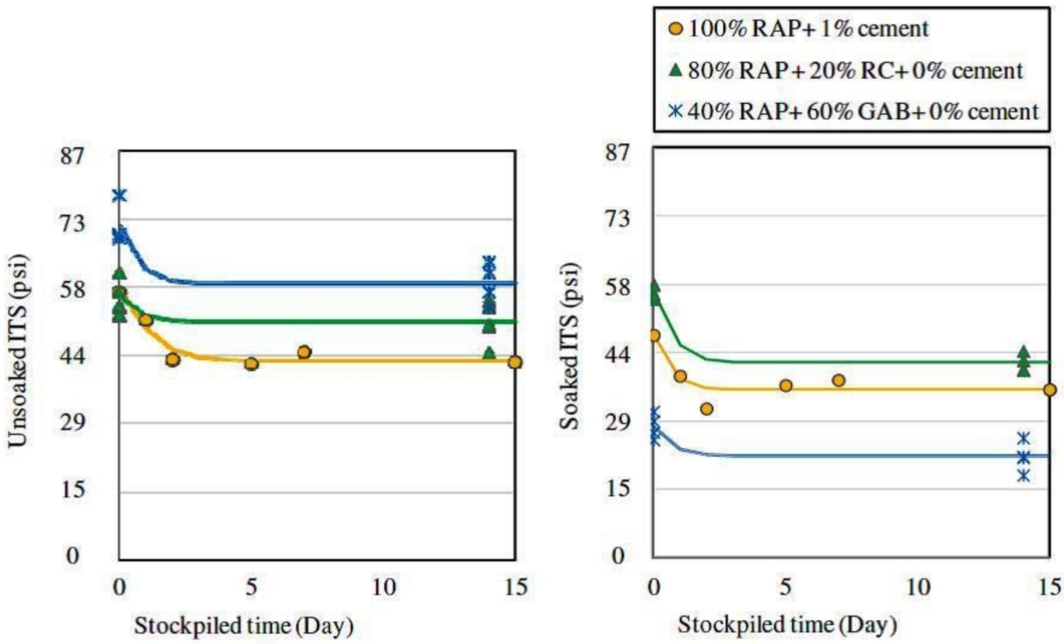


Figure 3.35 Un-Soaked and Soaked ITS Versus Stockpiling Time (Khosravifar et al. 2012)

- Permanent Deformation
  - Permanent deformation is affected by angularity (e.g., shape, hardness, and roughness) and maximum size of aggregates, compaction method (i.e., increased load sequence), and curing condition (i.e., temperature and curing time) (Wirtgen 2010).
  - Increasing foamed asphalt content increases the rutting and permanent deformation (Gonzalez et al. 2011, Kim et al. 2009).
  - Foamed asphalt stabilized with 100% RAP has a higher susceptibility to rutting compared to a foamed asphalt stabilized blend of 50% RAP with 50% soil cement. Unstabilized 100% RAP has the highest rutting resistance among the three materials (Mohammad et al. 2006).
  - Higher aging of RAP material contributes to moisture susceptibility and permanent deformation, even though resistance to permanent deformation is typically improved under dry condition (He et al. 2006).
  - Enhancing curing conditions (i.e., unsealed at 40ºC for 7 days) and adding cement significantly improved FASB resistance to permanent deformation even in the soaked condition (Fu et al. 2010b).
  - Addition of fly ash improves resistance to permanent deformation. Permanent deformation decreases with increasing fly ash content. (Wen et al. 2010).

- Foamability
  - Raising asphalt temperature and foaming water content can increase expansion ratio (ER), but decrease half-life, $t_{1/2}$ (Wirtgen 2010, Fu et al. 2011).
  - Optimum water content is obtained at the lowest asphalt temperature (320ºF) that can provide acceptable foaming characteristics (minimum requirement of ER and half-life $t_{1/2}$ is 8 and 6 seconds, respectively). In Figure 3.36, the optimum water content is shown in the top triangle (Schwartz et al. 2013).
  - Excessive fines (i.e., more than 12% passing a No.200 sieve) cause worsening dispersion of foamed asphalt and higher sensitivity to moisture (Fu et al. 2010a).

ENVIRONMENTAL PROPERTIES

- The properties include a pH within EPA groundwater limits, 6.5 to 8.5 (Edil et al. 2012). Figure 3.38a shows the pH of RAP leachate in batch tests as well as in the field. Concentrations of As, Se and Sb are slightly higher than corresponding USEPA groundwater maximum contaminant level (MCL), with peak As concentration of 37.9 μg/L, peak Se concentration of 113 μg/L and peak Sb concentration of 10.6 μg/L. Asphalt binder is probably associated with source of As, Se and Sb (Figure 3.38b-d; Edil et al. 2012).
- The pH of cement-treated RAP (no fibers) increases with increasing cement dosage, since soluble calcium hydroxide and/or portlandite are formed during hydration reactions of RAP cement with solution, raising its alkalinity (Figure 3.37; Hoyos et al. 2011).
- The pH decreases in cement-added RAP curing for longer periods (Hoyos et al. 2011).
- COD of water-soaked RAP decreases as cement dosage increases, since filterable fine materials come off the RAP sample as the cement dosage increases, reducing water impurities (Hoyos et al. 2011).

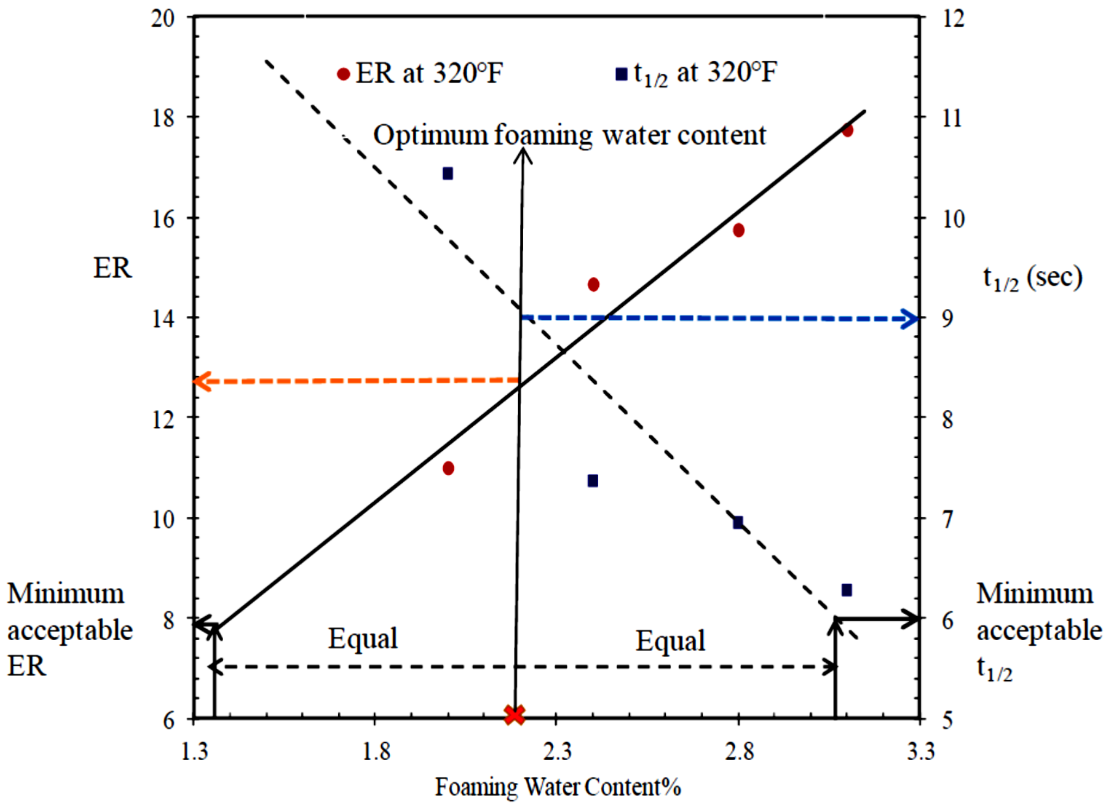


Figure 3.36 Expansion Ratio and Half-Life Tests (Schwartz et al. 2013)

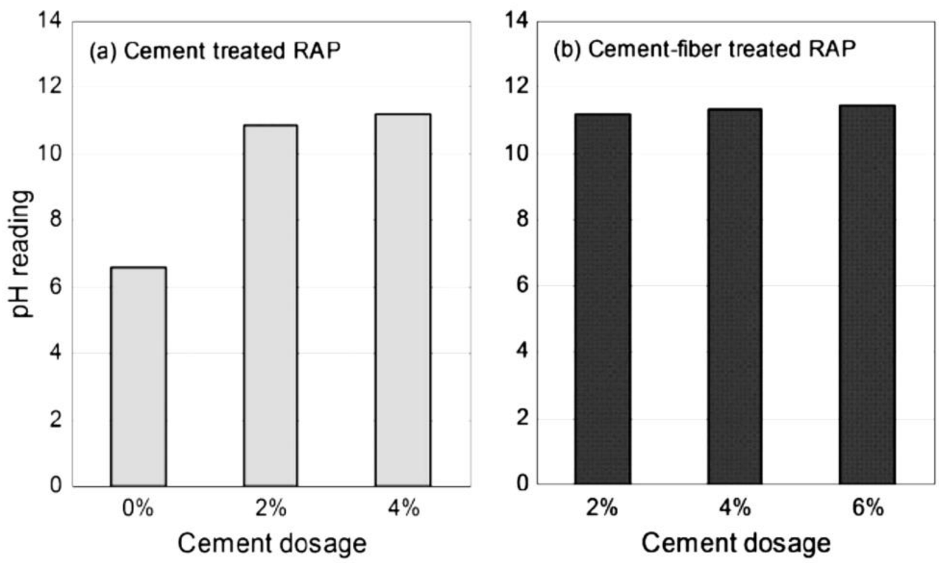
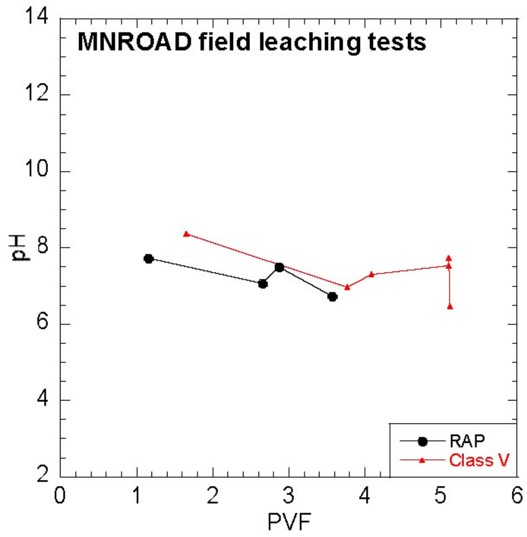
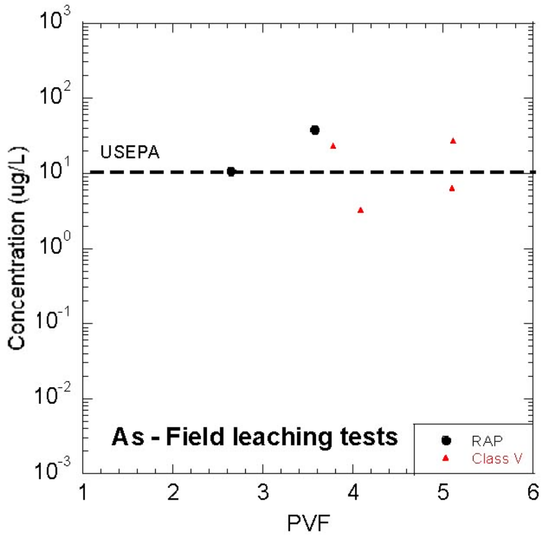


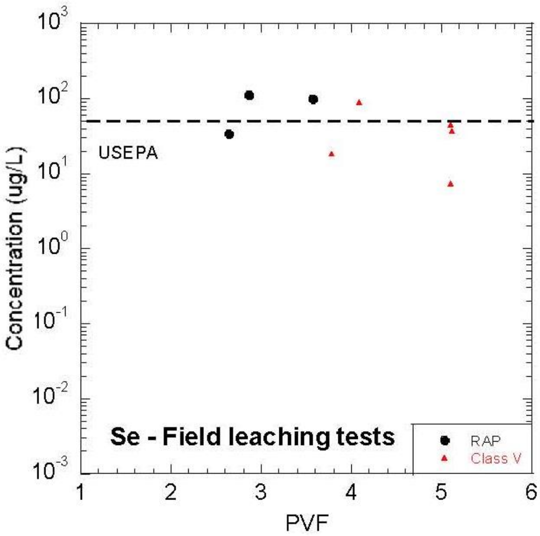
Figure 3.37 pH of Cement Treated and Cement-Fiber Treated RAP (Hoyos et al. 2011)



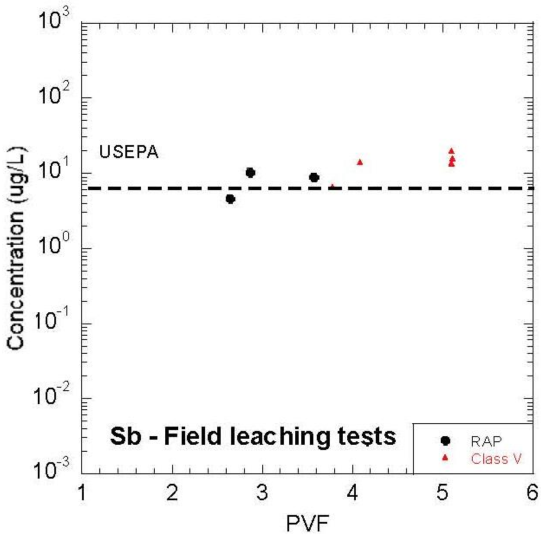
(a) MNROAD field leaching tests.



(b) As field leaching tests



(c) Se field leaching tests



(d) Sb field leaching tests

Figure 3.38 Leaching Results of RAP (Edil et al. 2012)

DESIGN RECOMMENDATIONS

- Asphalt emulsion stabilized RAP/aggregate blends must include a minimum of 50% approved base course aggregate. Amount and type of asphalt emulsion shall meet LBR strength requirement. Asphalt emulsion should not exceed 3.5% by weight (Cosentino et al. 2012).
- Portland cement stabilized RAP/aggregate blends must include a minimum of 50% approved base coarse aggregate. Amount and type of Portland cement shall meet LBR strength requirement. Portland cement content should not exceed 2% by weight (Cosentino et al. 2012).
- RAP can be blended with virgin aggregate or stabilized by cement and fly ash to increase its strength and

to reduce its creep and permanent deformations (Thakur and Han 2015).

- Several FASB mix design procedures were proposed including: ARRA (2001), Asphalt Academy (2002), Mohammad et al. (2003), Kim and Lee (2006), Wirtgen (2010) and others. Most methods are based on Marshall stability and a combination of Marshall stability and indirect tensile (IDT) strength under wet vs. dry conditions.
- For the AASHTO empirical pavement design procedure (AASHTO 1993), the structural layer coefficient is estimated from the dynamic modulus and ITS of the FASB materials. The structural layer coefficients proposed by Wirtgen (2010) are based on ITS (Figure 3.39), which represents the most widely used method for FASB structural design today.

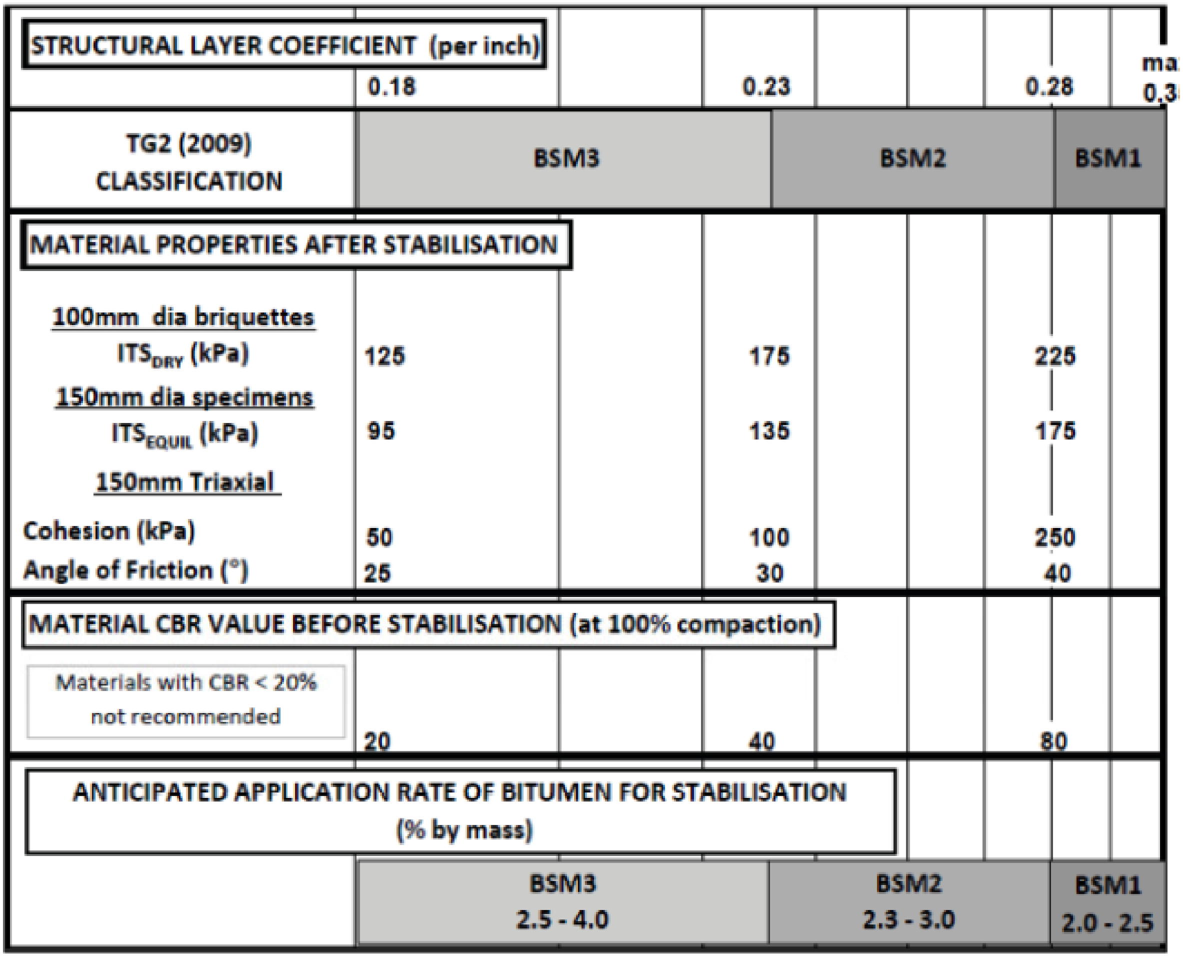


Figure 3.39 Suggested Structural Layer Coefficients for Bitumen Stabilized Materials (Wirtgen 2010)

Note. BSM=bitumen stabilized materials;

BSM1= well graded crushed stone or reclaimed asphalt with high shear strength, used as a base layer for design traffic applications of more than 6 million equivalent standard axles (MESA);

BSM2= graded natural gravel or reclaimed asphalt, moderately high strength, used as a base layer for design traffic applications of less than 6 MESA;

BSM3= soil-gravel and/or sand stabilized with higher bitumen contents, suitable for design traffic applications of less than 1 MESA;

TG2 (2009) is a guideline for design and construction of foamed bitumen treated material.

- Other studies suggest alternative structural layer coefficients based on different tests, such as average M_R and unsoaked ITS values (Table 3.23), and average M_R and unsoaked ITS minus one standard deviation (Table 3.24).

Table 3.23 Estimated Layer Coefficients Based on Average M_R and Average Unsoaked ITS (Schwartz et al. 2013)

Mixtures	FASB-H (cores)	FASB-A (F and B cores)	FASB-A (F cores)	FASB-A (B cores)
M_r at 68°F (|E*| at 5Hz,68°F), ksi	534	687	551	711
Unsoaked ITS, psi	53	76		
Methodology	Structural Layer Coefficient			
Based on asphalt layer method[1]	0.44 (0.47)[5]	0.44 (0.51)	0.44 (0.47)	0.44 (0.52)
Based on granular base method[2]	0.45	0.48	0.45	0.48
Based on Bituminous treated base method[3]	0.36	0.39	0.36	0.40
Based on ITS method for FASB[4]	0.35 (0.36)	0.35 (0.42)		

1. Figure 5.16 based on M_r.
2. $a_2 = 0.249*\log_{10}E$ (psi) - 0.977 for unstabilized base layers..
3. Figure 5.17 based on M_r
4. Figure 5.18 based on unsoaked ITS
5. Numbers in parenthesis show the layer coefficients extrapolated beyond the range of the charts.

Table 3.24 Estimated Layer Coefficients Based on Average M_R and Average Unsoaked ITS Minus One Standard Deviation (Schwartz et al. 2013)

Mixtures	FASB-H (cores)	FASB-A (F and B cores)	FASB-A (F cores)	FASB-A (B cores)
Mr at 68°F (|E*| at 5Hz,68°F) -σ, ksi	-	509	405	551
Unsoaked ITS- σ, psi	50	68		
Methodology	Structural Layer Coefficient			
Based on asphalt layer method[1]	-	0.44 (0.46)[5]	0.42	0.44 (0.47)
Based on granular base method[2]	-	0.44	0.42	0.45
Based on Bituminous treated base method[3]	-	0.35	0.32	0.36
Based on ITS method for FASB[4]	0.35	0.35 (0.40)		

1. Figure 5.16 based on M_r
2. $a_2 = 0.249*\log_{10}E$ (psi) - 0.977 for unstabilized base layers
3. Figure 5.17 based on M_r
4. Figure 5.18 based on unsoaked ITS
5. Numbers in parentheses show the layer coefficients extrapolated beyond the range of the charts.

- For stockpiled RAP, the following relation was suggested: $ITS_{soaked} = ITS_{initial\ soaked} + 27\% \times ITS_{initial\ soaked} \tanh(t)$.This predicts ITS values, where t is the stockpiling time in days and the initial soaked ITS is measured before stockpile (Khosravifar 2012).

FIELD RECOMMENDATIONS

- Initial stiffness, stiffening rate, and final stiffness should be monitored in QC/QA (Schwartz and Khosravifar 2013).
- For asphalt emulsions, evaluate laboratory curing temperature and time to determine what curing conditions give the highest field strength (Cosentino et al. 2012).
- Nuclear moisture and density gauge may be used to monitor the post-construction compaction level and field moisture content, but cannot capture stiffening of FASB during curing. Moisture corrections on the gauge are required (Schwartz et al. 2013).
- Falling weight deflectometer (FWD) measurements are appropriate for back calculating stiffness of cured FASB and other layers. However, it is not suitable for construction/immediate post construction QC/QA on the unpaved sections, since it induces excessive stress levels and plastic deformations (Schwartz et al. 2013).

BENEFITS

- FASB shows significantly better performance than bitumen asphalt in handling early traffic and resisting rain before placement of wearing course. Foamed asphalt mixes can improve flexibility and reduce brittleness of pavement (Ramanujam and Jones 2007).
- Foamed asphalt requires less curing periods, reducing cost of conventional flexible paving (Jenkins et al. 2000). Use of FASB can reduce the required thickness of pavement sections, resulting in cost savings (Schwartz and Khosravifar 2013).
- FASB may incorporate significant quantities of RAP into paving projects. Using increased amounts of fresh asphalt binder increases the energy use by 3% in MJ/tonne. By using warm mix technologies, energy consumption can be reduced by 4% in MJ/tonne and using 10% RAP will result in a 6% energy reduction (NCHRP 435).

SUGGESTED SPECIFICATIONS

Table 3.25 FASB Mix Requirements (Schwartz et. al. 2013)

Design Parameters	Value
Specimen compaction – either: (1) Marshall compaction (AASHTO T 245), number of blows (2) Gyratory compaction (AASHTO T 312), number of gyrations	 75 25
Indirect Tensile Strength (AASHTO T 283; no freeze-thaw cycle) (1) Minimum Wet Tensile Strength, psi (2) Minimum Tensile Strength Ratio (TSR), %	 50 70
Foamed Asphalt Expansion Characteristics @ 160, 170, & 180°C (1) Minimum Half-Life of Foamed Expansion, sec.[(1)] (2) Minimum Expansion Ratio [(2)]	 8 10

(1) Total time for foamed asphalt to settle to half of the maximum foamed volume.
(2) Maximum foamed asphalt volume divided by non-foamed asphalt volume.

Table 3.26 Gradation Requirements for FASB in Maryland (Schwartz et. al. 2013)

SIEVE SIZE	PERCENT PASSING
2”	100
1 ½”	90-100
¾”	60-100
No. 4	30-70
No. 200	5-15

Table 3.27 Summary of FASB Specifications (Schwartz et. al. 2013)

State	Expansion Ratio	Half-life (sec)	Gradation% <0.075mm	Marshall Compaction		Gyratory N	IDT Minimum		
				Flow	Stability		Dry (psi)	Wet (psi)	TSR %
Alaska									
Arizona	10	8	5-20	75	1625			45	30
FHWA	15	12		75				50	70
Hawaii				75					
Iowa	10	10		75		25		44	50
Maine		12				25	43		
Ohio			7-15				43	30	70
Ontario								22	50
Minnesota			7-15						
New Mexico	10	8	4-20	75	1625			45	50
Maryland	10	8	5-15	75				50	70
Virginia				75		30	45		70

Table 3.27 Summary of FASB Specifications, Schwartz et al. 2013 (continued)

State	Cure	Soak	Modified Compaction		Weather
			Density %	Moisture	
Alaska					Air≥ 40°F for 24 hours
Arizona	104°F to constant mass	77°F for 24 hours			Air≥ 10°C (2°C for 24 hours), surface≥ 2°C
FHWA			97		
Hawaii	104°F to constant mass		100 in average, none<98		≥ 50°F
Iowa	T283	T283	97		Air≥ 10°C, surface≥ 4°C
Maine	40°C for 72 hours	77°F for 20 min, 50 mm Hg for 45 min, 77°F for 10 min	92		
Ohio	140°F for 48 hours	77°F for 20 min, 50 mm Hg for 45 min, 77°F for 10 min	100 in average, none<98		Air≥ 60°F
Ontario	60°C for 72 hours		97		
New Mexico	104°F to constant mass	77°F for 24 hours	97		Air≥ 50°F, surface≥ 40°F, no rain, no temperature< 36°F expected for 24 hours
Maryland	104°F to constant mass	77°F for 20 min, 50 mm Hg for 50 min, 77°F for 10 min	95	OMC±2%	Air≥ 50°F, surface≥ 40°F, no rain, no temperature< 36°F expected for 24 hours
Virginia	40°C for 72 hours	25°C for 24 hours	98		Air≥ 50°F, no freezing temperature for 48 hours

3.2.3 RAP in Drainage/Fill

MECHANICAL PROPERTIES

- Gradation and Specific Gravity
  - According to the Unified Soil Classification System (USCS) classification, RAP is classified as well-graded gravel, while conventional fill material (CFM) is classified as poorly-graded gravel, (Figure 3.40; Cosentino et al. 2003, Rathje et al. 2001, Rathje et al. 2006, Soleimanbeigi et al. 2014).
  - RAP has similar gradation to that of reference materials suggested for structural fill construction, while conventional fill material consists of smaller particles (Rathje et al. 2001).

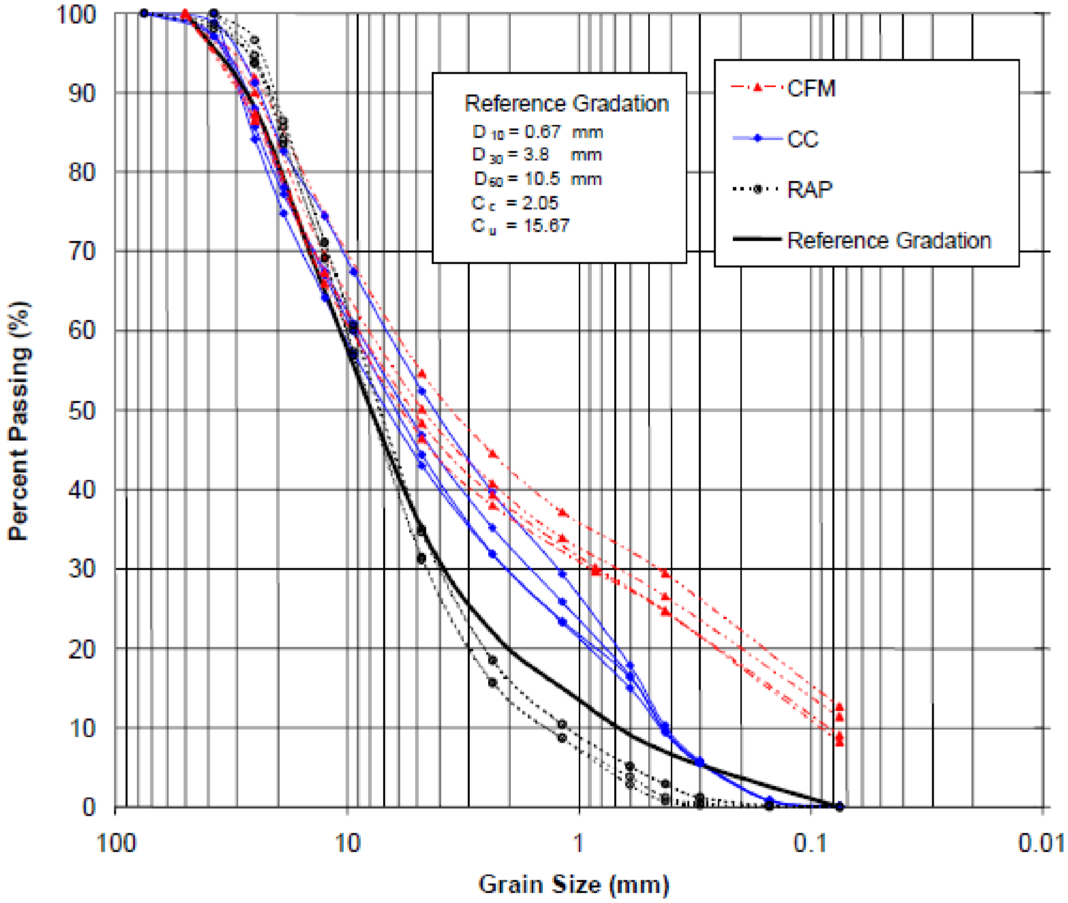


Figure 3.40 Proposed Reference Gradation for All Testing Materials (Rathje et al. 2001)
Note. Samples comply with proposed reference gradation to prevent grain size distribution affecting test results. CC=RCA; CFM=conventional fill materials.

  - According to the American Association for State Highway and Transportation Officials (AASHTO) classification system, RAP is classified as A-1-a, indicating good drainage (Doig 2000, Montemayor 1998).
  - The specific gravity of RAP is about 2.30, which is lower than that of conventional fill material, since the bitumen coating of RAP causes the formation of a large impermeable solid volume (Rathje et al. 2001).

◆ Drainage Properties

- Hydraulic conductivity (k) indicates how well water flows through a particular soil. RAP has a high k value, comparable to that of conventional fill materials (Table 3.28). RAP has good drainage characteristics, and is regarded as a freely drainable material (Rathje et al. 2006).
- Though RAP has high capacity for drainage, RAP-soil mixture is a poorly drained material. Hydraulic conductivity linearly decreases with increasing soil content (Figure 3.41). The fines in soil weaken the drainage capacity, since fines fill the intergranular voids, reduce effective pore size, increase friction and hence restrict flow through the material (Cosentino et al. 2003).

Table 3.28 Summary of Hydraulic Conductivity Results (Rathje et al. 2006)

Effective Confining Pressure, σ'_c (psi)	Hydraulic Conductivity, k (x 10^{-4} cm/s)		
	RAP	CC	CFM
5	38.4	1.6	13.8
20	25.7	0.84	21.8
30	27.7	0.18	6.4
40	12.7	0.12	4.5
50	5.5	0.11	6.0

Note. CC=RCA; CFM=conventional fill materials.

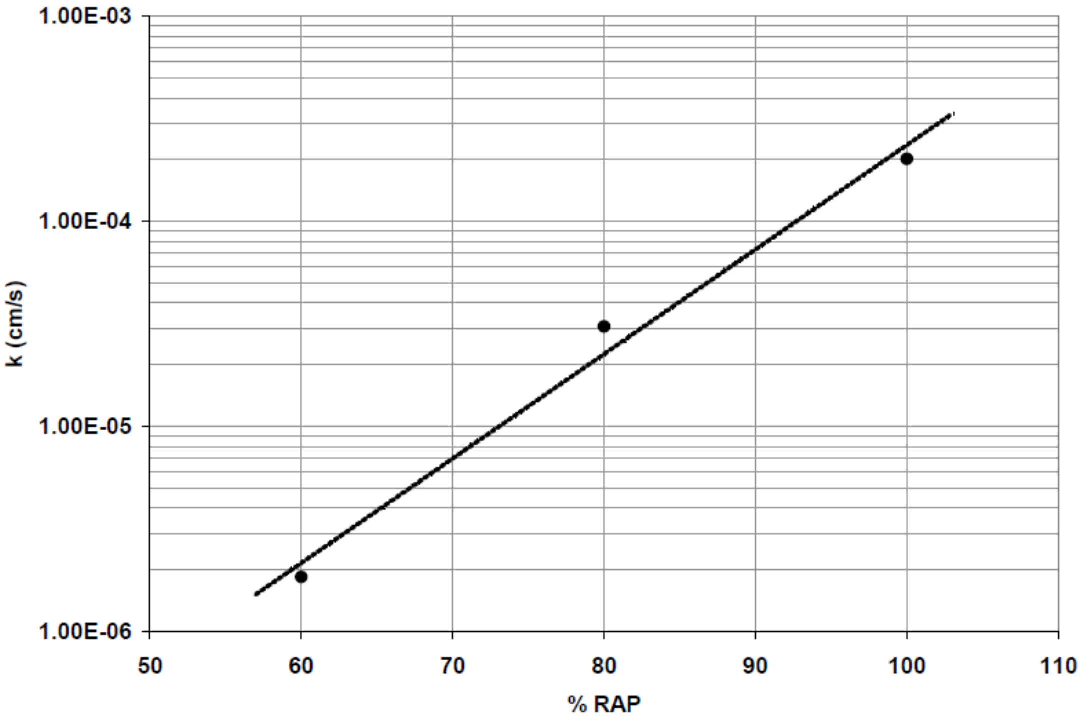
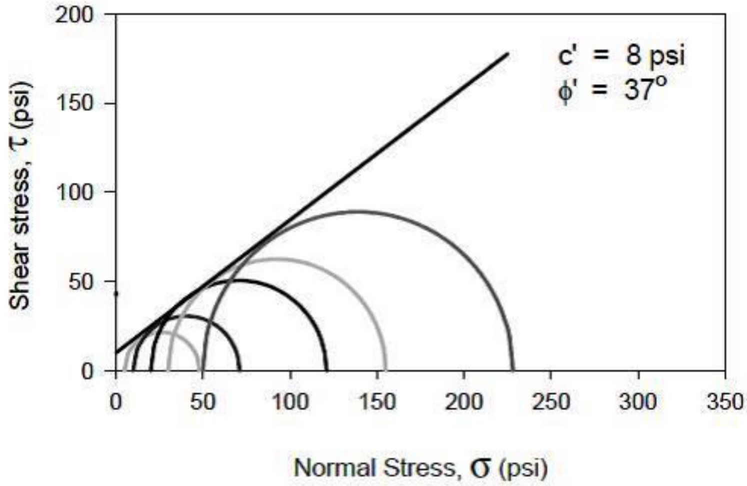


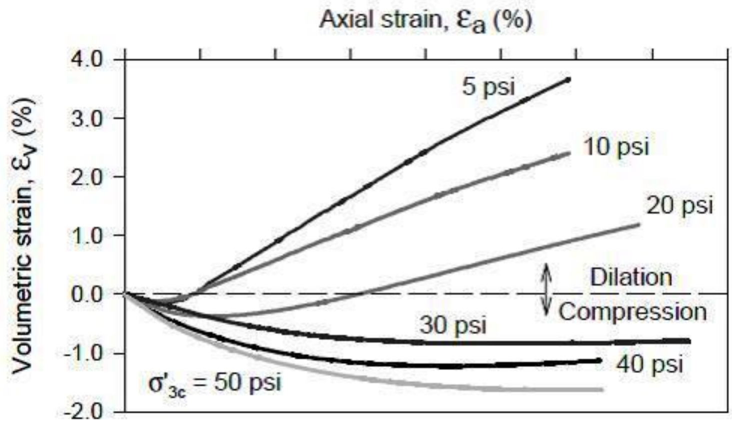
Figure 3.41 Permeability vs Percent RAP for RAP-Soil Mixtures (Cosentino et al. 2003)

◆ Strength and Stiffness

■ In triaxial compression tests, RAP showed strain-hardening behavior. RAP has an effective friction angle of 37° and effective cohesion of 8 psi, as a result of residual bitumen bonding effect (Figure 3.42a). Volumetric strains for RAP exhibited dilation at low confining pressures and contraction at higher confining pressures (Figure 3.42b; Rathje et al. 2006).



(a) Shear stress



(b) Volumetric strain

Figure 3.42 Consolidated-Drained Triaxial Test Results for RAP Specimens (Rathje et al. 2006)

■ In large-scale direct shear tests, excessive creep of RAP indicated that creep rupture, rather than shear failure, will come first. Thus, a direct shear test may not be applicable to RAP (Rathje et al. 2006).

■ Friction angles of RAP-soil mixtures decreased with increasing soil content, since soil may reduce the grain-to-grain contact and let larger particles float freely, creating a plane to facilitate particles slipping and dislocating under a load. Cohesion of RAP-soil mixtures increased as the percentage of soil increased, likely due to capillary pressures of soil particles (Cosentino et al. 2003).

■ One hundred percent RAP yielded the highest resilient modulus; however, 80/20 RAP-soil mixtures yield the highest triaxial compression strength. RAP usually experiences larger plastic deformations and smaller resilient strains, which contributes to higher resilient modulus and is an indicator of increased risk for rutting and creep (Bennert et al. 2000, Cosentino et al. 2003).

- Strength and stiffness of RAP is less susceptible to moisture than that of limerock (Cosentino et al. 2003).

◆ Compaction Properties

- Compaction can be evaluated by the maximum dry unit weight (density). Higher dry unit weight indicates better compressibility (Rathje et al. 2006). The maximum dry unit weight of compacted RAP is 19.4 kN/m^3, comparable to that of compacted sand (Soleimanbeigi et al. 2014).
- Dry unit weight of RAP is not sensitive to moisture, since bitumen coating of RAP forms a large impermeable volume of solids (Rathje et al. 2001, Soleimanbeigi et al. 2014). One hundred percent RAP material gained a maximum density of 117.8 lb/ft^3 at an optimum moisture content of 8.0%. 80/20 RAP-soil mixture had the highest maximum dry density of 121.7 lb/ft^3 at an optimum moisture content of 6.0% (Figure 3.43; Cosentino et al. 2003).

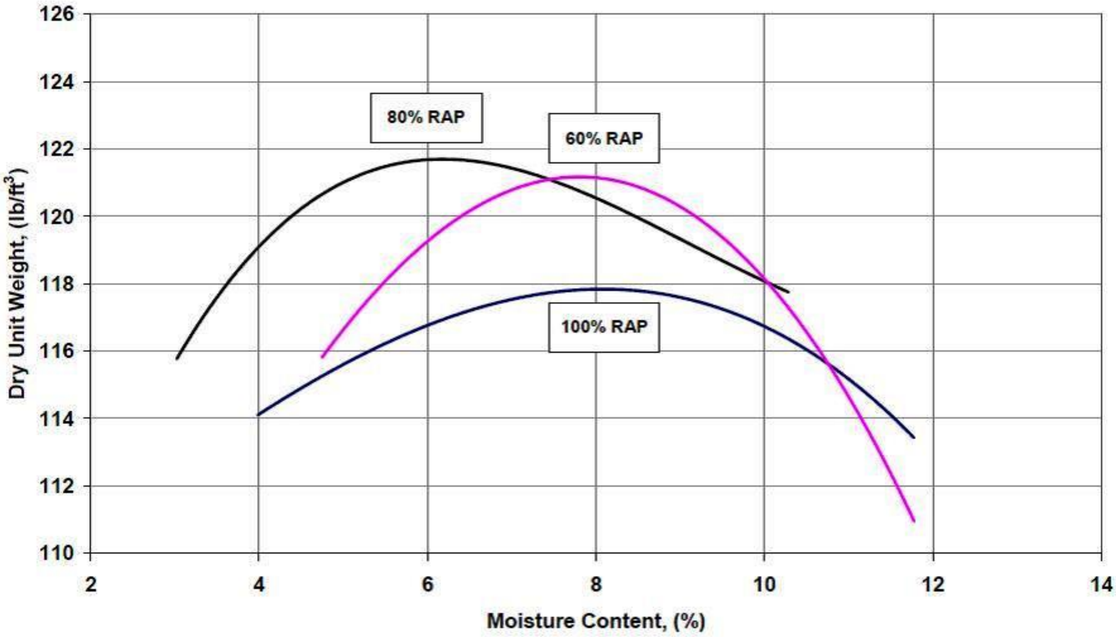


Figure 3.43 Moisture-Density Curve for RAP-Soil Mixtures (Cosentino et al. 2003)

- Density is an indicator of strength and stability of granular soil material, since densely compacted materials exhibit higher strengths with less deformation than the same loosely compacted materials. For RAP-soil mixtures, maximum density increases with RAP content until an optimal level; further increasing RAP content causes slight decrease in density (Figure 4; Cosentino et al. 2003).
- Soil content in RAP-soil mixtures also contributes to a high density, since soil consists of fine aggregates that increase density, as well as limerock bearing ratio (Cosentino et al. 2003). As the result, RAP particle breakdown during compaction also changes the density (Rathje et al. 2006). Limerock bearing ratio increases with increasing dry density. Higher limerock bearing ratio implies higher bearing strength (Cosentino et al. 2003).
- Though the compaction effort has a great influence on the maximum dry unit weight, the addition of

fine aggregates (i.e. passing the #40 sieve size), rather than double compaction effort, contributes more to high limerock bearing ratio. However, excessive fines can result in long-term total and differential settlement, leading to collapse (Rathje et al. 2006). Static compaction rather than the dynamic, vibratory or Proctor compaction is favorable to gain higher limerock bearing ratio (Cosentino et al. 2003).

- RAP has higher potential of collapse than conventional fill material and RCA, since bitumen coating prevents RAP from holding additional water, causing a low degree of initial saturation (Rathje et al. 2006). Low water content results in smaller dry unit weight, since internal capillary stresses resist the compaction of material (Morris and Delphia 1999). RAP particles are also less angular; cementation of conventional filler material and RCA further inhibits deformation of the particles and minimize its collapse potential (Rathje et al. 2006).
- Stress coefficient of compression (*n*) is an indicator of how much stress depends on compression. Higher *n* indicates the compressibility is more stress-dependent, rather than materials have higher compressibility. Compressibility of compacted RAP has higher dependency on stress level with an *n* of 0.33 (Soleimanbeigi and Edil 2015).
- Compressibility of RAP shows high sensitivity to temperature, since asphalt binder sustains applied stress by friction between particles and the viscosity of asphalt binder reduces with increasing temperature (Soleimanbeigi and Edil 2015).
- RAP compacted at high temperatures tends to gain higher stiffness and lower compressibility compared to RAP compacted at room temperature, since temperature rise increased compressive strain of compacted RAP, resulting in asphalt binder viscosity and therefore reducing void space (Soleimanbeigi and Edil 2015). Thermal preloading can effectively reduce compressibility of non-bituminous materials such as dredged material (Houston et al. 1985).

◆ <u>Permanent Performance</u>

- Creep usually consists of three stages: primary creep, secondary creep, and tertiary creep, followed by creep rupture (Figure 3.44). Primary creep occurs immediately after applying stress, but where strain rate decreases with time. In secondary creep, strain rate is at the minimum value ($\dot{\varepsilon}_{min}$) and keeps relatively constant. In the tertiary creep, strain increases again, which finally leads to complete creep rupture. Creep failure can be defined as soil rupture at the end of tertiary creep. Alternatively, some researchers define creep failure at the end of secondary creep (Rathje et al. 2006).
- Confining pressure affects creep behavior, with more significant creep deformations and more rapid creep rupture under smaller confining pressures (i.e., 5 psi and 10 psi) (Figure 3.45). Creep rupture occurs at higher stress due to increasing pore pressures caused by creep deformations. Smaller values of creep parameter (m) indicate more severe creep potential. Creep parameters for RAP are generally less than 1.0, which is comparable to a creep parameter of 0.7 for clays (Rathje et al. 2006).
- RAP with larger asphalt content may experience more severe creep. The time required to reach creep rupture decreases with increasing shear stress level. RAP generally ruptures more quickly than clay (Rathje et al. 2006).
- Settlements primarily occur within one year after completion of embankment construction. The long-term settlement of the embankment constructed with RAP is below 70 mm, lower than the allowable limit of 150 to 300 mm, if settlement is uniformly distributed along the length of the embankment, and the maximum settlement is between 300 and 600 mm (Stark et al. 2004).

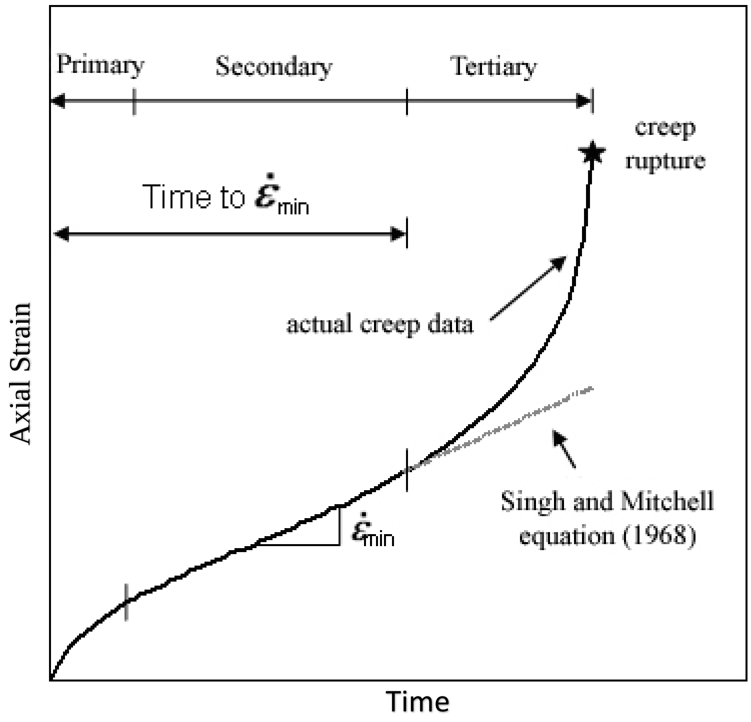


Figure 3.44 Time-Dependent Creep Deformation Under a Constant Stress Level (Rathje et al. 2006)

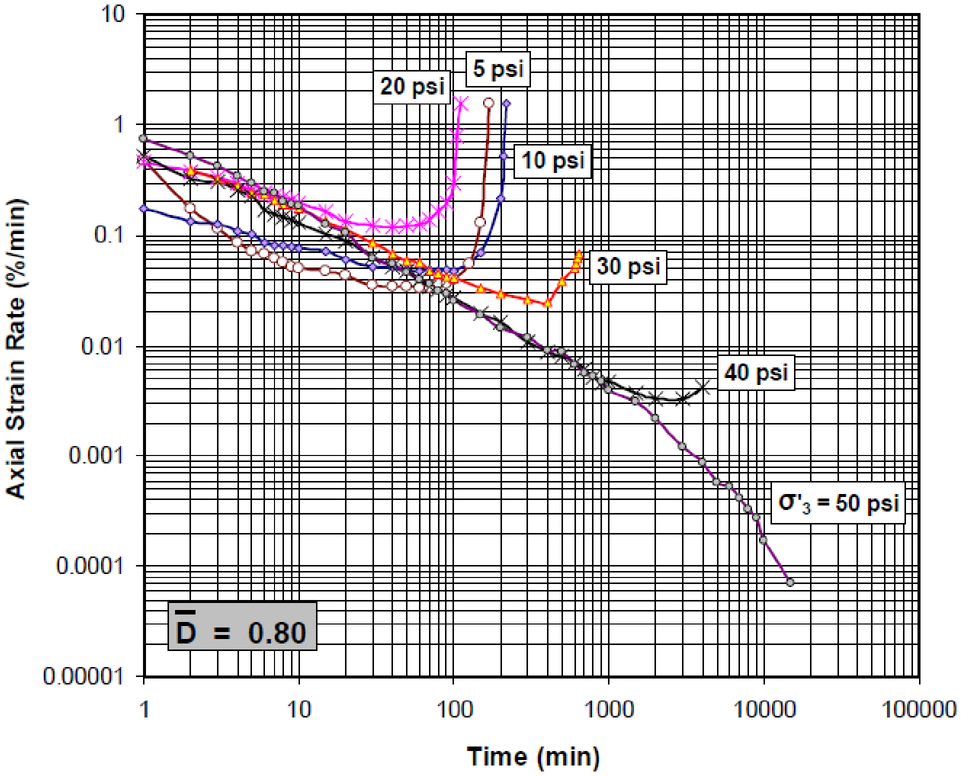


Figure 3.45 Axial Strain Rate Versus Time for RAP at Different Confining Pressure (Rathje et al. 2006)

Note. $\overline{D}$=0.80 indicates 80 percent of the ultimate strength (soil failure stress determined in strength tests). In this test, samples are designated to reach the same creep deformation at the same stress level $\overline{D}$.

ENVIRONMENTAL PROPERTIES

- RAP does not pose any threat to the environment, and most of the trace metal and PAH concentrations remain below the detection limit of the equipment used (Cosentino et al. 2003, Legret et al. 2005). Field samples collected from surface waters and groundwater, as well leachates collected from laboratory column leaching tests at different pHs, yielded concentrations far below EPA limits for drinking water (Cosentino et al. 2003).
- One out of four RAP samples from the State of Maryland showed a slight excessiveness of Al concentration in the water leaching test, according to EPA secondary-enforceable drinking water regulations (Table 3.29). The Cd concentration in the four RAP samples were all found above the limit of EPA for aquatic life and human health in fresh water and drinking water, as well as MD ATL (Maryland State aquatic toxicity limits) for fresh water. Cu concentrations were above chronic Maryland ATL, but lower than acute MD ATL. Two out of four RAP samples exhibited higher concentration of Pb with respect to chronic EPA water quality limit and chronic MD ALT for fresh water; only one sample had a Pb concentration above the acute EPA water quality limit and acute MD ALT (Aydilek and Mijic 2015).

RECOMMENDATIONS

- RAP has good drainage capacity that does not require additional drainage measures.
- The large-scale direct shear tests, which are force controlled, cannot be successfully performed on RAP because of the creep fracture of RAP prior to shear failure.
- Creep is a concern for RAP used in a structural fill; recycled hot mix asphalt, asphalt content, asphalt performance grade, aging and aggregate type all affect creep level. RAP with more asphalt content tends to experience more creep (Rathje et al. 2006).

BENEFITS

- Recycled materials replacing conventional natural aggregates helps to reduce consumption of energy and natural resources, and reduce greenhouse gas emissions associated with mining and production of natural aggregates (Gambatese and Rajendran 2005, Carpenter et al. 2007).
- More than 60 million tons of asphalt pavement material is reclaimed each year and mainly consumed in producing hot mix asphalt. However, of the amount of reclaimed asphalt outweighs what is needed by the hot mix asphalt industry. To deal with the remained RAP, other applications of RAP such as fill materials have been encouraged (Cosentino et al. 2003).

Table 3.29 Inorganic Component Concentration Analysis (Aydilek and Mijic 2015).

Pollutant	U.S. EPA MCL (mg/L)	U.S. EPA WQL (mg/L)	MD ATL (mg/L)	RAP 1 (mg/L)	RAP 2 (mg/L)	RAP 3 (mg/L)	RAP 4 (mg/L)
Aluminum	0.2	0.75	NA	0.271	0.162	0.153	0.236
Arsenic	0.05	0.15	0.15	0.00145	0.00747	0	0.00334
Boron	NA	0.75	NA	0	0	0	0
Barium	2	NA	2	0	0	0.0902	0
Calcium	NA	NA	NA	0	1.14	2.51	0.184
Cadmium	0.005	0.002 (acute)	0.00025 (chronic)	0.002 (acute)	0.00025 (chronic)	0.00741	0.00894
Cobalt	NA	NA NA	0 0	0.00469	0	0.00700	0.00682
Chromium	0.1	0.011 (Cr(VI), chronic)	0.011 (Cr (VI, chronic)	0.00669	0.00384	0.00346	0.00429
Copper	1	0.003873-0.06036	0.013 (acute)	0.009 (chronic)	0.0283	0.191	0.0115
Iron	0.3	1 (chronic)	-- 0.011	0	0.00115	0.00100	0.0113
Mercury	0.002	0.00077 (chronic)	0.00077 (chronic)	0	0	0	0
Potassium	NA	NA	NA	0	0.279	0	0
Lithium	NA	NA	NA	0	0	0	0
Magnesium	NA	NA	NA	0	0	0	0
Manganese	0.05	NA	NA	0	0	0	0
Sodium	NA	NA	NA	283	259	266	266
Nickel	NA	0.052	0.052	0	0	0	0
Phosphorus	NA	NA	NA	0	0	0	0
Lead	0.15	0.065 (acute)	0.0025 (chronic)	0.065 (acute)	0.0025 (chronic)	0	0
Silicon	NA	NA NA	0.907 0.827	0.0709	0.755	0.0290	0.0788
Vanadium	NA	NA	NA	0	0	0	0
Zinc	5	0.12	0.12	0	0	0	0
pH	6.5-8.5	6.5-9	NA				

Note. MCL=maximum contaminant levels for drinking water; MCL for Al is based on a secondary drinking water regulation; WQL=water quality limits for protection of aquatic life and human health in fresh water; MD ATL=Maryland State aquatic toxicity limits for fresh water; NA=not available.

SUGGESTED SPECIFICATIONS

Table 3.30 Summary of Laboratory Tests and Procedures (Cosentino et al. 2003)

Test	Procedure	Description
Sieve Analysis	AASHTO T27	Sieve analysis of fine and coarse aggregates.
Atterberg Limits	AASHTO T89 AASHTO T90	Determine the liquid limit of soils. Determine the plastic limit and plasticity index of soils.
Specific Gravity	AASHTO T100	Specific gravity of soils.
Dry Rodded Unit Weight	ASTM C29	Standard test method for unit weight and voids in aggregate.
Permeability	AASHTO T215 ASTM D5084	Permeability of granular soils (constant head). Standard test method for measurement of hydraulic conductivity of saturated porous materials using a flexible wall permeameter.
Static Triaxial Compression	ASTM D4767	Standard test method for consolidated undrained triaxial compression test for cohesive soils.
Resilient Modulus	LTTP Protocol P46	Resilient modulus of unbound granular base/subbase materials and subgrade soils.
Creep Test	ASTM D1557	Measure the creep failure strength.
Proctor Compaction Test	ASTM D698	Compact samples.
Hydraulic Conductivity Test	ASTM D5084	Measure drainage properties.
Column Leaching Test	ASTM D2434 ASTM D4874	Permeability of granular soils (constant head). Leaching solid material in a column apparatus.

3.2.4 RAP in HMA

MECHANICAL PROPERTIES

◆ Stiffness

- HMA mixtures with 100% RAP replacement provide the highest stiffness values regardless of testing frequency, moisture condition and asphalt type (Figure 3.46). Moisture negatively affects mixture's stiffness (Reyes-Ortiz et al. 2012).

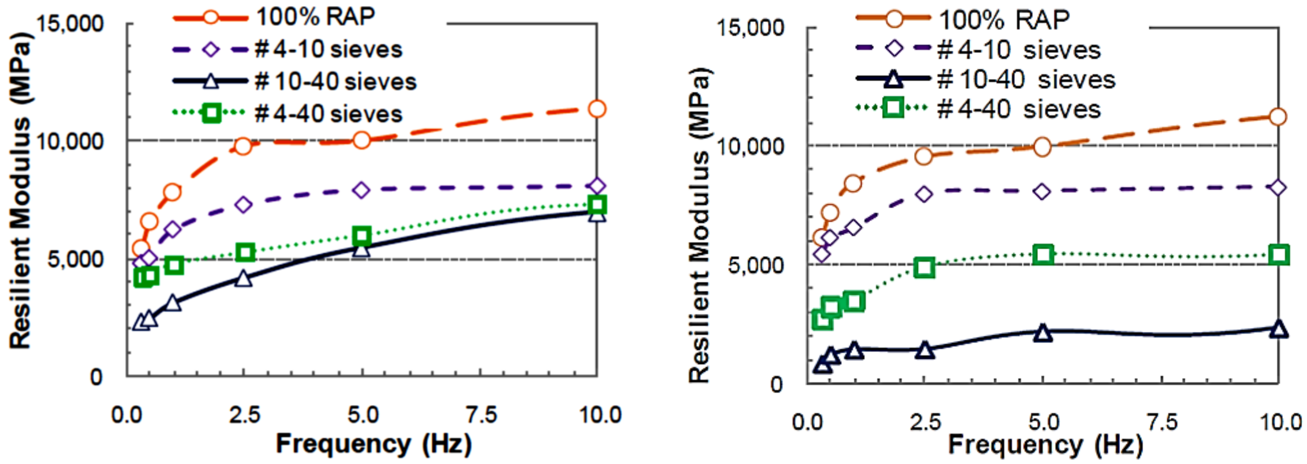


Figure 3.46 M_R of Specimens in Dry (left) and Wet (right) Condition (Reyes-Ortiz et al. 2012)

- With an increasing RAP percentage, asphalt mixture stiffness increases (Figure 3.47). Blending of RAP binder with virgin binder improves mixture properties. Testing variability increases with RAP content due to variability in RAP binder content and gradation, especially in coarse RAP fraction (Colbert et al. 2012). Higher percentages of fine RAP fraction can result in less variability of bitumen content and gradation (Don and Richmond 2007). Stiffness of asphalt mixtures increase as temperatures decreases (Colbert et al. 2012).

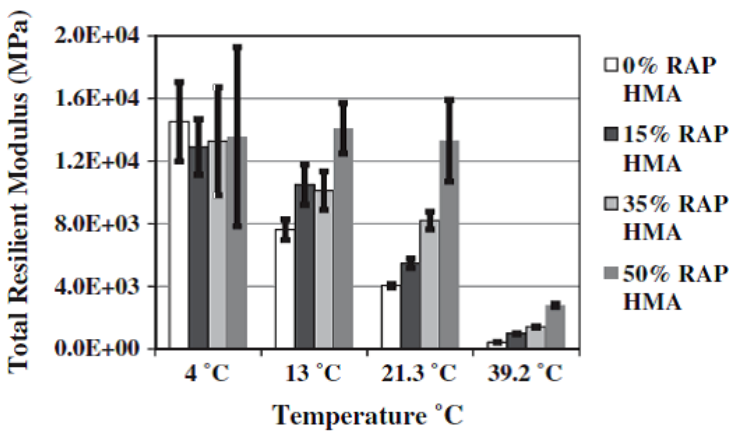


Figure 3.47 M_R Determined from the Average of Three Asphalt Mixture Specimens (Colbert et al. 2012)

- RAP mixtures have higher dynamic modulus than mixtures with virgin material. Loading frequency affects dynamic modulus (Li et al. 2008). Large modulus variability for high percentages of RAP is typically observed (Colbert et al. 2012).

- Use of rejuvenators (i.e., motor oil, OIL, ACF Iterlene 1000) can improve flexibility of RAP mixtures by decreasing stiffness modulus and increasing the phase angle (Silva et al. 2012).
- Crumb rubber (i.e., ground crumb rubber, cryogenic ground rubber) can increase resilient modulus of RAP mixtures (Xiao et al. 2009).

◆ <u>Indirect Tensile Strength</u>

- RAP replacement of 50% or more has higher ITS (Indirect Tensile Strength) compared to conventional HMA mixtures (Pereira et al. 2004, Celauro et al. 2010), due to higher dissipated energy for recycled mixtures (Valdes et al. 2011).
- 100% RAP mixtures have the highest ITS regardless of testing frequency, moisture condition and asphalt type (Figure 3.48). Water has negative effects on the mixture's ITS (Reyes-Ortiz et al. 2012).

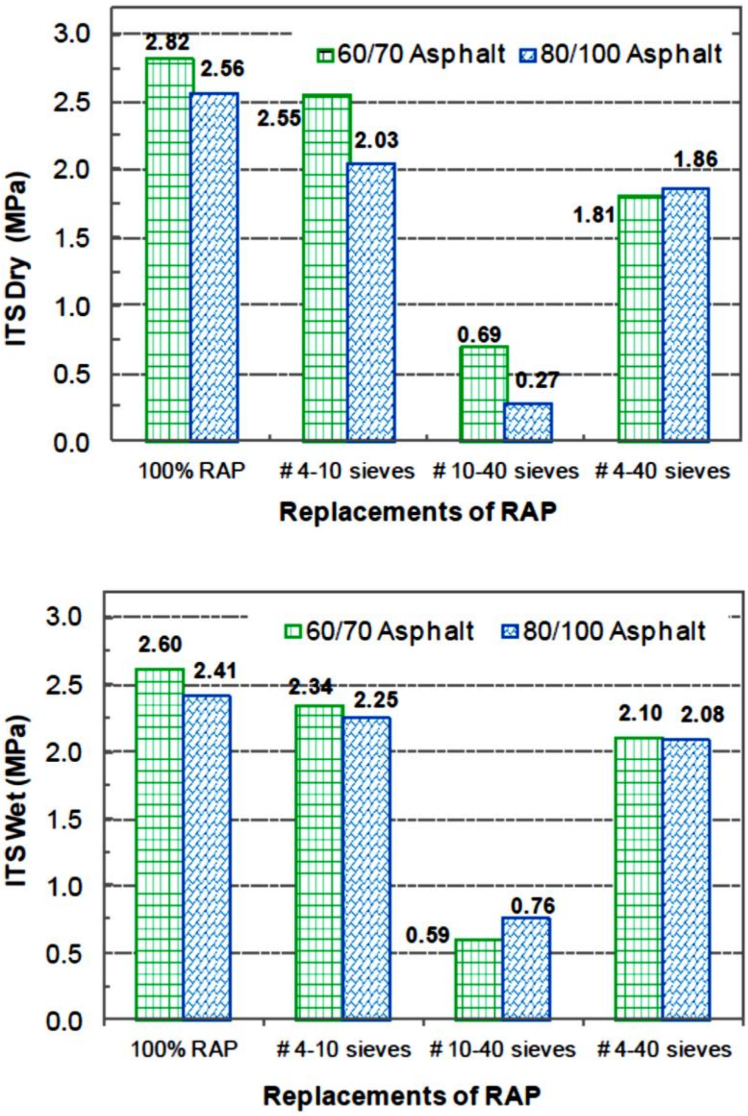


Figure 3.48 ITS for Specimens Tested in Dry (top) and Wet (bottom) Conditions (Reyes-Ortiz et al. 2012)

Note. 60/70 asphalt and 80/100 asphalt penetration grades (AASHTO M 20 and ASTM D 946).

- Rejuvenator additives in RAP-asphalt mixtures improve fracture resistance, since deformation on failure increases. However, ITS decreases at the same time (Figure 3.49). Rejuvenators reduce air void content in RAP-asphalt mixtures, because of degraded viscosity, improved workability, and raised binder content (Silva et al. 2012).

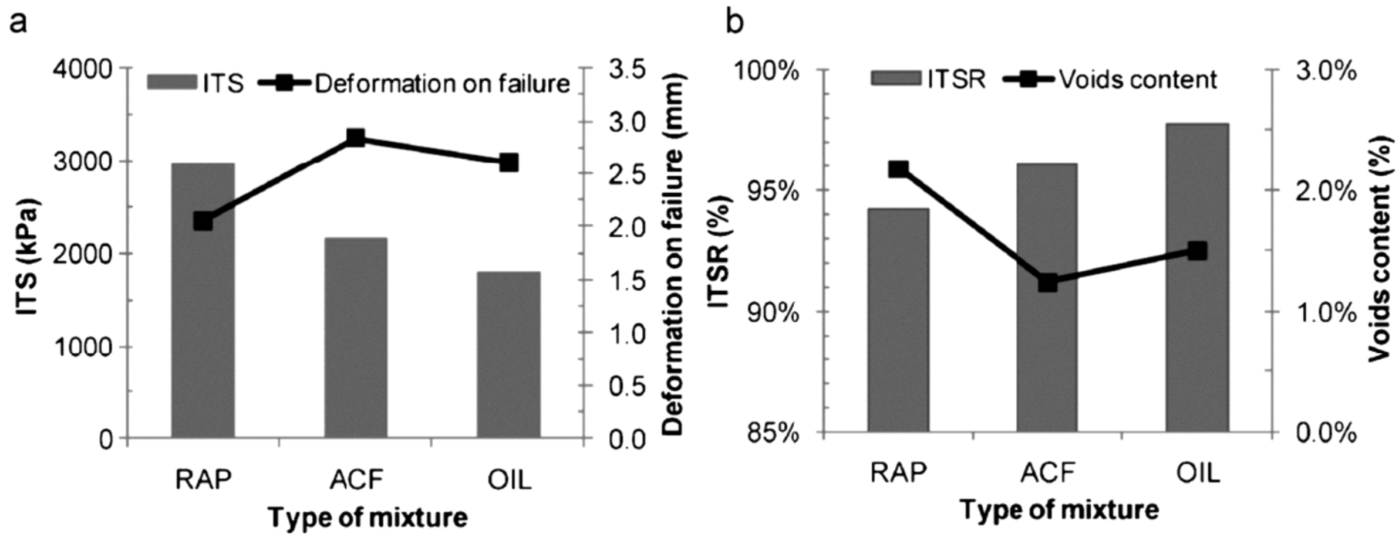


Figure 3.49 Results of (a) Tensile Strength (ITS vs. Deformation on Failure) and (b) Water Sensitivity Tests (ITSR vs. Air Voids Content), (Silva et al. 2012)

- Permanent Deformation
  - Higher content of RAP (up to 50%) improves rutting resistance (Figure 3.50; Colbert et al. 2012).

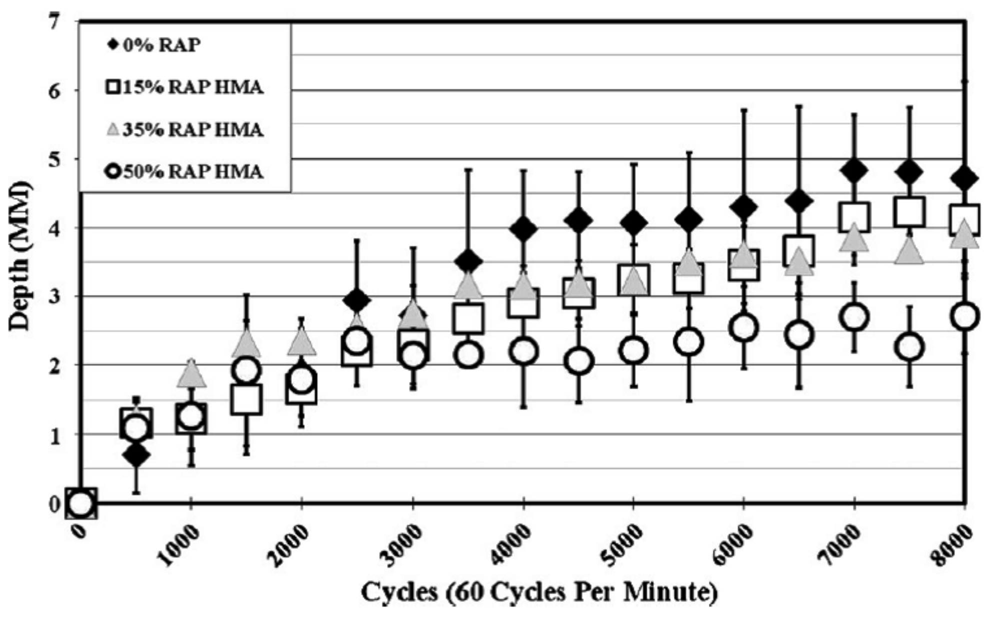


Figure 3.50 Rutting Results for Different Percentages of Asphalt Mixtures (Colbert et al. 2012)

- Use of rejuvenators (ACF and OIL) increase rutting (Figure 3.51), since rejuvenators increase binder content, reducing mixture viscosity (Silva et al. 2012). Crumb rubber additives improve rutting resistance (Xiao et al. 2009).
- RAP mixtures with rejuvenators (ACF and OIL) are more susceptible to aging than unmodified RAP mixture, since the binder of unmodified RAP is already hardened and unable to change properties at service temperature (Silva et al. 2012).

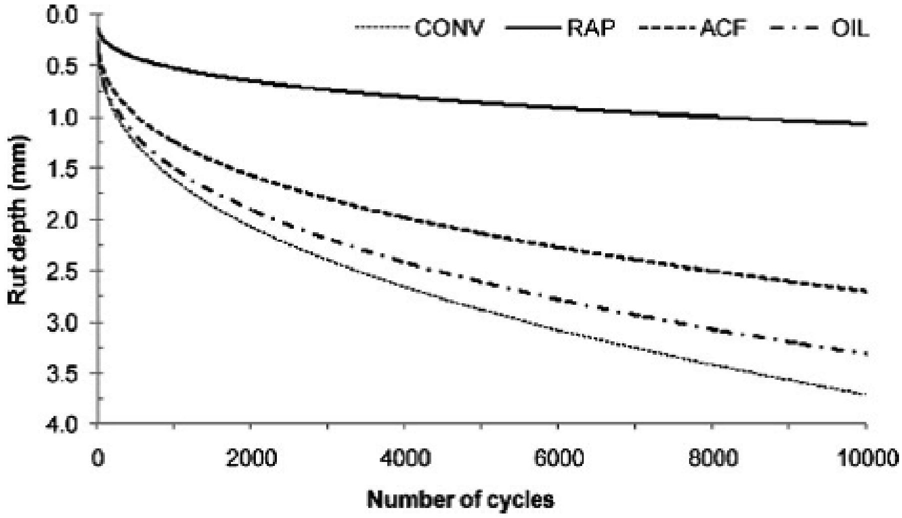


Figure 3.51 Wheel Tracking Test Results for Different Rejuvenators (Silva et al. 2012)

- Fatigue Cracking Resistance
  - HMA mixtures with 100% RAP have higher fatigue resistance compared to conventional HMA, due to high fines content produced by milling operations. However, high fines content will exacerbate rutting (Silva 2005).
  - Aged asphalt binder exhibits high resistance to low temperature cracking and fatigue cracking. Aged binder in RAP forms a layered system coating to aggregate particles, reducing stress concentration, and serving as a cushion layer between the hard aggregate and the soft binder mastic (Figure 3.52), and hence improving fatigue resistance. However, moisture may diffuse into the binder and weaken the layered system, reducing the long-term fatigue performance (Huang et al. 2005a).

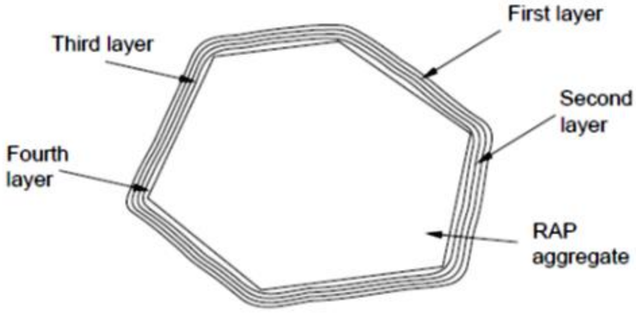


Figure 3.52 Layers of Asphalt Binder Coating RAP Aggregate (Huang et al. 2005)

- At low temperature, viscosity increases and phase angle decreases with increasing RAP binder percentage, due to the low viscosity and elasticity of the binder. Thus, ductility decreases and fatigue resistance decreases as well (Lee et al. 2002).
- Use of rejuvenators (ACF and OIL) increases flexibility and fatigue resistance (Silva et al. 2012). The use of crumb rubber in RAP mixtures compensates for loss of fatigue (Xiao et al. 2009).

ENVIRONMENTAL PROPERTIES

- All heavy metals were found to be below detection limits (BDL), except chromium (Table 3.31). Chromium was measured at 0.1 mg/l, 50 times below the level considered hazardous per RCRA (Resource Conservation Recovery Act). The leached Chromium is associated with slag, which is added in producing asphalt (Townsend 1998).

Table 3.31 TCLP Metals in Asphalt Mixture (Townsend 1998).

Parameter	Result (mg/L)	Detection limit (mg/L)
Barium	BDL	2.00
Cadmium	BDL	.020
Chromium	0.10	.010
Lead	BDL	.200
Silver	BDL	.040
Arsenic	BDL	.005
Selenium	BDL	.005
Mercury	BDL	.005

- Chromium and Lead were below the maximum concentration of contamination for TCLP (5 mg/L and 5 mg/L respectively), but testing results indicated leachate of Chromium and Lead did not meet drinking water standards (0.1 mg/L and 0.015 mg/L respectively) (Table 3.32). Lead contamination is possibility associated with leaded gasoline or crankcase oil. Chromium is related with wearing metal on vehicles or from slag aggregate (Townsend 1998).

Table 3.32 TCLP Metals in Six RAP Samples (Townsend 1998)

Parameter	Sample #1, (mg/L)	Sample #2, (mg/L)	Sample #3, (mg/L)	Sample #4, (mg/L)	Sample #5, (mg/L)	Sample #6, (mg/L)	Detection limit (mg/L)
Barium	BDL	.400	.360	.330	BDL	BDL	2.00
Cadmium	BDL	BDL	BDL	BDL	BDL	BDL	.020
Chromium	BDL	.520	BDL	BDL	BDL	BDL	.050
Lead	BDL	1.80	BDL	BDL	BDL	BDL	.200
Silver	BDL	BDL	BDL	BDL	BDL	BDL	.040
Arsenic	BDL	BDL	BDL	BDL	BDL	BDL	.005
Selenium	BDL	BDL	BDL	BDL	BDL	BDL	.005
Mercury	BDL	BDL	BDL	BDL	BDL	BDL	.005

- Volatile organic compounds (VOCs) and semivolatile organic compounds were BDL in HMA mixtures with RAP. Polycyclic aromatic hydrocarbons (PAHs), a part of the semivolatile organic compounds, were BDL except Naphthalene. Naphthalene was detected at 0.25 mg/L but still well below the regulatory guideline of 7.5 mg/L (Table 3.33; Townsend 1998).
- Polychlorinated biphenyls (PCBs) and semivolatile organic compounds were BDL in RAP samples. Polycyclic aromatic hydrocarbons, part of the semivolatile organic compounds, were below detection limits (Table 3.34; Townsend 1998).

Table 3.33 PAHs in HMA Mixture (Townsend 1998)

Parameter	Result, (μg/L)	Detection limit (μg/L)
Naphthalene	.25	.096
Acenaphthylene	BDL	.150
Acenaphthene	BDL	.194
Fluorine	BDL	.023
Phenanthrene	BDL	.033
Anthracene	BDL	.015
Fluoranthene	BDL	.037
Pyrene	BDL	.040
Benz(A)Anthracene	BDL	.048
Chrysene	BDL	.017
Benzo(B)Fluoranthene	BDL	.020
Benzo(K)Fluoranthene	BDL	.022
Benzo(A)Pyrene	BDL	.023
Dibenzo(A,H)Anthracene	BDL	.018
Benzo(G,H,I)Perylene	BDL	.036
Indeno(1,2,3-CD)Pyrene	BDL	.021

Table 3.34 PAHs in Six RAP samples (Townsend 1998)

Parameter	Sample #1, (µg/L)	Sample #2, (µg/L)	Sample #3, (µg/L)	Sample #4, (µg/L)	Sample #5, (µg/L)	Sample #6, (µg/L)	Detection limit, (µg/L)
Naphthalene	.490	BDL	.490	.300	BDL	BDL	.130
Acenaphthylene	BDL	BDL	BDL	BDL	BDL	BDL	.200
Acenaphthene	.140	BDL	.140	BDL	BDL	BDL	.130
Fluorine	BDL	BDL	BDL	BDL	BDL	BDL	.015
Phenanthrene	BDL	BDL	BDL	BDL	BDL	BDL	.130
Anthracene	BDL	BDL	BDL	BDL	BDL	BDL	.017
Fluoranthene	BDL	BDL	BDL	BDL	BDL	BDL	.017
Pyrene	BDL	BDL	BDL	BDL	BDL	BDL	.060
Benz(A)Anthracene	BDL	BDL	BDL	BDL	.017	BDL	.017
Chrysene	BDL	BDL	BDL	BDL	BDL	BDL	.033
Benzo(B)Fluoranthene	BDL	BDL	BDL	BDL	BDL	BDL	.023
Benzo(K)Fluoranthene	BDL	BDL	BDL	BDL	.050	BDL	.017
Benzo(A)Pyrene	BDL	BDL	BDL	BDL	BDL	BDL	.240
Dibenzo(A,H) Anthracene	BDL	BDL	BDL	BDL	BDL	BDL	.068
Benzo(G,H,I)Perylene	BDL	BDL	BDL	BDL	BDL	BDL	.110
Indeno(1,2,3-CD) Pyrene	BDL	BDL	BDL	BDL	BDL	BDL	.022

DESIGN RECOMMENDATIONS

- The mix property variability increased with increasing RAP content, therefore requiring a higher number of samples for quality control and quality assurance (NCHRP 435).
- Central plant recycling high RAP content and/or using improper virgin binder grade easily leads to accelerated fatigue and thermal cracking (NCHRP 435).
- Large and conical RAP stockpiles are preferred, since low, horizontal and flat stockpiles are subject to greater moisture accumulation than tall, conical stockpiles. Covering RAP stockpile is recommended to prevent moisture. It is also suggested to avoid condensation under the trap. Crush and screen the RAP to derive consistent properties and meet the gradation and volumetric requirements (NCHRP 435).

FIELD RECOMENDATIONS

- Binder content and gradation should be verified. Moisture content of the RAP should be verified if moisture in the mixture becomes a concern (NCHRP 452).
- A minimum stockpile frequency of testing is recommended, based either on the amount of RAP used or on days of production. Additional tests are needed if the RAP stockpile changes mixture properties (NCHRP 452).

BENEFITS

- Use of RAP provides energy savings. Using increased amounts of virgin asphalt binder implies higher energy use, in MJ/tonne. Using 10% RAP resulted in a 6% reduction in fuel cost. About 13% less energy was necessary to produce and place the lower lifts (i.e., binder course). Increasing the amount of RAP in HMA reduces the energy use. Using 50% RAP in HMA applications reduces energy consumption to about the level to produce cold mix asphalt (Table 3.35); CIPEC 2005.

- Use of RAP can eliminate disposal problems, reduce land use, and save natural materials and good quality aggregates (Olard et al. 2008).
- Use of RAP in HMA mixtures can produce a stable pavement structure at a lower cost than conventional materials (Olard et al. 2008).

Table 3.35 Energy Use for Various Roadway Applications (CIPEC 2005)

Product	Energy Use, MJ/tonne						% Reduction in Energy Use
	Binder	Aggregate	Manufacture	Transport	Laydown	Total	
Hot Mix Asphalt Concrete	279	38	275	79	9	680	0
High Modulus Hot Mix Asphalt Concrete	284	38	289	79	9	699	−3
Warm Asphalt Mix	294	38	234	80	9	654	4
Binder Course Hot Mix Asphalt	196	36	275	75	9	591	13
Recycled Hot Mix Asphalt Concrete with 10% RAP	250	35	275	73	9	642	6
Recycled Hot Mix Asphalt Concrete with 20% RAP	157	33	275	64	9	538	21
Recycled Hot Mix Asphalt Concrete with 30% RAP	137	30	275	58	9	510	25
Recycled Hot Mix Asphalt Concrete with 50% RAP	98	25	275	47	9	454	33
Emulsion-Based Cold Mix Asphalt	314	36	14	86	6	457	33

SUGGESTED SPECIFICATIONS

Table 3.36 AASHTO Test Methods (NCHRP 435)

AASHTO M320	Standard specification for performance-graded asphalt binder
AASHTO PP19	Standard practice for volumetric analysis of compacted hot mix asphalt
AASHTO R30	Standard practice for mixture conditioning of hot mix asphalt
AASHTO T164	Quantifiable extraction of bitumen from bituminous paving mixtures
AASHTO T166	Bulk specific gravity of compacted hot mix asphalt mixtures using saturated surface dry specimens
AASHTO T170	Standard method of test for recovery of asphalt binder from solution by Abson method
AASHTO T180	Standard method of test for moisture density relations of soils using a 4.54 kg (10 lb) rammer and a 457 mm (18 in.) drop
AASHTO T209	Theoretical maximum specific gravity and density of hot mix asphalt paving mixtures
AASHTO T240	Test method for effect of heat and air on a moving film of asphalt (rolling thin film oven test)
AASHTO T283	Standard method of test for resistance of compacted for mix asphalt (HMA) of moisture induced damage
AASHTO T312	Standard method of test for preparing and determining the density of hot mix asphalt (HMA) specimens by means of the Superpave gyratory compactor
AASHTO T313	Standard method of test for determining the flexural creep stiffness of asphalt binder using the bending beam rheometer (BBR)
AASHTO T315	Test method for determining rheological properties of asphalt binder using a dynamic shear rheometer
AASHTO T319	Quantitative extraction and recovery of asphalt binder from asphalt mixtures
AASHTO T321	Standard method of test for determining the fatigue life of compacted hot mix asphalt (HMA) subjected to repeated flexural bending
AASHTO T322	Determining the creep compliance and strength of hot mix asphalt (HMA)
AASHTO T99	Standard method of test for moisture-density relations of soils using a 2.5 kg (5.5 lb) rammer and a 305 mm (12 in.) drop
AASHTO TP2	Method for the quantitative extraction and recovery of asphalt binder from hot mix asphalt (HMA)
AASHTO TP31	Standard test method for determining the resilient modulus of bituminous mixtures by indirect tension
AASHTO TP62	Standard method of test for determining dynamic modulus of hot mix asphalt (HMA)
AASHTO TP7	Standard test method for determining the permanent deformation and fatigue cracking characteristics of hot mix asphalt (HMA) using the simple shear test (SST) device
AASHTO TP9-96	Standard test method for determining the creep compliance and strength of hot mix asphalt (HMA) using the indirect tensile test device

3.2.5 RAP in PCC

MECHANICAL PROPERTIES

- Properties of RAP
  - Specific gravity of RAP is lower than that of virgin coarse or fine aggregate (Brand et al. 2012).
  - Unit weight of milled or processed RAP is slightly lower than that of virgin aggregate and ranges from 120 to 140 pcf. Unit weight of RAP is largely determined by the recycled asphalt pavement of origin and the moisture content of the stockpile (Berry et al. 2013).
  - Water absorption for fine RAP is 1.2%, slightly lower than that of fine aggregate (Huang et al. 2005b).
  - Moisture content of RAP varies between 5%- 8%, depending on the stockpiled conditions, such as location, length of time stockpiled, and weather (FHWA 1997).

- Fresh Concrete Properties
  - At the same water/cement ratio, RAP concrete is less workable than natural aggregate concrete (Table 3.37), due to the high viscosity of asphalt-mortar coating on the aggregate. RAP is also rough and irregular in shape compared to gravel aggregate. However, RAP concrete still has satisfied workability; it can easily be mixed and the concrete consolidated (Okafor 2010, Huang et al. 2005b).

Table 3.37 Workability Test Results (Okafor 2010)

Mix proportion	Water/cement ratio	Slump, (mm)	
		Gravel	RAP
1:2:4	0.50	70	33
1:2:4	0.60	84	45
1:2:4	0.70	100	74
1:3:6	0.50	41	17
1:3:6	0.60	50	30
1:3:6	0.70	80	40

Note. 1:2:4 and 1:3:6 are the mix ratio between cement, sand, and RAP by weight.

  - As RAP content increases, slump decreases, indicating poorer workability of concrete (Huang and Shu 2005, Brand et al. 2012).
  - Slump of concrete made with only coarse or fine RAP is lower than that of concrete without RAP (Table 3.38), due to the high viscosity of asphalt binder. However, concrete made with both coarse and fine RAP has higher slump than that of concrete without RAP, since asphalt coating of both coarse and fine RAP reduces water absorption (Huang et al. 2005b).

Table 3.38 Mix Variants and Fresh Concrete Properties (Huang et al. 2005b)

Mix	Coarse aggregates	Fine aggregates	Air content (%)	Slump (cm)
1	Fresh	Fresh	1.60	16.5
2	RAP	Fresh	1.20	14.0
3	Fresh	RAP	2.50	7.5
4	RAP	RAP	2.00	20.0

◆ Hardened Concrete Properties

- Concrete made with RAP has lower compressive strength than concrete made with natural gravel, since asphalt is softer than virgin aggregate and the bond between asphalt and cement paste is weak (Huang et al. 2005b, Okafor 2010).
- Compressive strength decreases with increasing RAP content (Okafor 2010, Delwar et al. 1997). For example, 35% coarse RAP replacement meets the compressive strength requirement of 3500 psi at 14 days, while 50% coarse RAP replacement was 0.3% below the required strength (Brand et al. 2012).
- For concrete made with both fine and coarse RAP, 25% fine and 50% coarse RAP replacement reached 75% of the compressive strength of concrete without RAP after one year, while 50% fine and 100% coarse RAP replacement reached 53% of the compressive strength (Berry et al. 2013).
- Compressive strength of concrete made with RAP as both coarse and fine aggregate decreased more than concrete made with only coarse or fine RAP, as coarse aggregate and fine aggregate (Figure 3.53), respectively. Strength of concrete with RAP as coarse aggregate decreased the least. This was associated with the softer asphalt film around the RAP particles and the weak bonding between asphalt film and concrete matrix/aggregate (Huang et al. 2005b, Okafor 2010).

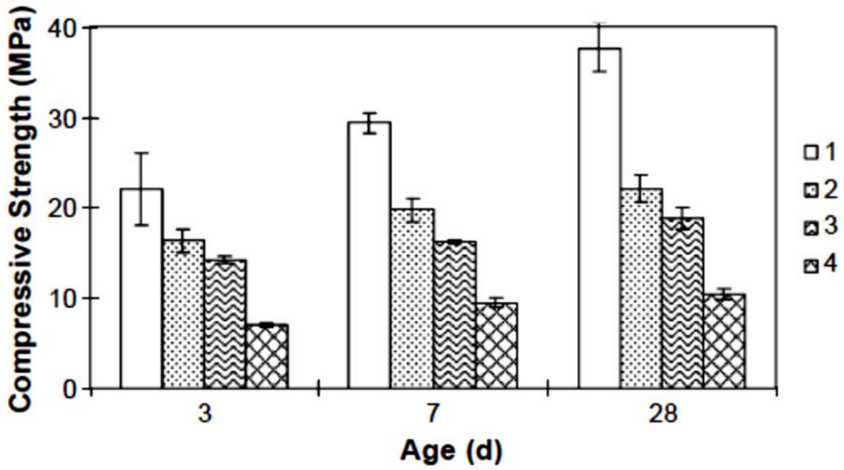


Figure 3.53 Compressive Strength at Different Days (Huang et al. 2005b)

Note. 1. Concrete with virgin aggregate; 2. concrete with RAP as coarse aggregate; 3. concrete with RAP as fine aggregate; 4. concrete with RAP as both fine and coarse aggregate.

- Compressive strength of concrete made with RAP increases with age, and the rate of strength gain decreases gradually (Berry et al. 2013).
- Similar to conventional concrete, high water-cement ratios yield lower compressive strength, since higher water/cement ratio leads to a reduction in cement mortar and bond strengths (Okafor 2010, Delwar et al. 1997). The highest compressive strength was found at a water/cement ratio of 0.50 (Figure 3.54; Okafor 2010).

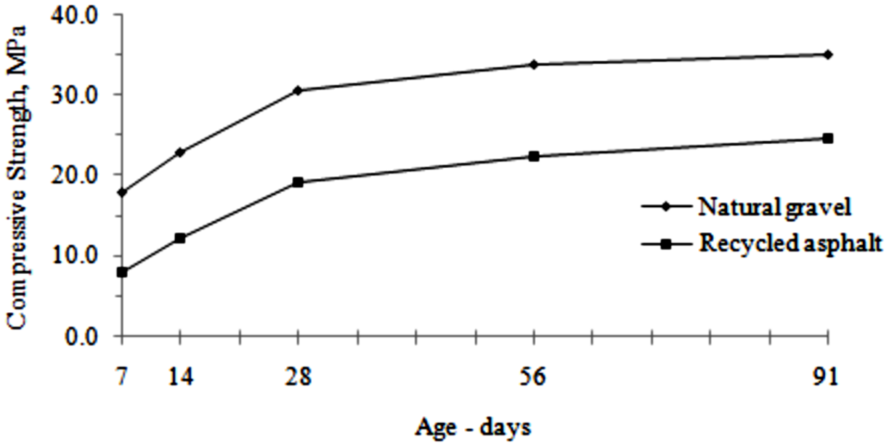


Figure 3.54 Compressive Strength at Water/Cement Ratio of 0.50 (Okafor 2010)

- Tensile strength decreases with increasing RAP content (Figure 3.55; Berry et al. 2013). The reduction in split tensile strength was lower than that of the compressive strength (Huang et al. 2005b).

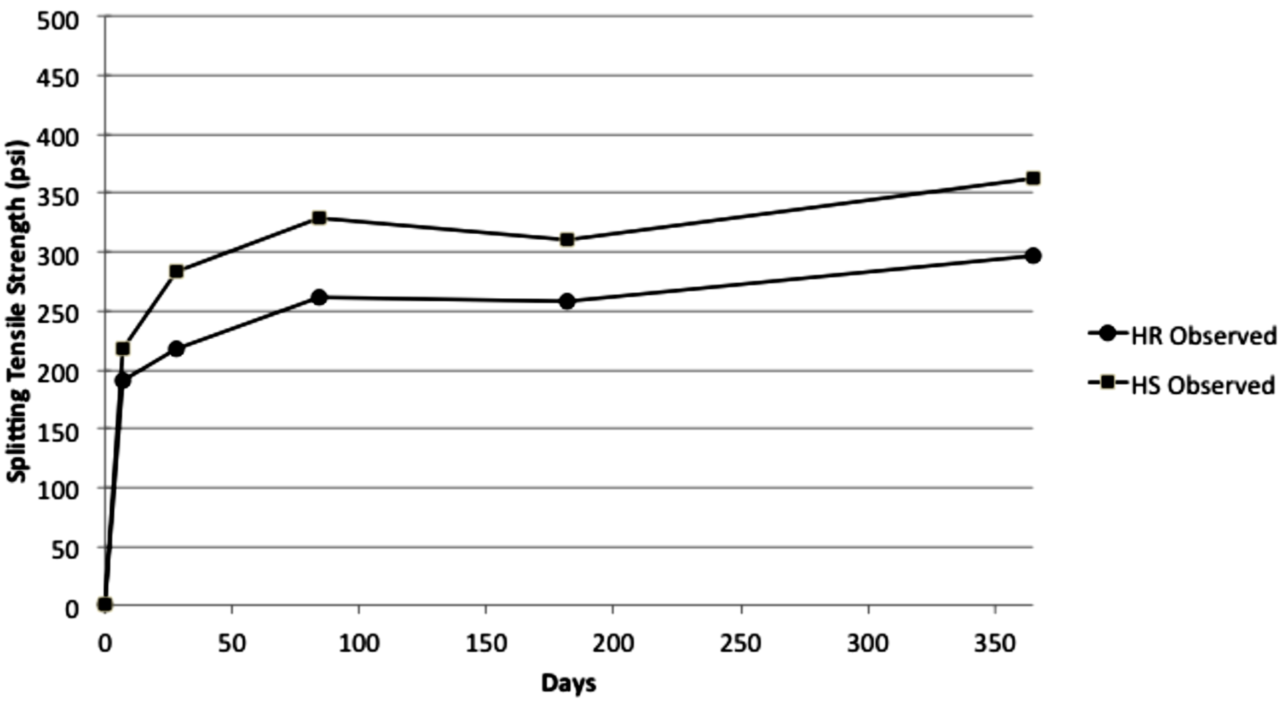


Figure 3.55 Splitting Tensile Strength for Concrete with RAP (Berry et al. 2013)

Note. HS=fine RAP replacement of 25% and coarse RAP replacement of 50% in volume; HR= fine RAP replacement of 50% and coarse RAP replacement of 100% in volume.

- Tensile strength of concrete made with both coarse and fine RAP decreases more than concrete made with only coarse or fine RAP. Strength of concrete with RAP as coarse aggregate decreases the least (Huang et al. 2005b).
- Flexural strength decreases with increasing RAP content (Berry et al. 2013, Okafor 2010). Flexural

strength depends more on the bond strength of asphalt-mortar attached to the aggregate particles; thus, changing the water/cement ratio (i.e., from 0.5 to 0.7) has little effect on the flexural strength of RAP concrete (Okafor 2010).

- Addition of silica fume has little effect on the performance of concrete with RAP, likely due to low slump and a short curing time of 28 days. A water reducing agent can improve strength and elastic modulus of concrete containing RAP (Huang and Shu 2005).
- Elastic modulus of concrete generally increases with time, and decrease with increasing RAP content, Figure 3.56. Prediction of elastic modulus with ACI method is affected by RAP content, with underestimate as concrete without RAP, and significant overestimate as RAP content is high (Berry et al. 2013).
- Concrete with RAP is more flexible than that of conventional concrete, with decreasing stiffness as RAP content increases (Delwar et al. 1997).

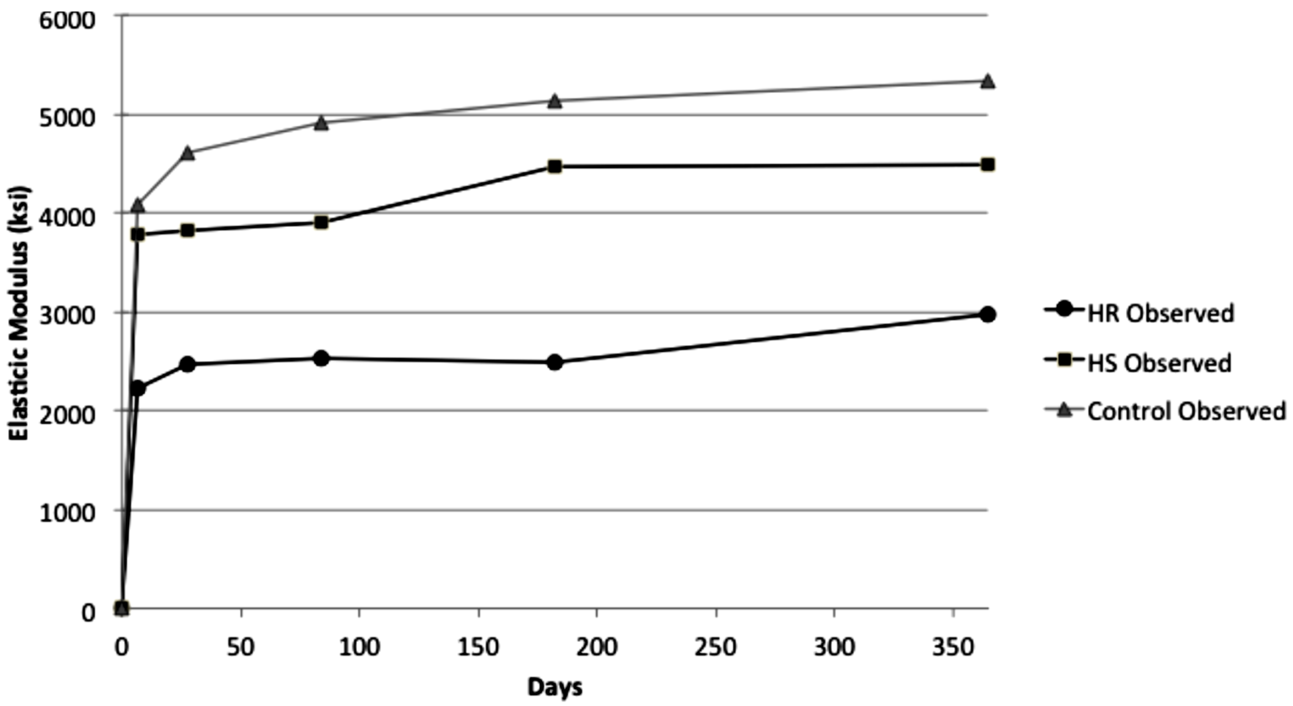


Figure 3.56 Elastic Modulus of Concrete with RAP (Berry et al. 2013)

Note: HS=fine RAP replacement of 25% and coarse RAP replacement of 50% in volume; HR= fine RAP replacement of 50% and coarse RAP replacement of 100% in volume; Control=PCC without RAP.

- Concrete with higher RAP content experienced more creep and shrinkage over time (Berry et al. 2013). Creep strains were slightly larger than shrinkage strains over time (Hossiney 2008).
- Concrete with higher RAP content has a higher creep coefficient (creep strain divided by initial elastic strain) at every time step (Figure 3.57), indicating higher creep potential (Berry et al. 2013).
- Creep predicted by the AASHTO method is lower than that in practice (Figure 5), because of the residual asphalt that is susceptible to creep. In addition, concretes containing considerable paste tend to creep more. The addition of fly ash may delay curing, resulting in inaccurate prediction for creep and shrinkage by the AASHTO methodology (Berry et al. 2013).

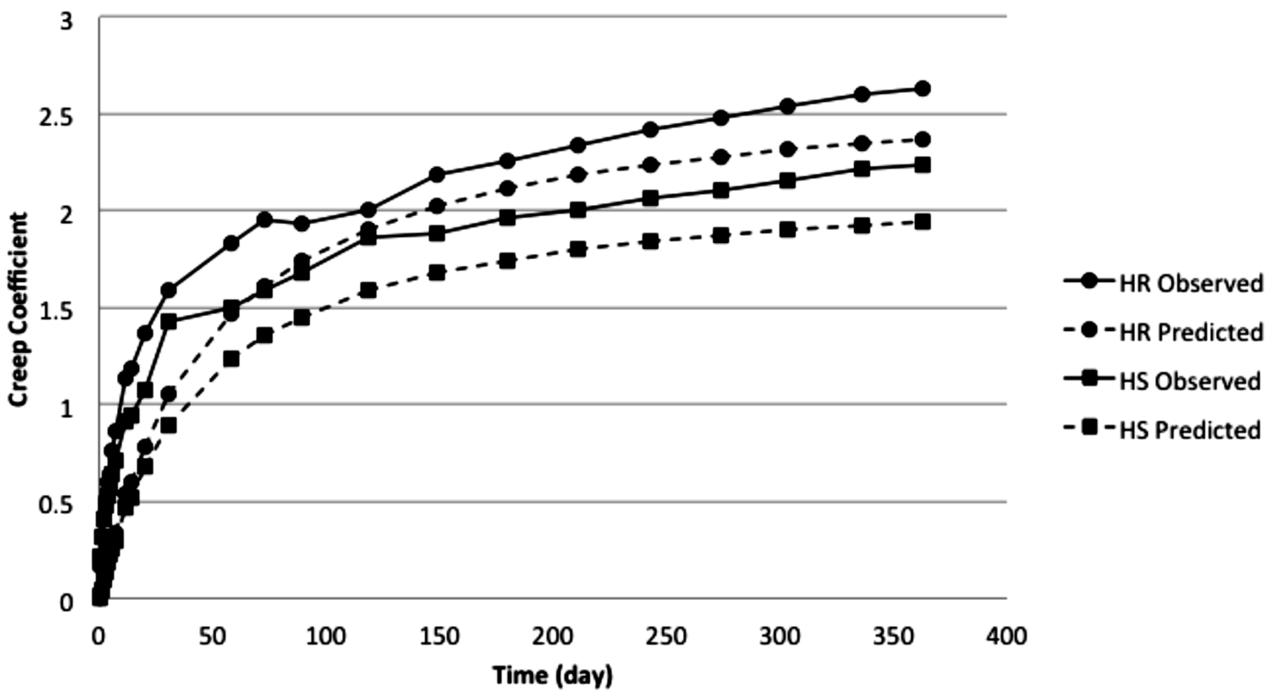


Figure 3.57 Creep Coefficient vs. Timefor Concrete Made with RAP (Berry et al. 2013)

Note. HS=fine RAP replacement of 25% and coarse RAP replacement of 50% in volume; HR= fine RAP replacement of 50% and coarse RAP replacement of 100% in volume. Creep coefficient predicted by AASHTO method is also plotted in the graph.

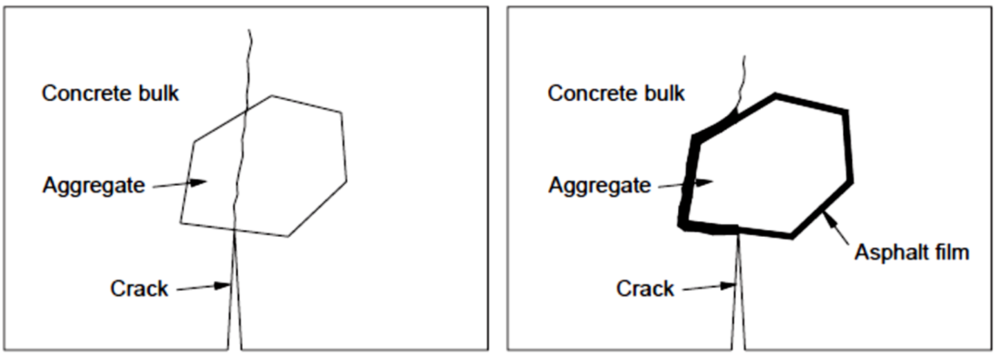


Figure 3.58 Crack Propagation in Concrete (left) and Concrete with RAP (right) (Huang et al. 2005b).

- Another study indicated that free shrinkage is independent of RAP content. Under ring restraint, concrete with 50% coarse RAP replacement showed lower shrinkage than concrete without RAP, exhibiting greater stress relaxation at later ages (Brand et al. 2012).
- The addition of RAP increased the toughness of concrete, since RAP aggregate can arrest crack propagation, making final product more resilient. However, concrete without RAP will disintegrate suddenly, as seen in Figure 3.58 (Huang et al. 2005b).
- Toughness of concrete with fine RAP was comparable to that of concrete without RAP (Figure 3.59). Concrete with coarse RAP or both coarse and fine RAP exhibited much higher energy absorption than

concrete without RAP (Huang et al. 2005b).

- Coarse RAP has greater effect on improving toughness of concrete mixtures than fine RAP. Fine RAP has a more adverse effect on concrete performance than coarse RAP (Huang et al. 2005b).

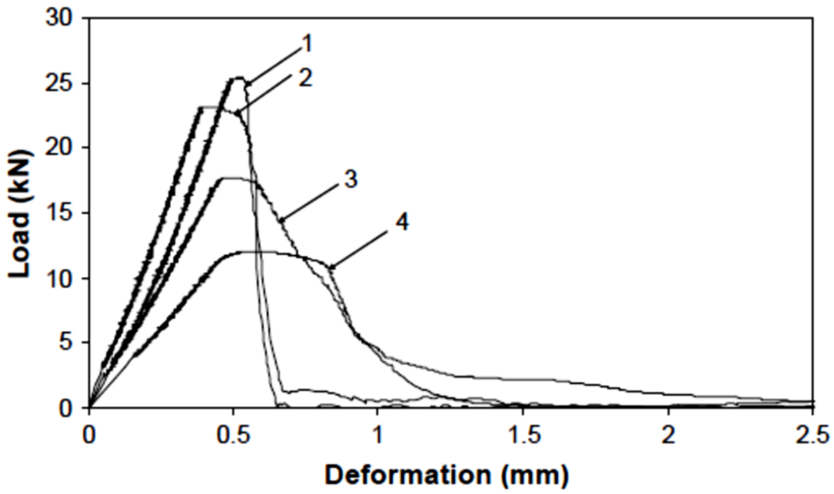


Figure 3.59 Load-Deformation Curves of Concrete Under Split Tensile Strength Test at 14 days (Huang et al. 2005)

Note. Figure 1 shows concrete with virgin aggregate; figure 2 is concrete with RAP as coarse aggregate; figure 3 is concrete with RAP as fine aggregate; and figure 4 is concrete with RAP as both fine and coarse aggregate.

◆ Durability

- The coefficient of thermal expansion is not affected by the addition of RAP (Hossiney 2008).
- Air void content is an indicator of concrete durability. Air content of concrete with RAP is comparable to that of concrete without RAP. Air content is independent of RAP content (Huang et al. 2005b, Huang and Shu 2005, Brand et al. 2012). Air content of concrete with 25% fine and 50% coarse RAP replacement and concrete with 50% fine and 100% coarse RAP replacement were 12.0% and 11.7%, respectively (Berry et al. 2013).
- Alkali-silica reactivity tests revealed that RAP and virgin coarse aggregate were non-reactive, while fine aggregate sand was mildly reactive (Brand et al. 2012).
- RAP has little influence on the abrasion resistance of concrete, since high paste content and low water-to-cement ratio contributed to higher abrasion resistance (Berry et al. 2013).
- Concrete with RAP has low chloride permeability. Increasing the RAP content slightly increases chloride ion penetrability, leading to lower durability. There are also studies indicating that RAP has little effect on the rapid chloride penetration (Brand et al. 2012).
- Although increasing RAP content slightly degrades freeze-thaw resistance of concrete, concrete with 50% coarse RAP replacement maintained adequate durability after 300 freeze-thaw cycles (Brand et al. 2012, Berry et al. 2013).
- Adding RAP to concrete hardly affects initial and total fracture energy, compared to concrete without RAP, although the critical stress intensity factor is reduced (Brand et al. 2012).

ENVIRONMENTAL PROPERTIES

- Leached concentrations (Ammonium and Sodium) from concrete made with precast waste aggregate and Trent Valley gravel are slightly higher compared to those leached from concrete made with limestone and RAP (Table 3.39; Erdema and Blanksonb 2014).
- Acidic compounds (i.e., Nitrate and Ammonium) are leached in large quantities from concrete made with RAP and are probably associated with the extra cement inherited from the old mortar. Therefore, concrete made with RAP has a higher capacity of acid-neutralization (Erdema and Blanksonb 2014).
- Certain metals (chloride, nitrate) tend to leach out in high concentrations from concrete with RAP, since high pH leads to increased solubility of these chemicals from RAP (Erdema and Blanksonb 2014).

Table 3.39 Leaching Analysis Results (Erdema and Blanksonb 2014)

Solution	Trent Valley	Limestone	Waste precast concrete	Recycled asphalt pavement
Concentrations (mg/L)				
Chloride	0.0255	0.0265	0.0159	0.0634
Cadmium	0.00045	0.00102	0.00051	0.00061
Nitrate	3.4845	3.8336	3.8642	4.1677
Ammonium	2.0220	2.1488	3.2844	1.8840
Sodium	82.4195	81.3634	91.1912	80.7211

- Electrical conductivity and pH values of the four different concrete specimens (Table 3.40), are similar. Concrete made with RAP has similar leaching performance to concrete made with virgin materials (Erdema and Blanksonb 2014).

Table 3.40 Electrical Conductivity and Results (Erdema and Blanksonb 2014)

Mix ID	Conductivity (µs)	pH	Temperature (°C)
Trent Valley concrete	5.19	12.36	21.6
Limestone concrete	6.01	12.43	21.6
Waste precast concrete	5.90	12.42	21.6
Recycled asphalt pavement	4.87	12.37	21.6

RECOMMENDATIONS

- Up to 35% coarse RAP replacement can meet required fresh concrete properties, strength, and durability. RAP does not need to be washed (contained a higher amount of fine particles passing the #4 sieve) in order to achieve required workability and strength (Brand et al. 2012).
- Strength loss caused by incorporating RAP into concrete can be mitigated by improving strength and modulus of asphalt by aging, and improving bonding between asphalt and aggregate (Brand et al. 2012).

BENEFITS

- Every year, over 100 million tons of RAP is reclaimed to construct the nation's roads (Huang et al. 2005b), which exceeds the demand of the HMA industry. The beneficial use of RAP in PCC can address the additional RAP available (Berry et al. 2013).
- Virgin aggregate partly replaced with RAP to produce concrete pavements is both efficient and environmentally friendly (Berry et al. 2013).

SUGGESTED SPECIFICATIONS

Table 3.41 Properties of PCC and Tests (Berry et al. 2013)

Properties	ASTM Test Method
Gradation	C136
Unit Weight	C29
Specific Gravity and Absorption	Coarse: C127 Fine: C128
Slump	C143
Air Content	C231
Compressive Strength	C39
Splitting Tensile Strength	C496
Elastic Modulus	C469
Modulus of Rupture	C78
Shrinkage	C512
Creep	C512
Alkali Silica Reactivity	C1260
Absorption	C642
Abrasion	C944
Chloride Permeability	C1202
Freeze-Thaw	C666
Scaling	C672

3.3 Foundry Sand (FS)

3.3.1 FS in Crack Sealant & HMA

MECHANICAL PROPERTIES

- Marshall Mix Design
  - Hot mix asphalt (HMA) typically comprises coarse aggregates, fine aggregates and asphalt binder (FIRST 2004).
  - FS can replace 8%- 25% of fine aggregate in HMA (FHWA 2004). For high volume roadways, the replacement can vary between 10% and 15%.
  - A 15% FS replacement may provide satisfactory HMA performance. When FS replacement is higher than 15%, the asphalt mix may become more sensitive to moisture damage (stripping and pavement deterioration), since silica in FS prompts stripping of HMA (FIRST 2004, Yazoghli-Marzouk et al. 2014).
  - The density of asphalt cement concrete decreases with increasing FS content. Without the addition of FS, density of HMA is about 2.4 g/cm^3; density decreased to 2.28 g/cm^3 at 20% FS replacement, as seen in Figure 3.60 (Bakis et al. 2006).
  - As the percentage of FS increases from 0% to 20%, Marshall stability of HMA decreased from 12.1 to 9.7 kN (Figure 3.61). While adding FS lowers stability, limiting FS to less than 10% of the total aggregate by weight may actually improve stability, in this case to 10.9 kN (Bakis et al. 2006).
  - The optimum asphalt content for HMA with FS is comparable (5%-6.2%) to conventional HMA (Miller et al 2001, Tikalsky et al 2004). However, another study indicated an increase in design binder content for HMA with FS (6%-6.5%), versus conventional HMA (5.5%), Braham 2002.
  - Using FS in HMA generally meets Superpave requirements for volumetric design. However, higher FS content requires additional asphalt binder (Braham 2002).

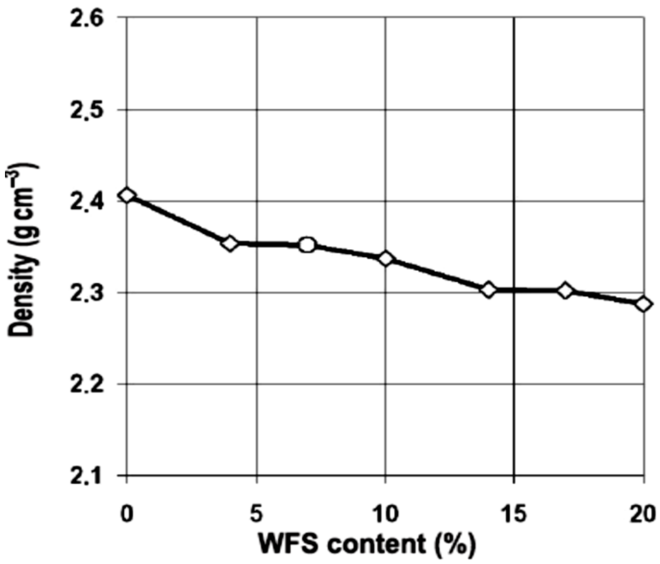


Figure 3.60 Density Values of FS-Asphalt Cement Specimen Tested (Bakis et al. 2006)

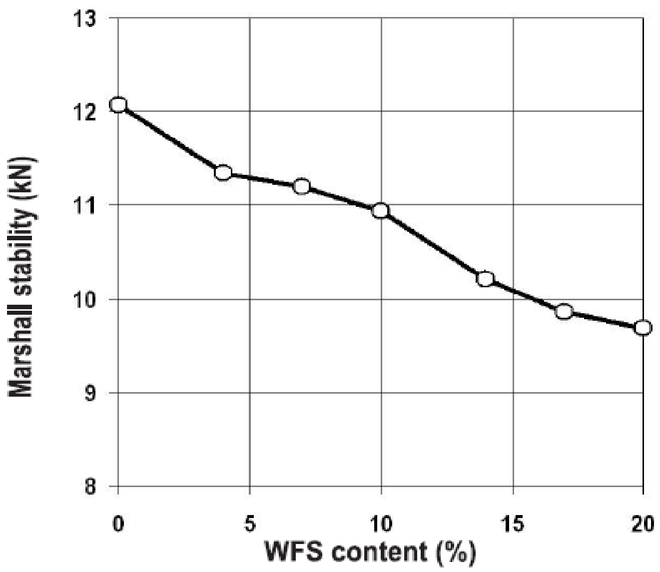


Figure 3.61 Marshall Stability of FS–Asphalt Cement Mixtures (Bakis et al. 2006)

◆ <u>Strength and Stiffness</u>

- ■ Indirect tensile strength of the asphalt cement mixtures decrease with increasing FS content. For example, indirect tensile strength varies from 13.9 kPa with 0% FS to 9.4 kPa with 20% FS (Figure 3.62; Bakis et al. 2006).

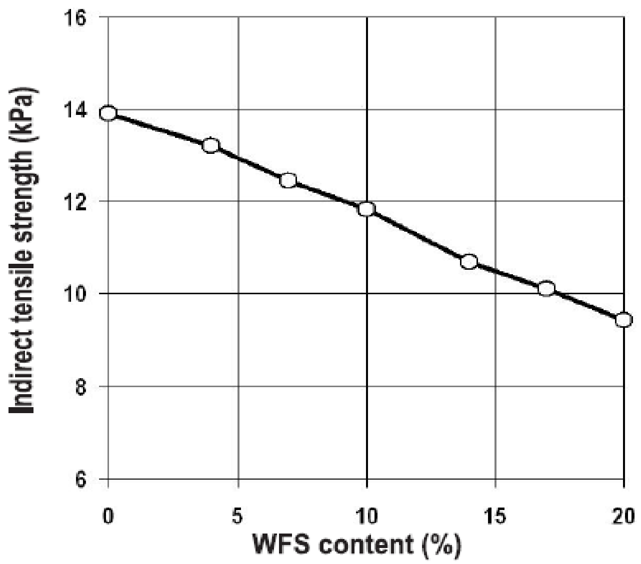


Figure 3.62 Indirect Tensile Strength of FS–Asphalt Cement Mixtures (Bakis et al. 2006)

- ■ Tensile strength of HMA with FS is slightly lower than that of conventional HMA, in both wet or dry conditions. Tensile strength ratio of HMA with FS may be lower than 0.70 (representing the recommended value by Wisconsin State DOT). Low tensile strength may be associated with the clay content in FS (Braham 2002).
- ■ In moist conditions, adding an anti-stripping agent into HMA with FS increases tensile strength (Braham 2002).

- Strength of HMA with FS may be not influenced by the absorption, angularity, and fines content of FS, since clay in FS may be dominant. An FS content of less than 20% may have a lower effect on the overall performance of HMA (Braham 2002).

- Stability and Durability
  - FS is generally non-plastic and has low absorption. Moisture resistance of FS depends on the clay content and organic additives used (FIRST 2004, Braham 2002). Clay-bonded FS (green sands) may typically be more sensitive to moisture (AFS).
  - Flow values decrease with increasing FS replacement of natural sand in asphalt concrete mixtures, for example, from 3.48 mm for 0% FS to 2.4 mm for 20% FS (Figure 3.63), since an increased fine content (due to FS) reduces permeability (Bakis et al. 2006).
  - Stability of HMA with recycled FS can be higher than that of HMA with conventional sand (Delange et al 2001).
  - HMA made with FS has shown good durability with resistance to weathering (Emery 1993).

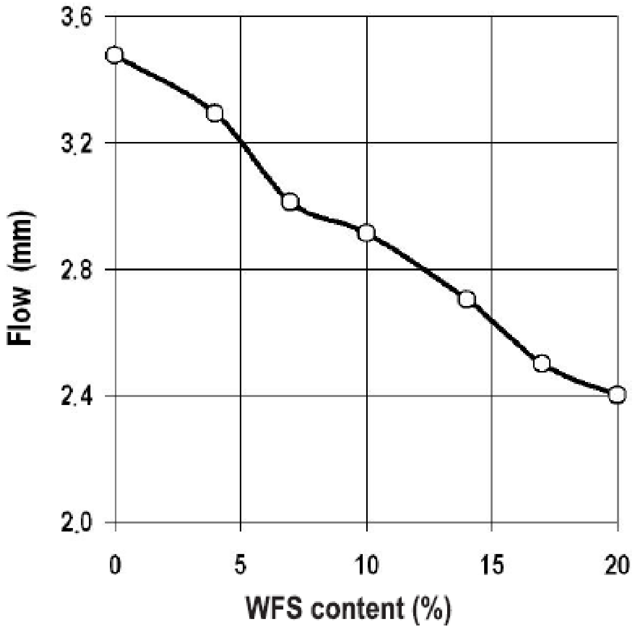


Figure 3.63 Flow Values of FS–Asphalt Cement Mixtures (Bakis et al. 2006)

ENVIRONMENTAL PROPERTIES

- Bituminous mixture containing FS does not release hazardous substances in the environment (Ideraldo et al. 2003).
- Addition of ferrous or aluminum FS to HMA has not shown any harm to the environment. Ferrous and aluminum FS are safe substitutes for virgin sands in construction applications (AFS).

DESIGN RECOMMENDATIONS

- Since specification (AASHTO M29) limits materials passing the No. 200 sieve to be between 5% and 10% in HMA, most FS with a higher percentage of fine aggregates need to be screened prior to blending, or by limiting FS content in HMA (FIRST 2004).
- Clay content and organic-based additive should be quantified and limited in producing an asphalt mix. For most FS, the sand equivalent test is not applicable, but methylene blue test is encouraged for measuring clay content. Organic based additives should be tested in loss on ignition test (FIRST 2004).
- FS should be free of thick coatings of burnt carbon, binders and mold additives, since these contents degrade adherence of asphalt cement binder to FS. Clay clumps can be removed by screening and/or washing, and iron and rubbish can be removed with magnets and/or hand separation (Benson and Bradshaw 2011).
- Properties of recycled FS are largely determined by the type of original FS (green or resin). For example, chemically bonded FS is drier and has a lower fines content than green FS (Hughes 2002). Each sand should be treated separately (Tikalsky et al 2004). Identify the type of FS and how the sand streams separate, comingle, etc., prior to use (Hughes 2002).
- The AASHTO pavement design method could be used to design asphalt pavements incorporating FS as fine aggregate (Benson and Bradshaw 2011).
- To further dry FS (less than 5% moisture), a pugmill (batch plants only) or a recycled asphalt feed (drum plants) can be used to dry the sand by already heated conventional aggregates (D'Allesandro et al 1990).
- Bentonite and organic binder can prolong the time required for drying FS and increase the load on the hot mix plant dust collection system. Bentonite should be processed to reduce fine contents. Coal and organic binders should be combusted (Benson and Bradshaw 2011).

FIELD RECOMMENDATIONS

- FS needs to be preprocessed into a consistent, high-quality product comparable to virgin sand. There are three steps needed in preprocessing FS (Hughes 2002):
  - Remove refuse and other contaminants
  - Remove metals
  - Processing and sizing
- Sizing green FS may result in an excess of minus 0.075 mm fines (HMA has requirements for fines content), which should be monitored and prevented (NCHRP 435).
- HMA producers should conduct an immersion Marshall test to evaluate the stripping potential of HMA with FS and incorporate anti-stripping agents (i.e., lime), if needed (AFS).
- The same field-testing procedures used for conventional HMA mixes should be used for mixes containing FS. Mixes should be sampled in a manner consistent with AASHTO T 168. The methods and equipment used for conventional HMA pavement are suitable to pavements containing FS (Benson and Bradshaw 2011).

BENEFITS

- Landfill disposal costs are escalating due to excessive transportation and landfill operations. This also causes landfill sites to be less available. Performance of FS degrades during the casting process, and eventually FS are removed to be landfilled. An ultimate solution to this issue is to beneficially reuse foundry byproducts (Benson and Bradshaw 2011).

- Energy spent on handling and reclaiming foundry byproducts can save up to 50 million MBtu for exploration of virgin materials, disposal of foundry products, construction of landfill, etc. (Tikalsky 2000).
- Beneficial reuse of FS is an effective way to reduce emissions (i.e., greenhouse gas), conserve landfill capacity and save virgin sands, which may no longer need mining or dredging (Benson and Bradshaw 2011).
- A case study for gray iron FS used in HMA showed that using 4,000 tons of FS saved 75% (about a $50,000 savings for the foundry) over the typical tipping fee costs. The FS made up about 10% by weight of the HMA aggregate (FIRST 2003).

SUGGESTED SPECIFICATIONS

Table 3.42 Tests for FS in HMA (Bakis et al. 2006)

Test	Standards
Marshall Stability	ASTM D1559
Loss of Soundness	AASHTO T104
Indirect Tensile Strength	AASTHO T283
Flow Value Test	ASTM D1559
Sand Equivalent Test	ASTM D2419
Non-Plastic Index Test	AASHTO T90
Loss on Ignition Test	(AASHTO T267-86

3.3.2 FS in Drainage/Embankment & Base

MECHANICAL PROPERTIES

- Gradation and Specific Gravity
  - Foundry Sand (FS) can be categorized to green sand and resin sand and is typically sub-angular to round in shape (Benson and Bradshaw 2011). Generally, green sand is made up of high-quality silica sand, 5%-10% bentonite clay, 2%-5% water, and less than 5% sea coal. Resin sand is comprised of high-quality silica sand, organic binder and catalysts.
  - FS has relatively uniform grain size distribution, with about 85%- 95% of particles between 0.6 and 0.15 mm (No.30 and No.100 sieve) and 5%- 12% of particles smaller than 0.075mm (No.200 sieve), as seen in Figure 3.64(Benson and Bradshaw 2011).

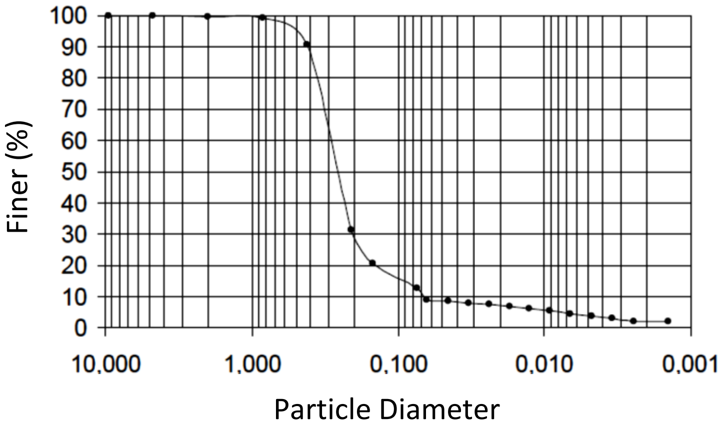


Figure 3.64 The Curve of Grain Size Distribution for FS (Benson and Bradshaw 2011)

  - According to the Unified Soil Classification System (USCS), FS is designated as well graded sand (Soleimanbeigi et al. 2014). According to AASHTO's soil classification, FS may be referred as A-3, A-2, or A-2-4 soil type (Gedik 2008, FIRST 2004).
  - FS is a non-plastic or low plasticity sand with little or no fines. Plastic behavior is associated with the clay content. With 6%- 10% clay, FS shows a liquid limit greater than 20 and a plastic index greater than 2 (FIRST 2004).
  - The specific gravity of FS ranges from 2.39 to 2.70. Variance is caused by different fines and additive contents (Federal Highway Administration 2004, Javed and Lovell 1994).
  - Compacted FS has a maximum dry unit weight of 11 kN/m^3, which classifies it as a lightweight material. Dry unit weights of FS are not sensitive to variations in moisture content (Soleimanbeigi et al. 2014).
  - FS has low water absorption, and absorption varies with different binders and additive types (Javed and Lovell 1994).
  - The loss on ignition values are relatively higher for green FS than other sands, due to combustible additives, such as seacoal (Gedik et al. 2008).

- <u>Drainage Properties</u>
  - FS has hydraulic conductivity of $2.7x10^{-3}$ cm/s at a hydraulic gradient of 0.5, high enough to provide good drainage capacity for structural fill applications (Soleimanbeigi et al. 2014).
  - Green sands with fines less than 6% as well as chemically bonded sands have permeability values ranging from $6x10^{-4}$ to $5x10^{-3}$ cm/sec. With bentonite clay more than 6%, permeability value of FS decreases significantly and ranges between $1x10^{-7}$ cm/s and $3x10^{-6}$ cm/sec (FIRST 2004).
  - Lime addition improves hydraulic conductivity of FS more than three orders of magnitude, indicating better capacity of drainage in winter conditions (Guney et al. 2006).

- <u>Strength</u>
  - Compacted FS has sufficient shear strength to provide stability for typical highway embankment fills (Soleimanbeigi et al. 2014, FIRST 2004). The friction angle of FS ranges from 30°- 36°, comparable to that of conventional sands. Typically, cohesion for FS is 3,700 psf (FIRST 2004).
  - FS has a comparable resilient modulus and California bearing ratio (CBR) to typical highway subbase materials (Kleven et al. 2000). CBR of FS is typically higher than that of granular sands, ranging between 11 and 30. CBR increases with increasing water content up to optimum water content, and then drops further increasing with additional water content (FIRST 2004).
  - The unconfined compressive strength and CBR of fully hydrated (i.e., cured for 7 days) FS-crushed rock mixtures can be improved by adding lime or cement (Figure 3.65), since the reaction of cement or lime causes the agglomeration of FS (Guney et al. 2006).
  - The unconfined compressive strength and CBR of cement or lime-amended, FS-crushed rock mixtures increase with increasing curing time, due to the time required for Portland cement to release calcium hydroxide ($Ca(OH)_2$) and quicklime to release free lime (CaO). In addition, the silica in FS is consumed to form calcium silicate hydrates, hardening the specimen (Guney et al. 2006).
  - Cement stabilized FS exhibits higher compressive strength and CBR than that of the same content of lime-stabilized FS in the first seven days, and the trend continues to increase until 6 months. Cement and lime additions at 8% and 10% by weight showed significant increase in unconfined compressive strength and CBR, especially at three and six months (Gedik et al.2008).
  - Higher compactive efforts increase the strength of the FS. Water content has great effect on unconfined compressive strength; therefore, intrusion of excess water should be prevented in the field and rain should be considered at the time of compaction (Guney et al. 2006).
  - Under a freeze-thaw cycle test, the loss of unconfined compressive strength is dominant by the first cycle (Figure 3.66). The effect of freeze-thaw on strength of FS mixtures depends on its influence on cementitious reactions. Freezing action retards the cementitious reactions, causing reduction in strength; accelerating the cementitious reactions causes an increase in strength. Between freeze-thaw cycles, freezing and thawing compensate each other, resulting in minimal variation in unconfined compressive strength (Guney et al. 2006).

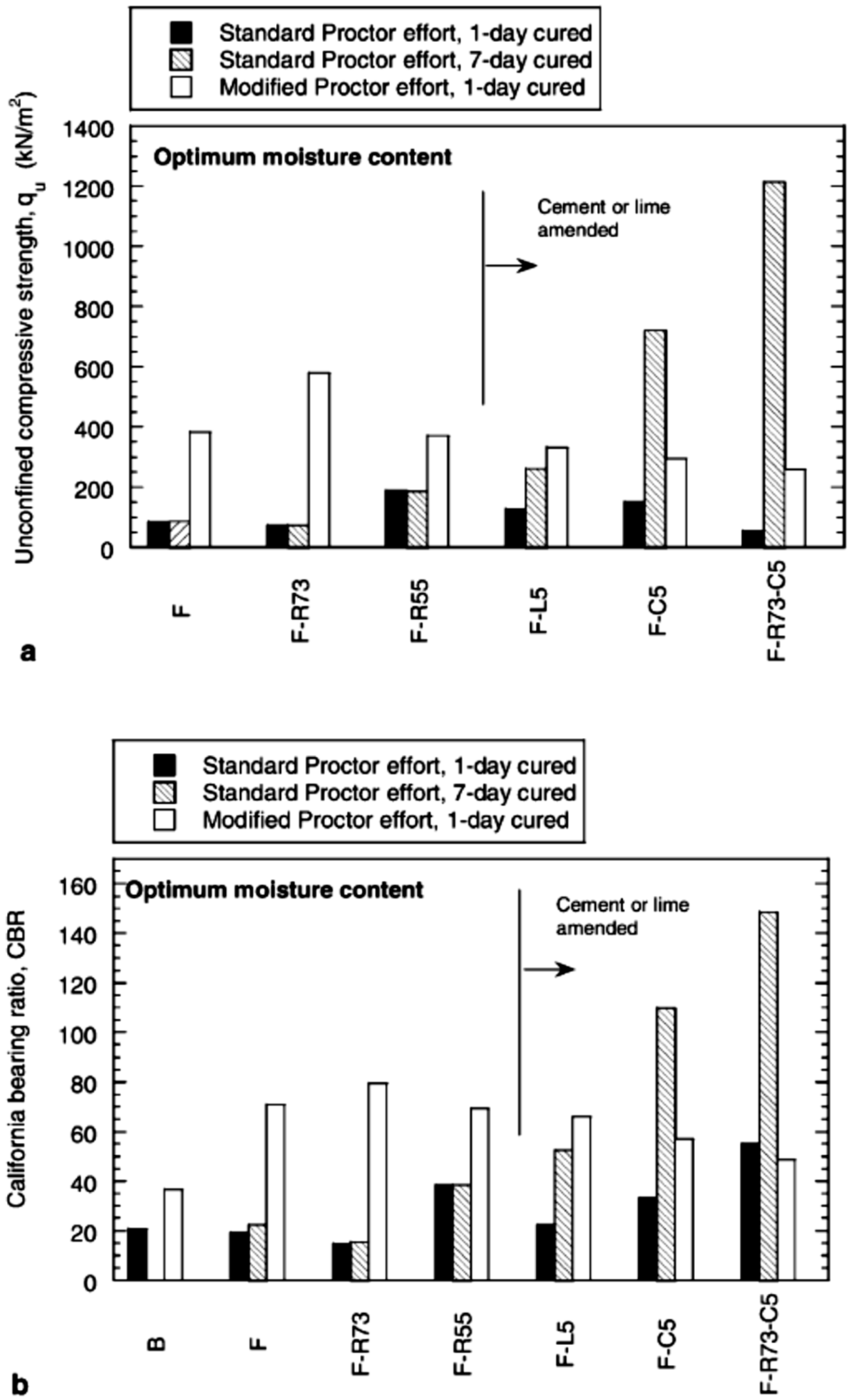


Figure 3.65 Effect of Curing Period and Cement or Lime Addition on (a) Strength and (b) CBR (Guney et al. 2006)

Note F: foundry sand; B: reference subbase; R55 and R73 designate the specimens with 55% and 73% crushed rock, respectively; L5 and C5 designate the specimens with 5% lime and cement, respectively.

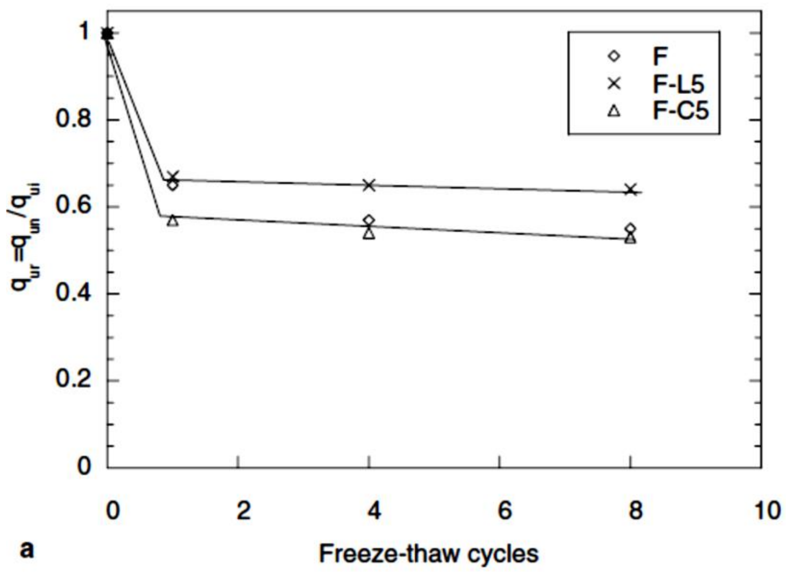


Figure 3.66 Effect of Winter Conditions on Unconfined Compressive Strength (Guney et al. 2006)

Note: L5 and C5 designate the specimens with 5% lime and cement, respectively.

- Compaction Properties
  - FS has satisfied compressibility for use as an embankment material (Mast and Fox 1998). FS is more compressible than natural sand due to binder and additives surrounding FS particles (Gedik et al.2008).
  - Owing to a weaker binder, compared to bulky sand grains, stress concentrations at the particle contacts tends to cause the crush of binder (Gedik et al.2008, Javed and Lovell 1994). FS has sufficient strength to resist breakdown under compaction (FIRST 2004).
  - Coarse grains of FS easily spread apart under compression, increasing fine grains content and inter-friction between fine grains (i.e., from 35%- 40.9%), therefore influencing mechanical properties (Thevanayagam et al. 2002).

- Permanent Performance
  - Embankment made of FS and 9% clay particles (<0.005 mm) has a plasticity index of 6, a friction angle of 38° and a settlement less than 7 mm, comparable to that of clean sand (Mast and Fox 1998).
  - Swell is negligible for FS, even for those with the highest bentonite content of 4.7-10.5% (Kleven et al. 2000).
  - Higher cement ratios may create fragility in cement-stabilized FS, leading to premature cracks in the pavement layer, which can be reflected to the upper layers (Gedik 2008).

ENVIRONMENTAL PROPERTIES

- Studies of Deng (2009) and of Dungan and Dees (2009) indicated that FS do not cause groundwater or surface water contamination, since the measured concentrations are significantly below the EPA maximum concentration limits.
- The study by Lee and Benson (2002) indicated that concentrations of Zinc (Zn), Lead (Pb), Chromium (Cr), and Iron leaching from FS may exceed the EPA limits. However, the difference is only 10%, which may be considered acceptable.

- TCLP (Toxicity Characteristic Leaching Procedure) extracts of FS, without any additives, had high concentrations of Copper (Cu), Pb, and Zn, over the limits of 5 mg/L. Adding Iron to the TCLP extraction of FS decreases Cu and Pb concentrations (Douglas 2003).
- Ji et al. (2001) reported that four different types of FS (green sands, furan/acid sand, phenolic sands and silicate sands) all contain Poly-Aromatic Hydrocarbon (PAHs) compounds. The PAHs in green sands are much higher than those in chemical binder FS. Phenolic/ester sands have higher PAHs than furan/acid and silicate sands (Table 3.43). The leached metal concentrations are very low in all waste FS (Table 3.44), and leached Cr concentrations increase with increasing pH of the eluted solution.
- Metal concentration decreases gradually with time passing (i.e., 48 hr. and 72 hr.), indicating the potential of excessive leachates at the construction stage (Guney et al. 2006).
- Lime or cement-amended FS mixtures show lower metal concentrations, possibly due to decreased solubility of these chemicals at high pH values, or decreased hydraulic conductivity because of agglomeration between the FS particles (Guney et al. 2006).

RECOMMENDATIONS

- As for structural fill, FS containing clays should be compacted to optimum water content. Resin sands have good drainage, but high bentonite green sands may have problematic drainage issues. FS may need to be screened or crushed prior to use. Consistent moisture content should be maintained to achieve the proper compaction in the field (AFS 2010).
- Engineers should investigate and check physical characteristics of the specific FS before applying in embankment use. Shear strength of FS is the key to design embankments because stability of slope depends on shearing strength. Plasticity index and moisture density should be investigated before designing the fill (AFS 2010).
- FS typically does not require special handling equipment or procedures, and is transported, placed, and compacted with conventional construction equipment. Green sands may require moisture during transportation and placement in case of dusting (AFS 2010).

BENEFITS

- Discarded FS typically has more consistent composition and higher quality compared to natural sands used in construction (Benson and Bradshaw 2011).
- Recycling FS can save energy by reducing the need to mine virgin materials, and may reduce costs for both producers and end users (Benson and Bradshaw 2011).
- Use of FS as a fine aggregate in construction applications meets the requirement of green sustainable construction by reducing the carbon footprint (Benson and Bradshaw 2011).

Table 3.43 Concentrations of PAHs in Different Types of FS (Ji et al. 2001)

Parameters	Sand types										
	Green sands			Furan/acid sands			Phenolic/ester sands				Silicate sands
	1	2	3	4	5	6	7	8	9	10	11
pH	9.5	9.7	9.8	4.4	4.9	3.2	7.8	9.2	9.3	9.7	10.1
Free phenol	7	12	3	0.5	1.1	0.7	5	10	3	0.8	<0.05
Free formaldehyde	<2	<2	<2	<5	<5	<5	<10	<10	<10	<10	<1
PAHs	9.36	28.7	18.2	0.22	0.68	0.24	1.47	2.44	1.23	1.99	0.36
Diphenylmethanediisocyanate	<1	<1	<1	NA	NA	NA	<1	<1	<1	<1	NA
Isoforonediisocyanate	<1	<1	<1	NA	NA	NA	<1	<1	<1	<1	NA

Note: NA=not available.

Table 3.44 Concentrations of Leached Metal in Different Types of FS (Ji et al. 2001)

Sample number	Sand type										Silicate sands
	Green sands			Furan/acid sands			Phenolic/ester sands				11
	1	2	3	4	5	6	7	8	9	10	
As	0.013	0.01	0.02	0.010	0.020	0.042	0.023	0.061	0.020	0.10	0.003
Ba	0.062	0.28	0.3	0.002	0.003	0.630	0.006	0.033	0.030	0.87	0.078
Cd	0.051	0.18	0.06	0.004	0.004	0.026	0.001	0.054	0.060	0.04	0.071
Cr	0.154	0.20	0.05	0.113	0.070	0.025	0.073	0.056	0.012	0.73	0.580
Pb	0.056	0.04	0.1	0.032	0.018	0.156	0.066	0.003	0.056	104	0.005
Hg (μg/L)	0.189	0.10	0.2	0.219	0.200	0.520	0.154	0.434	0.200	0.01	0.320
Se	0.042	0.02	0.1	0.033	0.002	0.410	0.054	0.050	0.170	0.10	0.023
Ag	0.064	0.03	0.01	0.059	0.010	0.031	0.038	0.010	0.010	0.10	0.043
Cu	0.057	0.06	251	0.053	0.002	0.080	0.083	0.020	0.100	NA	0.070
Zn	0.084	0.10	0.21	0.074	0.140	0.542	0.034	0.040	0.200	75.00	0.004

SUGGESTED SPECIFICATIONS

Table 3.45 Tests for Physical Properties of FS (Benson and Bradshaw 2011)

Property	**Test Method**	**Application**
Specific Gravity	ASTM D845-06	Embankment
Bulk Relative Density, lb/ft^3	AASHTO T084	Embankment
Absorption, %	ASTM C128-07a	
Moisture Content, %	ASTM D2216-05	Embankment
Clay Lumps and Friable Particles, %	ASTM C142-97, AASHTO T112	
Hydraulic Conductivity, cm/sec	ASTM D2434-68, ASTM D5084-03, AASHTO T215	Embankment
Plastic Index	ASTM D4318-05, AASHTO T090	Embankment

Table 3.46 Tests for Mechanical Properties of FS (Benson and Bradshaw 2011)

Property	**Test Method**	**Application**
Micro-Deval Abrasion Loss, %	ASTM D6928-06	
Magnesium Sulfate Soundness Loss, %	ASTM C88-05	
Internal Friction Angle (drained)	ASTM D4767-04, ASTM D3080	Embankment
Cohesion Intercept (drained), lb/ft^2	ASTM D4767-04, ASTM D3080	Embankment
Permeability	AASHTO T215, ASTM D5084	
Resilient Modulus	AASHTO T294-94	Base
California Bearing Ratio, %	ASTM D1883-05	Base
Unconfined Compressive Strength, lb/ft^2	ASTM D2166	Base

3.3.3 FS in Flowable Fill/SCC

MECHANICAL PROPERTIES

◆ Workability & Flowability

- Foundry Sand (FS) decreases workability of SCC. The higher FS content , the lower workability (Figure 3.67), due to the fact that: the fineness of FS increases surface area for water absorption; FS made up of angular particles decreases flowability; and hydrophilic silica sand contained in FS tends to attract water to it surface (Prabhu et al. 2015, Sahmaran et al. 2011).
- Workability of SCC with FS decreased as time elapsed (Prabhu et al. 2014). Requirement of superplasticizer increases with the increasing FS content, due to more fine grains in FS (Sahmaran et al. 2011).

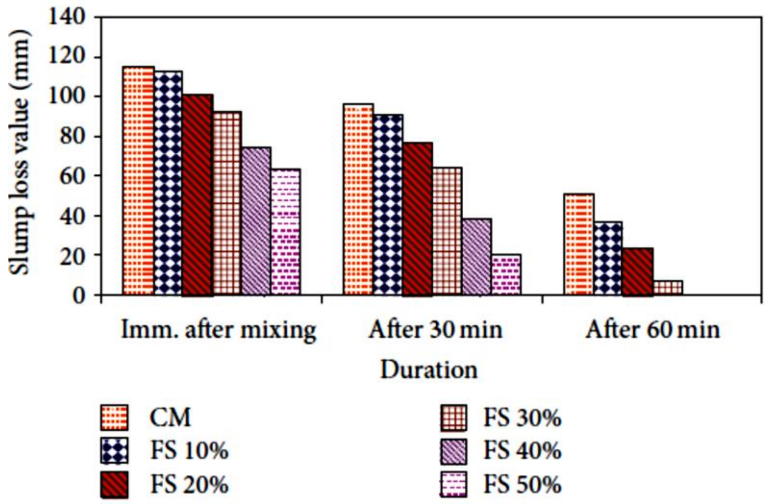


Figure 3.67 Workability of All Concrete Mixtures (Prabhu et al. 2015)

- Viscosity increases with increasing FS content, especially beyond 50% replacement of sand. SCC without fly ash has longer V-funnel flow time and slump flow time than the mixtures with fly ash, due to low viscosity of SCC with fly ash (Sahmaran et al. 2011).
- Flowability of FS is determined by gradation, particle shape and water content. Narrow particle gradation and prevailing round/sub-angular particle shape contribute to better flowability. Round particles facilitate flowability, yet with lower strength, compared to angular particles. Since FS is a composite of angular particles, regular, rounded sand has better flowability than FS. Water lubricates grains to improve flowability. However, excessive water leads to bleedings and volume instability, prolongs setting time and lowers quality (Deng and Tikalsky 2008).

◆ Strength

- Concrete mixtures with 30% FS replacement of natural sand have equal compressive strength with control concrete (CM). Compressive strength decreases with increasing FS replacement of natural sand (Figure 3.68), since higher water absorption diminishes workability and weakens consolidation effects, resulting in the formation of a higher number of small pores close to the aggregate surfaces. Additionally, clay, sawdust and wood flour included in FS may reduce specific density of concrete and create air voids in the concrete, further reducing density (Prabhu et al. 2014, Prabhu et al. 2015).

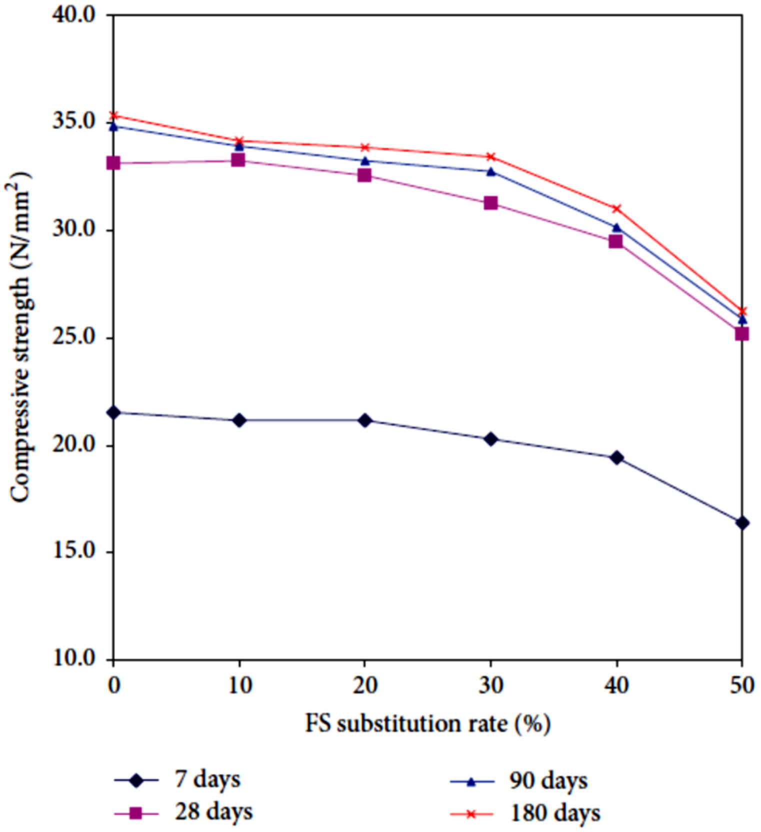


Figure 3.68 Comparison of Compressive Strength Value of all Mixtures at Different Ages (Prabhu et al. 2015)

- A study by Guney et al (2010) indicated that concrete with 10% FS replacement of fly ash has higher compressive strength at the age of 56 days.
- Temperature has little effect on compressive strength (Figure 3.69). Compressive strength rises slightly as temperature elevates from 200℃ to 300℃, since water migrates into pores, causing cement paste rehydration. Increasing fly ash content (up to 50%) and/or water-to-cement ratio reduces compressive strength (Pathak and Siddique 2012).
- Aging effect slightly improves compressive strength of concrete mixtures with FS (Prabhu et al. 2014).
- The addition of red mud (up to 4%) improves compressive strength of SCC mix with FS. When red mud content exceeds 4%, compressive strength decreases with additional red mud (Shetty et al. 2014).
- Flexural and tensile strength of concrete mixtures with FS is comparable to those of concrete mixtures without FS. Strength increases with concrete curing age, since many pores caused by fineness and dust particles in FS lead to lower density of concrete mixture (Prabhu et al. 2014).
- Splitting tensile strength increases with increasing FS content up to 20%, as seen in figure 3.70 (Siddique et al. 2009, Siddique and Kaur 2013). Concrete with 15% FS replacement has the highest splitting tensile strength among a 0%-20% FS substitution (Siddique and Kaur 2013).

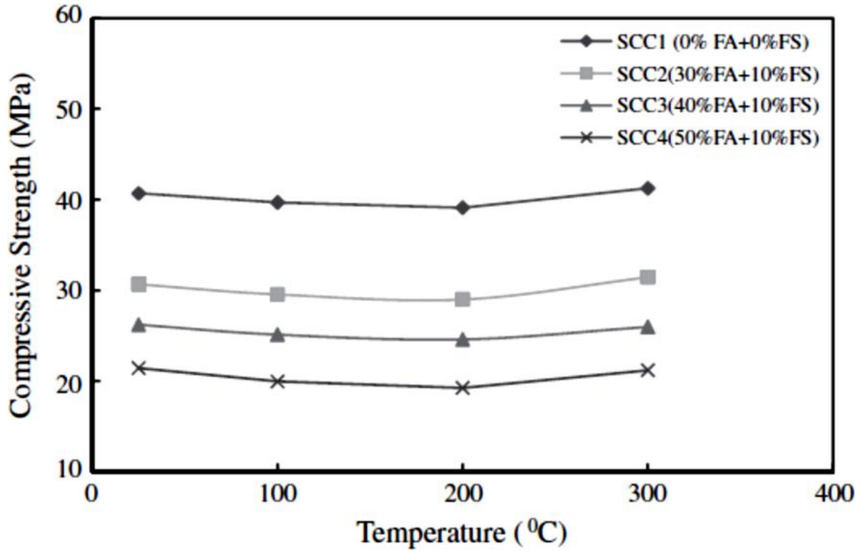


Figure 3.69 Compressive Strength Versus Temperature at 28 days (Pathak and Siddique 2012)

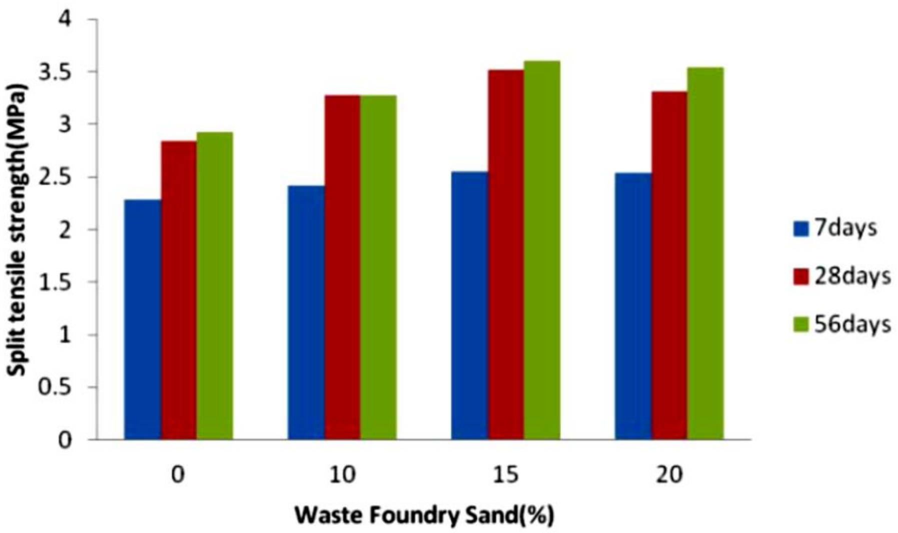


Figure 3.70 Splitting Tensile Strength of FS Concrete (Siddique and Kaur 2013)

- However, Guney et al. (2010) indicated that splitting tensile strength of 5% and 15% FS concrete mixes is lower than that of the concrete mixes without FS, while splitting tensile strength of 10% FS concrete is slightly higher than that of concrete mixes without foundry sand.
- For a 10% FS substitution, 4% red mud addition shows the highest split tensile strength at 28 days, and the 1% red mud addition achieves the highest flexural strength at 28 days. Adding red mud enhances flexural strength of the mixtures (Shetty et al. 2014).
- The splitting tensile strength decreases as fly ash content, water-to-cement ratio, and/or temperature increases (Figure 3.71). Strength loss in higher temperature is attributed to decomposition of hydration products and thermal incompatibility between aggregates and cement paste (Pathak and Siddique 2012).

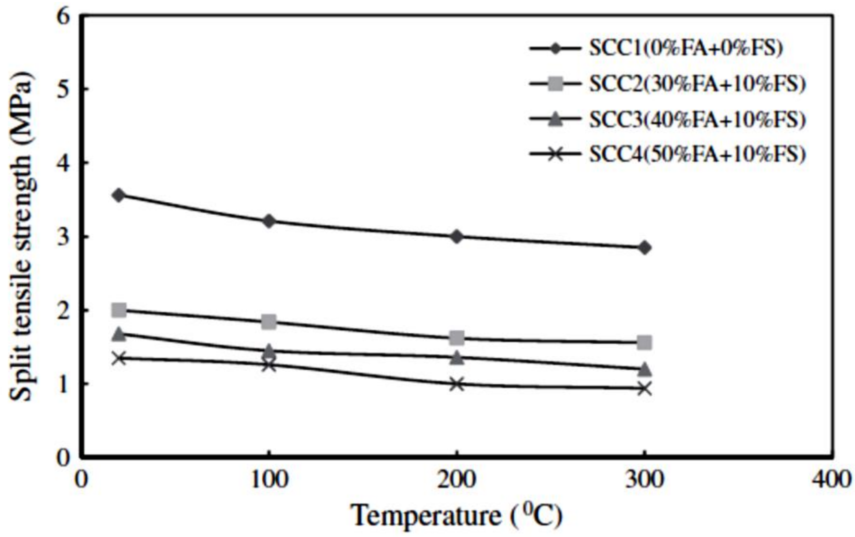


Figure 3.71 Split Tensile Strength Versus Temperature at 28 days (Pathak and Siddique 2012)

- Durability
  - Specific gravity and density of FS are about 2.38-2.72 and 1052-1554 kg/m^3, respectively. The variation is likely caused by sand mineralogy, particle gradation, particle shape and fine content (Deng and Tikalsky 2008).
  - FS is finer than typical fine aggregates (i.e., natural sand), which limits mixture segregation and provides a favorable flow in comparison to conventional flowable fill materials (Deng and Tikalsky 2008).
  - Water absorption of FS is about 0.38%-4.15%, higher than that of normal sand due to components of ashes and wood particles (Prabhu et al. 2015, Deng and Tikalsky 2008). Higher absorption corresponds to higher fine contents, since finer particles with higher specific surface area favor the absorption of water (Deng and Tikalsky 2008).
  - Drying shrinkage increases with the increase in FS replacement of sand (Figure 3.72), due to fineness and high water absorption of FS. Drying shrinkage increases over time. Using fly ash significantly reduces drying shrinkage. More drying shrinkage is reduced with increasing fly ash replacement of Portland cement, since fly ash particles are larger than those of FS (Sahmaran et al. 2011). Larger particles tend to store water, which slow the drying of concrete (Sahmaran et al. 2009, Sahmaran et al. 2011).
  - FS that replaces natural sand in concrete enhances the resistance to chloride penetration. The enhancement is proportional to the FS substitution rate, as the replacement rate exceeds 30% (Prabhu et al. 2015).
  - Coulomb value decreases with increasing FS content up to 15% (Figure 3.73), indicating the density of concrete increasing with FS content up to 15%. Coulomb charge at 91 days is less than that of 28 days, indicating density of concrete is increasing with age (Singh and Siddique 2011).

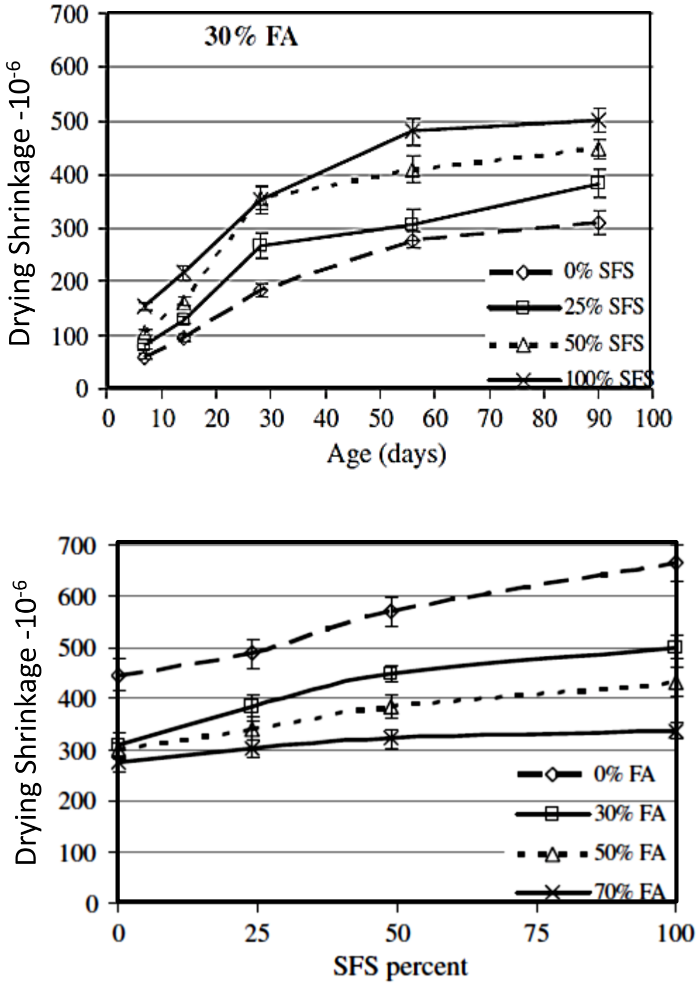


Figure 3.72 Effect of FS and Fly Ash on Drying Shrinkage: (top) 30% Fly Ash; (bottom) Drying Shrinkage at 90 Days (Sahmaran et al. 2011)

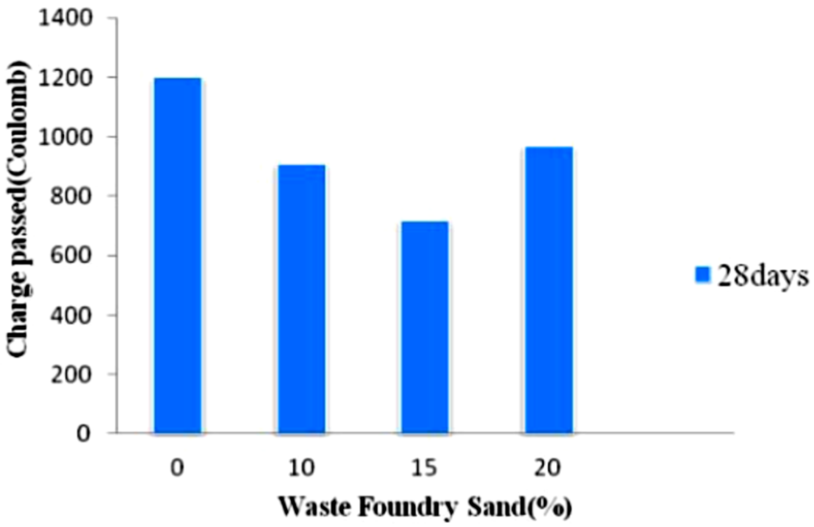


Figure 3.73 Chloride Penetration for SCC Mixes at Various FS Content (Siddique and Kaur 2013)

- Use of fly ash significantly reduces chloride permeability of hardened concrete mixtures. Reduction is more than 80% for fly ash replacing 50% and 70% Portland cement, since fly ash is finer than Portland cement and therefore is a more effective filler compacting internal structure. Pozzolanic reactions of fly ash further reduce pore size and micro-cracking in transition zones between aggregates and surrounding cementitious matrix (Figure 3.74; Kuroda et al. 2000, Mehta et al. 2006, Sahmaran et al. 2010).
- For FS replacement of sand up to 50%, the volume of permeable pores did not change significantly, therefore no effects on durability are expected (Sahmaran et al. 2010).
- Chloride-ion permeability decreases with increasing FS content (Figure 3.75), since fine particles of FS act as a filler, improving the internal structure of concrete (Siddique and Kaur 2013). Permeability decreases over time due to the hydration of Portland cement and pozzolanic reactions of fly ash (Sahmaran et al. 2010).
- Fly ash substitution of cement can reduce alkali ions and associated hydroxyl ions in concrete pore solution, diminishing electrical conductivity (Shehata et al. 1999).

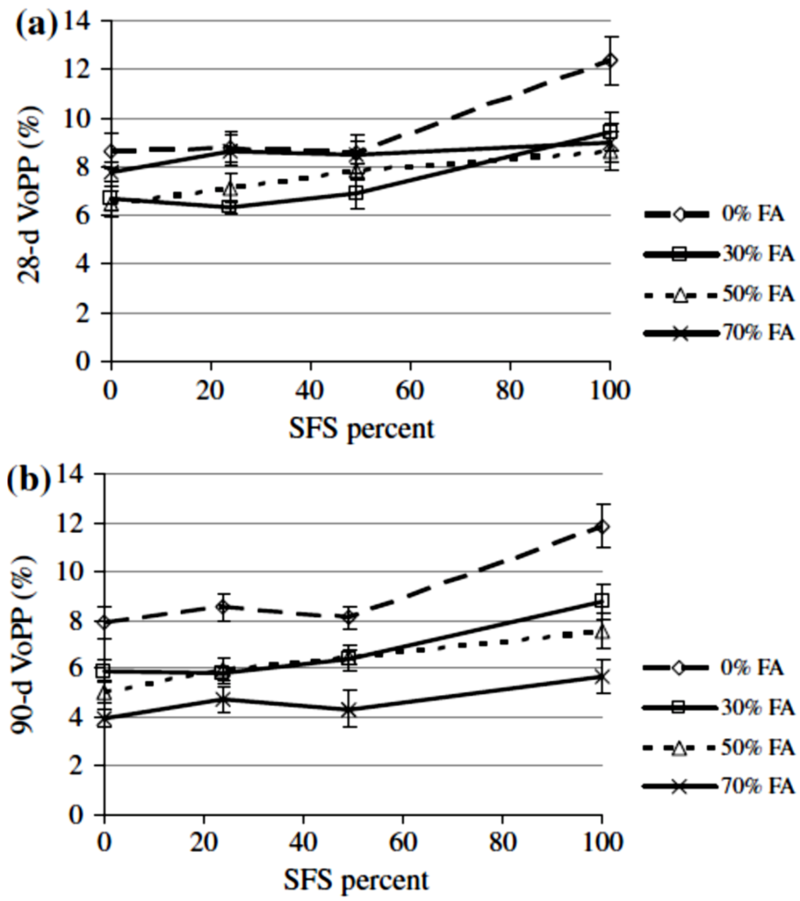


Figure 3.74 Volume of Permeable Pores a) at 28 Days; b) at 90 Days (Sahmaran et al. 2010)

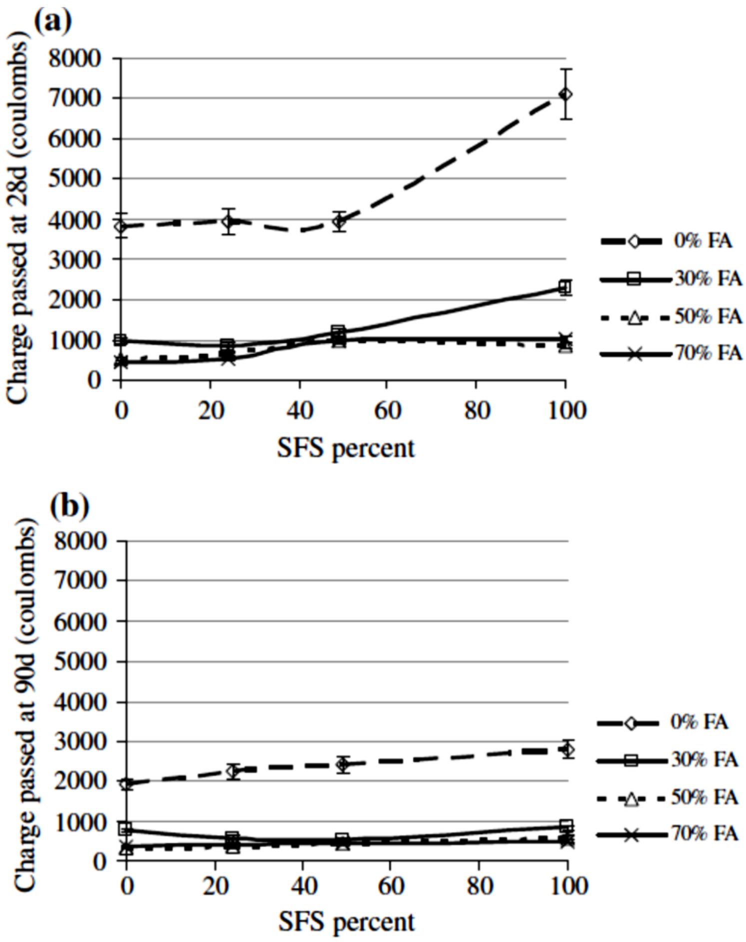


Figure 3.75 Rapid Chloride Permeability a) at 28 Days; b) at 90 Days (Sahmaran et al. 2010)

- The carbonation depth of concrete increases with an increasing FS content (Figure 3.76). This is due to the poor workability of concrete with FS, resulting in poor consolidation and high pores. In addition, carbon content in FS reacts with water, producing CO which reacts with calcium from calcium hydroxide and calcium-silicate hydrate to form calcite (Prabhu et al. 2015).

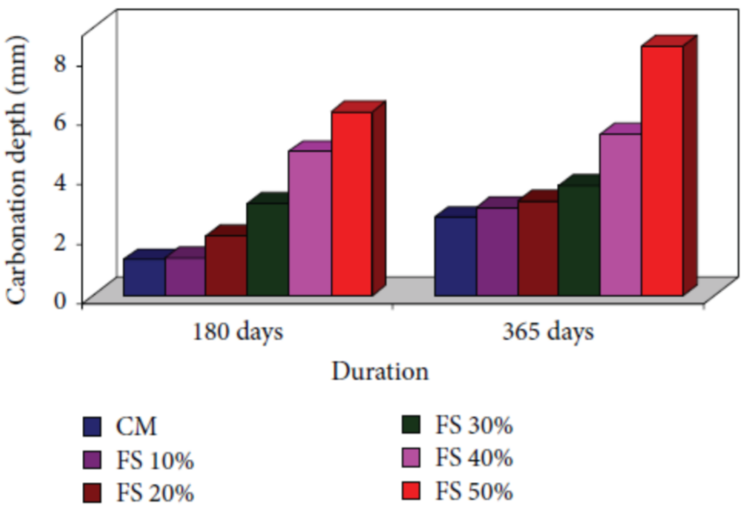


Figure 3.76 Carbonation Depth Values at Various Ages (Prabhu et al. 2015)

- Carbonation depth proportionally increases over time. Concrete with a FS substitution of less than 30% shows desirable resistance to carbonation, since carbonation coefficient does not exceed the value of 6 mm/month$^{0.5}$ (Prabhu et al. 2015, Castroa et al. 2000). Concrete with a substitution rate beyond 30% is not advisable for structural concrete, since the carbonation depth can approach the cover of reinforcing steel bars (Prabhu et al. 2015).
- Minimum electrical resistivity value is 20kΩ-cm, beyond which, corrosion cannot occur (Limeira et al. 2011, Chao-Lung et al. 2011). The resistivity value of concrete mixtures, with up to a 30% substitution of FS for sand, is beyond 20 kΩ-cm in all ages. The electrical resistivity value of concrete mixtures decreases with increasing FS substitution (Figure 3.77), due to poor workability, resulted in a large amount of pores (Prabhu et al. 2015).

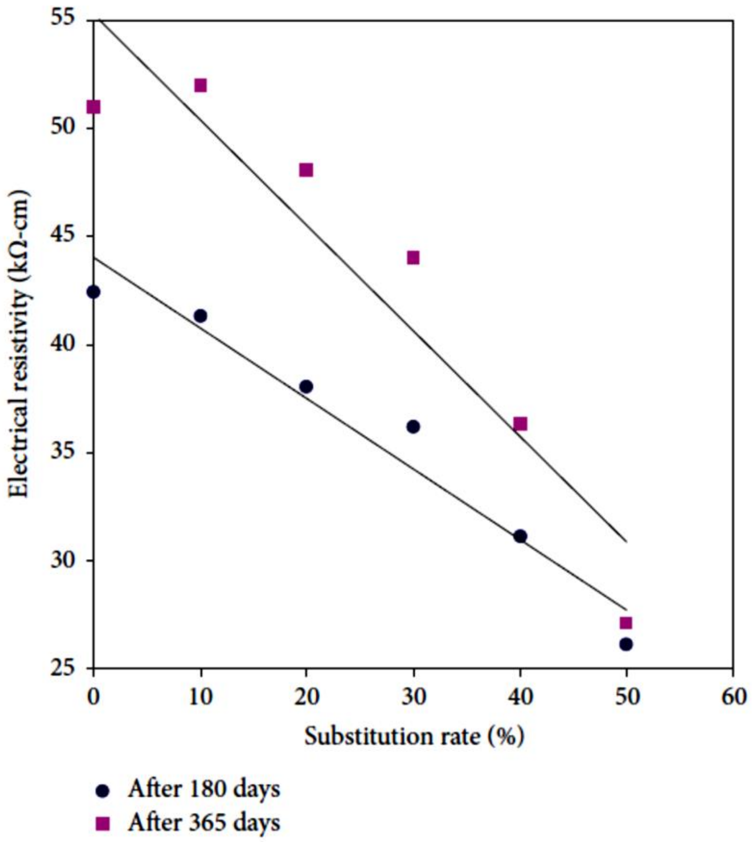


Figure 3.77 Electrical Resistivity Values at Various Ages (Prabhu et al. 2015)

- With aging effect, electrical resistance of concrete decreases (Prabhu et al. 2015).
- The sulphate resistance of concrete decreases with an increasing FS substitution for natural sand (Figure 3.78). Increasing FS contents significantly reduce compressive strength, especially for an FS substitution rate beyond 30%, due to sulphate attack in FS. SO_3 may also form ettringite, causing concrete deterioration (Prabhu et al. 2015).
- Concrete mixtures containing 10% FS experienced an increase in strength at all ages, compared to concrete mix without FS, even after being immersed into a magnesium sulphate solution. This indicates that 10% FS is optimum in resisting sulphate attack (Siddique and Kaur 2013).

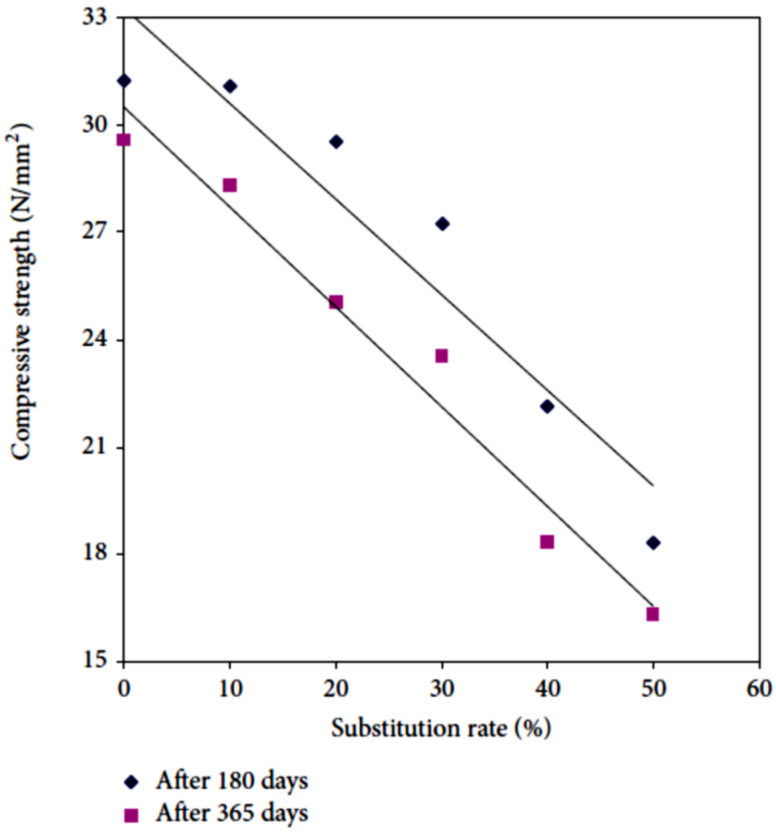


Figure 3.78 Sulphate Resistance Values at Various Ages (Prabhu et al. 2015)

ENVIRONMENTAL PROPERTIES

- Leachate from FS used in producing iron, steel and aluminum are below the regulatory limits for hazardous waste (Tikalsky et al. 2004, Dungan and Dees 2007).
- The pH increased when cement or lime is used. Electrical conductivity decreased due to the encapsulation process during cement stabilization. Leaching concentration of different metals (nickel, chromium, lead, copper, zinc and cadmium) decreased gradually over time (Guney et al. 2006).
- The leaching levels of iron, barium, magnesium, zinc, arsenic, chromium, lead, selenium, cadmium, mercury and chloride from flowable fill materials with 85% FS are below the enforcement standards of the Wisconsin Department of Natural Resources ground-water quality standards. Levels also meet drinking water standards (Naik and Singh 2001).
- Metal concentrations from flowable fill materials with FS are lower than EPA maximum limits. Organic remains contained in organic binders were burned or shaken away in casting processes. Acetone and naphthalene were below the EPA TCLP toxicity criteria (Table 3.47). The other organic compounds are not detectable and are below EPA TCLP toxicity criteria (Deng and Tikalsky 2007).

Table 3.47 Bleed Water Contaminants and TCLP Results of WFS Flowable Fills (Deng and Tikalsky 2007)

Constituents	Bleed water (μg/kg)				TCLP analyses (μg/kg)				US EPA TCLP criteria (μg/kg)
	WFS02	WFS12	WFS13	WFS16	WFS02	WFS12	WFS13	WFS16	
Arsenic	73.3	18.3	31.4	378	57.2	<50	<500	<50	5000
Barium	620	505	278	289	78.4	291	338	<10	100,000
Cadmium	6.4	6.4	7.7	10.2	<10	<10	<10	<10	1000
Chromium	75.8	48.5	189	681	25	60.9	72.2	<10	5000
Lead	26.7	23.1	13.7	93.6	<30	<30	<30	<30	5000
Mercury	<.2	<.2	<.2	<.2	<.2	<.2	<.2	<.2	200
Selenium	100	31.7	34	26.9	<50	<50	<50	<50	1000
Silver	.6	<.3	<.3	2	<50	<50	<50	<50	5000
Acetone	–	41	56	1540	86	86	100	115	–
Benzene	–	<5	<5	<5	<25	<25	<25	<25	500
Carbon tetrachloride	–	<5	<5	<5	<25	<25	<25	<25	500
Chlorobenzene	–	<5	<5	<5	<25	<25	<25	<25	100,000
Chloroform	–	<5	<5	<5	<25	<25	<25	<25	6000
1,4-Dichlorobenzene	–	<5	<5	<5	<25	<25	<25	<25	7500
1,2-Dichloroethane	–	<5	<5	<5	<25	<25	<25	<25	500
1,1-Dichloroethene	–	<5	<5	<5	<25	<25	<25	<25	700
Ethyl benzene	–	<5	<5	<5	<25	<25	<25	<25	
Methyl ethyl ketone	–	<10	<10	<10	<50	<50	<50	<50	200,000
Methylene chloride	–	<5	<5	<5	<25	<25	<25	<25	–
Naphthalene	–	619	180	115	<25	616	527	<25	–
Styrene	–	<5	<5	<5	<25	<25	<25	<25	–
Tetrachloroethene	–	<5	<5	<5	<25	<25	<25	<25	700
Toluene	–	<5	<5	<5	<25	<25	<25	<25	–
1,1,1-Trichloroethane	–	<5	<5	<5	<25	<25	<25	<25	–
Trichloroethene	–	<5	<5	<5	<25	<25	<25	<25	500
Vinyl chloride	–	<10	<10	<10	<50	<50	<50	<50	200
M,P-xylene	–	<5	<5	<5	<25	<25	<25	<25	–
Xylene-total	–	<10	<10	<10	<25	<25	<25	<25	–

"–" results not available. "<5" constituent nondetectable, in which "5" represents detection limit.

DESIGN RECOMMENDATIONS

- Structural design procedures for flowable fill materials are similar to conventional earth backfill materials (Benson and Bradshaw 2011).
- FS can be combined with natural sand (i.e., round sand) to achieve performance. Blended with natural sands, any organic material in FS may affect the dosage and effectiveness of air entraining agents (Benson and Bradshaw 2011).
- Cementitious materials can be a combination of Portland cement with fly ash, red mud, etc. Sodium silicate binder systems are not desirable in Portland cement (Benson and Bradshaw 2011).
- Retarders and water reducers can moderate high absorption of FS to improve the workability and strength of concrete. Trial mixtures should be examined for any potential compatibility problems (Benson and Bradshaw 2011).

FIELD RECOMMENDATIONS

- The methods and equipment used to mix, transport, and place flowable fill with conventional aggregates are also feasible to flowable fill with FS (Benson and Bradshaw 2011).
- FS should be screened and crushed to obtain the desired gradation when used in SCC. Magnetic particles should be separated prior to using FS. FS from green sand molding is black or gray and may affect concrete color, which can be addressed by replacing 15% or less of fine aggregates with FS (Benson and Bradshaw 2011).
- Properties of FS can affect the quality of concrete. Therefore, performance tests should be performed on the FS source—which largely determines the properties of FS—before exploring the FS use (Benson and Bradshaw 2011).
- When used in unbound applications, FS need to be pre-wet and at optimum moisture content on the first round of compaction, as the clay additive content tends to prohibit further compaction after re-wetting (NCHRP 435, 2013).
- Flowable fill with FS can be produced at a central concrete mixing plant in accordance with ASTM C94 and delivered by concrete truck mixers or by a mobile, volumetric mixer for small jobs (Benson and Bradshaw 2011).

BENEFITS

- Concrete with FS can achieve the required fresh and hardened properties. FS can be obtained from foundries with lower material cost; thus, the cost of fine aggregate reduction provides savings (Sahmaran et al. 2011).
- Disposal cost of these waste materials is reduced through recycling FS, as well as some other waste materials (i.e., fly ash, red mud), in concrete. Carbon dioxide emission in the cement plants can be reduced with the use of fly ash as a cement replacement (Sahmaran et al. 2011).
- The longer service life of structures using such concrete mixtures implies a reduction in repair costs (Sahmaran et al. 2011).

SUGGESTED SPECIFICATIONS

Table 3.48 Geotechnical and Leaching Property Tests of FS Flowable Fills (Deng and Tikalsky 2007)

Material phases	Properties	Testing specifications
Fresh	Fresh density	ASTM D6023
Fresh	Flowability	ASTM D6103
Fresh	Bleeding characteristics	ASTM C232
Fresh	Setting time and PR	ASTM C403
Fresh	Bleed water contaminants	EPA SW-486
Hardened	Hydraulic conductivity	ASTM D2434
Hardened	UC strength	ASTM D4832
Hardened	TCLP toxicity	EPA SW-486

Table 3.49 Physical Property Tests of FS Samples (Deng and Tikalsky 2007)

Properties	Testing standards
Particle gradation	AFS 1105
Grain shapes	AFS 1107
Grain fineness number	AFS 1106
Adsorption	ASTM C128

3.3.4 FS in PCC

MECHANICAL PROPERTIES

◆ Properties of Foundry Sand

- Foundry Sand (FS) aggregates are generally sub-angular to round in shape. FS has a comparatively uniform grain size, with 85%- 95% of the grain size between 0.6mm and 0.15mm, and 5%- 12% of grain size probably smaller than 0.075mm (Siddique and Noumowe 2008).
- FS showed lower fineness modulus and bulk density than regular sand (Aggarwal and Siddique 2014). The specific gravity of FS varies between 2.39 and 2.55 (Siddique and Noumowe 2008).
- FS has a low water absorption capacity of 0.45% and high permeability of 10^{-3}~10^{-6} (Siddique and Noumowe 2008). Water absorption and void percentage of FS are higher than those of regular sand (Siddique and Noumowe 2008, Siddique et al. 2009).
- Friction angle of FS varies between 33° and 40°, comparable to that of natural sands (Javed and Lovell 1994).

◆ Fresh Concrete Properties

- Increasing FS content decreases the slump value of fresh concrete (Figure 3.79), possibly due to clay-type fine materials in FS that reduce the fluidity of the fresh concrete (Guney et al. 2010, Khatib et al. 2012). Slump dropped almost linearly, from 200mm for the concrete without FS, to zero for concrete with an 80% and 100% FS replacement for natural sand (Khatib et al. 2012).
- Concrete containing FS and bottom ash has a higher water requirement compared to concrete containing only regular sand, which is necessary to maintain workability within a specified range, i.e., slump, at 30 mm (Table 3.50; Aggarwal and Siddique 2014).
- FS reduces workability of both mortars and concrete; therefore, a higher amount of superplasticizer is required to maintain desirable workability. The dosage of superplasticizer depends on the w/c ratio, among other factors. Unit weight and entrapped air content of concrete are not affected by FS content (Monosi et al. 2010).

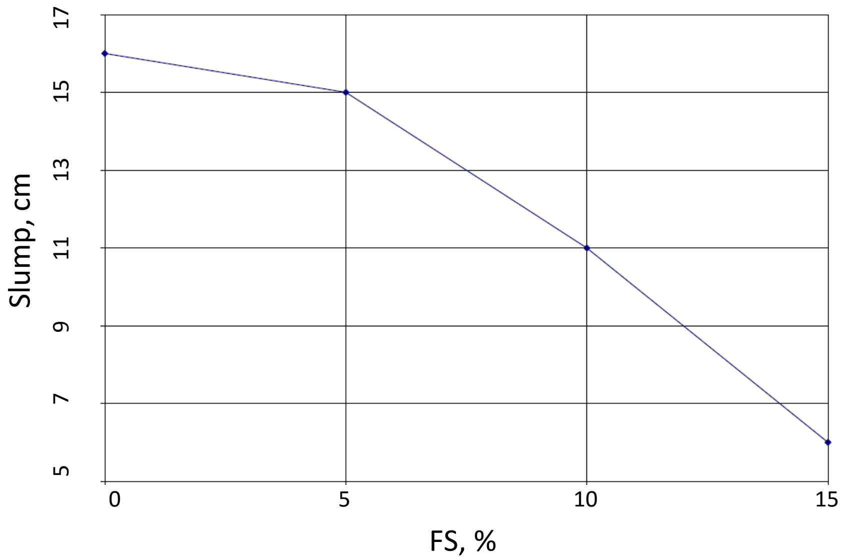


Figure 3.79 Slump of Concrete Containing FS (Guney et al. 2010)

Table 3.50 Fresh Concrete Properties with Bottom Ash & FS (Aggarwal and Siddique 2014)

Mix no.	CM	FB10	FB20	FB30	FB40	FB50	FB60
Cement (kg/m^3)	350	350	350	350	350	350	350
Foundry Sand (%)	0	5	10	15	20	25	30
Foundry Sand (kg/m^3)	0	30.25	60.50	90.75	121.00	151.25	181.50
Bottom ash (%)	0	5	10	15	20	25	30
Bottom ash (kg/m^3)	0	30.25	60.50	90.75	121.00	151.25	181.50
Water (kg/m^3)	175	180.30	185.60	190.90	201.50	212.12	238.63
W/C	0.5	0.52	0.53	0.55	0.58	0.61	0.68
Sand SSD (kg/m^3)	605	544.5	484.0	423.5	363.0	302.5	242.0
Fine aggregate (kg/m^3)	605	605	605	605	605	605	605
Coarse aggregate (kg/m^3)	1260	1260	1260	1260	1260	1260	1260
Superplasticizer (kg/m^3)	1.75	1.75	1.75	1.75	1.75	1.75	1.75
Slump (mm)	30	30	30	30	30	30	30
Compaction factor	0.83	0.81	0.78	0.81	0.78	0.78	0.81
Vee-bee consistometer (sec)	5.98	5.20	6.42	5.54	6.44	6.68	5.26
Air temperature (°C)	23	25	24	26	25	25	34
Concrete temperature (°C)	25	25	25	26	25	27	28
Air content (%)	2.1	2.6	2.6	2.7	2.7	2.9	3.4
Fresh concrete density (kg/m^3)	2392	2397	2402	2408	2418	2428.87	2455.38

Note: CM=control material, whose fine aggregate consists of natural sand; FB=FS and bottom ash, which replace fine aggregate (sand) at a certain percentage by weight.

- The water absorption of concrete with 5% FS is higher than conventional concrete, however absorption decreases when FS makes up more than 5%. Void content of concrete with 5% FS is higher than conventional concrete, however void content decreases when FS is more than 5%. (Guney et al. 2010).
- Another study indicated that water absorption increased with increasing FS content in concrete (Figure 3.80). Higher water absorption also implies a higher volume of pores, which is due to the unimodal grain size distribution of FS. The distribution resulted in low consolidation, and hence large volume of pores after consolidation (Khatib et al. 2012).

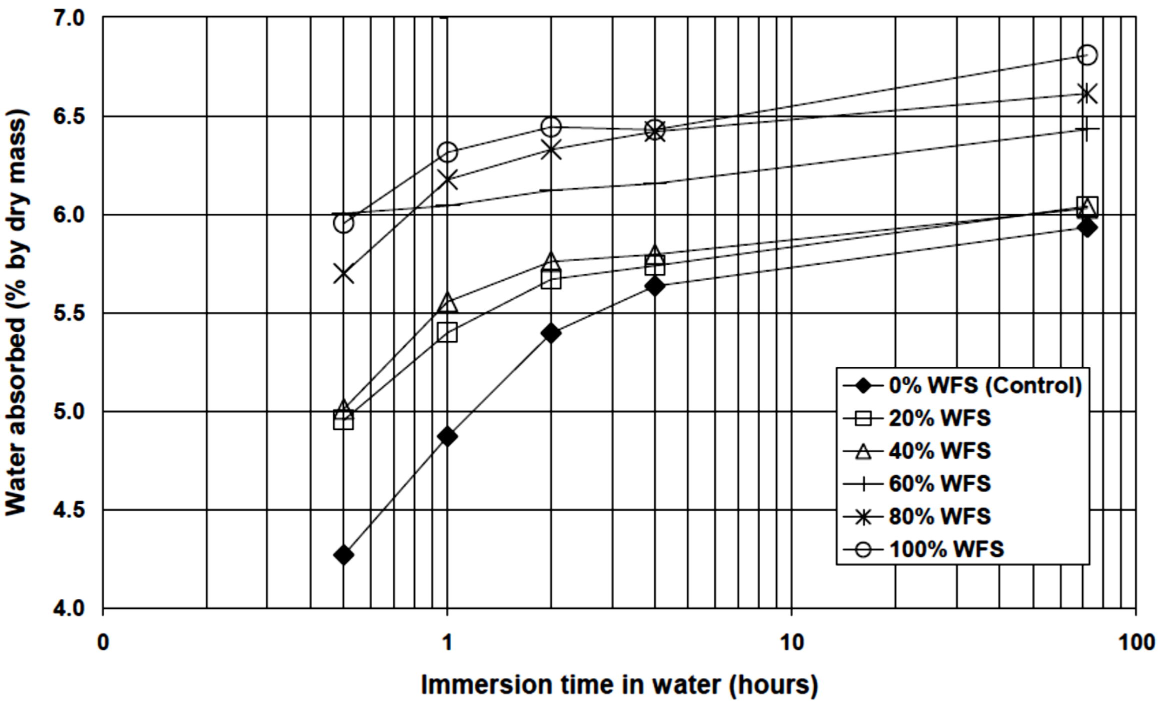


Figure 3.80 Effects of FS on Water Absorption of Concrete at 28 days of Curing (Khatib et al. 2012)

Note: WFS=waste foundry sand.

- <u>Hardened Concrete Properties</u>
  - Concrete made with green foundry sand (high-quality silica sand with clay binder) and chemical foundry sand (sand with one or more organic binders in conjunction with catalysts) obtains higher compressive strength than conventional concrete, when the concrete is produced with high w/c ratio (Etxeberria et al. 2010).
  - The study of Siddique et al. (2009) indicated that compressive strength of concrete increases slightly with the inclusion of FS (Figure 3.81); since FS is finer than regular sand, concrete made with FS is denser. The silica content in FS further improves the compressive strength. Compressive strength of concrete also increases with aging.
  - The study of Khatib et al. (2013) indicated that compressive strength decreases with increasing FS content, since fine particles in FS increase surface area and lead to weak interfacial zone.

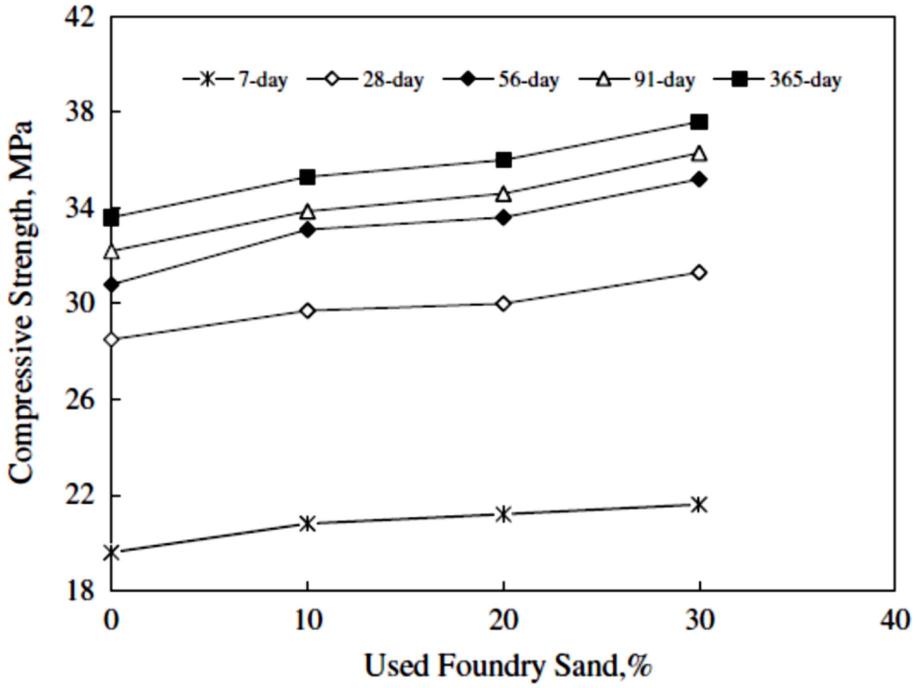


Figure 3.81 Compressive Strength in Relation to FS Content and Curing Age (Siddique et al. 2009)

- The study of Singh and Siddique (2012) indicated that compressive strength of concrete increases with increasing FS content up to 15% of partial replacement of sand, but reduces with 20% FS replacement. The former increase is due to fine particles in FS improving concrete density; the latter reduction is due to a large surface of fine particles reducing water cement gel in concrete matrix, and hence restricting the binding process of coarse and fine aggregate.
- The study of Guney et al. (2010) indicated that concrete with 10% FS shows comparable strength with conventional concrete, whereas concrete with other percentages of FS exhibits lower compressive strength. This may be related to the fact that particle size distribution of the concrete mixture with 10% FS results in more adherence, compared to other concrete mixtures with FS.
- The study of Aggarwal and Siddique (2014) indicated that compressive strength of concrete decreases when replacing natural sand at any percentage with FS and bottom ash in the same percentage. The maximum strength of concrete is obtained with the replacement of 30% natural sand, using 15% FS and 15% bottom ash (Figure 3.82). Compressive strength increases with aging, regardless of the percentage of FS and bottom ash.
- The study of Guney et al. (2010) indicated that for a 30 MPa compressive strength concrete, FS replacing 10%, 20%, and 30% of fine aggregate shows a higher compressive strength than the concrete without FS, at all ages. Compressive strength increases slightly with increasing FS content.
- The study of Siddique et al. (2015) indicated that the maximum compressive strength of concrete was observed at 15% FS replacement of fine sand. At 15% replacement, an M20 grade concrete (28-day compressive strength of 30 MPa) showed a higher strength increase than M30 grade (28-day compressive strength of 40 MPa) of concrete at any age, since M20 grade of concrete has more voids between particles filled by fine particles of FS.

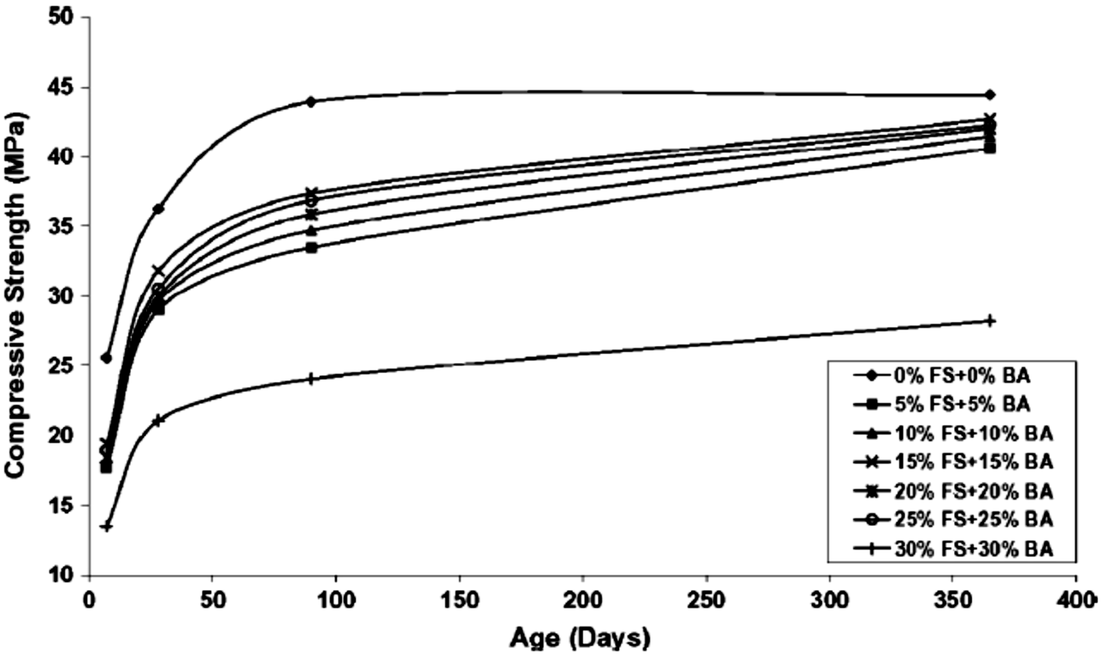


Figure 3.82 Compressive Strength of Concrete with FS and Bottom Ash, BA (Aggarwal and Siddique 2014)

- There is a linear relationship between compressive strength, Y, and water absorption coefficient, X (Figure 3.83). The water absorption coefficient is the rate of initial water absorption (in first 5 minutes), calculated with weight gain per unit area, divided by square root of time (Khatib and Clay 2004). The relationship seems to be independent of curing age and FS content (Khatib et al. 2013).

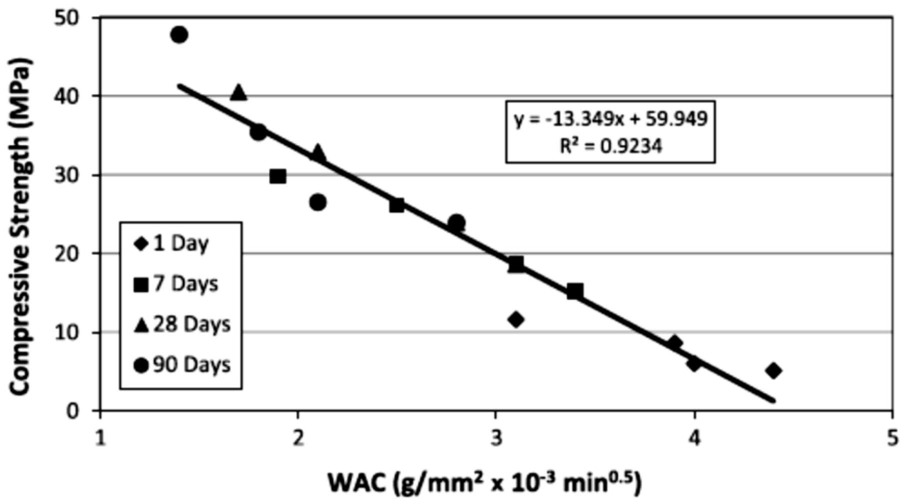


Figure 3.83 Relationship Between Compressive Strength and Water Absorption Coefficient, WAC (Khatib et al. 2013)

- The addition of FS improves splitting tensile strength of concrete for high w/c ratio (Etxeberria et al. 2010). As FS content increases, splitting tensile strength increases at all ages. Splitting-tensile strength also increases with increasing FS replacement (Siddique et al. 2009).

- Concrete with FS and bottom ash (in the same percentage) showed higher splitting tensile strength than conventional concrete. The maximum strength was obtained at a replacement of 30% (15% FS and 15% bottom ash). Splitting tensile strength increases with age, regardless of the percentage replacement of FS and bottom ash (Aggarwal and Siddique 2014).
- The maximum splitting tensile strength was achieved at 15% FS replacement of sand. At 15% replacement, the M20 concrete achieved higher increase in splitting tensile strength compared to the M30 (Siddique et al. 2015).
- Another study indicated that splitting tensile strength of concrete with 10% FS is slightly higher than that of concrete without FS, while the strength of concrete with 5% and 15% FS are lower than that of concrete without FS (Guney et al. 2010).
- Flexural strength of concrete mixtures increases slightly with increasing FS content. Flexural strength also increases with age (Siddique et al. 2009).
- The flexural strength of concrete with FS and bottom ash (BA) is lower than conventional concrete (Figure 84). FB30 (15% FS and 15% bottom ash) exhibited the highest strength among all FS and BA mixes at any age. Flexural strength of FS and bottom ash mixes increased with age (Aggarwal and Siddique 2014).

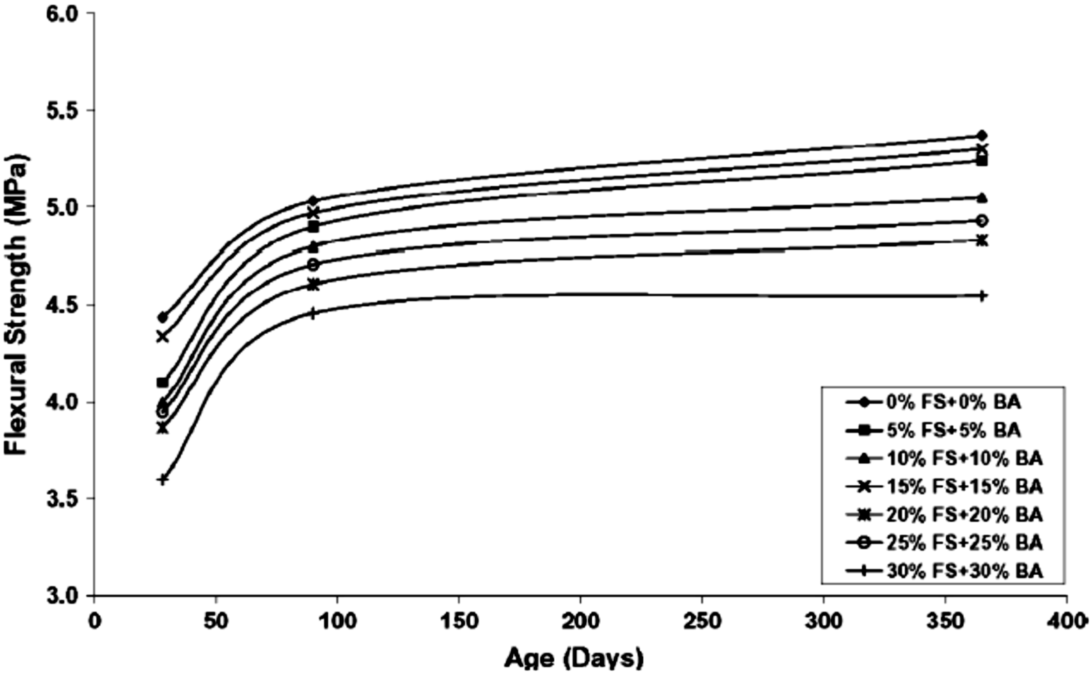


Figure 3.84 Flexural Strength of Concrete with FS and Bottom Ash, BA (Aggarwal and Siddique 2014)

- Addition of FS in concrete increases the modulus of elasticity at any age (Figure 3.85; Singh and Siddique 2012, Siddique et al. 2009). Modulus of elasticity also increases with increasing FS replacement. The modulus increase varies between 5.2% and 12%, depending on the FS content and curing time (Siddique et al. 2009).

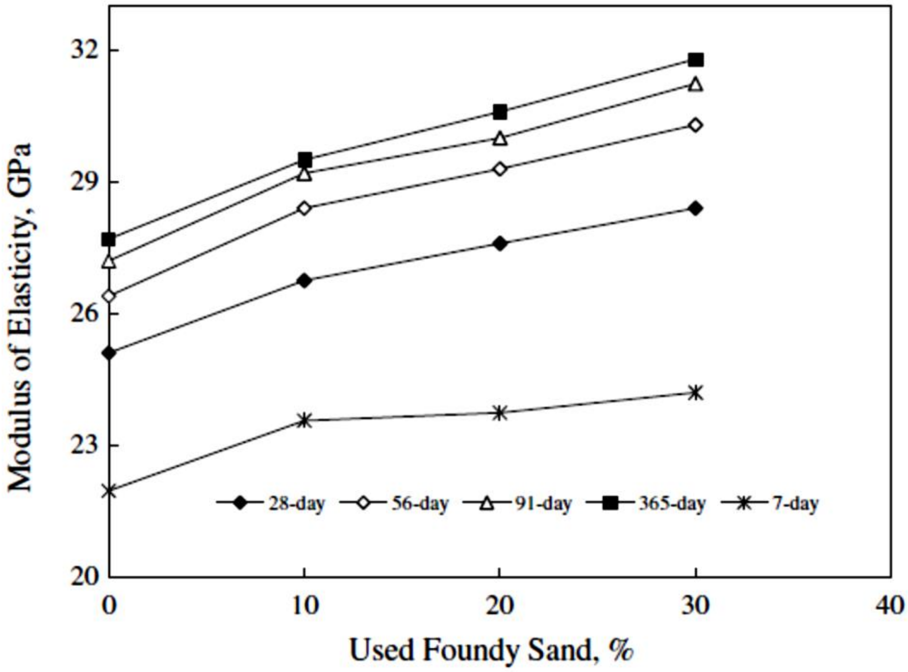


Figure 3.85 Modulus of Elasticity in Relation to FS Content and Curing Time (Siddique et al. 2009)

- Static modulus of elasticity increases with increasing compressive strength and vice versa, since the static modulus of elasticity is a function of the compressive strength (Guney et al. 2010, Siddique et al. 2015). The following relationship was proposed for these concrete mixtures: $E = 0.043 \times W^{3/2} \times \sigma^{1/2}$. E represents the modulus of elasticity in MPa, W is the concrete density in kg/m^3, and σ is the unconfined compressive strength in MPa (Guney et al. 2010).
- Inclusion of FS improved the modulus of elasticity of the M20 grade concrete at a higher rate than M30. Maximum increase of modulus was found at 15% FS replacement for both grades of concrete (Siddique et al. 2015).
- Dynamic modulus of elasticity for concrete with FS is lower than that of conventional concrete (Table 3.51). However, minor differences (within 6%) were observed for curing time of 28 days (Monosi et al. 2010).

Table 3.51 Dynamic Modulus of Elasticity (MPa) for Concrete Mixtures (Monosi et al. 2010)

MIX	C1	C1-7	C1-10	C2	C2-10
dynamic elastic modulus	40167	40052	37632	41920	39046

Note: Concrete (C1, C2) are proportioned with a water-cement ratio of 0.46 and 0.50; C1-7 indicates 7% mass of natural sand (fine aggregates) in Concrete 1 is replaced by FS; C1-10 indicates 10% mass of natural sand (fine aggregates) in Concrete 1 is replaced by FS.

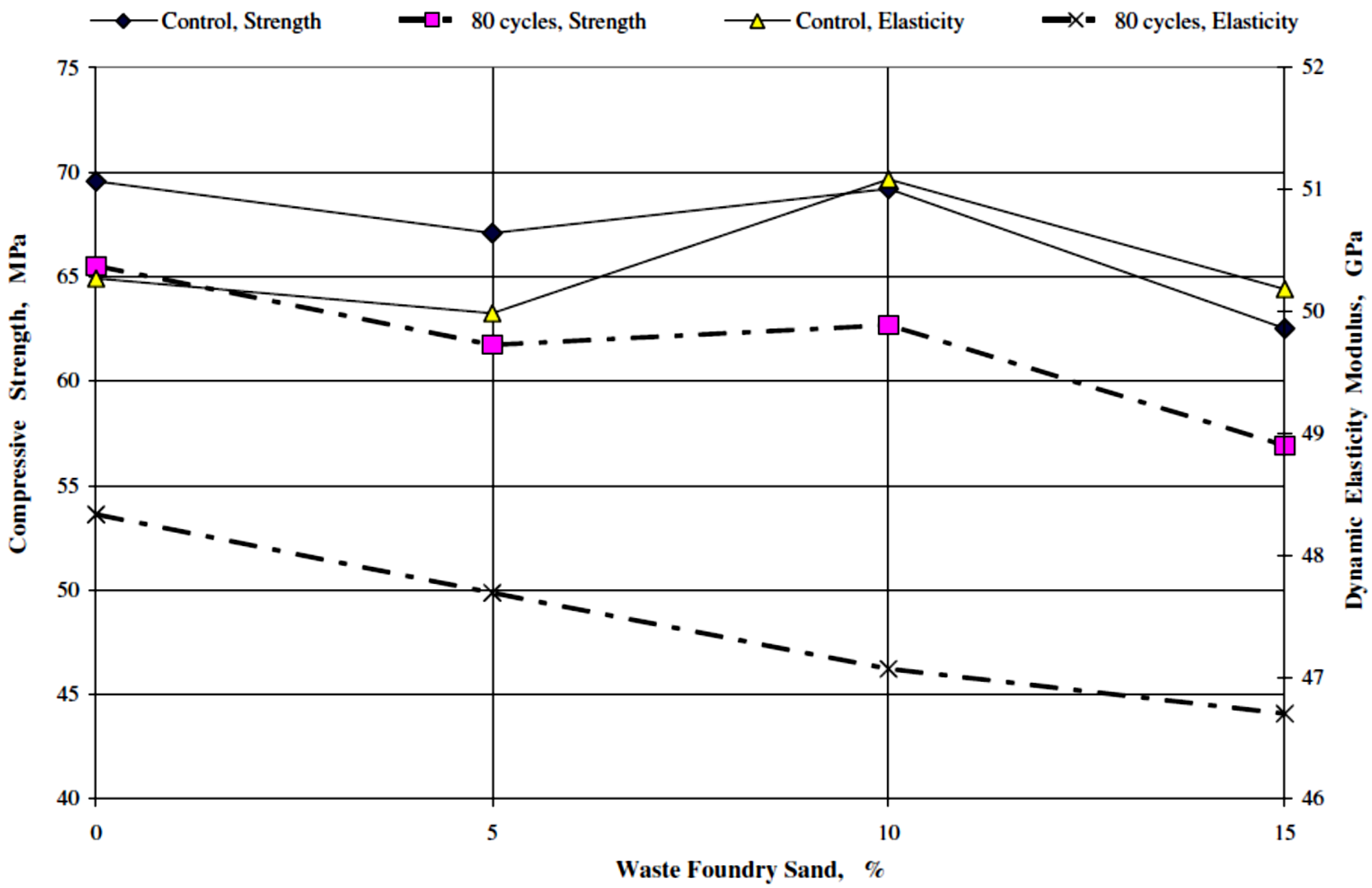


Figure 3.86 Freezing–Thawing on Compressive Strength and Dynamic Elasticity Modulus (Guney et al. 2010)

◆ Durability Properties

- Both compressive strength and dynamic modulus of the elasticity of concrete decrease with the freezing and thawing cycles, regardless of the FS content (Figure 3.86). The concrete with 10% FS is less influenced by freezing and thawing cycles, compared to the other FS concrete mixtures (Guney et al. 2010).
- A chloride permeability test showed that concrete with or without FS has low permeability, i.e., between 1000 and 2000 Coulombs (Figure 3.87). Chloride permeability decreased with increasing FS content up to 15%, then increased slightly with additional FS content. Decreased permeability implies higher density of concrete (Singh and Siddique 2012).
- Cement type, w/c ratio, curing condition, and testing age affect the chloride permeability of concrete. Resistance to chloride permeability decreases with aging, since finer particles of FS act as a good filler material to strengthen the internal structure of the concrete matrix (Aggarwal and Siddique 2014, Siddique et al. 2015, Singh and Siddique 2012).

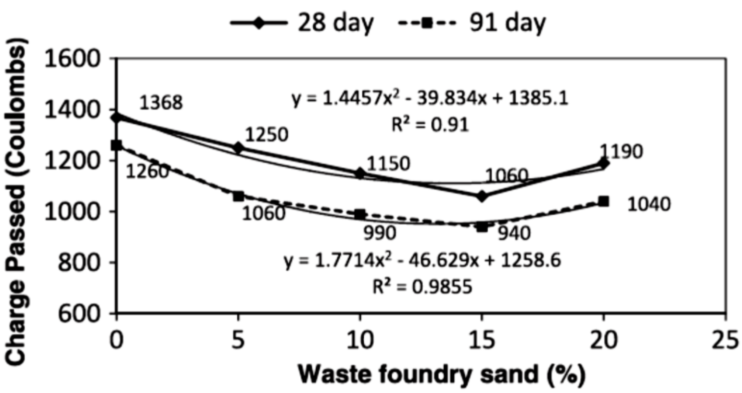


Figure 3.87 Effect of FS Content on Chloride Ion Penetrability (Singh and Siddique 2012)

- In another study, concrete with FS and bottom ash had a higher resistance to chloride penetration than concrete with only natural sand, with maximum resistance to permeability achieved by FB60 (30% FS and 30% bottom ash), Table 3.52. However, concrete with FS and bottom ash is classified as very low resistance to chloride penetration, according to ASTM C1202, i.e., less than 750 coulombs at 90 days and 500 coulombs at 365 days (Aggarwal and Siddique 2014).

Table 3.52 Chloride Permeability for Concrete with FS and Bottom Ash (Aggarwal and Siddique 2014).

Mix	Charge passed in coulombs (90-day)	Charge passed in coulombs (365-day)	Chloride ion penetrability
CM	578	323	Very low
FB10	628	357	Very low
FB20	616	306	Very low
FB30	600	321	Very low
FB40	664	383	Very low
FB50	652	377	Very low
FB60	741	486	Very low

- Ultrasonic pulse velocity (UPV) increases with increasing FS content in concrete, since fine particles of FS provide higher packing between particles, leading to lower permeability, and therefore a reduction in the transit time of the ultrasonic wave (Siddique et al. 2015).
- As FS replacement increases, UPV for M20 grade concrete increases more significantly than that of M30, since the addition of FS enhances the density of concrete and strengthens the internal micro-structure. The maximum increase of UPV was observed for the M20 concrete at 15% FS replacement (Siddique et al. 2015).
- However, another study indicated that UPV decreases with increasing FS content (Figure 3.88; Khatib et al. 2013).

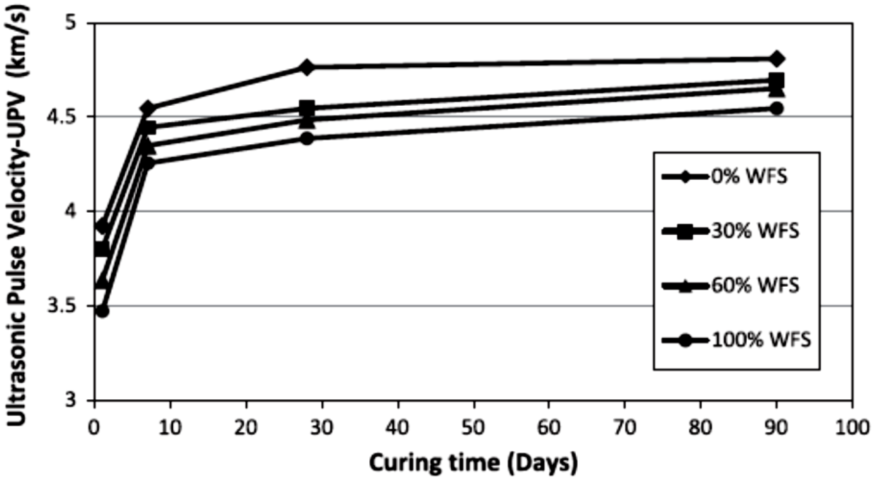


Figure 3.88 UPV vs Different FS Replacement at Different Curing Ages (Khatib et al. 2013)

- Higher UPV implies higher compressive strength (Figure 3.89). The relationship seems to be independent of the curing time or the FS content (Khatib et al. 2013).

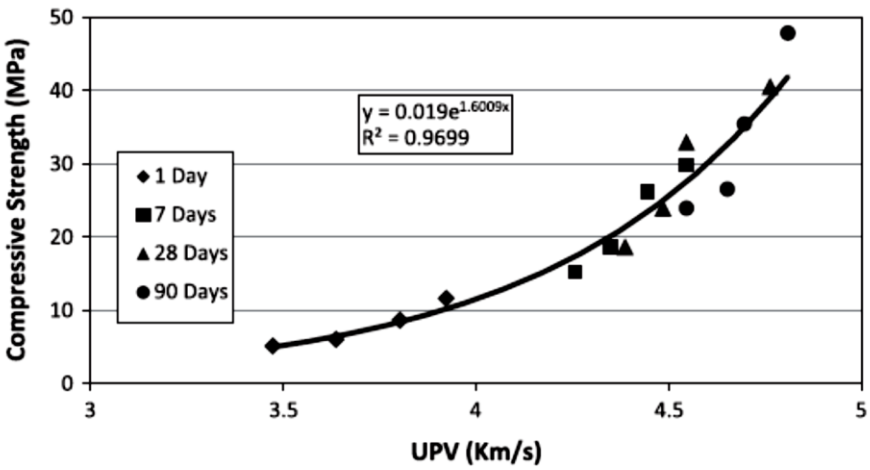


Figure 3.89 Relationship Between Compressive Strength and UPV (Khatib et al. 2013)

- Carbonation depth increases over time (Figure 3.90; Corinaldesi and Moriconi 2009, Siddique et al. 2011). FS replacement exacerbates carbonation. For every 10% increase of FS replacement, an average increase of 0.17 mm and 0.33 mm in carbonation depth occurs at 90 days and 365 days, respectively. The maximum carbonation depth occurs in the F60 mix (60% FS replacement) (Siddique et al. 2011).

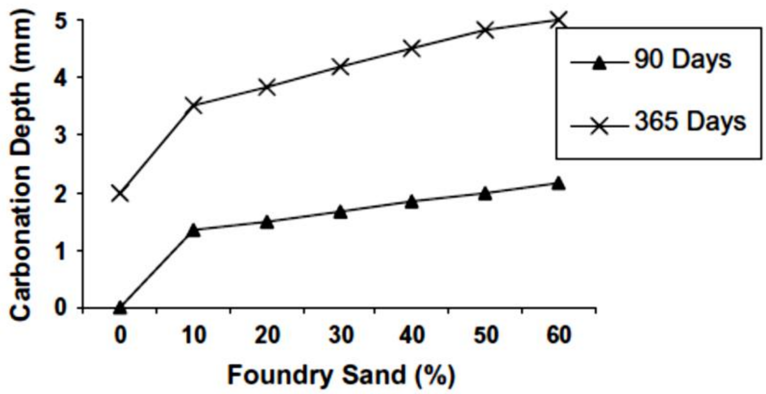


Figure 3.90 Carbonation Depth at Different Ages (Siddique et al. 2011)

- FS exacerbates drying shrinkage of concrete due to water loss (Figure 3.91). Shrinkage increases with increasing FS replacement (Khatib et al. 2012, Monosi et al. 2010).

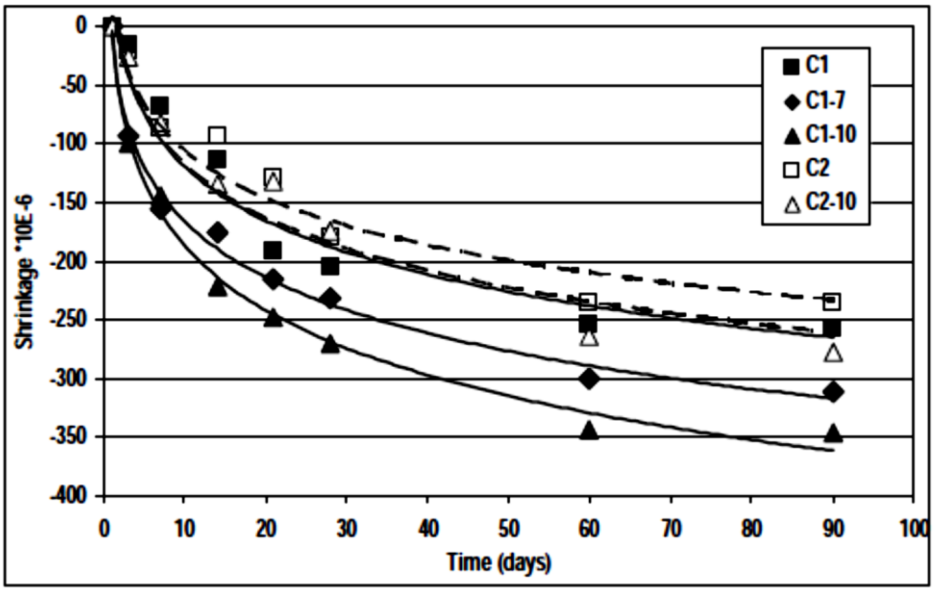


Figure 3.91 Concrete Drying Shrinkage vs Time (Monosi et al. 2010)

Note: Concretes (C1, C2) are proportioned with a water-cement ratio of 0.46 and 0.50; C1-7 indicates 7% mass of natural sand (fine aggregates) is replaced by FS in Concrete 1; C1-10 indicates 10% mass of natural sand (fine aggregates) is replaced by FS in Concrete 1.

- Shrinkage increases slightly both at short and long term curing times, since cement hydration may be delayed due to carbon (graphite) particles and/or a loosening of the bond between aggregate and cement paste (Monosi et al. 2010).
- Paste porosity, aggregate type and volume, and modulus of elasticity can affect drying shrinkage. The increase or decrease of drying shrinkage is consistent with compressive strength and modulus of elasticity (Monosi et al. 2010).

ENVIRONMENTAL PROPERTIES

- Metal concentrations (Ag, As, Ba, Be, Cd, Cu, Cr, Hg, Ni, Pb, Sb, Se, and Zn) tested by TCLP (Toxicity Characteristics Leaching Procedure) are below the thresholds for hazardous waste, according to the Resource Conservation and Recovery Act (RCRA). It is likely, though, that the metals released from FS are absorbed by organic matter and/or oxides, reducing the risk of metal leaching (Basta et al. 2005, Winkler and Bolshakov 2000).
- SPLP (Synthetic Precipitation Leaching Procedure) leaching results indicated that Ag, Be, Cd, Cr, Ni, Pb, and Sb were below their respective detection limits. As, Ba, Cu and Zn were the only metals that could be detected in SPLP. For As, 4 out of 43 samples slightly exceeded the National Primary Drinking Water Standard of 0.01 mg L^{-1}, while Ba, Cr, and Cu were lower than the National Primary Drinking Water Standard (Dungan and Dees 2009).
- The pH affects metals leaching from FS. The solution used for ASTM procedure and SPLP procedure are non-buffered; thus, the leaching results are similar (Dungan and Dees 2009).
- Most leachate was lower than requirements from Federal Drinking Water Standards. Metal concentrations are in the same order of magnitude to the concentration results of natural sand and sandy soils. FS from non-ferrous foundries (a combination of sand, dusts and slag) is occasionally found to have metal concentrations above RCRA thresholds (Winkler and Bolshakov 2000).
- Organic contaminants are often associated with binder. Green sand, which generally does not involve the use of organic binders, has lower potential for leaching organic compounds than chemically bonded sand. Organic compounds can be transformed into new hazardous compounds under incomplete combustion conditions. Organic compounds have not been found at significant concentrations in sand (FIRST 2004).
- Fungal treated concrete with FS shows a reduction in metal concentration, since fungi can remove both soluble and insoluble metal species from solutions (Burgstaller and Schinner 1993). Fungi can produce organic acids, which can solubilize metal and provide anions and protons for metal leaching (Sayer et al. 1997). Significant reductions in Cu, Cr, Hg, Li, Mg, Mn, Pb, and Zn were obtained in concrete made with fungal treated FS, with less reduction in Hg, Ba and Ni (Table 3.53; Kaur et al. 2013).

RECOMMENDATIONS

- Casting process evolves in various sands, inorganic or organic binders, and other additives. To avoid these excessive waste residues, screening systems and magnetic separators are needed to segregate usable sand from other wastes, and to separate particles of varying sizes prior to recycling (FIRST 2004).
- The casting cores are hardened by additives (i.e., epoxies, resins, organic binders) to form the inside part. Therefore, FS used to form the inside shapes needs further crushing, separation and screening before recycling (NCHRP 435).

Table 3.53 Metal Analysis of Leachate Obtained from Untreated and Fungal FS (Kaur et al. 2013)

Metal	WFS untreated	Fungal treated WFS	WHO standard limits (mg/l)	GWQS (mg/l)
Be	.02 ± .007	.0001 ± .00002	–	–
Ba	.03 ± .02	.02 ± .004	.3	.4
Cd	**.02 ± .01**	**.0005 ± .0002**	**.003**	**.0005**
Cr	**.19 ± .007**	**.05 ± .006**	**.05**	**.01**
Co	.01 ± .003	.0003 ± .0001	–	–
Cu	.051 ± .011	.01 ± .003	2	–
Fe	**.25 ± .04**	**.04 ± .02**	–	**.15**
Hg	.1 ± .08	.05 ± .02	.001	.0002
Li	.07 ± .02	.02 ± .007	–	–
Mo	**.10 ± .08**	**.005 ± .002**	**.07**	–
Mg	.05 ± .003	.02 ± .0z1	–	–
Mn	**.14 ± .06**	**.01 ± .004**	**.5**	**.02**
Ni	**.05 ± .04**	**.013 ± .006**	**.02**	–
Pb	**.19 ± .014**	**.01 ± .008**	**.01**	**.001**
Zn	.31 ± .11	.05 ± .02	3	2.5

WHO – world Health organization standard limits.
GWQS – Ground water quality standards.
Values in bold indicates metal reduction in fungal treated samples as compared to standard limits.

- Concrete, where up to 15% FS replaces fine aggregates, could be suitable for structural concrete (Singh and Siddique 2012).
- Since using alkyd urethane binder elevates Co and Pb concentrations, foundries are encouraged to use alternative binder systems with lower metal concentrations (Miguel et al. 2012).

BENEFITS

- Bhat and Lovell (1997) suggested that if clean sand was replaced by FS, which requires about 50% more cement, cost could still be reduced by 25% to $6.44/ton. A study from Italy indicated that treatment costs for recycling FS are justified by the savings in raw materials as well as the economic and environmental advantages from landfill use reduction. The savings can be up to 35,000 €/d (Fiore and Zanetti 2007).
- Heavy demand for concrete has resulted in the over-exploitation of river sand, causing an increase in riverbed depth, producing a lower water table and introducing salinity into rivers. Using FS can mitigate such effects (Prabhu et al. 2014).
- The restrictions associated with extracting sand from rivers increases the price of sand and has severely affected the stability of the construction industry (Dolage et al. 2013). Therefore, finding an alternative material to river sand has become imperative.

SUGGESTED SPECIFICATIONS

Table 3.54 Test Methods Used to Evaluate Byproducts and Highway Application Products (NCHRP 435)

Test Methods	Title
AASHTO Methods	
T112	Standard method of test for lightweight pieces in aggregate
T215	Standard method of test for permeability of granular soils
T90	Standard method of test for determining the plastic limit and plasticity index of soils
ASTM Methods	
C128	Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Fine Aggregates
C142	Standard Test Method for Clay Lumps and Friable Particles in Aggregates
C29	Standard Test Method for Potential Alkali Reactivity of Cement-Aggregate Combinations (Mortar-Bar Method)
C403	Standard Test Method for Time of Setting of Concrete Mixtures by Penetration Resistance
C88	Standard Test Method for Soundness of Aggregates by Use of Sodium Sulfate or Magnesium Sulfate
D1883	Standard Test Method for CBR of Laboratory Compacted Soils
D2216	Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
D2434	Standard Test Method for Permeability of Granular Soils (Constant Head)
D3987	Standard Test Method for Shake Extraction of Solid Waste with Water
D69	Standard Test Methods for Friction Tapes
D698	Standard Test Methods for Laboratory Compaction Characteristics of Soils Using Standard Effort
D854	Standard Test Method for Specific Gravity of Soil Solids by Water Pycnometer

Table 3.55 Regulatory Levels for Various Metals (Dungan and Dees 2009)

Element	Concentration (mg L^{-1})		
	TCLP regulatory levels	NPDWS	NSDWS
Ag	5.0		0.1
Al			0.05
As	5.0	0.01	
Ba	100.0	2.0	
Be		0.004	
Cd	1.0	0.005	
Cr	5.0	0.1	
Cu		1.3	1.0
Fe			0.3
Hg[a]	0.2	0.002	
Mn			0.05
Ni			
Pb	5.0	0.015	
Sb		0.006	
Se[a]	1.0	0.05	
Tl[a]		0.002	
Zn			5.0

TCLP, Toxicity characteristic leaching procedure.
NPDWS, National primary drinking water standard.
NSDWS, National secondary drinking water standard.
[a] Not quantified in this study.

3.4 Dredged Material (DM)

3.4.1 DM in Fill

MECHANICAL PROPERTIES

- Characteristics of DM
  - Baltimore Harbor sediment is classified as CH (Fat clay), with a liquid limit of 85, a plastic limit of 35, an average density of 10.8 kN/m^3 (68.49 pcf) and moisture content of 400%-600% (Crawford and Aydilek 2004).
  - The properties may differ depending on where the DM is collected. For instance, sediment from Mobile, Alabama's Mobile Harbor is classified as CL/ML (Lean clay/Silt), with a liquid limit of 96, a plastic limit of 28 and a specific gravity of 2.7 (Poindexter and Walker 1998). The New Jersey sediment is classified as MH/OH (Elastic silt/Organic clay or silt).
  - DM is usually composed of more silt and clay (< 0.063mm), compared to construction and demolition waste (Sheehan et al. 2008).
  - Denser soils have better weight-bearing capacities (Winfield and Lee 1999). Angular particles can bear more weight than rounded particles, since interlock between particles forms a stable, dense mass (Sheehan et al. 2008). The failure strain for angular-shaped particles is twice that for spherical particles.
  - Gradation and particle shape and size influence water-storage capacity, water-infiltration rates, aeration, fertility, ease of tilling and compressibility. Mineral and organic content, and moisture content in particles also affect these properties (Sheehan et al. 2008).
  - Plasticity of DM is associated with the types and amount of clay particles, water content and physicochemical interactions between clay particles. It influences compactibility, compressibility, shear strength or permeability of the material (Winfield and Lee 1999).
  - Permeability is related to mineralogy, particle size, gradation, void ratio and water content. Fine fractions (i.e., clay) usually have low permeability; however, high permeability is required when DM is used as fill materials.
  - Bulk unit weight is not significantly affected by cement or water content (Figure 3.92a). Bulk unit weight decreases slightly with increasing water content (Figure 3.92b); decreases strikingly with increasing air foam content, since a little air foam can generate large amount of voids (Figure 3.92c); and increases with increasing bottom ash content(Figure 3.92d; Kim et al. 2010).
  - Bulk unit weight linearly decreases if rubber is added to stabilize DM, as rubber has less specific gravity than DM. Rubber-added DM can achieve minimum weight fill. Rubber also works as thermal and buffer insulations in the fill material (Kim and Kang 2011).
  - DM contains organic matter with higher plasticity, shrinkage, compressibility, permeability, and lower shear strength. Other performances may also be improved, such as enhancing buffering capacity and immobilizing contaminants (Winfield and Lee 1999).

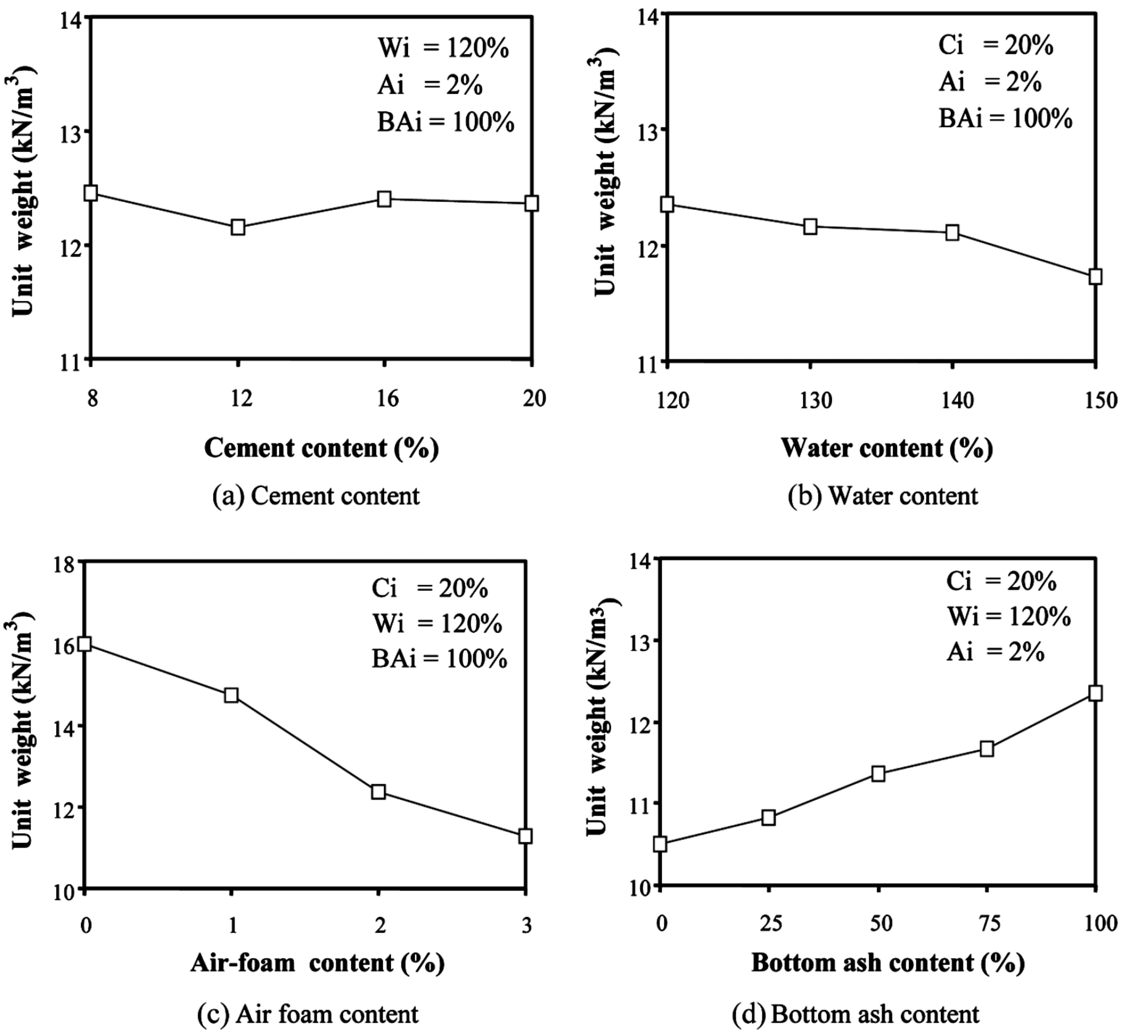


Figure 3.92 Bulk Unit Weight with Various Mixing Conditions (Kim et al. 2010)

Note: Ci=Cement content; Wi=Water content; Ai= Air foamed content; Bai=Bottom ash content.

- <u>In Flowable Fill</u>
  - DM has good to poor fill material characteristics (Mir et al. 2013). Good flowability of fill materials requires ability to self-level, self-fill and self-compact.
  - Air-foam stabilized DM has low weight and high flowing ability (Feng et al. 2001).
  - Flowability increases slightly with increasing air foam content (Figures 3.93c), decreases slightly with increasing cement and bottom ash contents (Figure 3.93a, Figure 3.93d), and rapidly increases with increasing water content (Figure 3.93b). Since air foam and water act as lubricants between particles, reducing the internal friction of the mixture, increasing these two can improve flowability (Kim et al. 2010).
  - Water content has the largest effect on flowability (Kim et al. 2010). However, higher water content results in reduced strength and aggregates segregation (Wu and Tsai 2009).

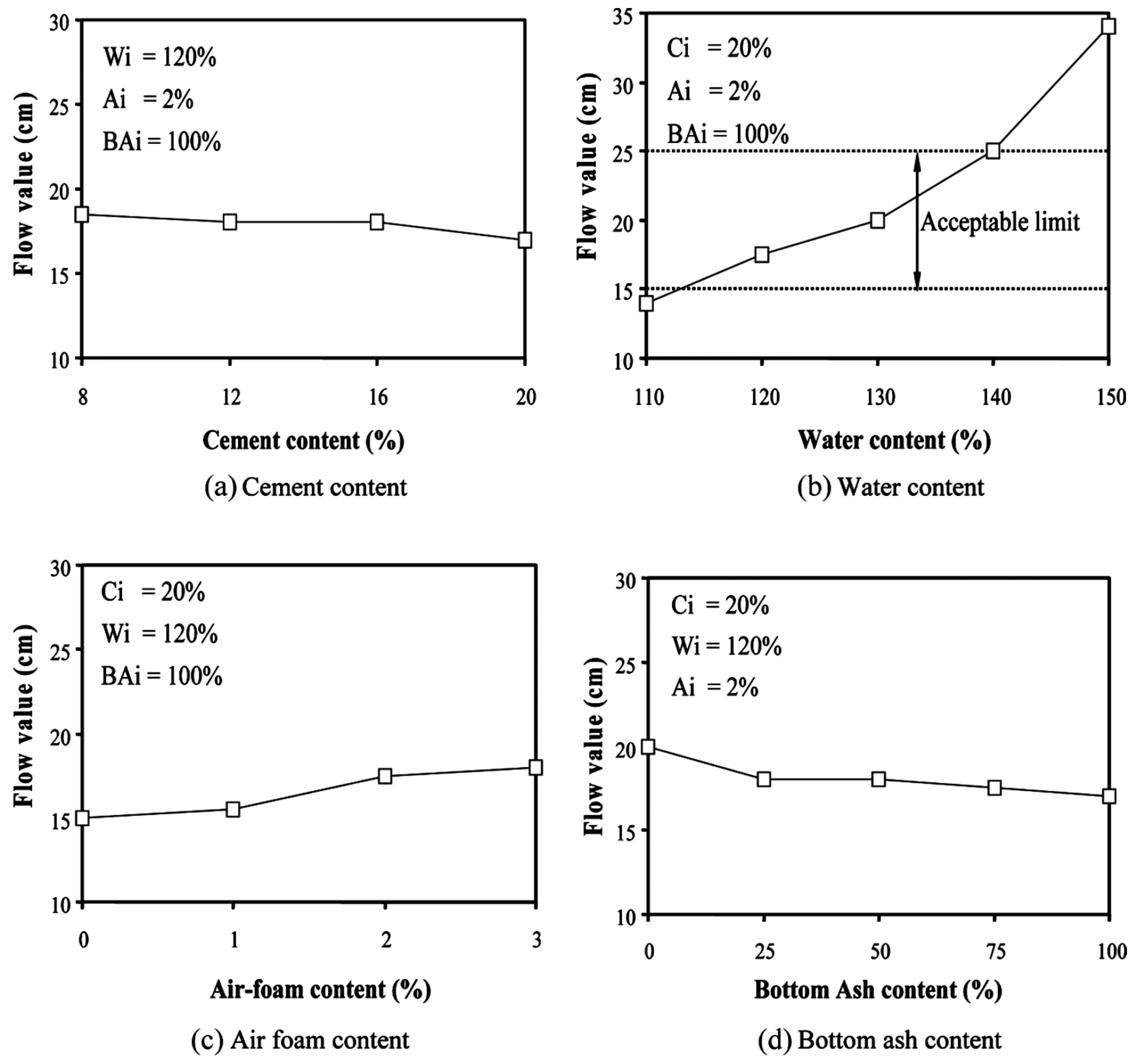


Figure 3.93 Flow Values with Various Mixing Conditions (Kim et al. 2010)

Note: Ci= Cement content; Wi= Water content; Ai= Air foamed content; Bi= Bottom ash content.
% =ratio of additives weight to the weight of dry soil.

- In the case of rubber addition for stabilization, flowability of DM decreases with increasing rubber content (Kim and Kang 2011). Rubber has poor gradation, high porosity and high permeability, which is unfavorable to the flowability of rubber-added lightweight soil (Wu and Tsai 2009).
- When rubber content is less than 50%, flowability increases with a higher water content. When rubber content exceeds 75%, adequate flow value (20 ± 5 cm) cannot be reached, regardless of the water content. At high rubber contents, water only drains out of a non-lubricated mixture (Wu and Tsai 2009).
- Acceptable flow value can be obtained by a combination of 140%-160% water with 0% rubber, 140%-180% water with 25% rubber, or 160%-200% water with 50% rubber (Figure 3.94; Kim and Kang 2011).
- The viscosity of Baltimore Harbor DM increases with bentonite stabilization, since bentonite is clay and denser than DM (Crawford and Aydilek 2004).

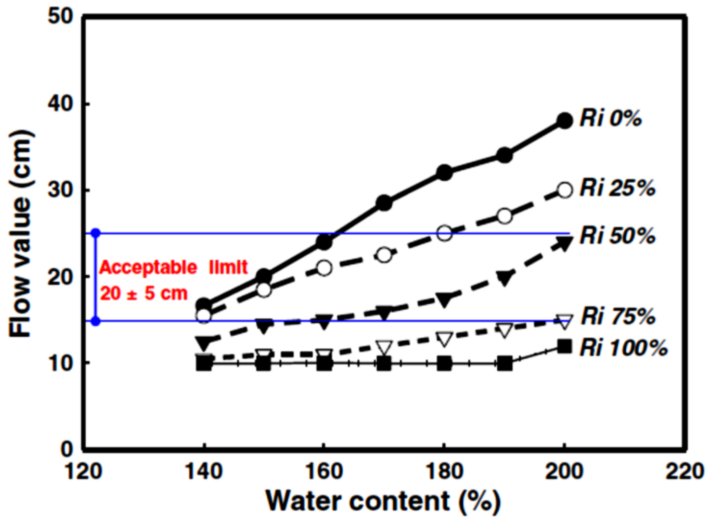


Figure 3.94 Flow Value with Rubber Content and Water Content (Kim and Kang 2011)

- Hydraulic conductivity of Baltimore DM decreases as bentonite content increases. Hydraulic conductivity decreases under greater pressure, since increased stress decreases the void ratio. Hydraulic conductivity increases with increasing fly ash content; fly ash attaching to the fines forms a better graded granular structure, thus increasing the void ratio (Crawford and Aydilek 2004).
- Grubb et al. (2007) stabilized DM with steel slag fines and found that hydraulic conductivity can be controlled by fines content and plasticity of the stabilized DM. The addition of 60%-80% steel slag fines (SSF) increases hydraulic conductivity of 100% DM by 1-3 orders of magnitude (Table 3.56; Grubb et al. 2007, Malasavage et al. 2012).

◆ In Embankment

- Adding cement to DM reduces ignition values. This indicates the reduction of organic content in DM, since cementitious matters from the chemical reactions of binders absorbs organic material in DM. Cement flocculates the fractions in soils, increasing the particle size and improving plasticity (Chan 2012).
- Cement improves ductility and prompts strain hardening of soil-cement mixture (Mostafa et al. 2002). Cement also contributes to increased shear strength due to cementation effect (Kim et al. 2010). A small dosage of cement is enough to solidify large amounts of soils, though a large dosage of fly ash is better than cement for strength enhancement (Chan 2012).
- Steel slag is approximately twice as effective in solidifying DM than that of cement-fly ash blend, since steel slag plays both roles of binder and filler and has large particle size, bonding the soil with slag particles and stiffening the structure of the mixture (Chan 2012).
- Cohesion of steel slag-stabilized DM is dependent on compaction-induced stresses and cementation during curing. Increasing steel slag fines content reduces compressibility and requires greater consolidation to obtain enough compressibility (Grubb et al. 2007, Malasavage et al. 2012).
- The addition of cement or fly ash improves the strength of DM mixtures, since they fill the voids within the soil and bind soil particles together. However, large fly ash contents are detrimental to the solidification process due to the presence of fine particles and unburned carbon in fly ash (Wang et al. 2011).

Table 3.56 Strength, Hydraulic Conductivity, and Consolidation Parameters of DM, SSF and DM-SFF blends (Malasavage et al. 2012)

Media tested[a]		CIŪ triaxial D4767 (2004a) c' [kPa (psf)]	ϕ' (°)	Hydraulic conductivity D5084 (2003) k (cm/s)	1D consolidation D2435 (2004b) c_v @400 kPa (cm²/s)	c_v @800 kPa (cm²/s)	P_c [kPa (psf)]	C_c (—)	C_r (–)
Dredged material (DM)		41(856)	27.3	1.06×10^{-8}	0.24	0.25	144 (3,000)	0.28	0.04
Blends	80/20 DM-SSF	48(1,003)	32.4	1.54×10^{-8}	0.29	0.24	129 (2,700)	0.28	0.04
	60/40 DM-SSF	70(1,462)	39.9	1.48×10^{-7}	0.3	0.26	125 (2,600)	0.21	0.02
	50/50 DM-SSF	48(1,003)	45.0	2.88×10^{-7}	0.23	0.25	129 (2,700)	0.21	0.02
	40/60 DM-SSF	62(1,295)	43.4	1.85×10^{-7}	0.39	0.39	144 (3,000)	0.19	0.04
	20/80 DM-SSF	104(2,172)	38.6	1.23×10^{-5}	0.4	0.4	168 (3,500)	0.18	0.01
Steel slag fines (SSF)[b]		48(1,003)	45.7	6.12×10^{-3}	1.46	1.78	144 (3,000)	0.12	0.008

Note: ASTM designations shown where relevant; c' = effective cohesion; ϕ' = effective friction; c_v = coefficient of consolidation; P_c = preconsolidation pressure; C_c = compression index; C_r = recompression index.

[a]Blend nomenclature shows DM content first, dry weight % basis. All specimens compacted to 95% relative compaction of maximum dry unit density per ASTM D1557 (2000a).

[b]Hydraulic conductivity for 100% SSF media per ASTM D2434 (1968).

- The addition of cement to DM obtains a higher strength than those with both fly ash and cement in the same percentage. Increasing cement content increases unconfined compressive strength of DM, since higher cement content facilitates a stronger pozzolanic reaction. Cement treatment also improves the ultimate strength and elastic modulus of DM mixtures due to pozzolanic activity (Chittoori et al. 2014).
- Unconfined compressive strength and initial slope of stress-strain curve for composite DM (with additives of cement, air foam and bottom ash) increases with increasing cement contents, but decreases with increasing water and air foam contents (Figures 3.95a-c). Most specimens exhibited shear failure, while a few specimens exhibited bulging failure because of low cement content, high water content or high air foam content (Kim et al. 2010).

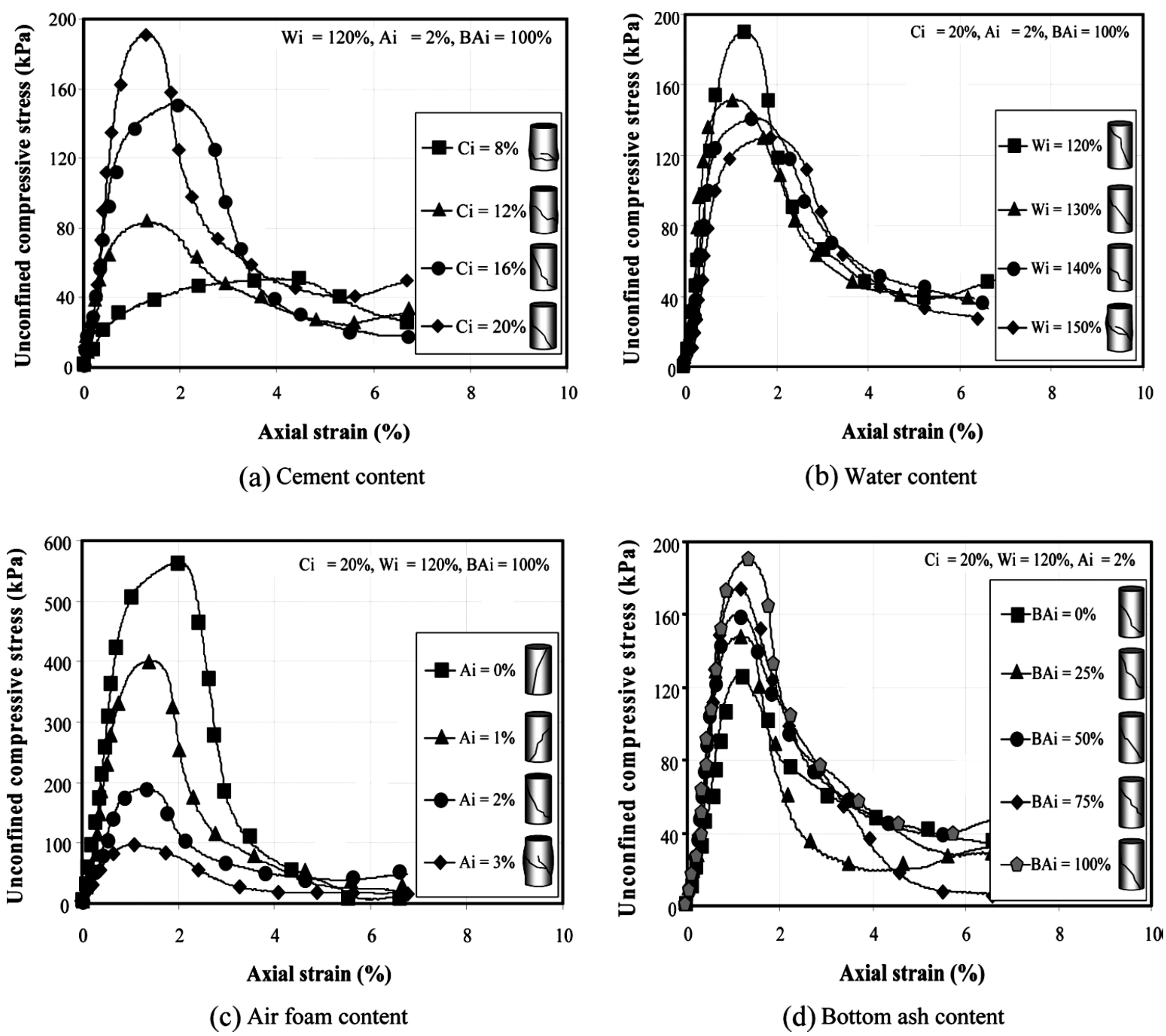


Figure 3.95 Stress-Strain Relationship with Various Mixing Conditions (Kim et al. 2010)

Note: Ci=Cement content; Wi=Water content; Ai=Air foamed content; Bi=Bottom ash content.

- Some cement-solidified DMs are able to recover strength lost with available calcium oxide, adequate temperatures, and a high pH environment. However, after the initial curing of DM, residual calcium oxide is almost depleted, resulted in permanent strength loss (Maher et al. 2006).
- The strength of air-foam stabilized DM increases with increasing cement content, but decreases with increasing air-foam content (Feng et al. 2001).
- Maximum compressive strength of composite DM increases with a higher bottom ash content (Figure 3.95d), since friction between aggregates improves shear resistance and pozzolanic reaction improves bond strength. Unconfined compressive strength of DM mixture increases linearly with a higher bottom ash content (Kim et al. 2010).
- Unconfined, compressive strength and initial slope of the stress-strain curve of rubber-added DM decrease with increasing rubber content (Figure 3.96). Shear strength reduces with increasing rubber component, due to loss of friction and bonding in the mixtures. Rubber promotes a light unit weight and ductile behavior of soil mixtures. However, high rubber content diminishes strength and stiffness because of fabric change and undesirable particle bonding (Kim and Kang 2011).

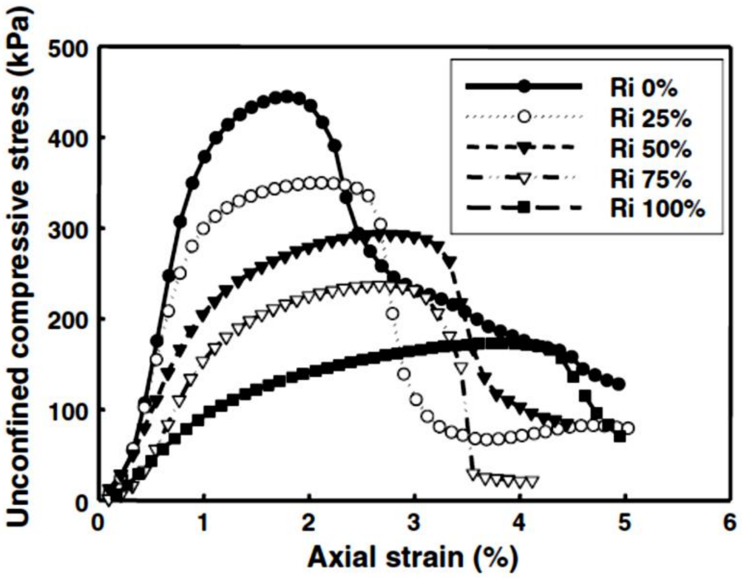


Figure 3.96 Stress-Strain Relationship with Rubber Content (Kim and Kang 2011).

- Steel slag fines blended with DM has much higher strength than crushed glass (CG)-blended DM, due to higher specific gravity of steel slag fines ($SG_{SSF}/SG_{CG}=1.4$). This difference affects blend unit weights, and reactivity (residual lime content in steel slag fines) associated with cementation (Grubb et al. 2013).
- Aging effect improves compressive strength of DM blended with steel slag fines, while slightly affects moisture content, indicating a relatively constant volume, density and moisture content throughout the curing period (Grubb et al. 2013).
- Shear strength increases by increasing normal stress and bottom ash content of stabilized DM, due to bond strength improved by the pozzolanic reaction of bottom ash and development of friction at the interface of mixture components. Cohesion increases with increasing bottom ash content. The internal friction angle increases slightly with an increase in bottom ash content (Kim et al. 2010).

- As steel slag fines content increases from 20% to 80%, CPT (cone penetrometer tests) tip resistance triples, while the same content change in crushed glass only doubles the CPT tip resistance (Grubb et al. 2006, Grubb et al. 2008).
- CPT sleeve resistance increases with aging of DM by an approximate factor of 2-4 and a decreasing DM content (Grubb et al. 2008). Although DM blended with crushed glass is not as strong as coarse materials (i.e., sands), they exceed the strengths of other stabilized fines, such as DM blended with ash (Grubb et al. 2013).
- Stiffness of DM with the addition of bottom ash is greater than that of untreated DM (Kim et al. 2010). Stiffness of rubber-added DM is less than that of bottom ash-added DM (Kim and Kang 2011).
- Air-foam stabilized DM has higher resilient modulus than original DM. The allowable number of load repetitions increases with increasing resilient modulus, cement content, or air-foam content (Park et al. 2014). Resilient modulus can be predicted by a linear relationship of compression test at 28 days curing, $Mr = 730q_u - 293000$, where q_u is unconfined compressive strength (Park et al. 2014).

ENVIRONMENTAL PROPERTIES

- Contaminants (metals, pesticides, polycyclic aromatic hydrocarbons (PAH) and polychlorinated biphenyls (PCB)) are a concern for using DM. The solubility, mobility and bioavailability of these contaminants reduce under anaerobic alkaline conditions. However, DM becomes oxidized and more acidic during dredging and placement. (Winfield and Lee 1999).
- Grubb et al. (2013) showed that less than 25% chromium was leached from 100% DM, meeting Maryland Department of the Environment criteria for chromium (Table 3.57).
- One hundred percent of steel slag fines did not leach Fe above a pH of 7.3 (<0.05 mg/L), which was far below the EPA secondary drinking water criteria of 0.3 mg/L. For DM- steel slag fines blends, Fe leaching was predicted to be less than 0.05 mg/L for a pH > 7. For 100% DM, Fe leaching increased with increasing acidification (Grubb et al. 2013).
- Although steel slag fines have a high capacity of fixing arsenic and 100% DM leached up to 125 mg/kg arsenic, 100% steel slag fines, 100% DM, or DM-steel slag fines blends did not exceed USEPA contamination limits (Grubb et al. 2010).
- Aged DM-steel slag fines blends leached up to 45 mg/kg arsenic, less than the Synthetic Precipitation Leaching Procedure detection limit of 0.056 mg/L (Grubb et al. 2011). Field arsenic concentrations for DM were 26 mg/kg, less than the Precipitation Leaching Procedure detection limit of 0.028 mg/L and almost matching the Toxicity Characteristic Leaching Procedure (TCLP) detection limit of 0.02 mg/L (Grubb et al. 2013).
- Dredged sediment barriers can serve as an effective containment and remediation system under appropriate conditions. Increasing bentonite content leads to an increasing adsorption of metals (cadmium, chromium, lead and zinc), while increasing fly ash content leads to a decreased adsorption of the metals. Larger barrier thickness improves adsorption. increased hydraulic gradient degraded adsorption and increased effective porosity has no effect on adsorption. Adsorption capacity depends on breakthrough time. A longer breakthrough time is associated with a higher adsorption capacity (Crawford and Aydilek 2004).

Table 3.57 Summary of Total Metal Concentration Results for 100% DM, 100% SSF and DM-SSF Blends, mg/kg (Grubb et al. 2013)

	Eastern U.S. soils[a]		MDE[b]	NJ[c]	PA[d]		DE[e]	365-day				
PPL metal	Range	Average	Non-Res	Non-Res	Clean	Regulated	URS NC RU	100% DM	80/20 DM-SSF	50/50 DM-SSF	20/80 DM-SSF	100% SSF
Antimony (Sb)	<1.0–8.8	0.76	41	19	12	53	82	<3.30	<2.91	<2.84	<2.43	<2.67
Arsenic (As)	<0.1–73	7.4	1.9	19	12	53	4	26.0	23.6	12.9	3.15	<1.84
Beryllium (Be)	<1.0–7.0	0.85	20	140	320	320	410	1.90	1.55	0.983	<1.74	<1.12
Cadmium (Cd)	ND-4.0	—	100	78	38	38	100	0.709	0.367	<0.966	<1.74	<2.10
Chromium (Cr tot)	1.0–1,000	52	310	NR	NR	NR	NR	132	363	612	908	1,133
Chromium (Cr III)	—	—	150,000	NR	190,000	190,000	310,000	—	—	—	—	—
Chromium (Cr VI)	—	—	310	6,100	94	190	610	—	—	—	—	—
Copper (Cu)	<1.0–700	22	4,100	45,000	8,200	36,000	8,200	221	371	170	71.4	49.5
Iron (Fe)	100–100,000	25,000	72,000	NR	NR	190,000	61,000	58,600	100,133	114,433	181,333	221,000
Lead (Pb)	<10–300	17	1,000	800	450	450	1,000	86.6	75.7	47.2	19.8	<19.9
Mercury (Hg)	<0.01–3.4	0.12	31	65	10	10	610	0.245	0.203	<0.143	<0.116	<0.101
Nickel (Ni)	<5.0–700	18	2,000	23,000	650	650	4,100	333	521	245	73	<18.0
Selenium (Se)	<0.1–3.9	0.45	510	57,000	26	26	1,000	2.42	1.59	0.933	<0.729	<1.38
Silver (Ag)	—	—	510	4,100	84	84	1,000	<2.15	1.85	<3.58	<3.06	<6.36
Thallium (Tl)	—	—	7.2	79	14	14	220	0.23	0.172	<0.142	<0.122	<0.230
Zinc (Zn)	<50–2,900	52	31,000	1,500	12,000	12,000	61,000	274	232	280	125	146

Note: Totals by USEPA 6000/7000 Method series; values below detection limit shown with "<" symbol; values are either numerical averages based on actual measurements, or the maximum non-detect limit for three replicates.

DESIGN RECOMMENDATIONS

- DM can be modified by adding pozzolanic admixtures, which gives the raw sediment the required strength and handling qualities to perform as well as traditional materials (Maher 2013).
- Additives, such as Portland cement (type I or II), lime, kiln dust, fly ash, coal burning residue, crushed glass, rubber and air foam, can react with sediment slurry to bind sediment particles together and effectively reduce its water content, improving the material's handling and compaction characteristics, as well as reducing the leaching potential of bound contaminants (Maher 2013).
- When selecting additives, consider the effectiveness in reduction of water content, regulatory requirements and restrictions, processing facility configuration, applicability to a wide range of sediments and chemical contaminants, availability and cost (Maher 2013).
- Quick lime can effectively solidify high water content soils; however, low availability and high cost prevents quick lime from being used widely (Samtani et al. 1994).
- Portland cement is an ideal additive because it is easily available and low in price. Cement takes more time to gain strength, allowing time for moisture conditioning and grading (Maher 2013).
- Fly ash has cementitious and pozzolanic properties, and is often used with Portland cement to improve workability, strength, and durability of DM (OCC 2010). Fly ash has the advantage of low price compared to other additives, though it may have high concentrations of heavy metals (Sadat Associates 2000).
- Lime kiln dust and cement kiln dust can be used to stabilize DM. Though these lime or cement byproducts are less expensive than lime or cement, the properties of byproducts are inconsistent, since they contain variable reactive chemicals (i.e., calcium oxide, silica, and alumina). The reactive capacity of the chemicals vary depending on fuel, kiln operations and the limestone feedstock, which makes it difficult to design a recipe for additive and sediment proportions (Maher 2013).
- Intense heat can destroy and transform the physical properties of DM to produce lightweight aggregate, glass, blended cement, etc. These products are free of contamination, and the metals remaining are not leachable. However, heat procession is expensive (in rotary kiln) is difficult to site (air pollution concerns) and has low productivity (prone to breakdowns) (Maher 2013).
- Contaminated sediment can be treated with a combination of chemical additives and separation technologies. Sediment washing by BioGenesis™ treatment technology segregates and destroys DM contaminants at either initial or final concentrations, which has unlimited capacity and productivity. Therefore, storage is required if dredging is proceeded at normal production rates and for final products as well (Maher 2013).

FIELD RECOMMENDATIONS

- Participants should draw up a Quality Assurance Project Plan and adhere to the pre-developed plan, which includes analytical methods, detection limits, frequency of testing, processing procedures, type and source of amendments, placement procedures, locations, depths, and acceptable criteria (Maher 2013).
- There are problems with DM procession because of heterogeneity or inadequate pre-dredging characterization of sediments. Therefore, frequent testing of DM and DM product and flexibility in processing rate and amendment ratios, is recommended to adjust the processing according to variability (Maher 2007).
- Volume of DM should be estimated to ensure sufficient capacity of processing facility and placement site.

- Pre-dredging project data should be reviewed to estimate the degree of in situ sediment heterogeneity and determine how heterogeneities affect processing and placement operations.
- Bench-scale tests should be used to ensure that DM placement meets all requirements and to determine type and ratio of the amendment(s) needed. The high organic matter content in DM should be considered for pozzolanic reactions. The pH and corrosive testing should be conducted on marine sediments if corrosion is a concern for the specific application (Maher 2007).
- Curing time should be recorded. Moisture conditioning and mixing performed at processing site should be noted. Unacceptable levels of water, debris or heterogeneity may require rejection/reprocessing of the DM or require a longer curing period (Maher 2007).
- DM stockpile should be checked to meet performance criteria before placement. The time of stockpiling and its purpose should be recorded. Shaping/grading or covering method to prevent moisture in DM stockpiles should be noted. For stockpile periods of more than two weeks, or in periods of much rain or snow, moisture content of DM should be retested and recorded (Maher 2007).
- Moisture content and ambient temperature greatly affect placement of DM. The amount of additive and the adequacy of mixing should be monitored carefully. Moisture content should be tested and controlled to meet criteria. Adjust and modify DM before final compaction by increasing additive used, increasing cure time in processing site and placement site, decreasing depth of each lift and increasing the time between lifts (Maher 2007).

BENEFITS

- DM has been deposited at Hart Miller Island, owned and operated by the Maryland Port Administration since 1984. On average, 1.5 million cubic yards of DM is removed each year from Baltimore harbor channels, anchorages, and berths. Until 2009, approximately 100 million cubic yards of DM has been stored (MIRC 2007). Using DM in highway applications will solve the storage, space and management problems of considerable DM (Randall et al. 2000).
- The cost of offshore disposal of DM is high. The processing costs are source dependent, involving dewatering of DM, crushing and grading cement and DM, and mixing or blending different source materials. Transportation for further processing is also costly (Sheehan et al. 2008). Though processing DM for highway applications is also costly, the products can make great profits.
- Virgin materials can be saved by using DM. Other waste materials (i.e., fly ash, cement dust, lime dust) can also be used as additives or modifiers to DM.

SUGGESTED SPECIFICATIONS

Table 3.58 Geotechnical Testing for DM Used in Non-Structural Applications (Maher 2013).

Compressive Strength	Unconfined Compressive Strength of Cohesive Soils	ASTM D2166
	Unconfined Compressive Strength Index of Chemical-Grouted Soils	ASTM D4219
Unit Weight	Unit Weight Voids in Aggregate	ASTM D29
	Standard Test Methods for Specific Gravity of Soil Solids by Water Pynometer	ASTM D854
	Standard Test Method for Density of Soil in Place by the Sand Cone Method	ASTM D1556
	Standard Test Method for Density of Soil and Soil Aggregate in Place by Nuclear Methods	ASTM D2922
Gradation	Particle Size Analysis of Soils	ASTM D422
	Sieve Analysis of Fine and Coarse Aggregate	ASTM D136
Moisture Density Characteristics	Standard Proctor Compaction for Optimum Moisture Content	ASTM D698
	Modified Proctor Compaction for Optimum Moisture Content	ASTM D1557
	Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils	ASTM D4318

Table 3.59 Geotechnical Testing for DM Used in Structural Applications (Maher 2013).

Unit Weight	Unit Weight Voids in Aggregate	ASTM D29
	Standard Test Methods for Specific Gravity of Soil Solids by Water Pynometer	ASTM D854
	Standard Test Method for Density of Soil in Place by the Sand Cone Method	ASTM D1556
	Standard Test Method for Density of Soil and Soil Aggregate in Place by Nuclear Methods	ASTM D2922
Compressive Strength	Unconfined Compressive Strength of Cohesive Soils	ASTM D2166
	Unconfined Compressive Strength Index of Chemical-Grouted Soils	ASTM D4219

Table 3.59 Geotechnical Testing for DM Used in Structural Applications, continued (Maher 2013).

Shear Strength	Standard Test Method for Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils	ASTM D2850
	Direct Shear Test of Soils Under Consolidated Drained Conditions	ASTM D3080
	Standard Test Method for Consolidated Undrained Triaxial Shear Test	ASTM D4767
Gradation	Particle Size Analysis of Soils	ASTM D422
	Sieve Analysis of Fine and Coarse Aggregate	ASTM D136
Moisture Density Characteristics	Standard Proctor Compaction for Optimum Moisture Content	ASTM D698
	Modified Proctor Compaction for Optimum Moisture Content	ASTM D1557
	Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils	ASTM D4318
Bearing Capacity	California Bearing Ratio (CBR) of Laboratory Compacted Soils	ASTM D1883
Permeability	Permeability of Hydraulic Conductivity of Saturated Porous Materials using Flexible Wall Permeater	ASTM D5084
	Permeability of Granular Soils by Constant Head	ASTM D2434
Durability	Standard Test Methods for Freezing and Thawing Compacted Soil-Cement Mixtures	ASTM D560
Consolidation	Standard Test Method for One-Dimensional Consolidation Properties of Soils	ASTM D2435

Table 3.60 Geotechnical Testing by Applications and Soil Types (Maher 2013)

	Silts and Clays	Sands
Flowable Fill (CLSM)		ASTM D421, 422, 4318, 698
Embankment Fill		ASTM D560
Roadway Subbase		ASTM D1883
Topsoil	ASTM D4318, 422, 4972, pH, Chloride Content, Organic Content	
Landfill Daily Cover	ASTM D4318, 422, 4972, 2434	ASTM D2434
Landfill Final Cover	ASTM D4318, 422, 4972, 2434	

Table 3.61 Characterization Tests for Chemical Properties of DM (Winfield and Lee 1999).

Analysis	Source
10. pH	ASA 1996 :Ch 16; CSSS: 16.2.1
11. Calcium Carbonate Equivalents	ASA 1996:Ch 16; CSSS 14.2 and 44.6
12. Cation Exchange Capacity	ASA 1996: Ch 40; CSSS 19.4
13. Salinity	ASA 1996: Ch 14; CSSS:18.2.2
14. Sodium	ASA 1996: Ch 19
15. Chloride	ASA 1996: Ch 31
16. Sodium Adsorption Ratio (SAR)	CSSS: 18.4.3
17. Electrical Conductivity	ASA 1996: Ch 14
18. Total Organic Carbon	ASTM D2974; D2974-87; ASA 1982: 29-4.2; CSSS 44.3
19. Carbon:Nitrogen Ratio	Analyses 19, 23, and 25 in this table
20. Total Kjeldahl Nitrogen	EPA-CRL-468
21. Ammonium Nitrogen	EPA-CRL-324
22. Nitrate-nitrogen	EPA-SW846-9200
23. Nitrite-nitrogen	EPA-SW846-9200
24. Total Phosphorus	EPA-CRL-435
25. Orthophosphorus	EPA-CRL-435
26 Potassium	ASA 1996: Ch 19
27. Sulfur	ASA 1996: Ch 33
28. Diethylene Triamine Pentaacetic Acid (DTPA) Metals	ASA 1982: 19-3.3; CSSS:1.3; Lee, Folsom, and Bates 1983
29. Total Metals *	EPA-SW846-200.9; ASA 1996: Ch 18-30
30. Pesticides (chlorinated)	EPA-SW846-8080
31. Polynuclear Aromatic Hydrocarbons (PAHs)	EPA- SW846-8270
32. Polychlorinated Biphenyls (PCBs) Congeners	EPA-CRL-8081
33. Dioxins	EPA-SW846-8290 and 1630
34. Leachate Quality Test	Myers and Brannon 1988
35. Surface Runoff Quality	Skogerboe et al. 1987

Notes: * Metals = arsenic, cadmium, chromium, copper, lead, mercury, silver, nickel, and zinc; Use EPA 1986 Method 245.6 for mercury determinations.

Methods:
ASA = American Society of Agronomy/Soil Science Society of America (Page, Miller, and Keeney 1982 and 1996).
CSSS = Canadian Society of Soil Science (Carter 1993).
ASTM = American Society for Testing and Materials (ASTM 1996).
EPA = USEPA (1986).

Table 3.62 Characterization Tests for Biological Properties of DM (Winfield and Lee 1999)

Analysis	Methods
36. Manufactured Soil Test	Sturgis et al. (1999)
37. Plant Bioassay	Folsom, Lee, and Preston 1981
38. Animal Bioassay	ASTM 1998, Standard Guide E 1676-97
39. Elutriate Bioassay	EPA 1991 (Method: 11.1.4) (USACE/USEPA 1991)
40. Pathogens (coliforms)	Standard Methods: 9221 E (Greensberg et al. 1992)

3.4.2 DM in Lightweight Aggregate/Bricks

MECHANICAL PROPERTIES

- <u>Properties of Brick</u>
  - Bulk density increases with increasing sintering temperature due to densification (Huang et al. 2005c). Mass density and porosity affect bulk density, since bulk density is the ratio of weight to total volume of the mass, plus open pores.
  - Water treatment residue (i.e, from the production of fresh water treatment) bricks requires a higher sintering temperature to meet the same bulk density compared to excavation waste soil (Figure 3.97), since excavation waste soil (i.e., from excavation of ground before construction) contains more Fe_2O_3, which lowers sintering temperature and increases melting of glass phase (Huang et al. 2005c).

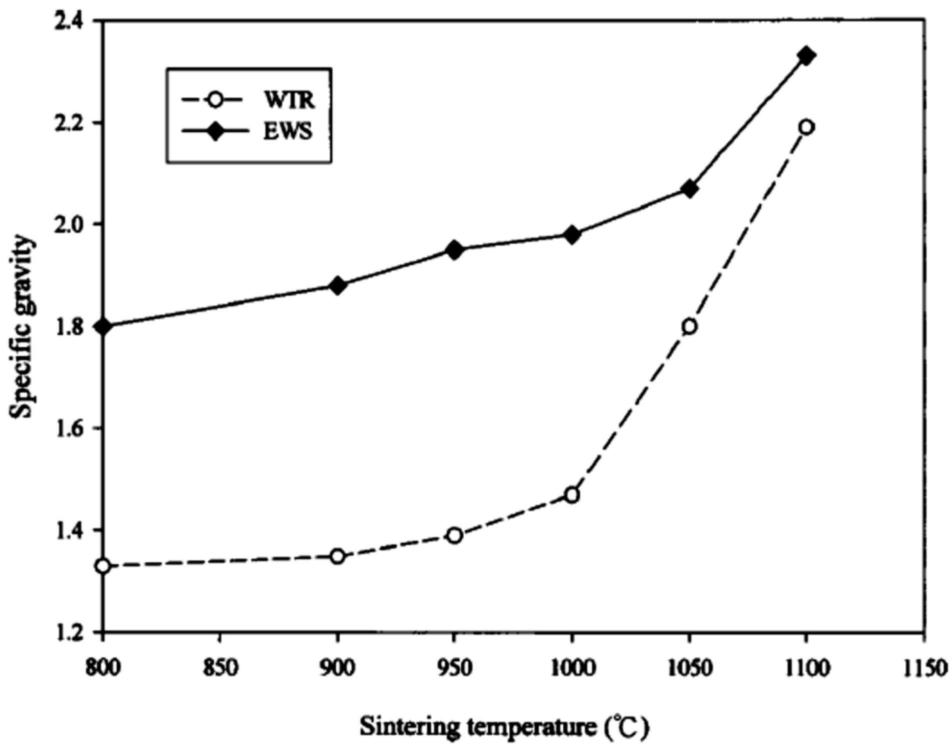


Figure 3.97 Bulk Density of Excavation Waste Soil, EWS, and Water Treatment Residual, WTR, Brick (Huang et al. 2005c)

Note: Bricks were made of 100% raw materials (EWS and WTR) without special treatment or additives.

  - For bricks made of reservoir sediment, a maximum density of 2.5 g/cm^3 is obtained without clay at a sintering temperature of 1100℃. At 1150℃, density decreases significantly with decreased clay replacement (less than 20%) due to thermal expansion of sintered specimens (Chiang et al. 2008).

- High water absorption is adverse to the durability of bricks, due to moss (i.e., a small plant) contamination and recrystallization of liquid $CaCO_3$ on brick surface (Huang et al. 2005c, Lafhaj et al. 2008).
- The water absorption of water treatment residue brick decreases with increasing sintering temperature, since sintering process closes open pores that absorb and store water (Huang et al. 2005c).
- Novosol® (developed and patented by the Solvay Company, stabilizing heavy metals by phosphatation and destructing organic matter by calcination) river sediment bricks are less porous and exhibit lower water absorption than standard brick, since quartz transformation in standard brick causes expansion that lead to micro-cracks (Samara et al. 2009).
- High porosity (48%-55%) of Novosol® river sediment brick is caused by two reactions. Calcite ($CaCO_3$) transforms to microporous calcium oxide (CaO) at temperatures around 800℃, increasing porosity (Moropoulou et al. 2001). Lime converts to portlandite ($Ca(OH)_2$), generating crystallization pressure in pores, resulting in cracks (Lafhaj et al. 2008).
- Water absorption coefficients of Novosol® river sediment bricks are all within regulatory limits (AFNOR 1983) and increase with increasing sediment addition (Table 3.63), since sediments decrease bond ability between particles and increase internal pore size of brick (Lafhaj et al. 2008).

Table 3.63 Water Absorption Coefficient of Brick Samples, by Percent (Lafhaj et al. 2008).

$F_{0\%}$	$F_{25\%}$	$F_{35\%}$	$F_{45\%}$	Regulatory limits
5.3	6.34	8.06	10.39	40

Note: $F_{0\%}$=Brick without sediment. $F_{25\%,\ 35\%,\ 45\%}$ =Brick made with 25%, 35%, 45% clay replaced by treated sediment on dry weight.

- Novosol® river sediment bricks require more sintering time than standard ones to achieve the same reduction in porosity; sintering rate is proportional to particle size and river sediment brick has larger particle size (Samara et al. 2009).
- High permeability has a negative effect on durability. High permeability facilitates water entering into pore structure and accelerates the deterioration when exposed to repeated freeze and thaw cycles (Samara et al. 2009).
- Novosol® river sediment bricks are less permeable than standard bricks, due to the present of quartz (particle size >30μm) in standard brick. Quartz transforms and expands in high temperatures, causing formation of micro-cracks. Quartz also decreases plasticity and facilitates de-flocculation (i.e., silicate makes clay particles repel each other), which further increases permeability of standard brick (Samara et al. 2009).
- Atterberg's test results indicate that plasticity index of brick mixture decreases proportionally with increasing Novosol® river sediments (Table 3.64). Brick mixture made with Novosol® river sediments is classified as low-plastic mixture, indicating lower plasticity and poorer bonding ability (Lafhaj et al. 2008).

Table 3.64 Effect of Sediment Proportion on the Plastic Nature of Brick Mixture (Lafhaj et al. 2008)

Mix-design	Liquid limit (%)	Plastic limit (%)	Plasticity index
$F_{0\%}$	39.2	20.8	18.4
$F_{25\%}$	37.8	20.75	17.05
$F_{35\%}$	36.1	23.7	12.4
$F_{45\%}$	34.25	23.7	10.55

Note: $F_{0\%}$=Brick without sediment. $F_{25\%, 35\%, 45\%}$ =Brick made with 25%, 35%, 45% clay replaced by treated sediment on dry weight.

- Water absorption of brick made of reservoir sediment (i.e., flowing sediments in river that sink to the bottom of a reservoir as the river is stilled behind a dam) decreases with increasing sintering temperature and decreased clay addition, because of lower open porosity (Figure 3.98). As temperature rises from 1000℃ to 1100℃, water absorption reduces by 80%, regardless of the clay content. However, when temperature exceeds 1100℃, water absorption is independent of clay content (Chiang et al. 2008).

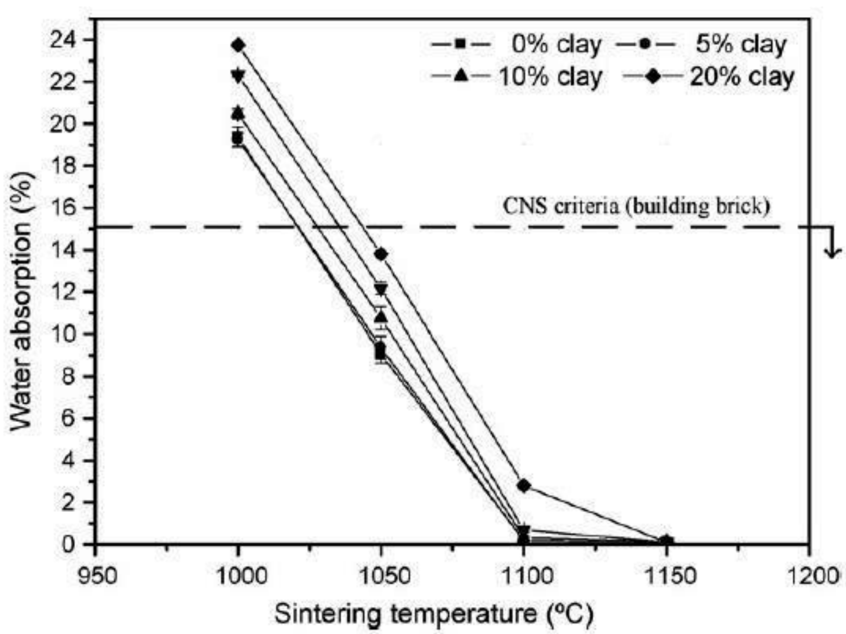


Figure 3.98 Effect of Sintering Temperature on Water Absorption of Bricks (Chiang et al. 2008)

- Compressive strength of water treatment residual brick increases with the increasing sintering temperature, especially when temperature exceeds 1000°C. When temperature is less than 900°C, there is no obvious growth in compressive strength (Huang et al. 2005c).

- Although the compressive strength of water treatment residual brick increases with increasing sintering time, the difference between three and six hours is so slight that three hours of sintering time is enough to achieve desirable strength (Huang et al. 2005c).
- Maximum compressive strength of bricks made of reservoir sediment occurs at 1100 °C sintering temperature with no clay replacement (Figure 3.99). Compressive strength decreases with increasing temperature from 1100°C to 1150°C with less than 20% clay, due to swelling of sintered specimens (Chiang et al. 2008).
- Average compressive strength of Novosol® river sediment bricks (36 MPa) is 63% higher than that of a standard brick (22 MPa), since river sediment is finer than coarse quartz sand, resulting in denser microstructure of brick. In addition, porosity of sediment-amended brick is lower than that of a standard one (Samara et al. 2009).
- Though quartz with a particle size of 10-30μm improves strength, large-size quartz particles weaken it. This fact is associated with volumetric changes as a result of quartz transformation at high temperature, which causes micro-cracks and tensile stress buildup, resulting in separation of quartz grains (Samara et al. 2009).
- Compressive strength decreases with increasing Novosol® river sediment content (Figure 3.100), since the addition of treated sediments increase internal pore size, making brick more porous (Lafhaj et al. 2008).

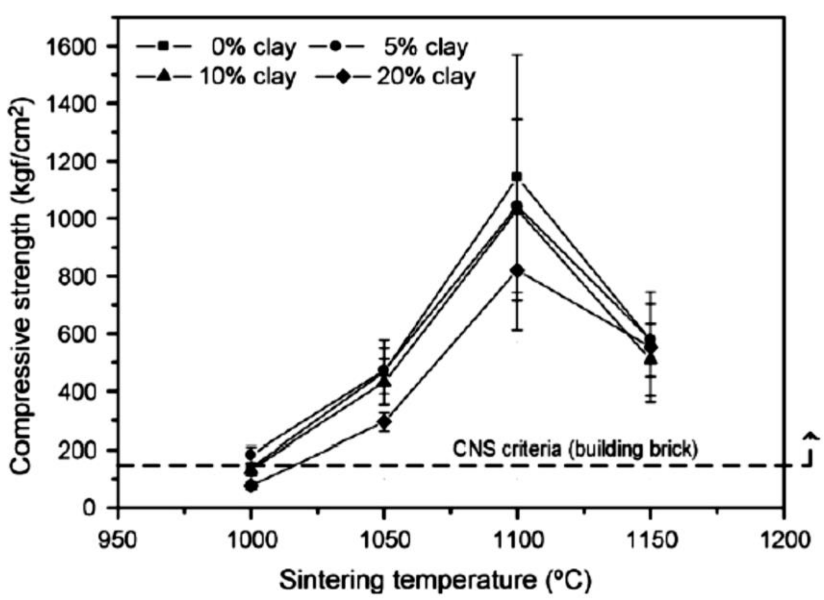


Figure 3.99 Effect of Sintering Temperature on Compressive Strength of Bricks (Chiang et al. 2008)

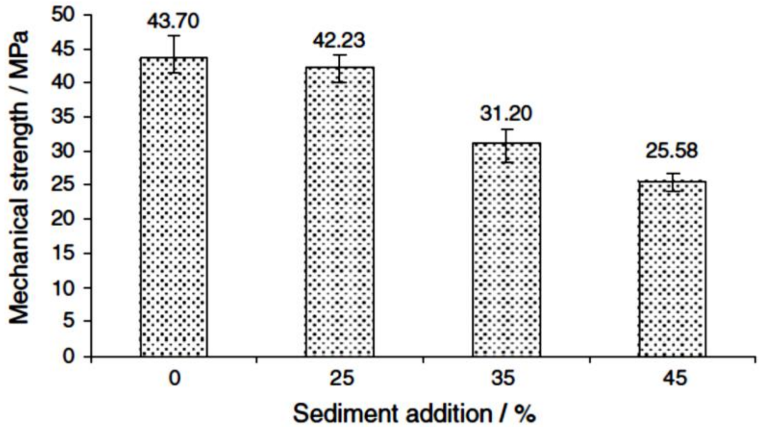


Figure 3.100 Effect of Sediment Proportion on Compressive Strength (Lafhaj et al. 2008).

- Excessive shrinkage can cause distortion and breakage of bricks. Significant shrinkage of water treatment residue brick begins to occur at 950℃ (Figure 3.101). Until 1100℃, volume is reduced by 45% due to firing shrinkage, much higher than the volume reduced by LOI (loss of ignition). This is due to the development of a new crystal (Huang et al. 2005c).
- Firing shrinkage (i.e., shrinkage from dry to fired, ASTM C326-09) of Novosol® river sediment bricks is higher (10%) than that of standard one (7%), since quartz in standard brick enhances the expansion coefficient, thus reducing linear shrinkage. However, Novosol® river sediment bricks require more sintering time, fineness and additional water (2% more) to achieve the desired plasticity, leading to higher shrinkage (Samara et al. 2009).
- For bricks made of reservoir sediment, shrinkage rate increases significantly with increasing sintering temperature. A maximum of 32% shrinkage occurs at sintering temperature of 1150℃ and 20% clay replacement. Although expansion exists in the meantime, good densification and high shrinkage is maintained throughout (Chiang et al. 2008).
- Pore size and distribution affects durability of bricks; in freezing state various pressures develop within the pore system because of water and in thawing state water further enter into the pores. Continuous cycles of freezing and thawing can eventually cause significant expansion and deterioration, such as cracking, spalling, or surface scaling (Lafhaj et al. 2008).
- Percentage of weight loss in Novosol® river sediment brick under freeze-thaw cycles is independent of sediment content (Table 3.65). Weight losses for all substitution ratios are less than 1%, the upper limit loss allowed by the French standard (AFNOR 1983). Neither cracking nor breakage occurs in bricks, indicating qualified freeze-thaw resistance (Lafhaj et al. 2008).
- A frost-resistance test revealed some micro-cracks within raw harbor sediment brick (Hamer and Karius 2002). Less micro-cracks can improve frost resistance as well as compressive strength (Hamer and Karius 2002). Micro-cracks are caused by organic substance and grain-size distribution, which can be compensated by optimizing burning temperature in the kiln (Okuno and Takahashi 1997).

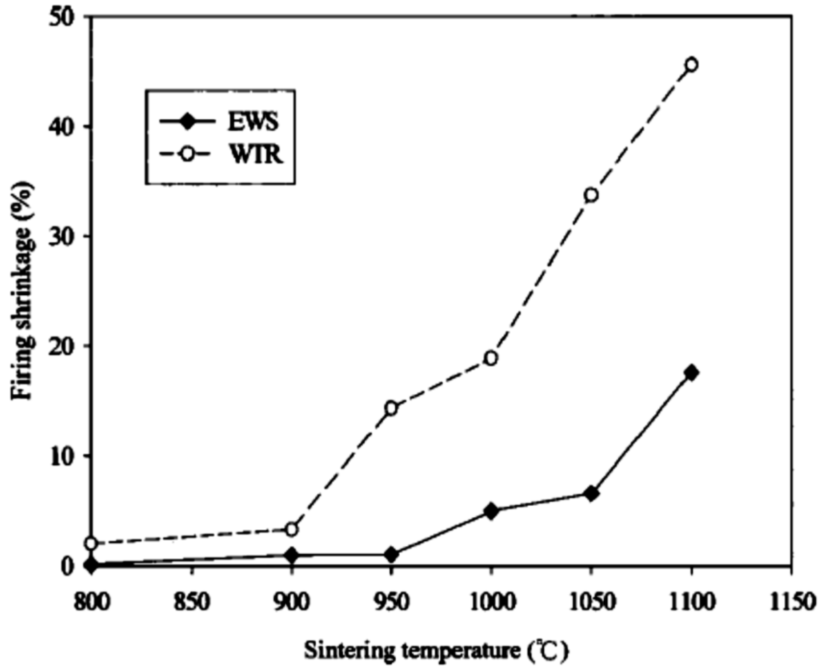


Figure 3.101 Firing Shrinkage of Excavation Waste Soil (EWS) and Water Treatment Residual (WTR) Brick (Huang et al. 2005c)

Note: Bricks were made of 100% raw materials (EWS and WTR) without special treatment or additives.

Table 3.65 Weight Loss After 25 Cycles of Freezing and Thawing (Lafhaj et al. 2008).

Mix-design	$F_{0\%}$	$F_{25\%}$	$F_{35\%}$	$F_{45\%}$
Average weight loss (%)	0.14	0.17	0.36	0.19

Note: $F_{0\%}$=Brick without sediment. $F_{25\%, 35\%, 45\%}$ =Brick made with 25%, 35%, 45% clay replaced by treated sediment on dry weight.

- Properties of LWA
  - Specific gravity of artificial aggregates made from water treatment residue ranges from 1.12 to 1.78 (Table 3.66), meeting the criteria (AFNOR 1983) for LWAs. Specific gravity increases with increasing sintering temperature (Huang et al. 2005c).

Table 3.66 Properties of LWA Made from Water Treatment Residual (Huang et al. 2005c)

Properties	Sintering temperature		
	1,000 °C	1,050 °C	1,100 °C
Specific gravity	1.12	1.71	1.78
Water absorption (%)	37	15.48	14.47

- Water absorption affects water availability during concrete mixing and the hardening process. A 37% water absorption occurs in sintered temperature of 1,000℃ (Table 2-65). Water absorption changes little when temperature exceeds 1050℃ (Huang et al. 2005c).
- Thermogravimetric analysis indicates that weight loss on ignition increases with increasing temperature. When temperature increases from 50℃ to 750℃, the material weight loses up to 7% due to evaporation of physically adsorbed water and crystal water in mineral. When temperature exceeds 750℃, a lower weight loss occurs (Tang et al. 2010).
- Density of manufactured aggregates using reservoir sediments ranges from 1010 to 1380 kg/m^3 (Table 3.67), significantly lower than that of natural aggregates. Water absorption at 30minutes increases with increasing bulk density, while water absorption at 24 hours slightly decreases with increasing bulk density (Tang et al. 2010).

Table 3.67 Properties of LWA Made from Reservoir Sediment (Tang et al. 2010)

Type of LWA	Dry loose bulk density (kg/m^3)	Particle density (kg/m^3)	Water absorption (%)		Crushing strength (MPa)
			30-min	24-h	
SA-600[a]	622	1010	5.5	12.3	7.2
SA-700[b]	713	1160	6.3	11.1	10.0
SA-800[c]	859	1380	6.6	10.4	13.4
CA-800[d]	844	1410	7.1	11.5	7.5

Notes: a, b, c: LWA made from reservoir sediment with different aggregate size and particle density;
d: commercially available LWA.

- Initial slump varies between 130 and 230 mm, indicating concrete made with reservoir sediment LWA as coarse aggregate possesses good workability (Table 3.68; Tang et al. 2010).
- Plastic lightweight concretes have lower densities than plastic normal density concrete. The densities of plastic lightweight concretes range from 1659 to 1745 kg/m^3 (Table 3.68), due to varied air content, water content, and LWA particle density (Tang et al. 2010).

Table 3.68 Fresh Properties of Concrete made with Reservoir Sediment LWA (Tang et al. 2010)

Mix no.	Initial slump (mm)	Unit weight (kg/m^3)
L600-40	130	1685
L600-55	210	1676
L600-75	180	1659
L800-40	200	1745
L800-55	230	1724
L800-75	230	1718

Note: 600/800 means average density of aggregates in lightweight concrete is about 600 or 800 lb/ft^3. 40/55/75 indicates w/c ratio of 0.4, 0.55, and 0.75, respectively.

- After a 28-day curing, density of concrete made with reservoir sediment LWA changed less than 0.5%, approximately 29%-35% lighter compared to normal density concrete (Table 3.69; Tang et al. 2010).
- Higher aggregate density and lower W/C ratio contribute to higher compressive strength (Table 3.69). 28-day compressive strength of the lightweight concrete ranges from 19.8MPa to 34.7 MPa, satisfying the strength requirement of 17 MPa, according to ASTM C 330 and ACI 318 (Tang et al. 2010).
- 28-day flexural strength ranges from 5.3MPa to 7.2 MPa, increasing with higher aggregate density and lower W/C ratio, Table 3.69 (Tang et al. 2010).
- Crushing strength (i.e., the maximum compressive load a material can withstand without fracturing, GB/T2842-81) of LWA increases with increasing bulk density (Table 3.69). LWA made from reservoir sediment (i.e., SA-800) shows better crushing strength than commercially available LWA (i.e., CA-800) and can serve as a structural aggregate (Tang et al. 2010).
- Electrical resistivity decreases with increasing W/C ratio, and increases with higher density (Table 3.69; Tang et al. 2010).

Table 3.69 Hardened Properties of Concrete made with Reservoir Sediment LWA (Tang et al. 2010).

Mix no.	Compressive strength (MPa)		Flexural strength (MPa)	Density (kg/m^3)	Electrical resistivity (kΩ cm)
	7-day	28-day			
L600-40	27.6	32.0	6.1	1550	10.1
L600-55	21.8	26.2	5.9	1515	9.1
L600-75	15.8	19.8	5.1	1490	7.6
L800-40	30.7	34.7	7.2	1566	10.6
L800-55	22.8	30.4	6.5	1544	10.1
L800-75	16.3	21.3	5.3	1492	7.6

Note: Concrete mixes cured at a relative humidity of 50 ± 5% and a temperature of 23 ± 2℃. L600/800-40/55/75 indicates lightweight concrete made with size 600/800 aggregate and w/c ratio of 0.4, 0.55, and 0.75, respectively.

ENVIRONMENTAL PROPERTIES

- Thermal treatment (1050℃) of contaminated sediments can destroy organic contaminants and transform remaining heavy metals into new minerals (Hamer and Karius 2002, Karius and Hamer 2001). However, Cr, V, As and Mo becomes even more mobile after thermal treatment (Karius and Hamer 2001).
- Leaching of bricks made of 50% (by weight) harbor sediments (Bremen, Germany) exhibited high concentrations (i.e., Zn, Cd, Pb and tributyltin) at acidic condition but low concentrations at neutral and alkaline condition. Small-size grains have higher concentrations due to large specific surface areas. Leachability of heavy metals from sediment brick is generally higher compared to commercial bricks (Karius and Hamer 2001).
- Grain sizes below 63 μm shows decreased leachability of V, Cr, Ni, As, Sr, Mo and Pb, due to absorption of sample material or precipitation (Karius and Hamer 2001).
- Leachate of Novosol® river sediment has a high pH value of 8.9, due to transformation of calcite ($CaCO_3$) into lime (CaO) during the sintering process. Concentrations of Cd, Cu, Ni, Pb, Zn from sediment-amended brick are below the regulatory limits (Samara et al. 2009).
- Quantities of metals leached out of bricks are less than those of Novosol® treated river sediment, since metals are either stabilized in glassy melt phase or transformed to low-solubility metal oxides during the sintering process. Sediment-amended brick can be considered as non-hazardous material, (Table 3.70; Samara et al. 2009).
- Leaching with acidic solution (at a pH of 4.92) revealed that metal concentrations from Novosol® river sediment brick are higher than those obtained by the French procedure regulated in AFNOR, 1998 (at a pH of 8.9), but still far below TCLP limits (Table 3.71; Samara et al. 2009, Lafhaj et al. 2008).
- A TCLP test undertaken on brick made with a different percentage of treated sediment indicated that metal concentrations increase with an increasing treated sediment content, but all mix-design are far below TCLP limits (Lafhaj et al. 2008).
- TCLP leachate concentrations from sintered specimens are less than those from reservoir sediment. TCLP leachate concentrations for the tested metals in all sintered specimens were far less than thresholds of Taiwan EPA regulatory (Table 3.72; Chiang et al. 2008).

Table 3.70 Leaching Results in Acetic Acid (Samara et al. 2009)

Element	Sediment-amended brick pH 4.92	BdN-standard brick pH 4.97	Regulated TCLP limit
Cd	<0.04	<0.04	1.00
Cu	0.1	0.2	15
Zn	3.7	3.3	25.00
Ni	<0.14	0.67	–
Pb	<0.4	<0.4	5.00

Note: Sediment-amended brick made with 15% clay replacement with treated sediment on dry weight.

RECOMMENDATIONS

- Injecting $Ca(OH)_2$ into flue gas stream is recommended to reduce SO_2 concentrations in exhaust gas stream during the brick manufacturing process (Hamer and Karius 2002).
- Adding $BaCO_3$ to raw sediment material can prevent bricks from possible efflorescence (Hamer and Karius 2002).

Table 3.71 Leachate of Brick in Acetic Acid (Lafhaj et al. 2008)

Element	$F_{0\%}$	$F_{25\%}$	$F_{35\%}$	$F_{45\%}$	Regulated TCLP limit
Cd	<0.02	0.08	0.1	0.16	1.00
Cu	0.1	0.52	0.76	1.2	15
Zn	1.63	3.06	3.28	4.92	25.00
Ni	0.33	0.34	0.56	0.92	–
Pb	<0.2	<0.5	<0.5	<0.5	5.00

Note: $F_{0\%}$=Brick without sediment. $F_{25\%, 35\%, 45\%}$ =Brick made with 25%, 35%, 45% clay replaced by treated sediment on dry weight.

Table 3.72 TCLP Metal Leachate Concentrations of Reservoir Sediment Brick, mg/l (Chiang et al. 2008)

	Sintering temperature 1100 °C				Sintering temperature 1150 °C			
	0% clay	5% clay	10% clay	20% clay	0% clay	5% clay	10% clay	20% clay
Pb	<0.009	0.02	<0.009	<0.009	<0.009	<0.009	<0.009	0.02
Cd	0.02	<0.008	<0.008	<0.008	0.03	<0.008	<0.008	<0.008
Cr	<0.006	<0.006	<0.006	<0.006	<0.006	<0.006	<0.006	<0.006
Cu	<0.005	<0.005	<0.005	0.01	<0.005	<0.005	<0.005	<0.005
Zn	0.86	0.44	0.13	0.18	0.98	0.67	0.17	0.11

Note: TCLP regulatory of Taiwan thresholds: Pb: 5mg/l; Cd: 1mg/l; Cr: 5 mg/l; Zn: 25 mg/l.

BENEFITS

- Producing bricks with harbor sediments can prevent overuse of natural clay resources and save sparse resources (Hamer and Karius 2002). Dredged material is not inevitability a waste, but can have added value in beneficial use (IADC 2009).
- Space slated for new landfills equipped with dewatering facilities and compensation areas can be preserved, especially for some cities with a limited landscape area for development (Hamer and Karius 2002). The challenge for disposal and storage of dredged material can also be eliminated (IADC 2009).
- Utilizing fine sediments to make LWA not only provides technical benefits, but also promotes increased use and applications of LWA in the construction industry (Tang et al. 2010).
- HarborRock® (use dredged material in high temperature kiln) LWA is believed to be the lightest LWAs with

a density of 37 tons/ft^3 (Francingues et al. 2011).

- Unit price of HarborRock® LWA is about \$57/ton, less than the medium cost of commercial LWA at \$67.5/ton (Francingues et al. 2011).

SUGGESTED SPECIFICATION

Table 3.73 Specification for LWA (Francingues et al. 2011).

Specification	Supplement
ASTM 330	Standard specification for LWA for structural concrete.
ASTM 331	Standard specification for LWA for masonry units.

3.4.3 DM in PCC/Cement

MECHANICAL PROPERTIES

- Fresh Concrete Properties
  - DM acts as either fine aggregate replacement or filler in PCC applications (Millrath 2003). DM can also serve as cement replacement in mortars or pastes (Aoual-Benslafa et al. 2015).
  - As a fine aggregate in PCC, DM dramatically reduces workability and requires additional water to meet target workability (Oh et al. 2011, Millrath 2003). For example, as DM content increases from 0% to 20%, flow is reduced from 72 mm to 32 mm at constant w/c ratio of 0.7. Alternatively, to maintain a constant flow of 47 mm, w/c ratio has to be increased from 0.45 to 0.88 (Millrath et al. 2001).
  - Superplasticizers helps to improve workability, since the water film around the particles experiences lower adhesive forces and change in surface charge. The addition of a superplasticizer can prevent particle agglomeration and swelling of concrete caused by clayey content in DM, facilitating homogenous distribution of particles (Millrath 2003).
  - When DM acts as fine aggregate in PCC, superplasticizer cannot reduce w/c ratio to an acceptable level at comparable flow. When DM acts as filler in PCC, w/c ratio can be reduced while maintaining comparable flow (Millrath 2003).
  - DM as filler (with more fines) in PCC significantly reduces flow, while concrete with treated DM filler has significantly less flow reduction. Flow reduction is caused by fines in DM, which have high water absorption capacity due to their large specific surfaces, expanding volume of material and increasing internal cohesion (Oh et al. 2011, Millrath 2003). Furthermore, fines interact with the superplasticizer, in terms of surface charges, causing agglomeration (Millrath 2003).
  - Density of concrete decreases significantly with increasing DM replacement of natural fine aggregates, since DM is lighter than other mixture components. Decreased density brings challenges in concrete consolidation (Millrath 2003).
  - Density of concrete increases slightly with either untreated or treated DM filler, since adequate fines fill voids between particles. However, consolidation of fresh concrete is still difficult due to lower workability (Millrath 2003).
  - Concrete containing untreated DM is slower to set and hydrate than concrete without DM, since organic matters and heavy metals (i.e., lead and zinc) retard or delay setting (Rossetti and Medici 1995). In addition, clay minerals have high adsorption capacity which further delay setting (Millrath 2003).
  - Treated DM has a much lower adsorption capacity than its untreated counterparts (Changling et al. 1995). Therefore, setting time is barely affected if treated DM is added to concrete (Millrath 2003).

- Hardened Concrete Properties
  - As w/c ratio increases, compressive strength keeps almost constant for concrete with DM replacement less than 15% (by mass of fine aggregate), whereas the strength of concrete with 20% DM replacement increases considerably (Millrath et al. 2001).

- At fixed w/c ratio, as DM replacement increases from 0% to 20% (by mass of fine aggregate), compressive strength is barely affected (Millrath et al. 2001).
- However, another study indicated that compressive strength increases with DM replacement/filler content up to 10% (Figure 3.102) and then decreases with additional DM replacement/filler content (Oh et al. 2011).
- Concrete with CUT treated DM (a treatment to physicochemical reorganize micro-structure) used as filler exhibited higher compressive strength than that of untreated DM (Millrath 2003).
- The addition of superplasticizer increases 7-day compressive strength at a lower w/c ratio, but the 28-day strength is barely affected. Superplasticizer acting as a surfactant and deflocculant increases the hardening of concrete at early age; however, long-term hardening is determined by releasing initially absorbed water, independent of superplasticizer (Millrath 2003).

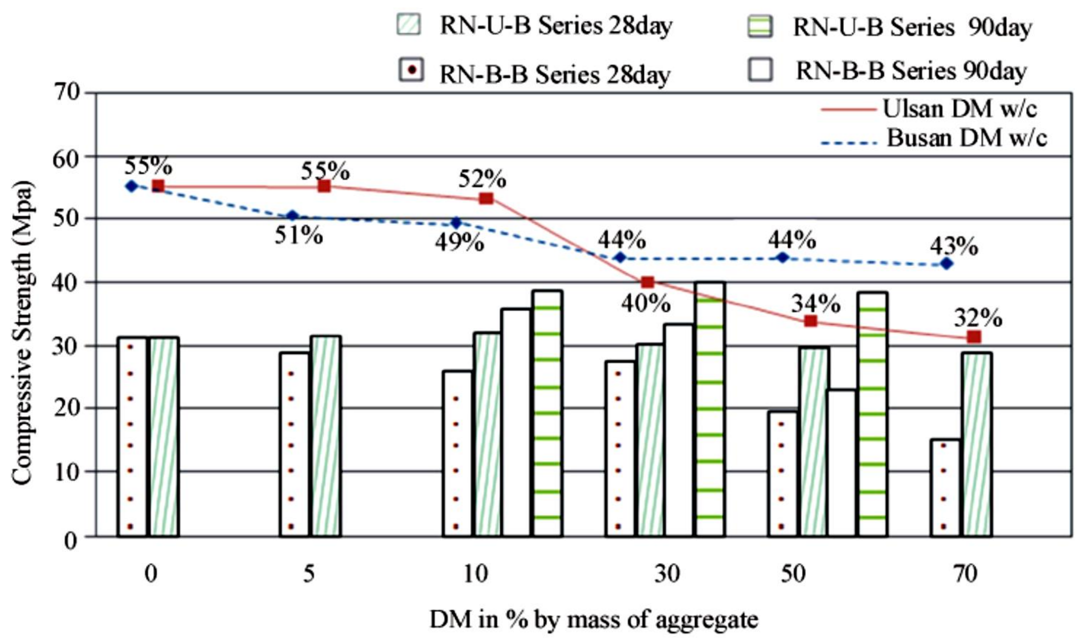


Figure 3.102 Compressive Strength Test Results (Oh et al. 2011)

Note: RN=replacement of aggregate with natural untreated DM; U or B=two origins of DM; B=test specimens.

- The addition of an air-entraining, water-reducing agent or naphthalene, high-performance water-reducing agent reduces the w/c ratio by more than 10% and improves compressive strength over 90 days (Millrath et al. 2001).
- Specimen size affects the compressive strength measured in test. A small specimen leads to underestimation of compressive strength, since large specimens have higher degree of homogeneity than smaller ones (Kumar and Monteiro 1993, Neville 1997).
- Tensile strength of concrete increases with the addition of clay minerals, and therefore increases with the addition of DM (Millrath 2003).

- A small amount (0.5%-1.0%) of salt or chloride content in DM acts as a mild accelerator, hastening heat evolution and strength gain of concrete at early ages (Limeira et al. 2012).
- Toughness increases with the addition of clay content into concrete (Millrath 2003). Clay content reduces volume of pores and facilitates homogeneity of the micro-structure, reducing the degree of anisotropy and improving ductility (Moukwa 1993).

◆ Durability

- DM is potentially corrosive to concrete due to its high pH, as well as its chloride and sulfate contents. Sulfate in excess of 0.3% and chloride in excess of 0.5% is considered severely or extremely corrosive (Oweis 1998). New York/New Jersey Harbor sediments have been tested with a sulfates content at 0.15% to 4.1% and a chlorides content at 0.36% to 5.7% (Maher 2013).
- Chloride concentrations slightly decrease with increasing DM content (Table 3.74), but are below the water soluble chloride limit in Portland cements to be used in reinforced concrete (0.15%), as well as the limit for pre-stressed concrete (0.06%). Therefore, DM will not increase chloride content of the final product, although it remains a practical manufacturing consideration (Dalton et al. 2004).

Table 3.74 Free Chloride Content Measured in Bench Scale Clinker Samples, Percent by Mass (Dalton et al. 2004)

Clinker sample	Free Cl^-
Control	0.018
Low DM	0.016
Medium DM	0.014
High DM	0.013

- Inclusion of chlorides can accelerate heat evolution (about 2-3 times) during early hydration and thermal movement in a structure can be increased consequently, especially in hot weather (Limeira et al. 2012).
- Clay minerals contained in DM increase absorption of water, which lead to porosity of concrete structure and poor durability and swelling (i.e., structural damage or even pop-outs) (Neville 1997).
- Organic contaminants in DM can affect the durability of concrete positively or negatively (Millrath 2003).
- Concrete without any filler exhibits higher expansion than concrete containing untreated or treated DM filler (Millrath 2003).
- Corrosion induced by microbes is not a concern for DM, due to high leaching pH and pozzolanic reaction, which consumes organic matter (Maher 2013).

◆ Properties of Cement

- In order to obtain normal consistency on pastes, w/c ratio has to be raised with increasing replacement of cement by DM. Higher DM replacement of cement requires higher w/c ratio—that is—a higher water content (Limeira et al. 2012).

- Increasing DM replacement of cement decreases fluidity of the paste, resulting in a prolonged flow time, Figure 3.103 (Limeira et al. 2012).
- Adding plasticizer to paste lowers flow time, indicating better fluidity. An amount of 2% plasticizer content is necessary for paste with 50% DM replacement to achieve a similar flow time to paste without DM (Limeira et al. 2012).

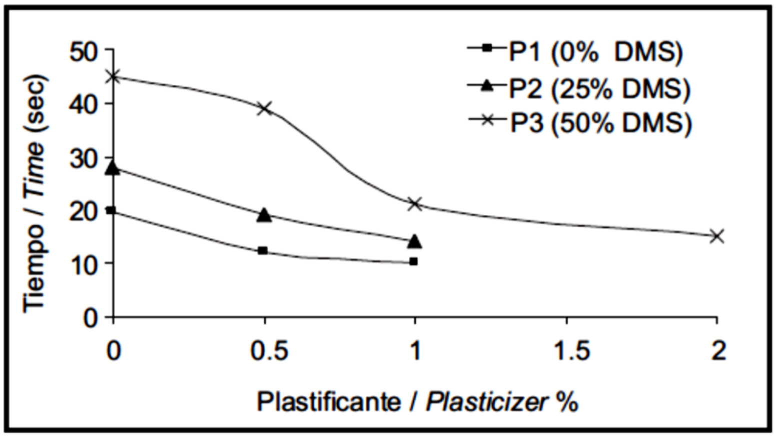


Figure 3.103 Flow Time in Pastes with w/c=0.5 (Limeira et al. 2012)

Note: Pastes P1, P2 and P3 include 0%, 25% and 50% of DM as partial substitution of raw sand 0~2 mm.

- Replacing natural sand with DM improves compressive strength of mortars, since, compared to natural sand, the finer grade of DM helps to modify granular skeleton (Limeira et al. 2012).
- During a 90-day curing, compressive strength of mortars decreases with an increasing phosphate treated DM replacement of cement, Figure 3.104 (Aoual-Benslafa et al. 2015).

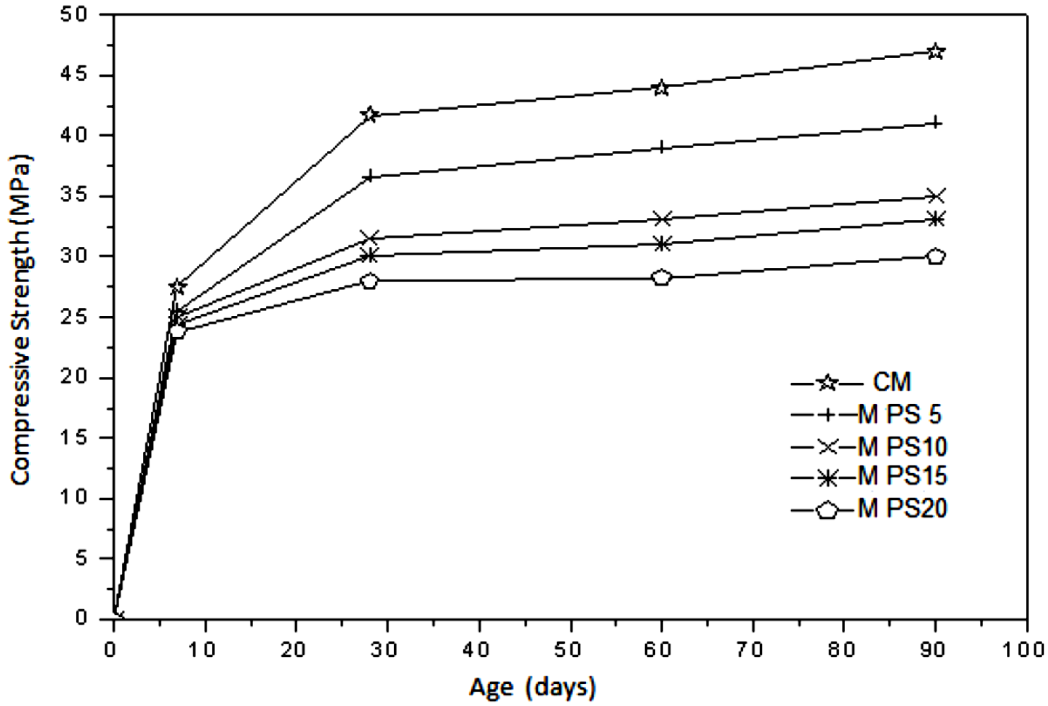


Figure 3.104 Compressive Strength for Mortars (Aoual-Benslafa et al. 2015)

Note: CM=control mortar without DM; MPS5, MPS10, MPS15, MPS20= mortar with 5%, 10%, 15% and 20% cement replaced with phosphate treated DM, respectively.

- Mortars with less than 25% DM replacement have a higher 28-day compressive strength compared to mortars without DM (Table 3.75); however, mixes with 25% DM replacement have comparable or less compressive strength compared to mortars without DM. Compressive strength decreases slightly when substitution reaches 25%, indicating that 25% could be the maximum content for DM (Limeira et al. 2012).

Table 3.75 Compressive Strength on Mortars (Limeira et al. 2012)

Resistencia a flexo-tracción en morteros (MPa) / *Flexural strength on mortars (MPa)*					
	DMS*	REF (0%)	DMS (5%)	DMS (15%)	DMS (25%)
7 días / *days*	DMS-A	29.27	34.35	34.31	29.98
	DMS-B	29.27	33.60	32.18	32.33
	DMS-C	29.27	31.62	33.43	32.50
28 días / *days*	DMS-A	37.67	38.11	39.31	35.84
	DMS-B	37.67	41.26	41.62	43.95
	DMS-C	37.67	38.93	40.63	39.45

- However, Agostini et al. (2007) reported that an addition of 33% of treated DM to mortars increases compressive strength by 20%, compared to mortars without DM.
- 28-day flexural strength increases slightly with increasing DM replacement to 15% (Table 3.76). Flexural strength of mortars with 15% DM replacement is 18% higher than that of mortars without DM (Limeira et al. 2012).
- Weight loss is greater for mortar immersed in HCL than in H_2SO_4 solution (Figure 3.105). Weight loss increases with an increasing percentage of DM (Aoual-Benslafa et al. 2015).

Table 3.76 Flexural Strength on Mortars (Limeira et al. 2012)

Resistencia a flexo-tracción en morteros (MPa) / *Flexural strength on mortars (MPa)*					
	DMS*	REF (0%)	DMS (5%)	DMS (15%)	DMS (25%)
7 días / *days*	DMS-A	6.39	7.10	6.28	5.70
	DMS-B	6.39	6.92	6.8	6.49
	DMS-C	6.39	6.43	6.7	6.46
28 días / *days*	DMS-A	6.63	6.52	6.56	6.16
	DMS-B	6.63	6.91	7.81	6.78
	DMS-C	6.63	6.64	7.83	7.66

Note: DMS-A, DMS-B and DMS-C are dredged sediments from four different places without any treatment, washing or drying. DMS-A, DMS-B and DMS-C were used on mortar production (partial substitution of raw sand 0~5 mm).

ENVIRONMENTAL PROPERTIES

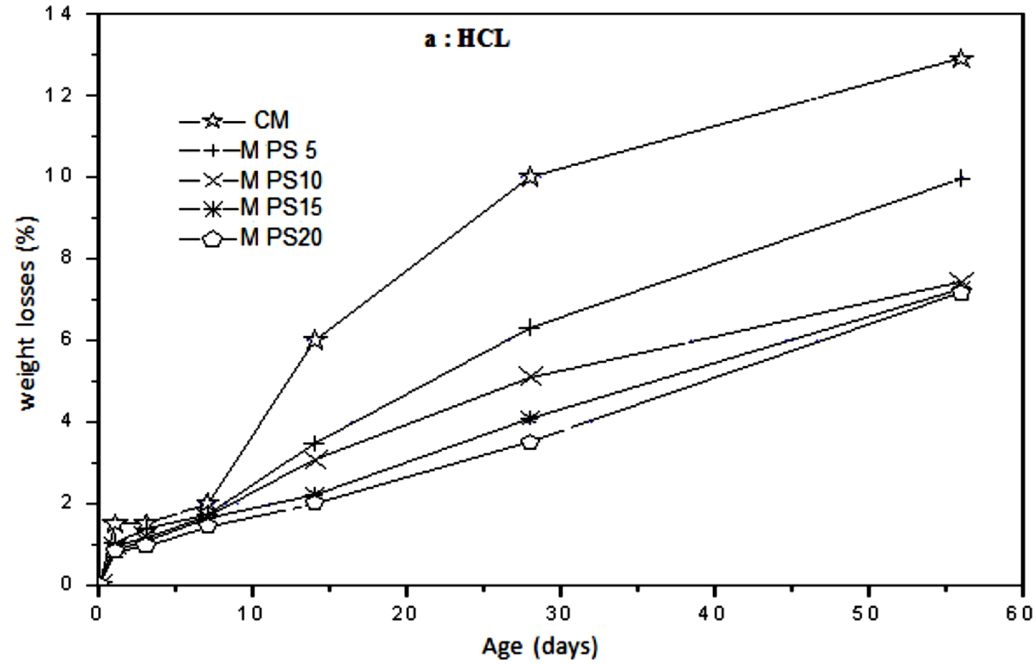
- DM contains heavy metals (e.g., lead and mercury), organics (i.e., pesticides and polychlorinated biphenyls), and E-Coli bacteria (Millrath et al. 2001).
- A TCLP test for New York/ New Jersey harbor DM revealed that metal concentrations from untreated sediments were below U.S. limits for classification as hazardous materials. Treatment such as phosphate addition, thermal processing and a combination of the two, can reduce leachate up to 89% (Figure 3.106; Ndiba and Axe 2009).
- Quantity of metals concentration is under the limit of the first level of action (Table 3.77). The level one to three is a set of concentration limits for toxic substances given by the Center for Studies and Experimentation of Public Work (CEDEX 1994) in Spain. The first level of action has the least allowance of metals and organics concentrations. Polychlorinated biphenyls (PCB's) value was less than 0.1 in all samples (dry sediment < 63 μm expressed by ng/g), lower than the limit of the first level of action (Limeira et al. 2012).

RECOMMENDATIONS

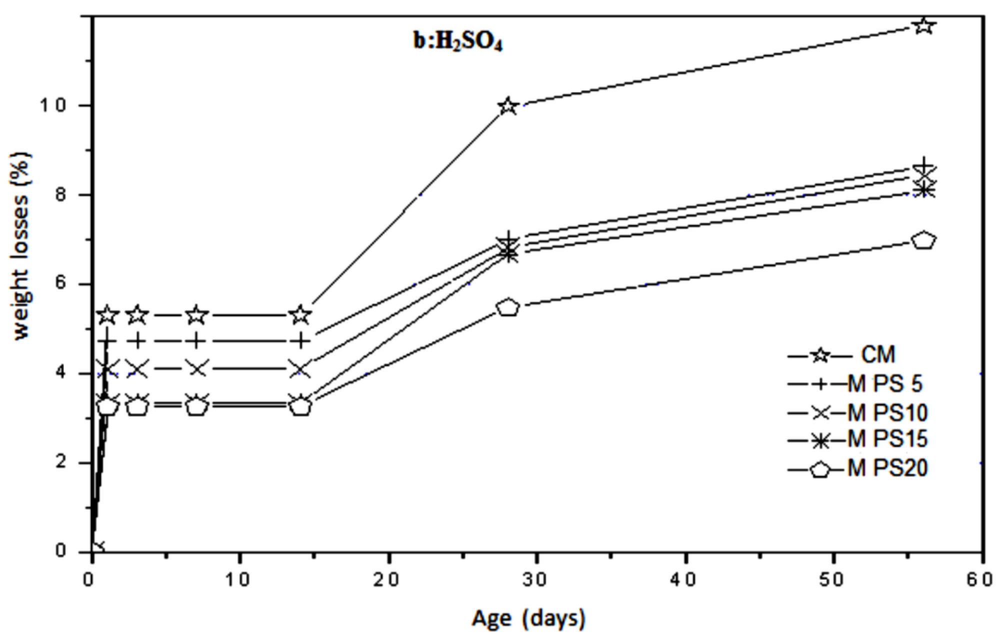
- DM is comprised of clays, silts, sand mingled with rocks, debris of variable sizes, and organic matter. Geology, mineralogy, morphology and composition of DM are associated with geographic location; therefore, properties of DM vary greatly and should be treated separately (Millrath 2003).
- Corrosion protection measures should be adopted where DM is added into cement or concrete, such as installation of a protective coating on steel or concrete, and the use of low-permeability or sulfate-resistant concrete (Maher 2013).
- Kiln operational conditions may have to be adjusted according to quartz content of DM, since a larger size of quartz crystals require higher maximum temperature or longer retention time to react. Increasing DM content means more quartz content, which hinders reaction between lime crystals and belite, resulting in lower alite contents, which determines effectiveness of cement (Dalton et al. 2004).

BENEFITS

- Every year, a large quantity of DM must be removed from harbor channels, anchorages and berths to be deposited and backfilled. Exploring a sustainable and economic way to reuse it should be a priority (MIRC 2007).
- Considerable space has been consumed by disposal and placement of DM. Consequently, environmental concerns such as the loss of open water and excessive sedimentation have become more and more important. Recycling DM can be both economically and environmentally friendly (MIRC 2007).



(a) Immersion in HCL



(b) Immersion in H_2SO_4

Figure 3.105 Weight Loss of Mortars According to Time of Immersion (Aoual-Benslafa et al. 2015)

Note: CM=control mortar without DM; MPS5, MPS10, MPS15, MPS20=mortar with 5%, 10%, 15%, 20% cement replaced with phosphate treated DM, respectively.

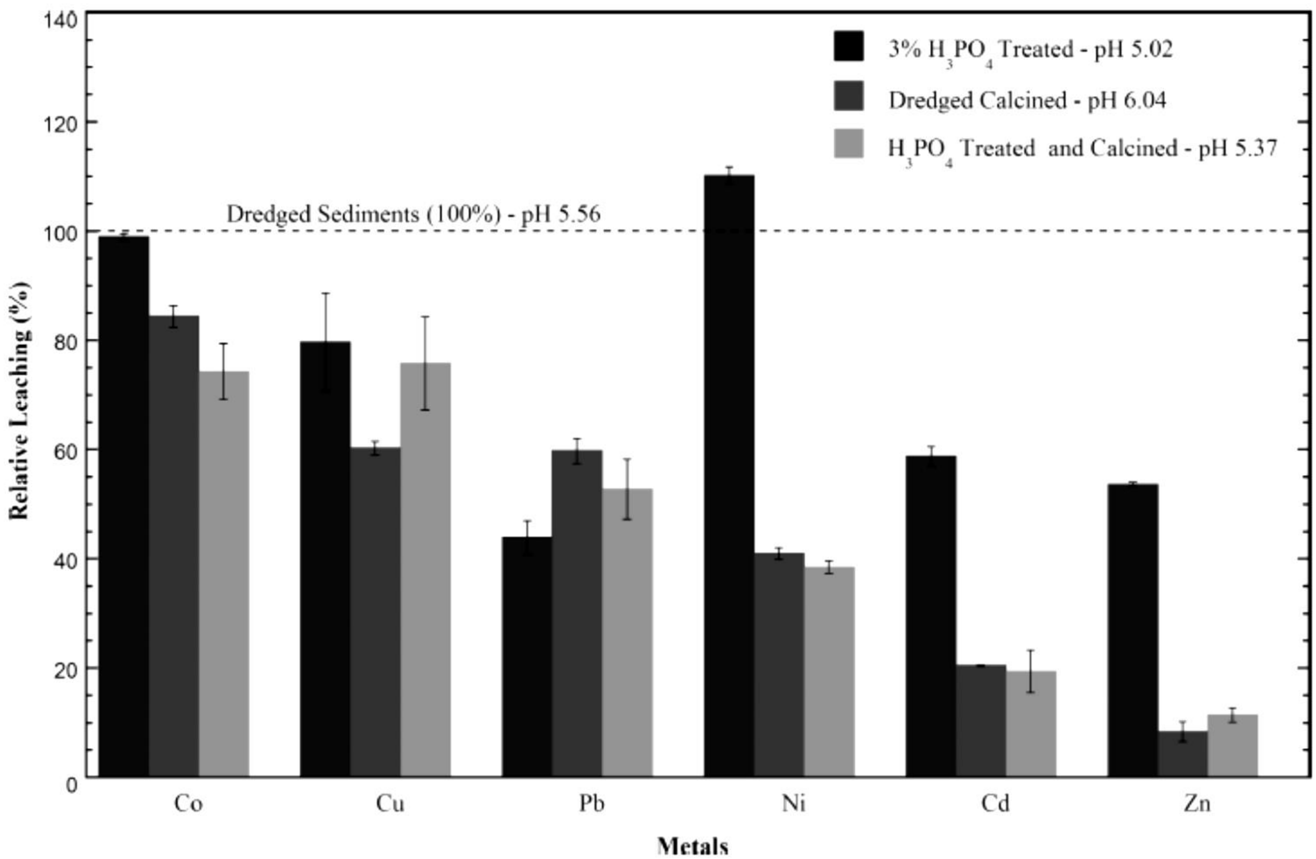


Figure 3.106 TCLP Leaching of Metals for Sediment Treatments Relative to Amount Leached from Raw Sediments (Ndiba and Axe 2009)

Note: Percentage leaching from calcined sediments is adjusted for loss of organic matter. Error bars indicate 2* standard error based on triplicate samples.

Table 3.77 Heavy Metals (μg/g), Total Organic Matter (OM) and Carbonates (Limeira et al. 2012)

Muestras / *Samples*	Cd	Cu	Zn	Cr	Ni	As	Hg	Pb	OM (%)	Carbonatos / *Carbonates* (% $CaCo_3$)
DMS-0	0.06	3.92	26.27	29.76	12.58	3.50	0.03	91.38	0.76	38.01
DMS-A	nd	12.07	34.77	42.13	13.10	nd	nd	7.37	0.77	39.75
DMS-B	nd	4.10	15.27	20.27	5.10	nd	nd	7.27	1.02	26.04
DMS-C	nd	6.47	13.77	20.83	6.20	nd	nd	7.40	0.88	29.31
Nivel de acción 1 / *Level of Action 1*	1	100	500	200	100	80	0.60	120	1%	na

Note: DMS-0, DMS -A, DMS-B and DMS-C are dredged sediments from four different places without any treatment, washing or drying. nd = not detected; na = not available.

Chapter 4: Constraints on the Use of Recycled Materials and Suggested Modifications to Current Specifications

4.1 Constraints on the Use of Recycled Materials

Based on the previous review of studies that examined the use of recycled materials in highway applications, constraints related to performance were identified (Tables 4.1 to 4.4). Such constraints and limitations need to be considered in further assessing the performance of highway materials in Maryland conditions and materials through pilot experimental studies, in order to develop the specific criteria and values to include in MSHA specs.

Table 4.1 Constrains of RCA in Highway Applications

Application		Constrains
GAB	Performance	◆ Strength • California Bearing Ratio of RCA is 40%-53% lower than that of the natural crushed rock typically used in highway bases. The range is caused by different moisture contents in base materials, with penetration values from 2.54 mm to 5.08 mm (Kolay and Akentuua 2014). ◆ Durability • Water absorption of RCA is two times higher than natural coarse aggregate (Kolay and Akentuua 2014), and three times higher than limestone (Cooley and Hornsby 2012). • Sodium sulfate soundness degradation of RCA is three times higher than natural coarse aggregate (Kolay and Akentuua 2014). • Los Angeles abrasion loss of RCA is 27%-41% higher than limestone (Cooley and Hornsby 2012, Cooley et al. 2007). The variability is due to the different sources of materials. • Permanent deformation is related to moisture content. When moisture content exceeds the optimum level content by 2%, permanent deformations double. It is recommended that field compaction meet the optimum moisture content (OMC) (Aydilek 2015).
	Environmental Properties	◆ Calcium carbonation and related tufa formation may reduce permittivity of drainage filter fabrics and weaken drainage capacity (Snyder and Bruinsma 1996). ◆ Effluent from drainage layers containing RCA are alkaline with a pH level of 11 to 12 (Snyder and Bruinsma 1996). ◆ High chloride content negatively affects de-icing salts used in winter maintenance operations (Chesner et al. 1998).

Table 4.1 Constrains of RCA in Highway Applications (continued).

Application		Constrains
Drainage /Fill	Performance	◆ Drainage • RCAs easily degrade and generate fines during transporting, stockpiling and placing. Los Angeles abrasion loss of RCA (meeting No.4 gradation) is about 15% higher than limestone (Nam et al. 2014). • Drainage material containing 4% fine RCA (meeting No.4 gradation) shows a significant decrease in drainage capacity with a reduction of 2.5-9 cm/s^2 in flow rate, as value of head varied from 3 to 30 in. Therefore, fine RCA should not exceed 4% by weight (Nam et al. 2014). ◆ Flowable fill • RCA replacing concrete sand requires more water to meet given flow value. To achieve 8 in. final flow value, 150-250 lb/yd^3 more water is required when the percentage of RCA varies from 50% to 100% (Lim et al. 2003). • Entrainment of air into flowable fill mixtures with RCA is not economical, since entrainment of 23% air needs more than 10 times the amount of air entraining agent, compared to concrete sand (Lim et al. 2003). ◆ Embankment • RCA is classified as poorly graded sandy gravel per the Unified Soil Classification System, and can be suitable for embankment construction (Rathje et al. 2006).
	Environmental Properties	◆ Initial laboratory pH of 12.5 decreases to a pH 12.3 in the first 24 hours, then keeps relatively constant at 12.1 (Nam et al. 2014). Even though laboratory column tests yield a pH of 11.0-12.5 (Schaertl et al. 2010), field tests show that leachate pH may be near neutral (6.5-8.0) after seven months, due to carbonation. ◆ More calcite precipitation is likely to occur with RCA than limestone (Nam et al. 2014).

Table 4.1 Constrains of RCA in Highway Applications (continued).

Application		Constrains
HMA	Performance	◆ Marshall design • Optimum asphalt content (OAC) for HMA with RCA is much higher than that of conventional mixtures. OAC of asphalt mixtures with RCA replacing both coarse and fine aggregate is about 7% in average; OAC of asphalt mixtures with RCA replacing all coarse aggregate is about 6.5% in average; OAC of asphalt mixtures with RCA replacing all fine aggregate is about 5.6% in average; OAC of conventional HMA mixtures is about 5.1% in average (Arabani et al. 2012). • With cement filler, OAC of HMA varied from 4.5% - 5.5% as the percentage of RCA ranged between zero and- 60%. With limestone filler, OAC of HMA varied from 4.3% - 5.5% as percentage of RCA ranged from zero to 60% (Perez et al. 2012). ◆ Durability • The addition of RCA reduces low-temperature flexibility of HMA. Bending strain energy of HMA with 100% RCA is 40% lower than conventional HMA. Bending stiffness moduli of HMA with 100% RCA is 21% higher than conventional HMA (Zhu et al. 2012). • RCA reduces moisture resistance of HMA (Pasandin and Perez 2015, Zhu et al. 2012). After water immersion, Marshall Stability of HMA with 100% RCA is 27% lower than that of conventional HMA. The moisture susceptibility can be moderated by adding anti-stripping agent (Bhusal and Wen 2013) or pretreatment (Zhu et al. 2012).

Table 4.1 Constrains of RCA in Highway Applications (continued).

Application		Constrains
PCC	Performance	◆ Fresh Properties • RCA use for coarse aggregate decreases workability (Amorim et al. 2012, Garber et al. 2011). Slump of concrete for a 28-day fc=40 MPa decreased from 17 cm to 5 cm, when percentage of RCA varied between 0%- 50%. However, concrete with 100% RCA had an increased slump value of 19 cm (Domingo-Cabo et al. 2009). ◆ Hardened Properties • The splitting tensile strength of concrete (28-day fc=4000 psi) drops by 12% for 50% RCA mix and by 29% for 100% RCA mix, compared to concrete prepared with conventional aggregate (Snyder 2006). • Modulus of rupture of concrete (28-day fc=4000 psi) drops by 12% for 100% RCA mix, compared to concrete prepared with virgin aggregate (Snyder 2006). • Fracture energy of concrete (28 day fc=4000psi) drops by 14% for 50% RCA mix and 22% for 100% RCA mix, compared to that of concrete prepared with virgin aggregate (Snyder 2006). ◆ Durability • Los Angeles abrasion loss of RCA is 5%-15% more than that of natural aggregates (Amorim et al. 2012). • Absorption capacity of RCA is 2.9%-5% higher than that of natural aggregates (Snyder, 2006). • RCA replacing fine natural aggregates increases shrinkage of concrete (28-day fc=4000 psi) by 20%-50%. RCA replacing both fine and coarse aggregates increases shrinkage of concrete by 70%-100% (Snyder 2006). • RCA originated from concrete that has experienced D-cracked or alkaline-silica-reaction (ASR) is more likely to have D-cracking or ASR experience (Cooley et al. 2007, Snyder 2006).
	Environmental Properties	◆ Water passing through an RCA layer can become highly alkaline, causing metal culvert and rodent guard corrosion, as well as vegetation kill near some drainage system outlets (Cooley et al. 2007). ◆ As, Cr, Pb, and Se may exceed USEPA MCL (maximum contaminant limit) in some States (Edil et al. 2012). Cu concentration may exceed USEPA MCL at acid condition, but in a natural environment, Cu leachate is lower than the limit (Lewis et al. 2015).

Table 4.2 Constrains of RAP in Highway Applications

Applications		Constrains
GAB	Performance	◆ CBR of RAP-based GAB is typically lower than GAB with natural aggregates. At a penetration value of 0.1 inch, CBR is reduced by 18% when RAP percentage increased up to 100%. At a penetration value of 0.2 inch, CBR is reduced up to 20% when RAP percentage is increased up to 100% (Bennett and Maher 2005). ◆ One hundred percent RAP cannot produce high-quality base courses due to its high deformation and creep (Puppala et al. 2012). Permanent strain of base varied from 0.68% - 5.63%, as RAP percentage increased from zero to 100% (Bennett and Maher 2005). Large deformations and high creep potential can be controlled by adding fly ash (Wen et al. 2010), using geocell reinforcement (Thakur et al. 2013), blending RAP with crushed stone, or stabilizing RAP with cementitious materials or foamed asphalt (Dong et al. 2014).
	Environmental Properties	◆ RAP has higher concentrations of total hydrocarbons and some PAHs (poly-aromatic hydrocarbons), in comparison to new conventional asphalt (Legret et al. 2005). However, peak PAH concentrations in deionized water or TCLP leachate is generally close or below the detection limit and groundwater intervention value (Shevidy et al. 2012). ◆ Concentrations of leached As, Se and Sb are slightly higher than their corresponding USEPA MCLs, with peak As concentration of 37.9 μg/L, peak Se concentration of 113 μg/L and peak Sb concentration of 10.6 μg/L. Asphalt binder is probably associated with the source of As, Se and Sb (Edil et al. 2012). ◆ Al concentrations in water leaching test may slightly exceed EPA secondary-enforceable drinking water limits. Cd concentration tends to exceed the limit of EPA for aquatic life and human health in fresh water and drinking water, as well as MD ATL (aquatic toxicity limits of Maryland State) for fresh water. Cu concentrations may exceed chronic Maryland ATL, but are within acute MD ATL. ◆ Pb concentrations probably exceed chronic EPA water quality limit and chronic MD ALT for fresh water, but are generally within the acute EPA water quality limit and acute MD ALT (Aydilek and Mijic2015).

Table 4.2 Constrains of RAP in Highway Applications (continued).

Applications		Constrains
FASB	Performance	◆ Excess fines (i.e., more than 12% passing No.200 sieve) lead to worse dispersion of foamed asphalt and higher sensitivity to moisture. FASB with 10% fines showed a lower fracture face asphalt coverage (FFAC) value of 5.8% - 9.0%, compared to FASB containing 8% fines with FFAC value of 29.8% - 32.4%. The range was caused by moisture content varying from 3% - 7% (Fu et al. 2010a). (FFAC is *a parameter to measure dispersion performance; higher value implies better dispersion and higher quality*)
Drainage /Fill	Performance	◆ RAP has higher potential of collapse in wet conditions than conventional fill material. Collapse index of RAP is up to 1.5%, while conventional material is about 0.2% (Rathje et al. 2006). ◆ Compressibility of RAP shows high sensitivity to temperature. Secondary compression ratio of RAP increased about 14 times as temperature was raised from 22ºC to 35ºC (Soleimanbeigi and Edil 2015). ◆ RAP has higher creep potential. Creep parameter for RAP is generally less than 1.0, comparable to clays, which have a creep parameter of 0.7 (Rathje et al. 2006).

Table 4.2 Constrains of RAP in Highway Applications (continued).

Applications		Constrains
PCC	Performance	◆ Concrete with RAP has lower compressive strength than conventional concrete. RAP replacing all coarse aggregate, all fine aggregate, both coarse and fine aggregates reduced 28-day compressive strength of PCC (28-day fc=5500 psi) by 34%, 50%, and 72%, respectively (Huang et al. 2005). After one year, 25% fine and 50% coarse RAP replacement showed 25% lower compressive strength; 50% fine and 100% coarse RAP replacement showed 47% lower compressive strength, compared to conventional PCC with a 28-day fc=3000 psi (Berry et al. 2013). ◆ Concrete with RAP has lower tensile strength than conventional concrete. RAP replacing all coarse aggregate, all fine aggregate, and both coarse and fine aggregate reduced splitting tensile strength of PCC (28-day fc=5500 psi) by 5%, 21%, and 50%, respectively (Huang et al. 2005). ◆ Addition of RAP decreases flexural strength. After one year, modulus of rupture for 25% fine and 50% coarse RAP replacement was 8% lower; 50% fine and 100% coarse RAP replacement was 25% lower, compared to conventional PCC with 28-day fc=3000 psi (Berry et al. 2013). ◆ Use of RAP decreases stiffness. After one year, 25% fine and 50% coarse RAP replacement had 16% lower elastic modulus; 50% fine and 100% coarse RAP replacement had 44% lower elastic modulus, compared to conventional PCC with 28-day fc=3000 psi (Berry et al). Concrete with higher RAP content experienced higher creep. Creep coefficients of PCC with 28-day fc=3000 psi and with 50% fine and 100% coarse RAP replacement, and 25% fine and 50% coarse RAP replacement, were at least 1.2 times higher than that of conventional concrete (Berry et al. 2013). ◆ Voids in PCC increases with higher RAP content. PCC with 28-day fc=3000 psi incorporating 25% fine and 50% coarse RAP showed 12% void content in volume, which is the upper limit of void content to gain desirable durability (Fick 2008, Berry et al. 2013).

Table 4.3 Constrains of FS in Highway Applications

Applications		Constrains
Crack sealant/ HMA	Performance	◆ When FS replacement is higher than 15%, the asphalt mix may become more sensitive to moisture damage (Yazoghli-Marzouk et al. 2014). After water immersion, indirect tensile strength (ITT) of HMA with 15% FS increased by 8%, comparable to conventional HMA (with an ITT value of 110.58 kPa); indirect tensile strength of HMA with 30% FS was lower by 16%, with an ITT value of 131.73 kPa (Javed et al. 1994). Moisture resistance of FS depends on the clay content and organic additives used (FIRST 2004, Braham 2002). Clay-bonded FS (green sands) may typically be more sensitive to moisture (AFS). ◆ FS reduces indirect tensile strength of HMA, decreasing from 13.9 kPa to 9.4 kPa as FS percentage increased from 0 to 20% (Bakis et al. 2006). ◆ FS reduced flow values of HMA, indicating lower plasticity and worse durability. Flow value reduced from 3.48 mm to 2.4 mm as percentage of FS increased from zero to 20% (Bakis et al. 2006).
Drainage/ Embankment /Base	Performance	◆ When bentonite clay content exceeds 6%, permeability value of FS decreases significantly, ranging between 1×10^{-7} cm/s and 3×10^{-6} cm/s (FIRST 2004). ◆ High cement ratios (>10% by weight) may make cement-stabilized FS more brittle, leading to cracking in base which can be reflected to upper layers (Gedik 2008).
	Environmental Properties	◆ TCLP (Toxicity Characteristic Leaching Procedure) extracts of FS without any additives may have high concentrations of copper, lead and zinc, over the limits of 5mg/L. However, adding iron to the TCLP extraction of FS can significantly decrease copper and lead concentrations (Douglas 2003).

Table 4.3 Constrains of FS in Highway Applications (continued).

Applications		Constrains
Flowable Fill/ SCC	Performance	◆ FS decreases workability of SCC. Slump value immediately after mixing reduced from 115 mm to 63 mm, as foundry sand percentage increases from 0 to 50% (Prabhu et al. 2015). Slump flow time decreased from 3.83s to 1.70s as FS content increased from zero to 100% (Sahmaran et al. 2011). ◆ Compressive strength decreases with increasing FS replacement of natural sand. The 28-day and 180-day compressive strength of 50% FS were 24% lower than concrete mixtures without FS (Prabhu et al. 2014, Prabhu et al. 2015). ◆ Carbonation depth of concrete increases with increasing FS content. At 180 days, carbonation depth of concrete mixtures with 10-50% FS was 6%-412% higher than concrete mixtures without FS. At 365 days, carbonation depth of concrete mixtures with 10%-50% FS was 12%-218% higher than concrete mixtures without FS (Prabhu et al. 2015). ◆ Substitution of FS increases permeability, but only significantly when the substitution rate exceeds 30%. Permeability coefficient of concrete mixtures with 50% FS were more than two times that of concrete without FS (Prabhu et al. 2015). ◆ Sulphate resistance of concrete decreases with increasing FS substitution of natural sand. At the age of 180 days, concrete mixtures with 50% FS showed a 37.7% decrease in compressive strength, while concrete mixtures without FS showed only a 6.2% decrease in compressive strength (Prabhu et al. 2015).
PCC	Performance	◆ Use of FS reduces the workability of concrete. Slump dropped almost linearly from 200 mm for concrete without FS (28-day fc=43.6 MPa) to zero for concrete with 80% and 100% FS, as replacement of natural sand (Khatib et al. 2012). ◆ Use of FS exacerbates carbonation of concrete (28-day fc=36 MPa). For every 10% increase of FS replacement, carbonation depth had an average increase of 0.17 mm and 0.33 mm at 90 days and 365 days, respectively (Siddique et al. 2011). ◆ FS exacerbates drying shrinkage of concrete in respect to conventional concrete (28-day fc=43.6 MPa). The 28-day shrinkage of concrete increased from 221.4 to 442.5 micro-strain as FS percentage increased from zero to 100% (Khatib et al. 2012).

Table 4.4 Constrains of Dredged Material (DM) in Highway Applications

Applications		Constrains
Flowable fill/ Embankment	Performance	◆ Crushed glass (CG) amended dredged material (CG-DM) • CG-DM blends are less strong than natural coarse aggregates (i.e., sand). The cone penetrometer test (CPT) value of the strongest embankment 80/20 CG-DM blend was six MPa, which had only 25% of the strength of 80/20 SSF-DM (Grubb et al. 2008, Grubb et al. 2013). ◆ Steel slag fines (SSF) amended dredged material • The addition of SSF requires more consolidation (i.e., compression) to obtain enough compressibility. Coefficient of consolidation decreases from 0.28 to 0.12 as SSF percentage increased from zero to 100%. Coefficient of reconsolidation decrease from 0.04 to 0.008 as SSF percentage increased from zero to 100% (Malasavage et al. 2012). ◆ Rubber amended dredged material • Unconfined compressive strength and shear strength decreased with increasing rubber content. Unconfined compressive strength decreased linearly from about 440 kPa to about 180 kPa as rubber content increased from zero to 100% (Kim and Kang 2011). • Flowability of DM decreases with increasing rubber content. Flowability with rubber content of zero, 25%, 50% was satisfied $(20 \pm 5cm)$ when water contents were between 140%-160%, 140%-180%, and 160%-200%, respectively (Kim and Kang 2011). ◆ Air-foam amended dredged material • The strength of air-foam stabilized DM decreases with increasing air-foam soil. Unconfined compressive strength decreased almost linearly from 310 kPa to 50 KPa as air foam content increased from zero to 3% (Kim et al. 2010).
	Environmental Properties	◆ Contaminant including metals, pesticides, polycyclic aromatic hydrocarbons (PAH), and polychlorinated biphenyls (PCB) is a concern for using DM. DM becomes oxidized and more acidic during dredging and placement (Winfield and Lee 1999).

Table 4.4 Constrains of DM in Highway Applications (continued).

Applications		Constrains
Lightweight aggregate/ Bricks	Performance	◆ Brick ■ Novosol® amended river sediment bricks • Firing shrinkage of Novosol® amended river sediment bricks (10%) is higher than that of standard bricks (7%) (Samara et al. 2009). • Novosol® amended river sediment brick is classified as a low-plastic mixture, indicating lower plasticity and poorer bonding ability (Lafhaj et al. 2008). ■ Water treatment residue brick • Water treatment residue brick requires higher sintering temperature to meet the same bulk density, compared to excavation waste soil brick. To achieve a specific gravity of 1.8, waste treatment residue brick requires at least 1050ºC, while excavation waste soil brick only needs 800ºC of sintering temperature (Huang et al. 2005).
	Environmental Properties	◆ Leachability of heavy metals from sediment brick was generally higher than commercial bricks (Karius and Hamer 2001).

Table 4.4 Constrains of DM in Highway Applications (continued).

Applications		Constrains
PCC/ cement	Performance	◆ PCC • DM replacing fine aggregate dramatically reduces workability of concrete. As DM content increased from zero to 20%, spread diameter in flow test of concrete (28-day fc=33MPa) reduced from 72 mm to 32 mm at a constant w/c ratio of 0.7. Inversely, to maintain a constant spread diameter of 47 mm, the w/c ratio must be increased from 0.45 to 0.88 (Millrath et al. 2001). • DM is potentially detrimental to concrete due to its high pH, as well as its chlorides and sulfates contents. New York/New Jersey Harbor sediments have been tested with sulfates content at 0.15-4.1% and chlorides content at 0.36-5.7% (Maher 2013). Sulfate in excess of 0.3% and chloride in excess of 0.5% is considered severely or extremely corrosive (Oweis 1998). ◆ Cement • The 28-day compressive strength decreases slightly when DM substitution ratio reached 25% (compared to mortar with DM less than 25%), indicating that 25% could be the optimum substitution ratio of DM for compressive strength (Limeira et al. 2012). • The 28-day flexural strength decreases slightly when DM substitution ratio reached 15% (compared to mortar with DM less than 15%), indicating that 15% could be the optimum substitution ratio of DM for flexural strength (Limeira et al. 2012).

4.2 Needed Modifications to Existing MSHA Specs

The research team reviewed the existing MSHA specifications for Portland cement concrete, HMA, GAB and Bricks/LWA (Tables 4.5, 4.7, 4.9, 4.11). Based on the findings and recommendations from past studies, the team explored the use of these recycled materials in highway applications and identified the MSHA specification areas that need to be revised to accommodate such materials (Tables 4.6, 4.8, 4.10, 4.12). The development of such modified specs will require exploratory studies assessing the impact of these recycled materials in current highway applications, and provide the required suggestions and design/performance requirements for modifying the specs and MSMTs.

4.2.1 Concrete Specs

Table 4.5 Current MSHA Specs Related to Concrete

PCC	Maryland Spec
Conventional PCC	➢ **Coarse aggregate (**AASHTO M80 Class A**)**: • AASHTO T104. Sodium Sulfate Soundness≤12%; • AASHTO T112. Clay Lumps and Friable Particles≤2%; • AASHTO T113. Chert; Less than 2.40 Specific Gravity≤3%; • AASHTO T112 and T113. Sum of Clay Lumps, Friable Particles and Chert≤3%; • AASHTO T113. Coal and Lignite≤0.5%; • AASHTO T11. Material finer than No. 200 sieve≤1% (1.5% if material passing No. 200 sieve is dust of fracture, free of clay or shale); • ASTM D4791. Flat and elongated≤ 12%; • AASHTO T96. LA abrasion≤50%. ➢ **Fine aggregate (**AASHTO M6 Class B**)**: • AASHTO T104. Sodium Sulfate Soundness≤10%; • AASHTO T112. Clay Lumps and Friable Particles≤3%; • AASHTO T113. Coal and Lignite≤1%; • AASHTO T11. Material finer than No. 200 sieve≤4% (5.0% for concrete not subject to surface abrasion); • AASHTO T21. Organic impurities≤ 3%. ➢ **Concrete Admixtures** • Prohibit the admixtures that contribute more than 200 ppm of chlorides (MSMT 610). • Do not use pozzolan and Type I (PM) or Type IP cement in the same mix. • Fly Ash (M 295), should be pozzolan Class C or F, except that the maximum permissible moisture content is 1.0% and when used in concrete Mix No. 3 and 6 the maximum loss on ignition is 3.0%.

Table 4.5 Current MSHA Specs Related to Concrete (continued)

<table>
<tr><th>PCC</th><th>Maryland Spec</th></tr>
<tr><td>Conventional PCC</td><td>

➢ **Aggregate Expansion due to Alkali Silica Reactivity** (MSMT 212):

- Expansion≤0.1% can be used without restriction;
- Expansion between 0.1% and 0.35% may only be used when one of the options at Table 902B are employed.
- Expansion≥0.35% is prohibited.

TABLE 902 B

OPTION	ALKALI CONTENT OF CEMENT % max	REPLACE CEMENT WITH		SPECIFICATION
		MATERIAL	% BY WEIGHT	
1	1.50	Class F Fly Ash	15 – 25	M 295
2	1.50	Ground Iron Blast Furnace Slag	25 – 50	M 302 Grade 100 or 120
3	1.50	Microsilica	5 – 7	C 1240
4	—	Blended Cement (a)	100	M 240
5	0.60 (b)	Low Alkali Cement	100	M 85

(a) Pozzolan content of 15 to 25 percent by weight of cement.
(b) For mixes (Mix 6 Modified, 12 Hour Patch Mix) used for portland cement concrete pavement repairs; the maximum allowable percentage of alkalies in portland cement is 0.70.

➢ **Chloride content** shall not exceed the following limits:

- Bridge Superstructure and Pre-stressed Concrete ≤500 ppm;
- Latex Modified Concrete ≤50 ppm;
- Other Concrete and Water Used in Curing ≤1000 ppm.
- Calcium chloride in solution shall contain a minimum of 30% salts.
- When used as a solution, Flakes shall contain 30-32% solids.

TEST PROPERTY	SPECIFICATION LIMITS
Magnesium Chloride $MgCl_2$, %	46.0 – 47.0
Calcium Chloride $CaCl_2$, %	2.0 – 3.0
Potassium Chloride KCl, %	0.5 – 1.0
Sodium Chloride NaCl, %	0.5 – 1.0
Sulfates, % max	0.05

</td></tr>
</table>

Table 4.5 Current MSHA Specs Related to Concrete (continued)

<table>
<tr><th>PCC</th><th>Maryland Spec</th></tr>
<tr><td>Conventional PCC</td><td>

Existing test and measurement methods:
MSMT 212, AASHTO M154, AASHTO M194, AASHTO M295, AASHTO M302, AASHTO C1240, AASHTO C116, AASHTO M240, AASHTO M144 (Type S, Grade I, Class A), AASHTO M85, AASHTO T309, AASHTO T152, AASHTO T196, AASHTO T23, AASHTO T26, AASHTO T27, AASHTO T96/ASTM C131, ASSHTO T21, AASHTO T11, AASHTO T113/ASTM C123, AASHTO T112, AASHTO T104, ASTM D4791, ASTM C227, AASHTO M92, AASHTO M92/ASTM E11, AASHTO M201/ASTM C511, AASHTO M210/ASTM C490, AASHTO T106/ASTM C109, AASHTO T162/ASTM C305, ASTM D512.
Concrete plants: AASHTO M157, ASTM C685, MSMT 558, MSMT 560

</td></tr>
<tr><td>Conventional Lightweight PCC</td><td>

Coarse aggregate (AASHTO M195):

AASHTO T112. Clay Lumps and Friable Particles≤2%;
ASTM D4791. Flat and elongated≤ 12%.

Fine aggregate (AASHTO M195):

AASHTO T112. Clay Lumps and Friable Particles≤2%;
AASHTO T21. Organic impurities≤ 3%.

Compressive strength≥4500 psi.
Shall compose of Type I Portland cement, an approved air entraining admixture, Type A or D chemical admixture, water, lightweight coarse aggregate, and fine aggregates.
Fly ash or ground iron blast furnace slag may be substituted for Portland cement.

<table>
<tr><th>PROPERTY</th><th>LIMIT</th><th>REMARKS</th></tr>
<tr><td>Cement Content</td><td>700 lb/yd^3, max</td><td>—</td></tr>
<tr><td>Average Density of Cured Concrete</td><td>118 lb/ft^3, max</td><td>—</td></tr>
<tr><td>Air Entrainment
(Entrapped Plus Entrained)</td><td>6 - 9%</td><td>Volumetric Method T 196</td></tr>
<tr><td>Slump
When coarse aggregate absorption >10 %
When coarse aggregate absorption ≤10 %</td><td>
3 in., max
2 - 5 in.</td><td>T 119
—
—</td></tr>
<tr><td>Water
Added to mix using saturated aggregates
Net (including absorbed water)</td><td>
0.45, max
0.75, max</td><td>
Water/cement ratio
Water/cement ratio</td></tr>
</table>

Existing test and measurement method:
ASTM C567, AASHTO T112, ASTM D4791, ASSHTO T21, AASHTO M195, AASHTO T27

</td></tr>
</table>

Table 4.5 Current MSHA Specs Related to Concrete (continued)

PCC	Maryland Spec
Portland cement	➢ Furnish certification in TC-1.03 ➢ **Existing test and measurement method:** AASHTO M85, AASHTO T131, AASHTO T153

Table 4.6 Potential Areas of Revisions to MSHA Specs for Concrete

PCC	Revision
RCA in PCC	1. **Los Angeles abrasion loss** of RCA is 5%~15% more than that of natural aggregates (Amorim et al. 2012).Thus, AASHTO T96. LA abrasion≤65%. 2. **Slump** of concrete (28-day fc=5800 psi) decreased from 17 cm to 5 cm when percentage of RCA varied form zero to 50%. However, concrete with 100% RCA had an increased slump value of 19 cm (Domingo-Cabo et al. 2009). 3. **Supplemental test and measurement method:** ➢ **Drying shrinkage:** ASTM C157. RCA replacing all fine natural aggregates increases shrinkage of concrete (4000 psi) by 20%~50%. RCA replacing all fine and coarse aggregates increases shrinkage of concrete by70%~100% (Snyder 2006). ➢ **Flexural strength:** ASTM C512. Modulus of rupture of concrete (4000 psi) decreases 12% for 100% RCA mix, compared to concrete with virgin aggregate (Snyder 2006). ➢ **Fracture crack:** ASTM C597. Fracture energy of concrete (4000psi) reduces by 14% for 50% RCA mix and 22% for 100% RCA mix, compared to concrete with virgin aggregate (Snyder 2006). ➢ **Resistance to deicing chemicals**: ASTM C672. ➢ **Sampling:** AASHTO T2 ➢ **Splitting tensile strength:** ASTM C496. The splitting tensile strength of concrete (4000 psi) decreases 12 % for 50% RCA mix and 29% for 100% RCA mix, compared to concrete with virgin aggregate (Snyder 2006). ➢ **Water absorption**: AASHTO T85/ASTM C127. Absorption capacity of RCA is 2.9%-5% higher than that of natural aggregates (Snyder 2006).

Table 4.6 Potential Areas of Revisions to MSHA Specs for Concrete (continues)

PCC	Revision
RAP in PCC	**1. Compressive strength:** RAP replacing all coarse aggregates, all fine aggregates, and both coarse and fine aggregate reduced 28-day compressive strength of PCC (5500 psi) by 34%, 50%, and 72%, respectively (Huang et al. 2005). **2. Supplemental test and measurement method:** ➢ **Creep deformation:** ASTM C512. Concrete with high RAP content experienced more creep than conventional PCC (Berry et al. 2013). ➢ **Flexural strength:** ASTM C512. 28-day modulus of rupture for PCC (3000 psi) with 25% fine and 50% coarse RAP replacement was 17% lower; 50% fine and 100% coarse RAP replacement was 31% lower, compared to conventional PCC (Berry et al. 2013). ➢ **Resistance to deicing chemicals:** ASTM C672. ➢ **Splitting tensile strength:** ASTM C496. RAP replacing all coarse aggregates, all fine aggregates, and both coarse and fine aggregate reduced 28-day splitting tensile strength of PCC (5500 psi) by 5%, 21%, and 50%, respectively (Huang et al. 2005). ➢ **Stiffness:** ASTM C469. 28-day elastic modulus of PCC (3000 psi) with 25% fine and 50% coarse RAP replacement was 17% lower; 50% fine and 100% coarse RAP replacement was 46.5% lower (Berry et al. 2013). ➢ **Void content:** AASHTO T19/ASTM C642. Void volume in PCC increases with higher RAP content. PCC (28-day fc=3000 psi) made with 25% fine and 50% coarse RAP showed 12% void content in volume, which is the upper limit of void content to gain desirable durability (Fick 2008, Berry et al. 2013).
FS in PCC	**1. Slump** dropped almost linearly from 200 mm for the concrete without FS (28-day fc=6000 psi) to zero for concrete with an 80% and 100% FS replacement of natural sand (Khatib et al. 2012). **2. Supplemental test and measurement method:** ➢ **Carbonation:** ASTM C876. For every 10% increase of FS replacement, carbonation depth of concrete (28-day fc=5000 psi) had an average increase of 0.17 mm and 0.33 mm at 90 days and 365 days, respectively (Siddique et al. 2011). ➢ **Drying shrinkage:** ASTM C157. 28-day shrinkage of concrete (28-day fc=6000 psi) increased from 221.4 to 442.5 micro-strain, as FS percentage increased from zero to 100% (Khatib et al. 2012).

Table 4.6 Potential Areas of Revisions to MSHA Specs for Concrete (continued)

<table>
<tr><th>PCC</th><th>Revision</th></tr>
<tr><td>FS in SCC</td><td>1. Compressive strength: Compressive strength decreases with increasing FS replacement of natural sand. 28-day compressive strength of 50% FS is 24% lower than concrete mixtures without FS (Prabhu et al. 2014, Prabhu et al. 2015).

2. Slump: Slump value immediately after mixing reduces from 115 mm to 63 mm, as foundry sand percentage increases from zero to 50% (Prabhu et al. 2015). Slump flow time decreased from 3.83s to 1.70s as FS content increased from zero to 100% (Sahmaran et al. 2011).

3. Supplemental test and measurement method:

➢ Carbonation: ASTM C876. At 180 days, carbonation depth of concrete mixtures with 10%-50% FS was 6%-412% higher than concrete mixtures without FS. At 365 days, carbonation depth of concrete mixtures with 10%-50% FS was 12%-218% higher than concrete mixtures without FS (Prabhu et al. 2015).
➢ Permeability: ASTM D2434. Substitution of FS increases permeability, but only significantly when the substitution rate exceeds 30%. Permeability coefficient of concrete mixtures with 50% FS was more than two times that of concrete mixtures without FS (Prabhu et al. 2015).
➢ Sulfate resistance: AASHTO T104/ASTM C88. Sulphate resistance of concrete decreases with increasing FS substitution of natural sand, leading to reduced compressive strength. Concrete with 50% FS substitution showed a 37.7% decrease in 180-day compressive strength, more than concrete without FS, which only showed a 6.2% decrease (Prabhu et al. 2015).</td></tr>
<tr><td>DM in PCC</td><td>1. Slump: As DM content increased from zero to 20%, spread diameter in flow test of concrete (28-day fc=4500 psi) reduced from 2.8 in. to 1.3 in. at a constant w/c ratio of 0.7. Inversely, to maintain a constant spread diameter of 1.85 in., w/c ratio has to be increased from 0.45 to 0.88 (Millrath et al. 2001).

2. Chlorides and sulfates contents: New York/New Jersey Harbor sediments have been tested with sulfates content at 0.15%-4.1% and chlorides content at 0.36%-5.7% (Maher 2013). Sulfate in excess of 0.3% and chloride in excess of 0.5% is considered severely or extremely corrosive (Oweis 1998).</td></tr>
<tr><td>DM in Cement</td><td>1. Compressive strength: Maximum substitution rate could be 25% in respect to compressive strength (Limeira et al. 2012).

2. Flexural strength: Maximum substitution ratio could be 15% in respect to flexural strength (Limeira et al. 2012).</td></tr>
</table>

4.2.1.1 Referenced Specs

AASHTO SPEC

1. AASHTO C1240. Standard specification for silica fume.
2. AASHTO M144. Standard specification for calcium chloride.
3. AASHTO M154. Specification for air-entraining admixture for concrete.
4. AASHTO M157. Standard specification for ready-mixed concrete (chemical limitations for mixing water).
5. AASHTO M194. Standard specification for chemical admixtures for concrete.
6. AASHTO M195. Lightweight aggregates for structural concrete.
7. AASHTO M201. Standard specification for mixing rooms, moist cabinets, moist rooms and water storage tanks used in the testing of hydraulic cements and concretes.
8. AASHTO M210. Standard specification for apparatus for use in measurement of length change of hardened cement paste, mortar and concrete.
9. AASHTO M240. Standard specification for blended cement.
10. AASHTO M295. Standard specification for coal fly ash and raw or calcined natural pozzolans for use in concrete.
11. AASHTO M302. Standard specification for slag cement for use in concrete and mortars.
12. AASHTO M85. Standard specification for Portland cement (chemical and physical).
13. AASHTO T104. Soundness of aggregate by use of sodium sulfate or magnesium sulfate.
14. AASHTO T106. Standard method of test for compressive strength of hydraulic cement mortar using 50 mm or 2 in. cube specimens.
15. AASHTO T112. Clay lumps and friable particles in aggregate.
16. AASHTO T113. Standard method of test for lightweight pieces in aggregate.
17. AASHTO T131. Standard method of test for time of setting of hydraulic cement by icat needle.
18. AASHTO T152. Standard method of test for air content of freshly mixed concrete by the pressure method.
19. AASHTO T153. Standard method of test for fineness of hydraulic cement by air permeability apparatus.
20. AASHTO T162. Standard method of test for mechanical mixing of hydraulic cement pastes and mortars of plastic consistency.
21. AASHTO T19. Standard method of test for bulk density (“unit weight”) and voids in aggregate.
22. AASHTO T196. Standard method of test for air content of freshly mixed concrete by the volumetric method.
23. AASHTO T2. Sampling of aggregates.
24. AASHTO T21. Organic impurities in fine aggregates for concrete.
25. AASHTO T23. Making and curing concrete test specimens in the field.
26. AASHTO T26. Quality of water to be used in concrete.
27. AASHTO T27. Sieve analysis of fine and coarse aggregate.
28. AASHTO T309. Standard method of test for temperature of freshly mixed Portland cement concrete.
29. AASHTO T85. Standard method of test for specific gravity and absorption of coarse aggregate.

Aggregate Grading Requirements Test Method AASHTO T27																	
Material		Sieve Size															
		2-1/2”	2”	1-1/2”	1”	3/4”	1/2”	3/8”	No. 4	No. 8	No. 10	No. 16	No. 30	No. 40	No. 50	No. 100	No. 200
Coarse Agg-PCC	57[b]	-	-	100	95-100	-	25-60	-	0-10	0-5	-	-	-	-	-	-	-
	67	-	-	-	100	90-100	-	20-55	0-10	0-5	-	-	-	-	-	-	-
	7	-	-	-	-	100	90-100	40-70	0-15	0-5	-	-	-	-	-	-	-
Fine Agg-PCC or Underdrain[b]		-	-	-	-	-	-	100	95-100	-	-	45-85	-	-	5-30	0-10	-
Coarse Agg-LPCC		-	-	-	100	90-100	-	10-50	0-15	-	-	-	-	-	-	-	-
Fine Agg-LPCC[a]		-	-	-	-	-	-	100	85-100	-	-	40-80	-	-	10-35	5-25	-

Note: PCC=Portland Cement Concrete; LPCC=Lightweight Portland cement Concrete.

(a) Fine aggregate includes natural or manufactured sand.

(b) When this material is used for drainage applications, recycled concrete is prohibited.

ASTM SPEC

1. ASTM C173. The volumetric method for determining air content can be used for concrete made with any type of aggregate.
2. ASTM C191. The set time of cement paste made with the questionable water, as measured using the Vicat apparatus, should not be 1 hour less than or 1-1/2 hours more than the set time of paste made with potable or distilled water.
3. ASTM C204. Standard test methods for fineness of hydraulic cement by air-permeability apparatus.

ASTM C227. Determine the potentially expansive alkali–silica reactivity of cement–aggregate combinations.

4. ASTM C231. The pressure method is widely used for determining air content. It takes less time than the volumetric method.
5. ASTM C469. Standard test method for static modulus of elasticity and Poisson's ratio of concrete in compression.
6. ASTM C496. The split-tension test measures the tensile strength of concrete.
7. ASTM C512. Standard test method for creep of concrete in compression.
8. ASTM C567. Standard test method for determining density of structural lightweight concrete.
9. ASTM C642. Standard test method for density, absorption, and voids in hardened concrete.
10. ASTM C672. Standard test method for scaling resistance of concrete surfaces exposed to deicing chemicals.
11. ASTM C685. Standard specification for concrete made by volumetric batching and continuous mixing.
12. ASTM C876. Standard test method for corrosion potentials of uncoated reinforcing steel in concrete.
13. ASTM C88. The soundness test simulates weathering by soaking the aggregates in either a sodium sulfate or a magnesium sulfate solution.
14. ASTM D2434. Standard test method for permeability of granular soils (constant head).

15. ASTM D4791. Standard test method for flat particles, elongated particles, or flat and elongated particles in coarse aggregate.
16. ASTM D512. Standard test methods for chloride ion in water.
17. ASTM E11. Standard specification for wire-cloth sieves for testing purposes.
18. ASTM C597. Standard test method for pulse velocity through concrete.
19. ASTM C157. Standard test method for length change of hardened hydraulic-cement mortar and concrete.

MSMT SPEC

1. MSMT 212. Accelerated detection of potentially deleterious expansion of mortar bars due to Alkali-Silica reaction aggregate or aggregate/pozzolans combination.
2. MSMT 560. Certification of concrete plant technician.
3. MSMT 558. Calibrating concrete mobile mixers.
4. The concrete mixes shall meet the following:

PCC Mixtures									
Mix No.	28 Day Specified Compressive Strength (psi)	Standard Deviation (psi)	Critical Value (psi)	Minimum Cement (lb/yd^3)	Coarse Agg Size M43	Maximum W/C Ratio by wt	Slump Range (in.)	Total Air Content %	Concrete Temperature F
1	2500	375	2430	455	57,67	0.55	2-5	5-8	70 ± 20
2	3000	450	3010	530	57,67	0.50	2-5	5-8	70 ± 20
3	3500	525	3600	580	57,67	0.50	2-5	5-8	70 ± 20
4	3500	525	3600	615	57,67	0.55	4-8	N/A	70 ± 20
5	3500	525	3600	580	7	0.50	2-5	5-8	70 ± 20
6	4500	675	4770	615	57,67	0.45	2-5	5-8	65 ± 15
7	4200	630	4420	580	57	0.50	1 1/2-3	5-8	70 ± 20
8	4000	600	4180	750	7	0.42	2-5	5-8	65 ± 15

4.2.2 HMA Specs

Table 4.7 Current MSHA Specs Related to HMA

HMA	Maryland Spec
Conventional HMA	➢ **Hot Mix Asphalt Superpave** (AASHTO M323) • AASHTO T104. Sodium Sulfate Soundness≤12%; • AASHTO T112. Clay Lumps and Friable Particles≤2%; • AASHTO T113. Chert; Less than 2.40 Specific Gravity≤3%; • AASHTO T112 and T113. Sum of Clay Lumps, Friable Particles and Chert≤3%; • AASHTO T113. Coal and Lignite≤0.5%; • ASTM D4791 (Dimensional ratio of calipers shall be 5:1; the test for flat and elongated particles (max/min) shall be conducted on the blend). Flat and elongated≤10%. • AASHTO T96. LA abrasion≤45%; • MSMT 411. PV≥5. Polish Value (PV) shall be 5.5 when any aggregate being blended has a PV less than 5.0. PV shall be 5.0 when the aggregate from each source has a PV of 5.0 or greater. PV shall be 9.0 when any aggregate being blended has a PV less than 8.0. PV shall be 8.0 when the aggregates from each source has a PV of 8.0 or greater. When carbonate rock is used, it shall have a minimum of 25% insoluble residue retained on the No. 200 sieve. Aggregate from no more than two sources may be blended. Determine proportions of blended aggregate under MSMT 416. When recycled asphalt pavement (RAP) is used, the PV shall be 4.0. ➢ **Gap Graded Hot Mix Asphalt Superpave** (AASHTO M323) • AASHTO T104. Sodium Sulfate Soundness≤12%; • AASHTO T112. Clay Lumps and Friable Particles≤2%; • AASHTO T113. Chert; Less than 2.40 Specific Gravity≤3%; • AASHTO T112 and T113. Sum of Clay Lumps, Friable Particles and Chert≤3%; • AASHTO T113. Coal and Lignite≤0.5%; • ASTM D4791 (Dimensional ratio of calipers shall be 3:1/5:1; test conducted on particles retained on the No. 4 sieve). Flat and elongated≤20/5%. • AASHTO T96. LA abrasion≤30%; • MSMT 411. PV≥8. PV shall be 9.0 when any aggregate being blended has a PV less than 8.0. PV shall be 8.0 when the aggregates from each source has a PV of 8.0 or greater. When carbonate rock is used, it shall have a minimum of % insoluble residue retained on the No. 200 sieve. When recycled asphalt pavement (RAP) is used, the PV shall be 4.0.

Table 4.7 Current MSHA Specs Related to HMA (continued)

HMA	Maryland Spec
Conventional HMA	➢ **Other requirement:** • Asphalt binder recovered from RAP (binder replacement) shall not be greater than 30% of the asphalt binder of the mix without further evaluation. If mixes contain more than 30% binder replacement with RAP, test and evaluate mixes in accordance with PP61 or R62. Testing should be approved by OMT/ATD (Office of material technology/Asphalt technology division) and the asphalt producer. • Allowable percentage and suitability for use of RAP shall be determined in conformance with MSMT 412 and M 323. Binder grade adjustment is not required when RAP≤20%. • The use of RAP, not to exceed 10%, may be considered for applications where higher polish value aggregates are required and in mixes requiring elastomer type polymer binder. • HMA shall have a Tensile Strength Ratio (TSR) of at least 0.85 when tested in conformance with D 4867. The freeze-thaw conditioning cycle is required. HMA mixes not meeting the minimum TSR requirement shall include an antistripping additive. ➢ **Existing test and measurement method:** AASHTO M323, MSMT 410, MSMT 412, MSMT 441, MSMT 733, MSMT 735, AASHTO T27, ASTM D4791 (for aggregate retained on the 4.75 mm sieve), AASHTO R35, AASHTO M231, AASHTO R9, AASHTO M320 (Table 1), AASHTO TP62 (when RAP in surface mixes≥20% and RAP in base mixes≥25%), ASTM D4867, ASTM C1097, AASHTO T104, AASHTO T112, AASHTO T113, ASTM D4791, AASHTO T96 ➢ **Existing test and measurement method for HMA plants:** AASHTO M156, MSMT 414, MSMT 453, MSMT 251, MSMT 735, AASHTO T255

Table 4.7 Current MSHA Specs Related to HMA (continued)

<table>
<tr><th>HMA</th><th>Maryland Spec</th></tr>
<tr><td>Conventional HMA</td><td>
TABLE 904 A – MIX TOLERANCES
<table>
<tr><th>PHYSICAL PROPERTY</th><th>TOLERANCE:PLANT SITE OR HAULING UNIT SAMPLES (b)</th><th>TOLERANCE:PROJECT SITE BEHIND THE PAVER SAMPLES(b)</th></tr>
<tr><td>Passing No. 4 (4.75 mm) sieve and larger, %</td><td>± 7</td><td>± 7</td></tr>
<tr><td>Passing No. 8 (2.36 mm) thru No. 100 (150 µm) sieve, %</td><td>± 4</td><td>± 5</td></tr>
<tr><td>Passing No. 200 (75 µm) sieve, %</td><td>± 2</td><td>± 2</td></tr>
<tr><td>Asphalt content, %</td><td>± 0.4</td><td>± 0.5</td></tr>
<tr><td>Ratio of dust to binder material</td><td>0.6 to 1.6 (a)</td><td>0.6 to 1.6 (a)</td></tr>
<tr><td>Mix temperature leaving plant versus mix design temperature, F</td><td>± 25</td><td>NA</td></tr>
<tr><td>Deviation of maximum specific gravity per lot versus design maximum specific gravity</td><td>± 0.030</td><td>± 0.040</td></tr>
<tr><td>Voids, total mix, (VTM), %</td><td>4.0 ± 1.2</td><td>4.0 ± 1.2</td></tr>
<tr><td>Voids, total mix, 4.75 mm mix (VTM), %</td><td>3 ± 2</td><td>3 ± 2</td></tr>
<tr><td>Voids in mineral aggregate, (VMA), %</td><td>± 1.2 from design target</td><td>± 1.2 from design target</td></tr>
<tr><td>Voids filled asphalt (VFA), %</td><td>Within spec</td><td>Within spec</td></tr>
<tr><td>Bulk specific gravity, G_{mb}, %</td><td>± 0.022</td><td>± 0.022</td></tr>
<tr><td>G_{mb} at N_{max}, %</td><td>+ 0.5</td><td>+ 0.5</td></tr>
</table>
(a) Not applicable to 4.75 mm.
(b) For mixes other than Gap Graded HMA.
</td></tr>
</table>

Table 4.8 Potential Areas of Revisions to MSHA Specs for HMA

<table>
<tr><th>HMA</th><th>Revision</th></tr>
<tr><td>RAP in HMA</td><td>None.</td></tr>
<tr><td>RCA in HMA</td><td>1. Optimum asphalt content: Optimum asphalt content (OAC) for HMA with RCA is much higher than that of conventional mixtures. OAC of asphalt mixtures with RCA, replacing both coarse and fine aggregate, is about 7% on average; OAC of asphalt mixtures with RCA replacing all coarse aggregate is about 6.5% on average; OAC of asphalt mixtures with RCA replacing all fine aggregate is about 5.6% on average; OAC of conventional HMA mixtures is about 5.1% on average (Arabani et al. 2012).

2. Supplemental test and measurement method:

➢ Moisture resistance: AASHTO T283/ ASTM D4867. RCA reduces moisture resistance of HMA. After water immersion, Marshall Stability of HMA with 100% RCA is 27% lower than that of conventional HMA (Pasandin and Perez 2015, Zhu et al. 2012).

➢ Fatigue resistance: AASHTO T321. Addition of RCA reduces low-temperature flexibility of HMA. Bending strain energy of HMA with 100% RCA is 40% lower than that of conventional HMA. Bending stiffness moduli of HMA with 100% RCA is 21% higher than that of conventional HMA (Zhu et al. 2012).</td></tr>
<tr><td>FS in crack sealant/HMA</td><td>Supplemental test and measurement method:

➢ Moisture resistance: AASHTO T283. When FS replacement is higher than 15%, the asphalt mix may become more sensitive to moisture damage (Yazoghli-Marzouk et al. 2014). After water immersion, indirect tensile strength of HMA with 15% FS increased by 8%, comparable to conventional HMA, with an ITT value of 110.58 kPa (Javed et al. 1994).

➢ Clay/silt content: ASTM D2419. Moisture resistance of FS depends on the clay content and organic additives used (FIRST 2004, Braham 2002). Clay-bonded FS (green sands) may typically be more sensitive to moisture (AFS).

➢ Indirect tensile strength: AASHTO T322. FS reduces indirect tensile strength of HMA, decreasing from 13.9 kPa to 9.4 kPa, as FS percentage increased from 0 to 20% (Bakis et al. 2006).

➢ Marshall flow: AASHTO T245/ ASTM D1559. FS reduced flow values of HMA, indicating lower plasticity and worse durability. Flow value reduced from 3.48 mm to 2.4 mm as percentage of FS increased from zero to 20% (Bakis et al. 2006).</td></tr>
</table>

4.2.2.1 Referenced Specs

AASHTO SPEC

1. AASHTO M156. Standard specification for requirements for mixing Plants for Hot-Mixed, Hot-Laid Bituminous Paving Mixtures.
2. AASHTO M231. Standard specification for weighing devices used in the testing of materials.
3. AASHTO M320. (Table 1) SUPERPAVE™ Binder Grade, PG: 70-28.

AASHTO M320 (Table 1) Binder requirement for PG: 70-28

Property			AASHTO test methods	Specifications
Original binder				
Specific gravity		15.6℃	T228	Report
Softening point			D36	Report
Penetration (100 grams, 5sec), dmm		25℃	T49	Report
Viscosity, Pa*s		135℃	T316	3.0 max
		165℃		Report
Separation, R&B difference, 48 hrs		163℃		
Top, 1/3, Softening point			D5892	Report
Bottom, 1/3, Softening point				
Difference				2(4) max
Dynamic shear, kPa		64℃	T35	1.0 min.
		82℃		
After RTFOT @135℃				
Mass change, %			T240	1.0 max.
Dynamic shear, kPa		70℃	T315	2.2 min.
		76℃		
MSCR	0.1kPa	64℃	TP 70-08	Report
	3.2kPa			
Pressure aging residue 100℃, 300psi, 20hr.			R28	
Dynamic shear, kPa		16℃	T315	Report
		28℃		5,000 max.
Creep stiffness	Stiffness, MPa (60sec)	-12℃	T313	300 max.
	M value			0.300 min.
	Stiffness, MPa (60sec)	-18℃		300 max.
	M value			0.300 min.

4. AASHTO M323. Superpave Mix Design Aggregate Gradation Control Points.

	Nominal Maximum Size (mm)					
Sieve Size, mm (in.)	**37.5**	**25**	**19**	**12.5**	**9.5**	**4.75**
50 (2 in.)	100	—	—	—	—	—
37.5 (1 1/2 in.)	90–100	100	—	—	—	—
25 (1 in.)	90 max	90–100	100	—	—	—
19 (3/4 in.)	—	90 max	90–100	100	—	—
12.5 (1/2 in.)	—	—	90 max	90–100	100	100
9.5 (3/8 in.)	—	—	—	90 max	90–100	95–100
4.75 (No. 4)	—	—	—	—	90 max	90–100
2.36 (No. 8)	15–41	19–45	23–49	28–58	32–67	—
1.18 (No. 16)	—	—	—	—	—	30–60
0.075 (No. 200)	0.0–6.0	1.0–7.0	2.0–8.0	2.0–10.0	2.0–10.0	6.0–12.0

5. AASHTO M332. Performance-graded asphalt binder using multiple stress creep recovery (MSCR).
6. AASHTO PP61. Practice for developing dynamic modulus master curves for hot mix asphalt (HMA) using the asphalt mixture performance tester (AMPT).
7. AASHTO R59. Recovery of asphalt from solution by Abson Method.
8. AASHTO R62. Developing dynamic modulus master curve for asphalt mixtures.
9. AASHTO R9. Standard recommended practice for acceptance sampling plans for highway construction.
10. AASHTO T104. Soundness of aggregate by use of sodium sulfate or magnesium sulfate.
11. AASHTO T11. Materials finer than No.200 sieve in mineral aggregate by washing.
12. AASHTO T164. Quantitative extraction of asphalt binder from HMA.
13. AASHTO T2. Sampling of aggregates.
14. AASHTO T209. Theoretical maximum specific gravity and density of HMA.
15. AASHTO T245. Standard method of test for resistance to plastic flow of bituminous mixtures using Marshall apparatus.
16. AASHTO T255. Standard method of test for total evaporable moisture content of aggregate by drying.
17. AASHTO T27. Sieve analysis of fine and coarse aggregate.

Table 901C. Asphalt Mix AGGREGATE GRADING REQUIREMENTS, PERCENTAGE PASSING FOR MIX DESIGN, TEST METHOD T 27										
Material	Sieve Size									
	19mm	12.5mm	9.5mm	4.75mm	2.36mm	1.18mm	600μm	300 μm	150 μm	75 μm
Hot Mix Asphalt Superpave - 4.75mm	-	-	100	80-100	36-76	-	-	-	-	2-12
Gap Graded Hot Mix Asphalt - 9.5mm	100	100	75-90	30-50	20-30	-	-	-	-	8-13
Gap Graded Hot Mix Asphalt - 12.5mm	100	90-99	70-85	28-40	18-30	-	-	-	-	8-11
Gap Graded Hot Mix Asphalt - 19.0mm	100	82-88	60max	22-30	14-20	-	-	-	-	9-11

18. AASHTO T283. Standard method of test for resistance of compacted for mix asphalt (HMA) of moisture induced damage.
19. AASHTO T308. Determining the asphalt binder content of HMA by the ignition method.
20. AASHTO T312. Preparing and determining the density of hot-mix asphalt (HMA) specimens by means of the Superpave gyratory compactor (AASHTO T 312-03).
21. AASHTO T315. Determining the rheological properties of asphalt binder using a dynamic shear rheometer (DSR).
22. AASHTO T316. Viscosity determination of asphalt binder using rotational viscometer.
23. AASHTO T321. Standard method of test for determining the fatigue life of compacted hot mix asphalt (HMA) subjected to repeated flexural bending.
24. AASHTO T322. Standard method of test for determining the creep compliance and strength of hot mix asphalt (HMA) using the indirect tensile test device.
25. AASHTO T342. Standard method of test for determining dynamic modulus of hot-mix asphalt concrete mixtures.
26. AASHTO TP62. Standard method of test for determining dynamic modulus of hot mix asphalt (HMA).

ASTM SPEC

1. ASTM C1097. Standard specification for hydrated lime for use in asphalt cement or bituminous paving mixtures.
2. ASTM D1559. Resistance to plastic flow of bituminous mixtures using Marshall apparatus.
3. ASTM D2171, ASTM D2170. Similar to the penetration test, the viscosity test is used to measure asphalt consistency. Two types of viscosity are commonly measured: absolute (ASTM D2171) and kinematic (ASTM D2170).

4. ASTM D2419. Standard test method for sand equivalent value of soils and fine aggregate.
5. ASTM D3497. The dynamic modulus test in triaxial compression has been used in the pavement community for many years (ASTM D3497). The test consists of applying an axial sinusoidal compressive stress to an unconfined or confined HMA cylindrical test specimen.
6. ASTM D4867/4867M. Standard test method for effect of moisture on asphalt concrete paving mixtures.
7. ASTM D5404. Recovery of asphalt from solution using the rotary evaporator to ensure that changes in the asphalt properties during the recovery process are minimized.
8. ASTM D6373. The performance-graded asphalt binder specifications are in ASTM D6373. See Table below.

MSMT SPEC

1. MAMT 251. Determination of moisture content of aggregates.
2. MSMT 410. Laboratory and field strip test for hot mix asphalt (HMA).
3. MSMT 412. Design procedure for asphalt mixes containing reclaimed asphalt pavement (RAP) and/or reclaimed asphalt shingles (RAS).
4. MSMT 414. Testing of asphalt release agents.
5. MSMT 453. Procedures for checking asphalt drum mix plants.
6. MSMT 733. Statistical analysis of material using quality level analysis for determination of pay factors.
7. MSMT 735. Procedure for evaluating bituminous materials for statistical compliance.

4.2.3 GAB/FASB and Base Specs

Table 4.9 Current MSHA Specs Related to GAB/FASB and Base Specs

GAB/FASB/Base	Maryland Spec
Conventional GAB	➢ AASHTO T90. PI≤6; ➢ AASHTO T104. Sodium Sulfate Soundness≤12%; ➢ ASTM D4791. Flat and elongated≤15%; ➢ AASHTO T96. LA abrasion≤50%. ➢ Existing test and measurement method: ASTM D2940, AASHTO T90, AASHTO T104, ASTM D4791, AASHTO T96
Conventional Base	➢ AASHTO T90. PI≤9; ➢ AASHTO T104. Sodium Sulfate Soundness≤12%; ➢ AASHTO T96. LA abrasion≤50%. ➢ Existing test and measurement method: MSMT 562, MSMT 251, MSMT 254, ASTM D140, AASHTO T2, AASHTO T27/ASTM C136, AASHTO T248, AASHTO T255, AASHTO M231, ASTM D2940, AASHTO T90, AASHTO T 104, AASHTO T96
Conventional FASB	➢ ASTM D1227, Type II, using ASTM D2939, modified by MSMT 423, Procedure B. ➢ Existing test and measurement method: MSMT 423, ASTM D1227, ASTM D2939 (Withdrawn 2012), ASTM D6690, AASHTO M6, AASHTO M85, AASHTO T48/ASTM D92, AASHTO T49/ASTM D5, AASHTO T53/ASTM D36, AASHTO T106, AASHTO T179/ASTM D1754

Table 4.10 Potential Areas of Revisions to MSHA Specs for GAB/FASB and Base Specs

GAB/FASB /Base	Revision
RCA in GAB	1. **Sodium sulfate soundness:** Suggest AASHTO T104. Sodium Sulfate Soundness≤36%. Sodium sulfate soundness degradation value of RCA is three times higher than that of natural coarse aggregate (Kolay and Akentuua 2014).

Table 4.10 Potential Areas of Revisions to MSHA Specs for GAB/FASB and Base Spec (continued)

GAB/FASB /Base	Revision
RCA in GAB	**2. Supplemental test and measurement method:** ➢ **California Bearing Ratio:** AASHTO T193/ASTM D1883. CBR of RCA is 40%-53% lower than that of natural crushed rock typically used in highway bases. The range is caused by different moisture contents in base materials, from penetration value of 2.54 mm to 5.08mm (Kolay and Akentuua 2014). ➢ **Water absorption:** ASTM C128/AASHTO T84. Water absorption of RCA is two times higher than that of natural coarse aggregate (Kolay and Akentuua 2014), and three times higher than that of limestone (Cooley and Hornsby 2012). ➢ **Moisture content:** ASTM D2216. When moisture content exceeds optimum moisture content (OMC) by 2%, permanent deformations double. Field compaction is suggested to make moisture content meet OMC (Aydilek 2015).
RAP in GAB	**Supplemental test and measurement method:** ➢ **California Bearing Ratio:** AASHTO T193/ASTM D1883. CBR of RAP is typically lower than natural aggregates. At a penetration value of 0.1", CBR reduced from 182% to 18% when RAP percentage ranged from zero to 100%. At a penetration value of 0.2", CBR reduced from 195% to 20% when RAP percentage increased from zero to 100% (Bennett and Maher 2005). ➢ **Permanent strain:** AASHTO TP46. Permanent strain of base varied from 0.68% to 5.63% as RAP percentage increased from zero to 100% (Bennett and Maher 2005).
RAP in FASB	**Supplemental test and measurement method:** ➢ **Fines content:** AASHTO T27/ASTM C136. Excess fines (i.e., more than 12% passing No.200 sieve) lead to worse dispersion of foamed asphalt and higher sensitivity to moisture. Therefore, the maximum fines content may be 12% passing No.200 sieve (Fu et al. 2010a).
FS in Base	1. **Cement content:** High cement ratios (>10% by weight) may make cement-stabilized FS more fragile, causing cracks in the pavement layer which can be reflected to upper layers. Therefore, cement content should be less than 10% (Gedik 2008). **2. Supplemental test and measurement methods:** ➢ **Permeability:** AASHTO T125 ➢ **Clay content:** AASHTO T112/ASTM C142 When bentonite clay content exceeds 6% by weight, permeability value of FS decreases significantly and ranges between 1×10^{-7} and 3×10^{-6} cm/sec. Therefore, bentonite clay content should be less than 6% (FIRST 2004).

4.2.3.1 Referenced Specs

AASHTO SPEC

1. AASHTO M231. Standard specification for weighing devices used in the testing of materials nineteenth edition.
2. AASHTO M6. Standard specification for fine aggregate for hydraulic cement concrete.
3. AASHTO M85. Standard specification for Portland cement (chemical and physical).
4. AASHTO T104. Soundness of aggregate by use of sodium sulfate or magnesium sulfate.
5. AASHTO T106. Standard method of test for compressive strength of hydraulic cement mortar using 50 mm or 2 in. cube specimens.
6. AASHTO T112. Standard test method for clay lumps and friable particles in aggregates.
7. AASHTO T125. Permeability of granular soils (constant head).
8. AASHTO T179. Standard method of test for effect of heat and air on asphalt materials (thin-film oven test).
9. AASHTO T193. Standard method of test for the California bearing ratio.
10. **AASHTO T2.** Sampling of aggregates.
11. AASHTO T248. Standard method of test for reducing samples of aggregate to testing size.
12. AASHTO T255. Standard method of test for total evaporable moisture content of aggregate by drying.
13. AASHTO T27. Sieve analysis of fine and coarse aggregate.
14. AASHTO T48. Standard method of test for flash and fire points by Cleveland open cup.
15. AASHTO T49. Penetration of bituminous materials.
16. AASHTO T53. Standard method of test for softening point of bitumen ring-and-ball apparatus.
17. AASHTO T84. Standard method of test for specific gravity and absorption of fine aggregate.
18. AASHTO T90. Standard method of test for determining the plastic limit and plasticity index of soils.
19. AASHTO T96. Standard method of test for resistance to degradation of small-size coarse aggregate by abrasion and impact in the Los Angeles machine.
20. AASHTO TP46. Standard test method for determining the resilient modulus of soils and aggregate materials.

ASTM SPEC

1. ASTM C128. Standard test method for relative density (specific gravity) and absorption of fine aggregate.
2. ASTM C136. Standard test method for sieve analysis of fine and coarse aggregates.
3. ASTM C142. Standard test method for clay lumps and friable particles in aggregates.
4. ASTM D1227. Standard specification for emulsified asphalt used as a protective coating for roofing.

Property	Type II Class 1		Type II Class 2	
	min	max	min	max
Weight per U.S. gallon, lb	8.2	9.0	9.2	9.5
Weight per litre, g	980	1080	1100	1140
Residue by evaporation, %	45	55	40	60
Ash content of residue, %	5	25	30	50
Water content, % [A]	...	55	40	60
Flammability			no tendency to flash or ignite	
Firm set, h	...	24	...	24
Heat test, 100 ± 3°C (212 ± 5°F)			no blistering, sagging or slipping	
Flexibility 0 ± $^{1}/_{2}$ °C (32 ± 1°F)			no cracking or flaking	
Resistance to water			no blistering or re-emulsification	
Direct flame test			coating shall char in place	

5. ASTM D140. Standard practice for sampling bituminous materials.
6. ASTM D2216. Standard test methods for laboratory determination of water (moisture) content of soil and rock by mass.
7. ASTM D2939. Standard test methods for emulsified bitumen used as protective coatings (withdrawn 2012).
8. ASTM D2940. Standard specification for graded aggregate material for bases or subbases for highways or airports.
9. ASTM D4791. Standard test method for flat particles, elongated particles or flat and elongated particles in coarse aggregate.
10. ASTM D6690. Standard specification for joint and crack sealants, hot applied, for concrete and asphalt pavements.

MSMT SPEC

1. MSMT 251. Determination of moisture content of aggregates.
2. MSMT 254. Field determination of the amount of stabilization agent in bases and subgrades.
3. MSMT 562. Certification of base course plant technician.

Table 901 A. Aggregate Grading Requirements Test Method AASHTO T27

Material	Sieve Size															
	2-1/2”	2”	1-1/2”	1”	3/4”	1/2”	3/8”	No. 4	No. 8	No. 10	No. 16	No. 30	No. 40	No. 50	No. 100	No .200
Graded Agg-Base(a)	-	100	95-100	-	70-92	-	50-70	35-55	-	-	-	12-25	-	-	-	0-8
Bank Run Gravel-Base	100	-	-	85-100	-	60-100	-	-	-	35-75	-	-	20-50	-	-	3-20

Note: (a) To establish target values for design.

4.2.4 Bricks/LWA Specs

Table 4.11 Current MSHA Specs Related to Bricks/LWA Specs

Brick/LWA	Maryland Spec
Conventional brick	1. Brick for paving shall conform to the requirements of ASTM (C62, Grade SW) for building brick or shale, with the following modifications: a. The absorption limits shall be from 5%- 12% for the average of five bricks. b. The compressive strength shall not be less than 41.4 MPa [6,000 psi]. c. The modulus of rupture shall not be less than 6.9 MPa [1,000 psi]. d. The bricks shall be No. 1, water struck type for paving. 2. The bricks shall be 57 mm x 90 mm x 190 mm [2¼ in x 3¾ in x 8 in] with permissible variations not to exceed 1.5 mm [1/16 in] in depth, 3 mm [1/8 in] in width and 6 mm [1/4 in] in length. 3. Before ordering new brick, samples shall be submitted in whole straps to show color range. 4. Existing test and measurement method: AASHTO M144, ASTM C62

Table 4.12 Potential Areas of Revisions to MSHA Specs for Bricks/LWA Specs

Brick/LWA	Revision
DM in brick	**Supplemental test and measurement method:** ➢ **Novosol® amended river sediment bricks** **Firing shrinkage:** ASTM C326. Firing shrinkage of Novosol® amended river sediment bricks (10%) is higher than that of standard bricks (7%) (Samara et al. 2009). ➢ **Water treatment residue brick** **Sintering temperature:** Water treatment residue brick requires higher sintering temperature to meet the same bulk density, compared to excavation waste soil brick. To achieve G_s= 1.8, waste treatment residue brick requires at least 1050℃, while excavation waste soil brick only needs 800℃ of sintering temperature (Huang et al. 2005).

4.2.4.1 Referenced Specs

AASHTO SPEC

1. AASHTO M144. Standard specification for calcium chloride.

ASTM SPEC

1. ASTM C326. Standard test method for drying and firing shrinkages of ceramic whiteware clays.
2. ASTM C62. Standard specification for building brick (solid masonry units made from clay or shale).

4.2.5 Drainage and Fill Specs

Table 4.13 Current MSHA Specs Related to Drainage and Fill Specs

Drainage/Fill	Maryland Spec
Conventional borrow	➢ Select borrow: A-2, A-3, or A-2-4 material as specified in the Contract Documents. The maximum dry density shall not be less than 105 lb/ft^3. ➢ Common borrow: A maximum dry density of no less than 100 lb/ft^3. ➢ Existing test and measurement methods: AASHTO T180 (Method C unless material with more than 35% retained on the No. 4 sieve, then Method D), AASHTO T27
Conventional fill material	➢ AASHTO T90. Performance Index$\leq$ 6; ➢ AASHTO T104. Sodium Sulfate Soundness$\leq$ 12%; ➢ ASTM D4791. Flat and elongated$\leq$ 15%; ➢ AASHTO T96. LA abrasion$\leq$50%. ➢ Existing test and measurement methods: ASTM D2940, AASHTO T90, AASHTO T104, ASTM D4791, AASHTO T96, AASHTO T27.
RAP in drainage/fill	➢ Allow in drainage. ➢ Less than 15%. ➢ Meet section TC-6.10; Need written approval by engineer. ➢ Prohibited for use within 1 ft of the surface in any area to be vegetated. ➢ Existing test and measurement methods: AASHTO T27

Table 4.13 Current MSHA Specs Related to Drainage and Fill Specs (continued**)**

Drainage/Fill	**Maryland Spec**
RCA in drainage/fill	➢ Allow in drainage. ➢ Soundness loss by five cycles of the magnesium sulfate test≤18%. ➢ Meet section TC-6.10; Need written approval by engineer. ➢ Prohibited for use within 1 ft of the surface in any area to be vegetated. ➢ Existing test and measurement methods: AASHTO T104, AASHTO T27

Table 901 A. Aggregate Grading Requirements Test Method AASHTO T27

Material	Sieve Size															
	2-1/2"	2"	1-1/2"	1"	3/4"	1/2"	3/8"	No. 4	No. 8	No.10	No.16	No.30	No.40	No.50	No.100	No.200
Crusher Run Aggregate CR-6	-	100	90-100	-	60-90	-	-	30-60	-	-	-	-	-		-	0-15

Note: Recycled asphalt pavement may be used as a component not to exceed 15% and is not subject to aggregate physical property requirements in TABLE 901 B.

Table 4.14 Potential Areas of Revisions to MSHA Specs for Drainage and Fill Specs

Drainage/Fill	**Revision**
RCA in drainage/fill	**1. Gradation:** Suggest No.4 gradation for drainage application, Table 1. Drainage material containing 4% fine RCA (meet No.4 gradation) shows significant decrease in drainage capacity with a reduction of 2.5-9 cm/s^2 in flow rate, as value of head varied from 3 in. to 30 in. Therefore, fine RCA should not exceed 4% by weight (Nam et al. 2014). **2. Los Angeles abrasion loss**: Suggest AASHTO T96. LA abrasion≤65%. RCA (meet) is about 15% higher than that of limestone (Nam et al. 2014). **3. Supplemental test and measurement methods:** ➢ **Flowability:** ASTM D4832**.** RCA replacing concrete sand in flowable fill requires more water to meet given flow value. To achieve an 8 in. final flow value, 150-250 lb/yd^3 more water is required when the percentage of RCA varied from 50% to 100% by weight (Lim et al. 2003).

Table 4.14 Potential Areas of Revisions to MSHA Specs for Drainage and Fill Spec (continued)

Drainage/Fill	Revision
RAP in drainage/fill	**Supplemental test and measurement methods:** ➢ **Compaction:** ASTM D698. Compressibility of RAP shows high sensitivity to temperature. Secondary compression ratio of RAP increased about 14 times as temperature was raised from 22 ℃ to 35℃ (Soleimanbeigi and Edil 2015). ➢ **Creep:** ASTM D1557. RAP has a higher potential of creep failure. Creep parameters for RAP is generally less than 1.0, which is comparable to clays with creep parameter of 0.7 (Rathje et al. 2006).
RAP in embankment	**Supplemental test and measurement methods:** ➢ **Collapse potential:** ASTM D4546. RAP has higher potential of collapse in wet conditions than conventional fill material. Collapse index of RAP is up to 1.5%, while that of conventional material is about 0.2% (Rathje et al. 2006). ➢ **Creep:** ASTM D1557. RAP has higher potential of creep failure. Creep parameter for RAP is generally less than 1.0, which is comparable to clays with a creep parameter of 0.7 (Rathje et al. 2006).
FS in drainage/ embankment	1. **Cement content:** High cement ratios (>10% by weight) may make cement-stabilized FS more fragile, causing cracks in the pavement layer which can be reflected to upper layers. Therefore, cement content should be less than 10% (Gedik 2008). 2. **Supplemental test and measurement methods:** ➢ **Permeability:** AASHTO T125, ASTM D5084. ➢ **Clay content:** ASTM C142/AASHTO T112 When bentonite clay content exceeds 6% by weight, permeability value of FS decreases significantly and ranges between $1x10^{-7}$ and $3x10^{-6}$ cm/s. Therefore, bentonite clay content should be less than 6% (FIRST 2004).
DM in flowable fill	**Supplemental test and measurement methods:** ➢ **Rubber amended dredged material** **Flowability:** ASTM D4832. Flowability of DM decreases with increasing rubber content. Based on test results, flowability with a rubber content of zero, 25% and 50% was satisfied (20±5cm) when water content was 140-160%, 140-180% and 160-200%, respectively (Kim and Kang 2011).

Table 4.14 Potential Areas of Revisions to MSHA Specs for Drainage and Fill Specs (continued)

Drainage/Fill	Revision
DM in embankment	**Supplemental test and measurement methods:** ➢ **Crushed glass (CG) amended dredged material** • **Cone penetrometer test:** ASTM D3441. CG-DM blends are not as strong as natural coarse aggregates (i.e., sand). The CPT value of the strongest embankment 80/20 CG-DM blend was six MPa (Grubb et al. 2008, Grubb et al. 2013). ➢ **Steel slag fines (SSF) amended dredged material** • **Compaction:** ASTM D698. The addition of SSF requires more consolidation (i.e., compression) to obtain enough compressibility. Coefficient of consolidation decreases from 0.28 to 0.12 as SSF is increased from zero to 100% by weight. Coefficient of reconsolidation decreased from 0.04 to 0.008 as SSF is increased from zero to 100% by weight (Malasavage et al. 2012). ➢ **Rubber amended dredged material** • **Unconfined compressive strength:** ASTM D2166. Unconfined compressive strength decreases linearly from about 440 kPa to about 180 kPa, as rubber content is increased from zero to 100% by weight (Kim and Kang 2011). ➢ **Air-foam amended dredged material** • **Unconfined compressive strength:** ASTM D2166. Unconfined compressive strength decreases almost linearly from 310 kPa to 50 KPa as air foam content is increased from zero to 3% by weight (Kim et al. 2010).

Suggested Aggregate Gradation for Drainage (ASTM D442; Nam et al. 2014)

Material	Sieve Size													
	2-1/2”	2”	1-1/2”	1”	3/4”	3/8”	No. 4	No. 8	No.10	No.16	No.30	No.40	No.50	No.100	No.200
No.4 Gradation	-	100	90-100	20-55	0-15	0-5	-	-	-	-	-	-	-	-	-

4.2.5.1 Referenced Specs

AASHTO SPEC

1. AASHTO T104. Soundness of aggregate by use of sodium sulfate or magnesium sulfate.
2. AASHTO T125. Permeability of granular soils (constant head).
3. AASHTO T180. Standard method of test for moisture density relations of soils using a 4.54 kg (10 lb) rammer and a 457 mm (18 in.) drop.
4. AASHTO T27. Standard method of test for sieve analysis of fine and coarse aggregates.
5. AASHTO T90. Determining the plastic limit and plasticity index of soils.

ASTM SPEC

1. ASTM D1557. Standard test methods for laboratory compaction characteristics of soil using modified effort (56,000 ft-lbf/ft^3 (2,700 kN-m/m^3)).
2. ASTM D2166. Standard test method for unconfined compressive strength of cohesive soil.
3. ASTM D3441. Standard test method for mechanical cone penetration tests of soil.
4. ASTM D4546. Standard test methods for one-dimensional swell or collapse of soils.
5. ASTM D4832. Standard test method for preparation and testing of controlled low strength material (CLSM) test cylinders.
6. ASTM D5084. Standard test method for measurement of hydraulic conductivity of saturated porous materials using a flexible wall permeameter.
7. ASTM D698. Standard test methods for laboratory compaction characteristics of soil using standard effort (12 400 ft-lbf/ft^3 (600 kN-m/m^3)).

Chapter 5: Life Cycle Analysis for Recycled Highways

5.1 Evaluation of Recycled Materials in Highway Application by PaLATE

5.1.1 Introduction

PaLATE is a popular pavement life-cycle assessment tool, which evaluates the economic and environmental effects of a highway project from initial construction to maintenance and, eventually, to the project's design life (Horvath 2004). The economic module within PaLATE predicts the life cycle cost of activities and materials (i.e., recycled materials) in a highway project. The environmental module estimates energy and water consumption, air emission (i.e., greenhouse gas) and fume pollution, as well as the discharge of metals (e.g., mercury and lead) and organics (e.g., PAH). The primary objectives of this study were to investigate the benefits of using recycled materials in highway applications and to examine the influence that different recycled material content in pavement applications have on cost and the environment. Pavements made with recycled materials are compared with conventional pavements containing only virgin materials. The results from PaLATE can assist on how to best utilize recycled materials and identify the optimum substitution rate in highway applications.

5.1.2 Project Description and Model Creation

5.1.2.1 Pavement Design

To model the life cycle of pavements, dimensions (i.e., the width, length, and depth of each layer) of the pavement structure should first be defined. According to the literature, the minimum lane width for most U.S. and state highways is 12 feet (3.7 m); therefore, it is assumed that the width for two lanes (two directions) is 24 feet. Shoulders cannot be included in the analysis because of their variability in width, thickness, and composition. For comparison a 1 mile pavement section can be used to represent the bases for the analysis. In regards to layer thicknesses, concrete and asphalt pavements have different requirements due to the differences in pavement mechanics and load distribution behavior. In pavement design, thickness of concrete, asphalt and base layers were selected at 8 in., 4 in. and 4 in., respectively, representing typical cross sections in the US. Table 5.1 presents the design pavement inputs for the analysis.

Once pavement design considerations are established, the volume of construction materials, their source (hauling distance), and pertinent construction and maintenance activities can be defined. The density of materials is shown in Table 5.2. The project site was assumed to be 10 miles away from the RCA, RAP and foundry sand suppliers, 30 miles away from the quarry of virgin aggregates, 5 miles away from bitumen plants, 10 miles from cement plants and 30 miles away from disposal landfills. The transportation distance of in-place recycling is assumed to be zero.

The Asphalt Pavement Alliance (APA) recommends that the service life of pavements should be no less than 40 years and should include at least one rehabilitation activity (APA 2010). FHWA recommends a minimum of 35 years for the analysis period. For these reasons, a service life of 40 years was selected for the analysis of this study.

Table 5.1. Summary of dimensions design.

Design	Material	Width (feet)	Length (mile)	Depth (inches)	Volume (yd^3)
PCC layer	PCC with RCA	24	1	8	3129
	PCC with RAP	24	1	8	3129
	PCC with FS	24	1	8	3129
	Conventional PCC	24	1	8	3129
HMA layer	HMA with RAP	24	1	4	1564.5
	HMA with RCA	24	1	4	1564.5
	HMA with FS	24	1	4	1564.5
	Conventional HMA	24	1	4	1564.5
Base layer	GAB with RCA	24	1	4	1564.5
	GAB with RAP	24	1	4	1564.5
	FASB with RCA & RAP	24	1	1.4	547.6
	Base with FS	24	1	4	1564.5
	Conventional GAB	24	1	4	1564.5
Embankment	Embankment with FS	24	1	200	105382.7
	Conventional Embankment	24	1	200	105382.7

Note: The slope ratio for embankment is typically 2H : 1V (Ramanathan et al. 2015).

Table 5.2. Density of materials suggested by PaLATE.

Material	Density (tons/yd^3)
RCA	1.88
RAP	1.85
Foundry Sand	1.50
Cement	1.27
Water	0.84
Bitumen	0.84
Virgin aggregate	1.25
FDR mixture	1.83

Note: Though studies provided different density value for these materials, the density listed here was used in the calculation. The “ton” is metric ton.

Treatment life is also a part of the economic assessment. While the time to first rehabilitation should be based on actual construction and pavement management data, timing may also be based on experience and observed performance. Information collected by APA (2010) from all 50 state highway agencies indicated that 20 years may be a reasonable period between initial construction and first rehabilitation, while the average interval was 15.7 years. FHWA (2000) also indicated that most asphalt overlays can last for over 15 years and many can work satisfactorily for more than 20 years. In this study, a 20-year interval was chosen between construction and the first rehabilitation for an asphalt pavement.

MDOT and MnDOT reported that concrete pavements normally have an average life span of 27.5 years. In a report by Weland and Muench (2010), a span of 20 years was suggested for diamond grinding of PCC overlay. ACPA (1998) indicated that PCC overlay has a service life of 25 years or more. A rubblized PCC base with an asphalt overlay has an average service life of 22 years (ACPA 1998). In this study, a 20-year interval was chosen between construction and the first rehabilitation for concrete pavement.

The base life span is assumed to be the same with the HMA overlay, since Full-depth Reclamation (FDR) will reclaim part of the base materials. Embankments have an expected life span of 19.2 years for a minimum traffic volume, and 18.7 years for a maximum traffic volume (Frangopol and Tsompanakis 2014). The life span of embankment is assumed at 20 years, which is the period between initial construction and first rehabilitation. The details of treatment life and activities are listed in Table 5.3 and Table 5.4, respectively.

Table 5.3. Summary of treatment life.

Types	Treatment Life (years)
Asphalt pavement	0, 20, 40
Concrete pavement	0, 20, 40
Base	0, 20, 40
Embankment	0, 20, 40

5.1.2.2 Initial Construction and Maintenance

Initial construction activities normally include installing pavement and hauling raw or processed materials to the site. Maintenance activities may be more complex to consider in the analysis, since they can include frequent repairs (i.e., patching, micro-surfacing, crack sealing, etc.). In this study, minor repairs are not considered while major rehabilitation it is. Rehabilitation activities include the handling of existing materials (i.e., landfill, recycling), hauling new pavement materials and paving operations. Table 5.4 lists the specific activities conducted during initial construction and maintenance. These activities vary, depending on the pavement type and base.

Rubblization is the process of breaking an existing Portland Cement Concrete slab into small fragments, ranging from sand-size particles to coarse aggregate particles that may be 100 mm (4 in.) to 200 mm (8 in.). Studies indicated that rubblized roads with an asphalt overlay have an average service life of 22 years and provide more than a 60% cost savings, compared to the tear out and replacement of concrete (ACPA 1998). Furthermore, the useful life of replaced concrete base is 80% shorter than the useful life of the rubblized concrete base (ACPA 1998). Therefore, rubblization was selected in this study as the rehabilitation option, with a service life of 20 years (ACPA 1998).

The concrete layer can be rehabilitated by either an unbound PCC overlay or removing and replacing the entire PCC slab (NAPA 2014). Weland and Muench (2010) proposed three methods to rehabilitate PCC pavements: replacing with a new PCC pavement, replacing with a new asphalt pavement or recycling the PCC pavement by a CSOL (crack, seal and overlay the existing PCC pavement with HMA) process. A CSOL process is more environment-friendly compared to replacing with a new pavement, since the old pavement does not need to be removed and landfilled.

FHWA (2015) stated that the full-depth reclamation (FDR) of asphalt road normally works well for 8-12 years with thin surface treatment and 15-20 years or longer with a hot asphalt concrete pavement layer. FDR with emulsified asphalt performs well for 7-10 years with thin surface treatment and 15-20 years or longer with a hot asphalt concrete overlay. Considering the potential advantages of FDR over conventional pavement replacement with new materials, FDR was adopted in the rehabilitation stage for recycled pavements.

Table 5.4. Activities in construction and maintenance.

Pavement/Base	**Initial Construction**		**Maintenance (Rehabilitation)**	
	Conventional	**Recycled**	**Conventional**	**Recycled**
Concrete pavement	Install concrete pavement; Virgin material from quarry	Install concrete pavement; RCM from concrete plant; RAP from asphalt plant; FS from factory	Rubblization	Rubblization
Asphalt pavement	Install asphalt pavement; Virgin material from quarry	Install asphalt pavement; RCA from concrete plant; RAP from asphalt plant; FS from factory	From site to landfill; Virgin material from quarry; Install asphalt pavement	FDR; Install asphalt pavement
Base	Install subbase & embankment; Virgin material from quarry	Install subbase & embankment; RCM from concrete plant; RAP from asphalt plant; FS from factory	Install subbase & embankment; Virgin material from quarry	FDR; Install subbase & embankment
Embankment	Install subbase & embankment; Virgin material from quarry	Install subbase & embankment; FS from factory	Install subbase & embankment; Virgin material from quarry	FDR; Install subbase & embankment

Maintenance costs are normally estimated based on procurement records. APA (2011) indicated that maintenance costs estimated in a Life Cycle Cost Analysis (LCCA) procedure should follow the historical documentation of actual pavement activities and expenditures. In this study, life cycle cost only comprises the expense in initial construction and first rehabilitation. The costs for minor pavement repairs (i.e., patching, crack sealant, etc.) were not considered, since these costs are small compared to the actual construction cost. A summary of the construction costs are provided in Table 5.5.

Table 5.5. Summary of construction costs.

Treatment	Unit	Unit Cost	Reference
Install 4-in. asphalt paving	$/yd^2	16.79	RS Means, 2015
Install 8-in. concrete paving	$/yd^2	34.44	NAPA, 2014
Rubblization	$/yd^2	1.5	
Install subbase & embankment	$/yd^2	16.50	RS Means, 2015
From site to landfill	$/ton	56.9	OC Waste & Recycling, 2015
FDR	$/ton	4.60	FHWA, 1998
FDR-emulsified asphalt	$/ton	5.45	FHWA, 1998

Environmental effects are estimated in PaLATE by adding the consumption and emission in each stage of pavement construction and maintenance. Energy use and air emissions are based on the productivity, fuel consumption rate and the engine size of the construction equipment. HTP (human toxic potential) is a normalized risk factor reflecting the potential harm that a chemical can cause when released into the water or air (Hertwich et al. 2001). HTP and RCRA (Resource Conservation and Recovery Act) hazardous waste are measured based on the type of materials and activities. In this study, various construction equipment were chosen during the construction and maintenance process. Table 5.6 provides information on the type, productivity and fuel consumption of this equipment.

Table 5.6. Summary of equipment characteristics.

Equipment	Engine capacity(hp)	Productivity (ton/h)	Fuel Consumption (l/h)	Fuel Type
Slipform Paver	106	564	19.7	Diesel
Texture curing machine	70	187	20.2	Diesel
Pneumatic roller	100	668	26.1	Diesel
Tandem roller	125	285	32.7	Diesel
Excavator	131	315	34.2	Diesel
Vibratory soil compactor	174	1832	27.6	Diesel
Multi head breaker	350	520	76.5	Diesel
Asphalt road reclamation	670	4800	120	Diesel
Excavator	131	225	34.2	Diesel
Wheel loader	135	225	35.3	Diesel
Dozer	285	225	71.4	Diesel
Generator	519	225	98.4	Diesel

5.1.2.3 Mix Design

The substitution rates of recycled materials were determined according to the literature review. The suggested mix designs proposed by various studies are listed in Table 5.7 for PCC (by weight) and Table 5.9 for HMA (percentage by weight). Since PaLATE's input for "initial construction" and "maintenance" requires the volume of each material, weight of materials should be transferred to volume. Table 5.8 and Tables 5.10-5.12 present the volume of materials used in PCC, HMA, base, and embankment, respectively.

Table 5.7. Suggested mix design for PCC.

Category	**RCA (lb/yd^3)**	**RAP (lb/yd^3)**	**FS (lb/yd^3)**	**Coarse Aggregate (lb/yd^3)**	**Fine Aggregate (lb/yd^3)**	**Cement (lb/yd^3)**	**Water (lb/yd^3)**	**Reference**
PCC with RCA 50%[1]	845.9	-	-	979.1	1252.7	535	214	Volz et al. 2014
PCC with RCA 100%[1]	1650.5	-	-	-	1441.6	535	192.6	Volz et al. 2014
PCC with RAP 40%[2]	-	1150	-	1072	788	500	250	Hossiney 2012
PCC with RAP 100%[2]	-	2922	-	-	-	500	250	Hossiney 2012
PCC with FS 20%[3]	-	-	207	1941	726	677	298	Singh and Siddique 2012
Conventional PCC	-	-	-	1958.2	1252.7	535	214	Volz et al. 2014

Note: 1. RCA takes up the percentage of coarse aggregates by weight.

2. RAP takes up the percentage of both coarse and fine aggregates by weight.

3. FS takes up the percentage of fine aggregates by weight.

Table 5.8. Volumes of materials in PCC layer for PaLATE input.

Category	**RCA (yd^3)**	**RAP (yd^3)**	**FS (yd^3)**	**Coarse Aggregate (yd^3)**	**Fine Aggregate (yd^3)**	**Cement (yd^3)**	**Water (yd^3)**
PCC with RCA 50%	494.4	-	-	794.8	1098.3	463.1	278.5
PCC with RCA 100%	1023.2	-	-	-	1345.5	491.3	266.0
PCC with RAP 40%	-	710.3	-	907.4	719.7	450.6	341.1
PCC with RAP 100%	-	2177.8	-	-	-	541.3	409.9
PCC with FS 20%	-	-	140.8	1476.9	597.6	547.6	366.1
Conventional PCC	-	-	-	1451.9	1001.3	422.4	253.4

Note: 1. Total volume of PCC is 3129 yd^3, as shown in Table 5.1.

2. Air content is ignored in the volume calculation.

3. Coarse aggregate and fine aggregate are combined as virgin aggregate in PaLATE input.

Table 5.9. Suggested mix design for HMA.

Category	**RAP% by weight[2]**	**RCA% by weight[2]**	**FS% by weight[2]**	**Bitumen% by weight**	**Virgin aggregate% by weight**	**Reference**
HMA with 25% RAP[1]	24.2%	-	-	4.4%	71.3%	Shirodkar et al. 2011
HMA with 35% RAP[1]	33.7%	-	-	4.4%	61.8%	Shirodkar et al. 2011
HMA with 45% RCA[1]	-	42.1%	-	6.5%	51.4%	Wong et al.2007
HMA with 10% FS[1]	-	-	9.5%	4.8%	85.7%	Bakis et al. 2006 and Braham 2002
Conventional HMA	-	-	-	5.3%	94.7%	Wong et al.2007

Note: 1. Recycled materials are based on the percentage (i.e., 25%) of total aggregates by weight.

2. Recycled materials are based on the percentage (i.e., 24.2%) of HMA mixture (bitumen is included) by weight.

Table 5.10. Volumes of materials in HMA layer for PaLATE input.

Category	RAP (yd^3)	RCA (yd^3)	FS (yd^3)	Bitumen (yd^3)	Virgin aggregate (yd^3)
HMA with 25% RAP	272.2	-	-	109.5	1184.3
HMA with 35% RAP	391.1	-	-	112.6	1060.7
HMA with 45% RCA	-	491.3	-	170.5	902.7
HMA with 10% FS	-	-	123.6	111.1	1329.8
Conventional HMA	-	-	-	120.5	1444.0

Note: 1. Total volume of HMA is 1564.5 yd^3, as shown in Table 5.1.

2. Air content is ignored in the volume calculation.

Table 5.11. Volumes of materials in base layer for PaLATE input.

Category	RAP (yd^3)	RCA (yd^3)	FS (yd^3)	Emulsified Asphalt (yd^3)	Cement (yd^3)	Virgin aggregate (yd^3)	Reference
Conventional GAB[1]	-	-	-	-	-	1564.5	Aydilek et al. 2015
GAB with 100% RCA[1]	-	1564.5	-	-	-	-	
GAB with 100% RAP[1]	1564.5	-	-	-	-	-	Bennett and Maher 2005
Cement-stabilized FS base[2]	-	-	1383.0	-	181.5	-	Gedik 2008
FASB with 40% RAP & 60% RCA[3]	152.8	306.2	-	345.0	-	-	Schwartz and Khosravifar 2013

Note: 1. Total volume of GAB is 1564.5 yd^3, as shown in Table 5.1.

2. Water usage is ignored in this calculation. The base material consists of 10% cement and 90% FS by weight. Total volume of cement-stabilized FS base is 1564.5 yd^3, as shown in Table 5.1.

3. The optimum asphalt content is 3% by weight. RAP and RCA replace 40% and 60% of natural aggregates by weight, respectively. Total volume of FASB is 547.68 yd^3, as shown in Table 5.1.

Table 5.12. Volumes of materials in embankment for PaLATE input.

Category	FS (yd^3)	Virgin aggregate (yd^3)	Reference
Embankment with FS[1]	105382.7	-	Yazoghli-Marzouk et al. 2014
Conventional Embankment[2]	-	105382.7	-

Note: 1. The optimum moisture content of FS is about 12.5%. Water usage is ignored in this calculation. The optimal density of FS (1.34 ton/yd^3) is a little lower than the value listed in Table 5.2. For consistency of the analysis the density shown in Table 5.2 is used.

2. Total volume of the designed embankment is 105382.7 yd^3.

PaLATE provides two different methods for performing life cycle cost analysis. The first method includes a sum of the cost of each activity. The cost of each activity is calculated by multiplying the unit cost of work (Table 5.5) with the total amount of work. The second method includes a sum of the cost of materials. The cost of each material (Table 5.10) is calculated by multiplying the unit cost of material with the total amount of materials. The latter method was utilized in the current study, since the activities of constructing or maintaining pavements with recycled materials are the same for each application (i.e., PCC, HMA, etc.).

Table 5.13. Cost of labor, equipment, and materials.

Material	Unit	Unit Price
RAP	\$/ton	6.18
RCA	\$/ton	6.23
FS	\$/ton	9.72
Virgin Aggregate	\$/ton	30
Cement	\$/ton	98.5
Bitumen	\$/ton	534
Water	\$/gal	6.7
Labor	\$	16,000
Equipment	\$	12,000
Overhead & Profit	\$	11,000

5.1.2.4 Economic Parameters

A discount rate is used in calculating the present value and annual equivalent value of a project. Discount rate typically varies from 1%- 8%. The selection of a discount rate can significantly affect the final results. Adjusting the discount rate can be a good solution for dealing with the uncertainty associated with future interest rates and inflation. Too high a discount rate would overemphasize the importance of the initial cost. According to a survey conducted by APA (2010), an average discount rate of 3.7% is used in the U.S. with a range between 2.3% and 6.0%. Twenty-three states used a discount rate of 4% when performing life cycle cost analysis (APA 2011). In this study, discount rates of 3% and 6% were used.

5.1.3. Results and discussion

5.1.3.1 Result of Economic Cost

◆ PCC

The results of LCCA are shown in Figure 5.1-5.3 for PCC layer, HMA layer and base, respectively. In this study, two discount rates were used (3% and 6%) in estimating net present value (NPV) and annual equavilent worth. A higher NPV or higher annual equivalent worth indicates higher cost. The range between NPV1 and NPV2 is the total cost with a deviation due to an uncertainty in inflation.

As seen in Figure 5.1, cost of PCC made with recycled materials are comparable to that of conventional PCC. PCC layer containing 20% foundry sand has the highest cost, due to high usage of cement and water (shown in Table 5.8). Cement has the highest unit price among all the components of PCC. The higher usage of cement will definitely raise the cost of PCC. Water has a low unit price, but high water usage will elevate the cost significantly. In addition, foundry sand replaces only a small amount of fine aggregates (20%) in PCC. The small cost savings contributed by the low price of recycled foundry sand is offset by the high cost of cement and water. However, a substitution rate greater than 20% restricts the mechanical performance (Singh and Siddique 2012). The study of Bhat and Lovell (1997) indicated that if clean sand was replaced by FS, which requires about 50% more cement, cost could still be reduced by 25% to $6.44/ton. The divergence may be due to the higher price of FS and cement used in this study or the different mix design.

Cost of PCC made with RAP is 10% higher than that of conventional PCC. The higher cost is the result of higher cement content, as shown in Table 5.8. As the RAP replacement rate increases from 40% to 100%, life cycle cost increases a little. The cost savings contributed by the low price of RAP is offset by the increased usage of cement (increased by about 65 yd^3). The addition of RAP generally worsens the performance of concrete (Hossiney 2012), but RAP added at reasonable amounts (40% by weight) can meet the requirements of mechanical properties. As a result, 100% RAP replacement should not be used in producing PCC.

PCC incorporating RCA can reduce the life cycle cost slightly at high RCA content (Figure 5.1). Even though PCC with 40% RCA replacement has a higher cost than conventional PCC, 100% RCA replacement reduces the cost by about 6% ($27,000~$37,000 per mile). For a project of 150 mile-long pavement, $4.8 millon can be saved; This is consistent with NCHRP 435, which indicated that costs saved from recycling PCC are as high as $5 million on a single project. When using RCA replacement at a certain ratio, the cost savings contributed by low price of RCA compensates the increased cost in cement and water usage.

◆ HMA

The lifecycle cost for HMA layer in asphalt pavement is presented in Figure 5.2. Asphalt pavements were rehabilitated by FDR, with the exception of conventional asphalt pavement. RAP addition significatly reduces the cost of HMA layer (by about 40%), with a slight reduction associated with increasing RAP content. The reason for the cost reduction can be attributed to the lower unit price of RAP compared to virgin aggregates. In addition, FDR technique greatly reduces the cost of maintenance activities, such as landfilling the waste asphalt concrete and transportation of new materials to the site. Finally, RAP requires less bitumen in producing HMA compared to virgin aggregates. Since higher bitumen is required by higher RAP content, cost reduces slightly when RAP content raises from 25% to 35%.

In Figure 5.2, HMA made with RCA and FS have a lower cost compared to conventional HMA (by about 23%), largely due to the FDR technique used in the rehabilitation stage. FIRST (2003) also indicated that 10% gray iron FS used in HMA showed a $50,000 savings when using 4,000 tons of FS. HMA with 45% RCA replacement has a similar cost to 10% FS replacement (the cost reduced is by about 23%). A higher percentage of RCA replacement does not reduce more cost, since RCA requires a large amount of bitumen in producing HMA (Table 10). FS has little influence on the cost, due to a much small replacement ratio of 10%, though 10% is the optimum replacement ratio in respect of mechanical poperties (Bakis et al. 2006 and Braham 2002).

◆ Base

The life-cycle cost for base layer in asphalt pavement is presented in Figure 5.3. Bases were rehabilitated by full-depth reclamation, with the exception of the conventional GAB base. Recycled GAB base either with 100% RCA or 100% RAP show the lowest cost (by about 50%) due to the lower price of recycled materials. Similarly, TFHRC (2010) indicated that incorporating 20%-50% RAP into base mixtures can save 14-34% per tonnage. For stabilized base, cement usage elevates the total cost significatly; however, the cost for cement-stabilized base with FS is still lower than conventional GAB (with a cost reduction of about 30%). FASB also shows great cost savings (about 47%), though emulsified asphalt costs more.

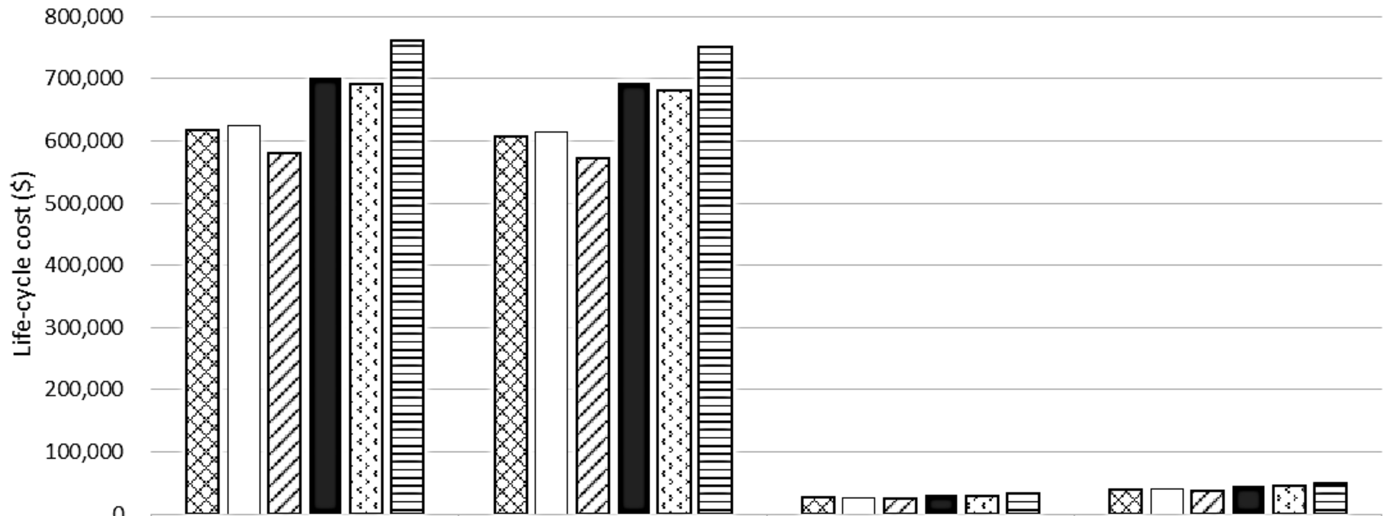
◆ Embankment

Table 5.14 lists the life cycle cost of an embankment constructed with two different geomaterials. Embankment with 100% foundry sand exhibits greater cost savings compared to a conventional embankment (by about 60%), due to the lower price of FS.

Table 5.14. Life-cycle cost for embankment made with recycled materials.

Materials	Virgin Embankment	100%FS|Embankment
NPV1	6,200,489	2,447,785
NPV2	5,235,218	2,066,722
Annual Cost 1	268,248	105,897
Annual Cost 2	347,941	137,358

Note: NPV=net present value; Annual cost=annual equivalent worth. NPV 1 and Annual cost 1 are calculated at discount rate of 3%. NPV 2 and Annual cost 2 are calculated at discount rate of 6%.

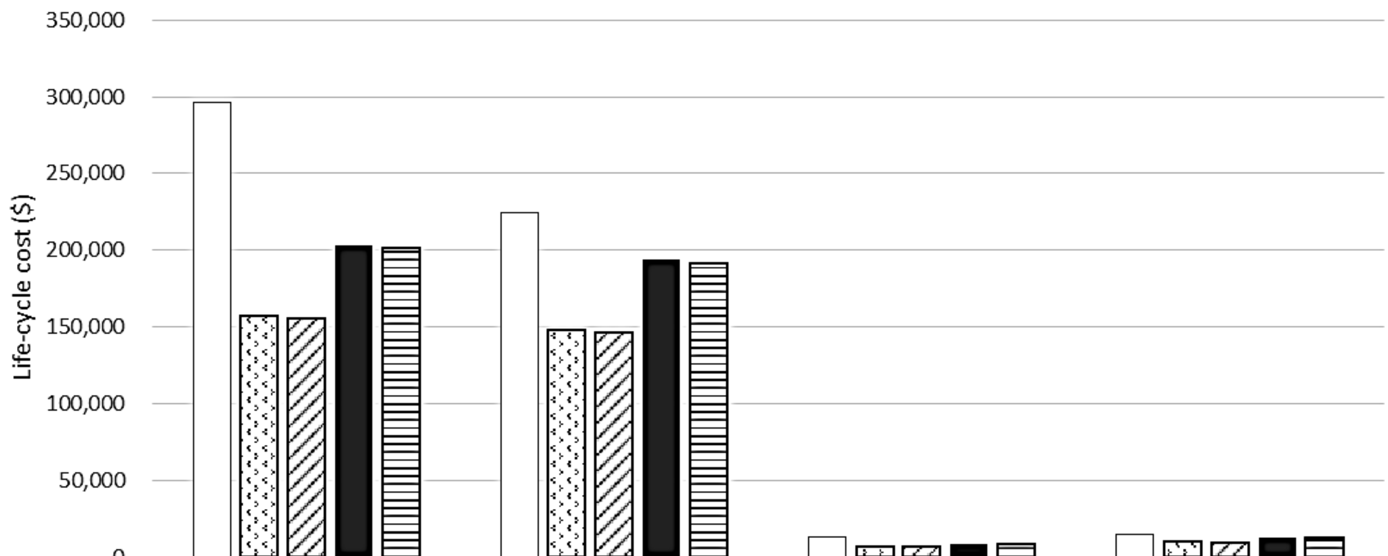


	NPV1	NPV2	Annual Cost1	Annual Cost2
Virgin	617,071	607,638	26,696	40,385
RCA 50%	624,041	614,608	26,997	40,848
RCA 100%	580,373	570,940	25,108	37,946
RAP 100%	702,450	693,017	30,390	46,059
RAP 40%	690,898	681,465	29,890	45,291
FS 20%	760,471	751,038	32,900	49,915

PCC

Figure 5.1. Life-cycle cost for PCC layer made with recycled materials.

Note: NPV=net present value; Annual cost=annual equivalent worth. NPV 1 and Annual cost 1 are calculated at discount rate of 3%. NPV 2 and Annual cost 2 are calculated at discount rate of 6%.

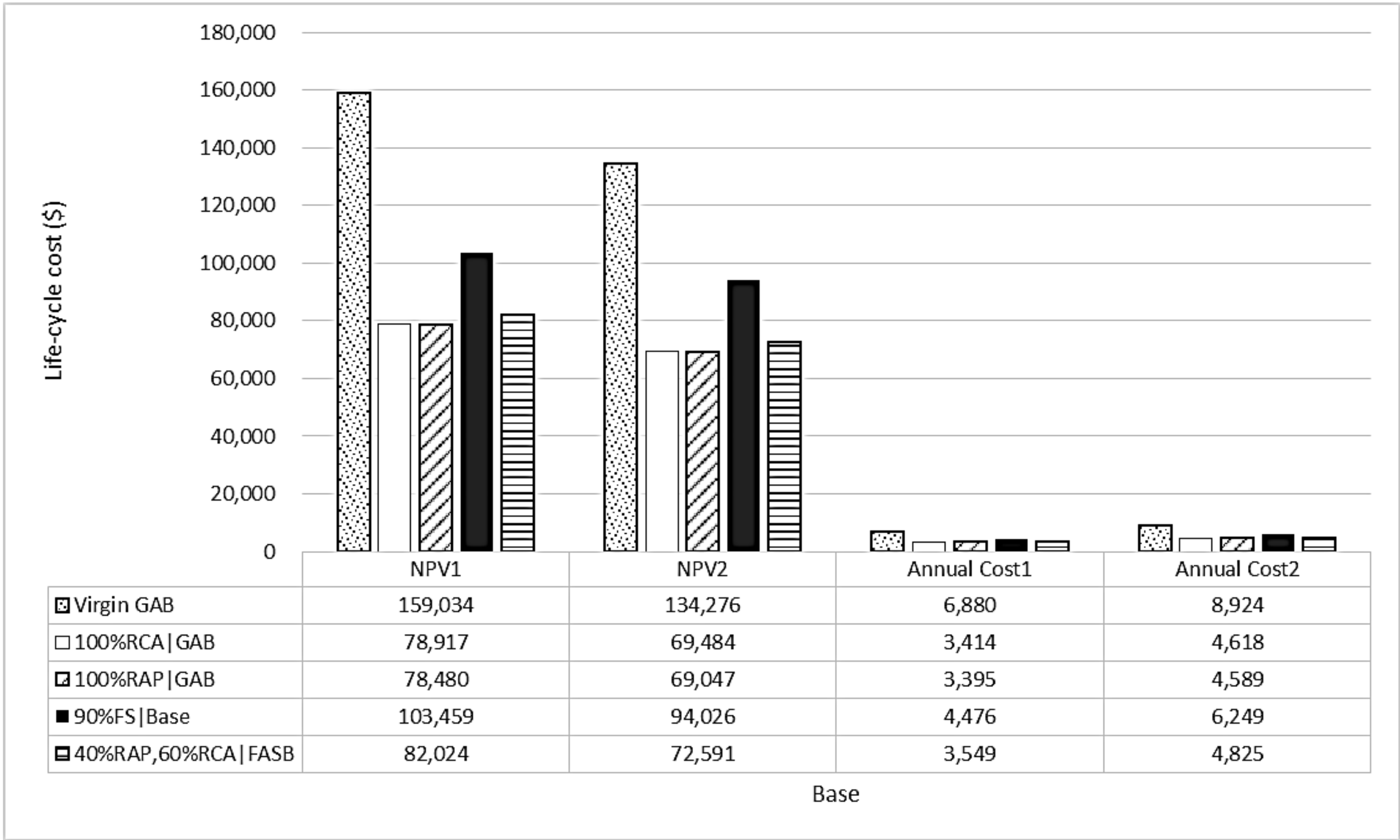


	NPV1	NPV2	Annual Cost1	Annual Cost2
Virgin	295,877	224,179	12,800	14,899
RAP 25%	157,239	147,806	6,803	9,823
RAP 35%	155,367	145,934	6,722	9,699
RCA 45%	203,197	193,764	8,791	12,878
FS 10%	201,137	191,704	8,702	12,741

HMA

Figure 5.2. Life-cycle cost for HMA layer made with recycled materials.

Note: NPV=net present value; Annual cost=annual equivalent worth.NPV 1 and Annual cost 1 are calculated at discount rate of 3%. NPV 2 and Annual cost 2 are calculated at discount rate of 6%.



	NPV1	NPV2	Annual Cost1	Annual Cost2
Virgin GAB	159,034	134,276	6,880	8,924
100%RCA|GAB	78,917	69,484	3,414	4,618
100%RAP|GAB	78,480	69,047	3,395	4,589
90%FS|Base	103,459	94,026	4,476	6,249
40%RAP,60%RCA|FASB	82,024	72,591	3,549	4,825

Figure 5.3. Life-cycle cost for base layer made with recycled materials.

Note: NPV=net present value; Annual cost=annual equivalent worth.NPV 1 and Annual cost 1 are calculated at discount rate of 3%. NPV 2 and Annual cost 2 are calculated at discount rate of 6%.

5.1.3.2 Results of Environmental Effect

◆ PCC

In concrete pavements, the environmental loads of conventional PCC and PCC layers made with recycled materials can be seen in Table 5.15. Most energy is consumed in material production, with a smaller portion consumed in transportation; process consumes the least energy. Gases emission and hazardous waste generation have the same trend. Materials production involves a large amount of chemical reactivities and physical activities like milling, cruching, heating, etc. Environmental loads of transportation are associated with the distance of hauling. Process is related to the construction of pavement.

RAP replacement reduces life cycle energy consumption of PCC slightly, while RCA replacement has comparable energy consumption (deviation within 1%), and FS replacement increases energy consumption (Figure 5.4). Though producing and transporting virgin aggregates is more energy-consuming compared to recycled materials, the high cement content required in recycled PCC leads to higher energy consumtion. Producing cement needs a significant amount of energy, even more than producing virgin aggregates. PCC with 100% RAP has the lowest energy consumption (reduced by 6%) among the six scenarios. Increasing the content of RAP can reduce more energy consumption. PCC made with 20% FS has the highest energy consumption (increased by 7%), which can be attributed to the higher cement content. In addition, FS replaces only a small amount of fine aggregates in PCC; therefore, energy saved by producing FS cannot offset the increased energy needed by cement.

Water consumption is higher for recycled PCC than conventional PCC (Figure 5.5), especially PCC with 100% RAP replacement and PCC with 20% FS replacement. Water consumption is determined by the mix design and distance of transportation in PaLATE. In this case, higher water consumption can be attributed to the higher water and cement content in mix design of recycled PCC (Table 5.8). Producing cement is water-consuming and needs about the same amount of water to produce PCC. Increasing RCA content hardly affects water consumption, while increasing RAP content significantly raises water consumption.

Greenhouse gas emission follows the same trend as energy consumption (Figure 5.6 and Figure 5.9). The only difference is that gas emission increases linearly with increasing RCA content, although it is contradicted with the findings of Evangelista and Brito (2007) that greenhouse gas emission reduces by 6.8%- 20.4% as RCA content increases from 30% to 50%. The reason for the divergence may be that Evangelista and Brito used fine RCA in PCC. Fine RCA can work as a filler in PCC, reducing cement content. NO_x emission is comparable between conventional PCC and recycled PCC (Figure 5.7). PCC with 100% RAP has a higher SO_2 emission than PCC with RCA replacement and conventional PCC (Figure 5.8). PCC with 20% FS has the highest amount of SO_2 emission, due to the high amount of cement used in producing PCC. SO_2 emission increases as RAP and RCA content increases. CO emission is comparable between conventional PCC and recycled PCC (Figure 5.9), except for 20% FS. Recycled PCC has lower fume emission of PM_{10} than conventional PCC (Figure 5.10), since fume emission is related to the production of virgin aggregates. Cement is an inferior source of the fume emission. Therefore, PM_{10} emission decreases as replacement ratios of recycled materials rise.

Harzardous discharge for PCC made with recycled materials is lower than that of conventional PCC (Figures 5.11-

5.13). Virgin aggregate and cement are the primary hazardous sources. PCCs made with recycled materials generally have higher cement content; therefore, recycled PCCs produce a comparable amount of RCRA hazardous waste as conventional PCC (Figure 5.11). PCC with 100% RAP shows the lowest hazardous waste generation. Human toxic potential (non-cancer) is reduced by 14%- 27% as the content of RCA increases from 50%- 100% (Figure 5.13), consistent with the findings of Evangelista and Brito (2007) which show a decrease in human toxicity of 6.8%- 20.7% as RCA content increases from 30%- 50%. Human toxic potential also falls as RAP content increases (Figures 5.12 and 5.13). PCC with 100% RAP shows the lowest human toxic potential.

◆ HMA

In asphalt pavement, HMA layers made with recycled materials have less environmental loads compared to conventional HMA (Table 5.16). These smaller environmental loads can be attributed to the FDR technology used in recycled HMA. Most energy is consumed in material production, while part of the energy is consumed in transportation; process consumes the least energy. Gas emission and hazardous waste generation follow the same trend.

HMA made with 25% RAP shows the lowest life-cycle energy consumption (reduced by 42%), compared to the energy consumption by HMA with 35% RAP (Figure 5.14). HMA with 45% RCA has higher energy consumption (reduced by 16%), higher water consumption (comparable to conventional HMA) and higher greenhouse gas (NO_x and CO emissions) than other recycled HMA, due to higher bitumen content (Figures 5.15-5.17 and Figure 5.20). Similarly, CIPEC (2005) indicated that using 50% RAP in HMA applications can reduce energy consumption by 33%. HMA with 10% FS has higher fume emission of PM_{10} than other recycled HMA, due to high content of virgin aggregate (Figure 5.18). SO_2 emission is comparable for the recycled HMAs, reduced by 50% compared to virgin HMA (Figure 5.19), though the source of SO_2 is different with different recycled materials in use.

HMA made with 45% RCA generates the highest amount of RCRA hazardous waste among the five scenarios of HMA, due to the high content of bitumen (Figure 5.21). In addition, RCA demolition and crushing produce much hazardous waste. HMA made with 45% RCA and 10% FS may have higher human toxic potential (Figure 5.22 and Figure 5.23), which is related to the bitumen content and virgin aggregate content. Since old asphalt pavement is typically exposed to both the natural environment and human activities, it absorbs detrimental matter in their serving period. When old asphalt pavement is processed to reuse, some chemicals may remain in the recycled materials. With an increase of RAP content from 25% to 35%, the hazardous discharge increases by about 3% (Table 5.16).

◆ Base

For the base layer, recycled GAB generally has less environmental load than conventional GAB (Table 5.17), since virgin aggregate has longer a transportation distance (one-way distance of 30 miles, compared to 10 miles for recycled materials). In addition, virgin aggregates (i.e., limestone) have higher potential to generate hazardous waste and toxic chemicals. Most energy is consumed in material production and transportation inferiors, while process consumes the least energy. Hazardous waste generation follows the same trend. Gases emission may be higher in transportation than materials production (i.e., NO_x).

A cement-stabilized base with 90% FS has the highest energy consumption (Figure 5.24), water consumption (Figure 5.25) and gas emission (Figure 5.26, 5.27, 5.30), which can be attributed to the presence of cement. However, a cement-stabilized base with 90% FS has the lowest hazardous discharge among the five scenarios (Figure 5.31-5.33), since FS and cement have low hazardous discharge compared to other materials. FASB with 40% RAP and 60% RCA has moderate energy consumption, water consumption and gas emission, but the highest SO_2 emission (Figure 5.29) and hazardous discharge, due to the present of emulsified asphalt. Recycled aggregate can reduce fume (PM_{10}) emission by 50% or more (Figure 5.28), since hauling distance is reduced (from 30 miles to 10 miles, based on assumption). In addition, producing virgin materials (i.e., milling) can release considerable fumes into environment.

◆ Embankment

As seen in Table 5.17, embankment made of 100% FS has 12% higher energy consumption and 6% higher greenhouse gas emission than conventional embankment. Water comsumption is comparable for recycled embankment and conventional embankment. Other gas emissions and hazardous discharges are lower for recycled embankment than for convenional embankment. Particularly, recycled embankment can reduce RCRA hazardous waste generation by up to 58% and NO_x emission by 47%.

Table 5.15. Recycled materials used in PCC.

Environment	Materials	Virgin	RCA 50%	RCA 100%	RAP 100%	RAP 40%	FS 20%
Energy consumption/ MJ	Materials Production	6,273,521	6,319,040	6,308,454	6,020,774	6,131,476	6,776,293
	Materials Transportation	185,520	155,049	126,059	50,881	139,574	161,598
	Processes (Equipment)	34,778	34,778	34,762	34,778	34,779	34,778
	Total	6,493,820	6,508,868	6,469,275	6,106,433	6,305,829	6,972,669
CO_2 Emission/Mg	Materials Production	437	449	458	423	429	473
	Materials Transportation	14	12	9	4	10	12
	Processes (Equipment)	3	3	3	3	3	3
	Total	454	464	470	430	442	487
NOx Emission/kg	Materials Production	5,330	5,543	5,707	5,736	5,419	5,823
	Materials Transportation	742	621	506	207	559	648
	Processes (Equipment)	58	58	58	58	58	58
	Total	6,130	6,222	6,270	6,001	6,036	6,528
Water Consumption /kg	Materials Production	2,391	2,479	2,510	2,675	2,485	2,760
	Materials Transportation	32	26	21	9	24	28
	Processes (Equipment)	3	3	3	3	3	3
	Total	2,426	2,509	2,534	2,687	2,512	2,791
RCRA Hazardous Waste Generated/kg	Materials Production	7,777	7,948	8,114	7,192	7,554	7,924
	Materials Transportation	1,337	1,117	908	367	1,006	1,164
	Processes (Equipment)	115	115	115	115	115	115
	Total	9,229	9,181	9,137	7,674	8,675	9,204

Table 5.15. Recycled materials used in PCC (continued).

Environment	Materials	Virgin	RCA 50%	RCA 100%	RAP 100%	RAP 40%	FS 20%
SO_2 Emission/kg	Materials Production	3,891	4,046	4,149	4,312	3,986	4,386
	Materials Transportation	45	37	30	12	34	39
	Processes (Equipment)	4	4	4	4	4	4
	Total	3,940	4,087	4,183	4,329	4,023	4,429
CO Emission/kg	Materials Production	2,824	2,878	2,913	2,926	2,841	2,995
	Materials Transportation	62	52	42	17	47	54
	Processes (Equipment)	12	12	12	12	12	12
	Total	2,899	2,942	2,968	2,956	2,901	3,061
Human Toxic Potential (cancer)	Materials Production	181,451	177,540	174,658	162,661	174,285	171,597
	Materials Transportation	164	137	111	45	123	143
	Processes (Equipment)	0	0	0	0	0	0
	Total	181,615	177,677	174,769	162,706	174,408	171,740
PM_{10} Emission/kg	Materials Production	2,269	2,110	1,945	1,507	2,003	2,265
	Materials Transportation	145	121	99	40	109	126
	Processes (Equipment)	4	4	4	4	4	4
	Total	2,418	2,235	2,048	1,551	2,117	2,396
Human Toxic Potential (non-cancer)	Materials Production	1,201,715,412	1,033,902,538	874,919,844	300,899,790	896,902,931	1,051,114,892
	Materials Transportation	7,003	5,853	4,759	1,921	5,269	6,100
	Processes (Equipment)	0	0	0	0	0	0
	Total	1,201,722,416	1,033,908,391	874,924,603	300,901,711	896,908,199	1,051,120,993

Table 5.16. Recycled materials used in HMA.

Environment	Materials	Virgin	RAP 25%	RAP 35%	RCA 45%	FS 10%
Energy Consumption/ MJ	Materials Production	4,198,677	2,485,873	2,513,975	3,619,547	2,765,550
	Materials Transportation	218,453	54,885	52,221	77,325	100,340
	Processes (Equipment)	21,769	14,440	14,430	14,431	14,431
	Total	4,438,899	2,555,198	2,580,626	3,711,303	2,880,322
CO_2 Emission/ Mg	Materials Production	205	124	126	198	143
	Materials Transportation	16	4	4	6	8
	Processes (Equipment)	2	1	1	1	1
	Total	223	130	131	205	152
NOx Emission/ kg	Materials Production	1,655	964	977	1,362	1,001
	Materials Transportation	870	219	208	308	400
	Processes (Equipment)	38	26	26	26	26
	Total	2,563	1,209	1,211	1,696	1,427
Water Consumption / kg	Materials Production	1,235	795	813	1,231	841
	Materials Transportation	37	9	9	13	17
	Processes (Equipment)	2	1	1	1	1
	Total	1,274	805	823	1,246	860
RCRA Hazardous Waste Generated/kg	Materials Production	49,567	32,575	33,469	50,938	33,309
	Materials Transportation	1,574	395	376	557	723
	Processes (Equipment)	70	70	70	70	70
	Total	51,212	33,041	33,916	51,565	34,102

Table 5.16. Recycled materials used in HMA (continued).

Environment	Materials	Virgin	RAP 25%	RAP 35%	RCA 45%	FS 10%
SO_2 Emission/ kg	Materials Production	65,475	32,860	32,873	33,160	32,885
	Materials Transportation	52	13	12	18	24
	Processes (Equipment)	3	2	2	2	2
	Total	65,530	32,875	32,888	33,180	32,911
CO Emission/ kg	Materials Production	714	459	469	721	487
	Materials Transportation	73	18	17	26	33
	Processes (Equipment)	8	6	6	6	6
	Total	795	483	492	752	526
Human Toxic Potential (cancer)	Materials Production	818,312	532,189	544,213	821,419	550,555
	Materials Transportation	193	49	46	68	89
	Processes (Equipment)	0	0	0	0	0
	Total	818,505	532,238	544,259	821,488	550,644
PM_{10} Emission/ kg	Materials Production	727	330	308	472	592
	Materials Transportation	171	42	40	60	78
	Processes (Equipment)	12	11	11	11	11
	Total	910	383	359	542	680
Human Toxic Potential (non-cancer)	Materials Production	696,981,483	292,480,939	263,953,963	432,919,401	548,194,476
	Materials Transportation	8,247	2,072	1,971	2,919	3,788
	Processes (Equipment)	0	0	0	0	0
	Total	696,989,730	292,483,011	263,955,934	432,922,320	548,198,264

Table 5.17 Recycled materials used in base and embankment.

Environmental effect	Materials	Virgin GAB	100%RCA| GAB	100%RAP| GAB	90%FS| Base	40%RAP,60 %RCA| FASB	Virgin Embank-ment	100%FS| Embank-ment
Energy consumption/MJ	Materials Production	651,445	112,092	110,303	1,189,913	618,688	40,631,507	48,757,808
	Materials Transportation	139,361	32,345	31,829	22,814	10,551	4,346,067	1,738,427
	Processes (Equipment)	18,720	13,035	12,827	10,215	4,467	1,167,599	1,401,119
	Total	809,526	157,472	154,960	1,222,942	633,706	46,145,173	51,897,355
CO_2 Emission/Mg	Materials Production	46	8	8	84	35	2,878	3,453
	Materials Transportation	10	2	2	2	1	650	260
	Processes (Equipment)	1	1	1	1	0	88	105
	Total	58	12	12	86	36	3,615	3,818
NOx Emission/kg	Materials Production	93	197	194	780	253	5,798	6,958
	Materials Transportation	555	129	127	91	42	34,620	13,848
	Processes (Equipment)	30	21	21	17	7	1,108	1,329
	Total	678	347	341	887	302	41,526	22,135

Table 5.17. Recycled materials used in base and embankment (continued).

Environmental effect	Materials	Virgin GAB	100%RCA| GAB	100%RAP| GAB	90%FS| Base	40%RAP,60 %RCA| FASB	Virgin Embank-ment	100%FS| Embank-ment
Water Consumption /kg	Materials Production	91	0	0	476	240	5,659	6,791
	Materials Transportation	12	6	5	4	2	1,480	592
	Processes (Equipment)	2	1	1	1	0	114	136
	Total	104	7	7	481	243	7,253	7,519
RCRA Hazardous Waste Generated/kg	Materials Production	757	808	795	749	10,433	28,331	28,331
	Materials Transportation	502	233	229	164	76	62,632	25,053
	Processes (Equipment)	67	94	92	74	32	4,207	5,048
	Total	1,327	1,135	1,117	987	10,541	95,170	58,432
SO2 Emission/kg	Materials Production	45	13	13	799	881	2,825	3,390
	Materials Transportation	33	8	8	5	3	2,077	831
	Processes (Equipment)	2	1	1	1	0	125	150
	Total	81	22	22	806	884	5,027	4,371

Table 5.17. Recycled materials used in base and embankment (continued).

Environ mental effect	Materials	Virgin GAB	100%RCA| GAB	100%RAP| GAB	90%FS| Base	40%RAP,60 %RCA| FASB	Virgin Embank-ment	100%FS| Embank-ment
CO Emission/kg	Materials Production	61	42	42	291	151	3,789	4,546
	Materials Transportation	46	11	11	8	4	2,885	1,154
	Processes (Equipment)	7	5	4	4	2	267	320
	Total	114	58	57	302	156	6,940	6,021
HTP (cancer)	Materials Production	61,797	43,025	42,377	4,645	171,519	193,376	193,376
	Materials Transportation	123	29	28	20	9	7,683	3,073
	Processes (Equipment)	0	0	0	0	0	0	0
	Total	61,920	43,053	42,405	4,665	171,529	201,059	196,449

Table 5.17. Recycled materials used in base and embankment (continued).

Environmental effect	Materials	Virgin GAB	100%RCA| GAB	100%RAP| GAB	90%FS| Base	40%RAP,60%RCA| FASB	Virgin Embank-ment	100%FS| Embank-ment
PM_{10} Emission/kg	Materials Production	661	14	14	462	35	41,234	49,481
	Materials Transportation	108	25	25	18	8	6,748	2,699
	Processes (Equipment)	4	2	1	1	1	134	161
	Total	773	41	40	481	44	48,117	52,341
HTP (non-cancer)	Materials Production	780,139,774	194,263,305	75,455,492	6,388,100	48,081,982	162,062,470	970,627,316
	Materials Transportation	5,261	1,221	1,202	861	398	328,126	131,250
	Processes (Equipment)	0	0	0	0	0	0	0
	Total	780,145,035	194,264,526	75,456,694	6,388,962	48,082,380	162,390,596	970,758,566

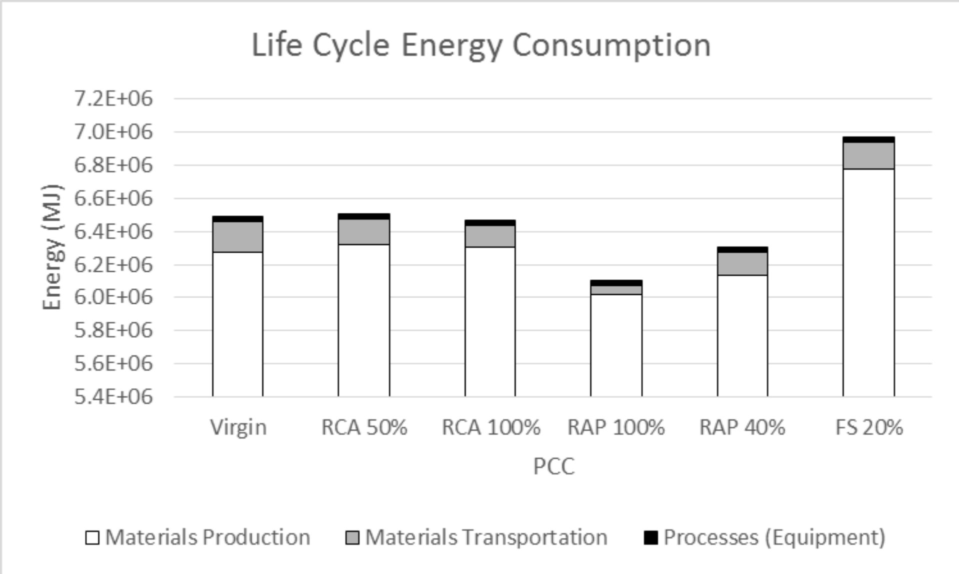


Figure 5.4. Life-cycle energy consumption for PCC made with recycled materials.

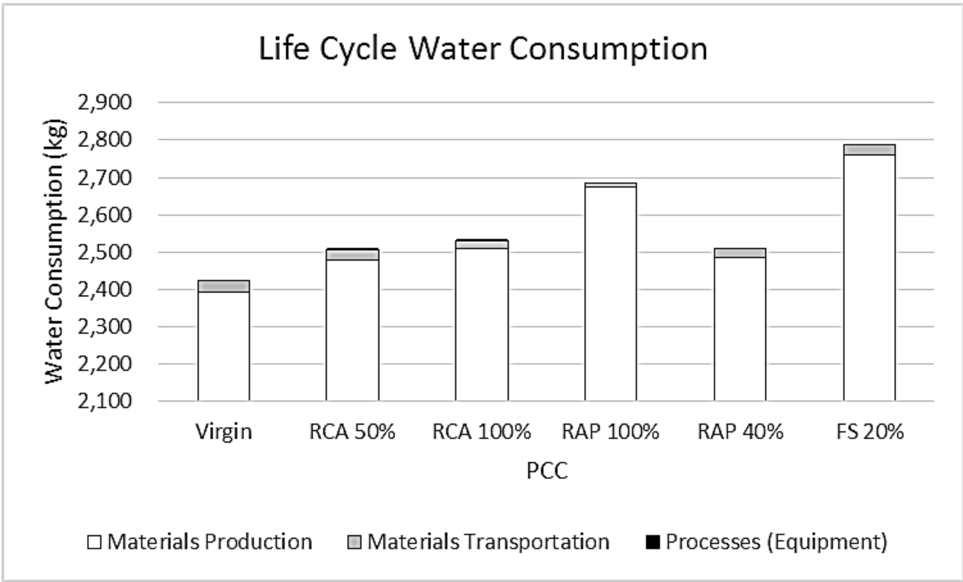


Figure 5.5. Life-cycle water consumption for PCC made with recycled materials.

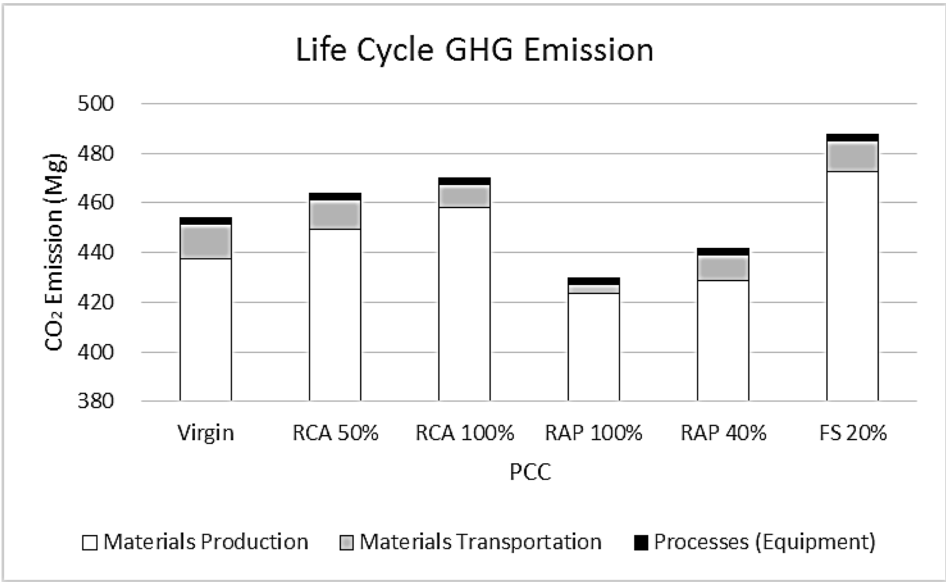


Figure 5.6. Life-cycle greenhouse gas emission for PCC made with recycled materials.

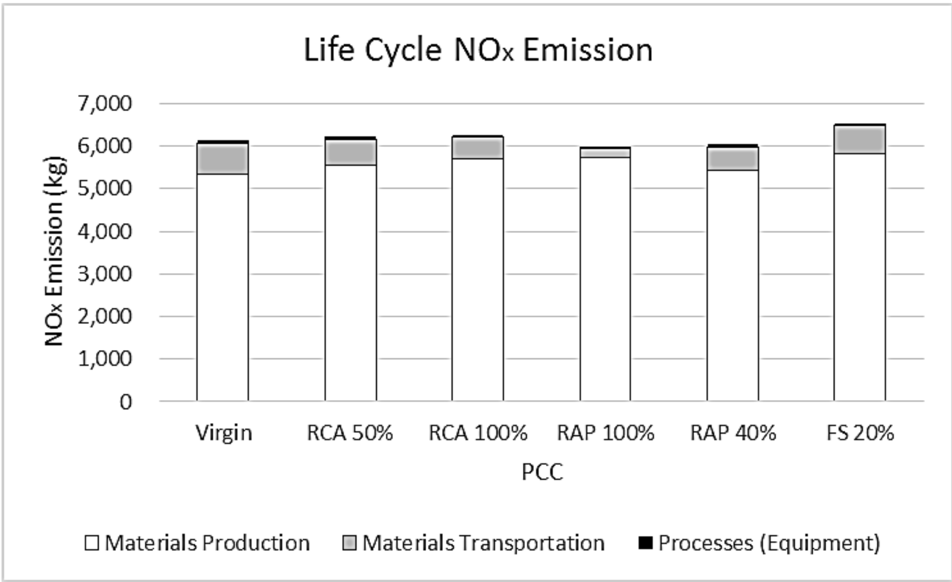


Figure 5.7. Life-cycle NO_x emission for PCC made with recycled materials.

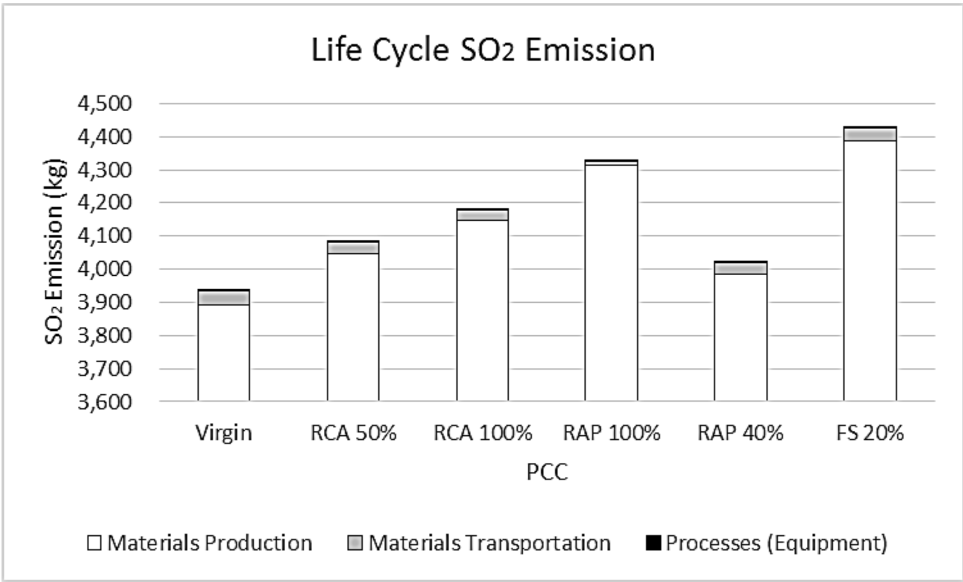


Figure 5.8. Life-cycle SO_2 emission for PCC made with recycled materials.

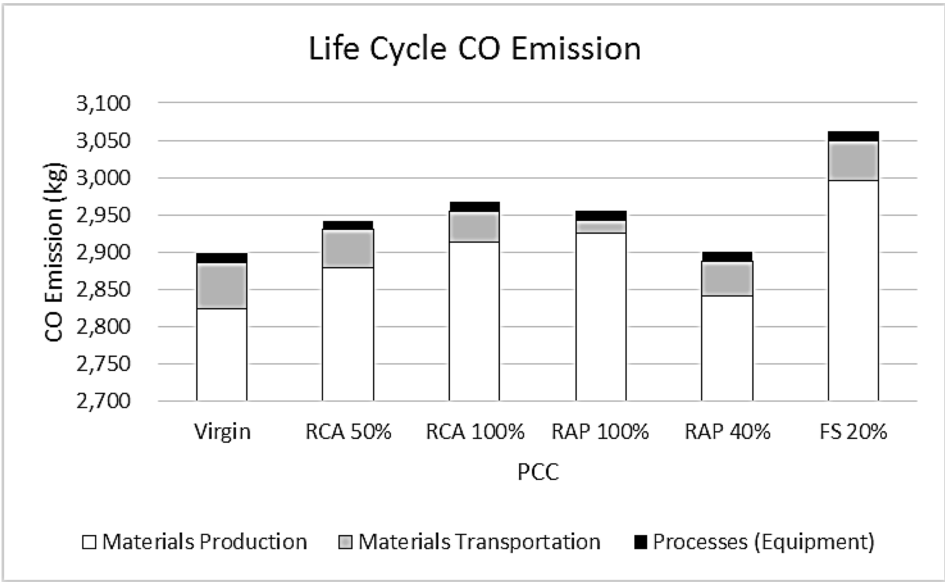


Figure 5.9. Life-cycle CO emission for PCC made with recycled materials.

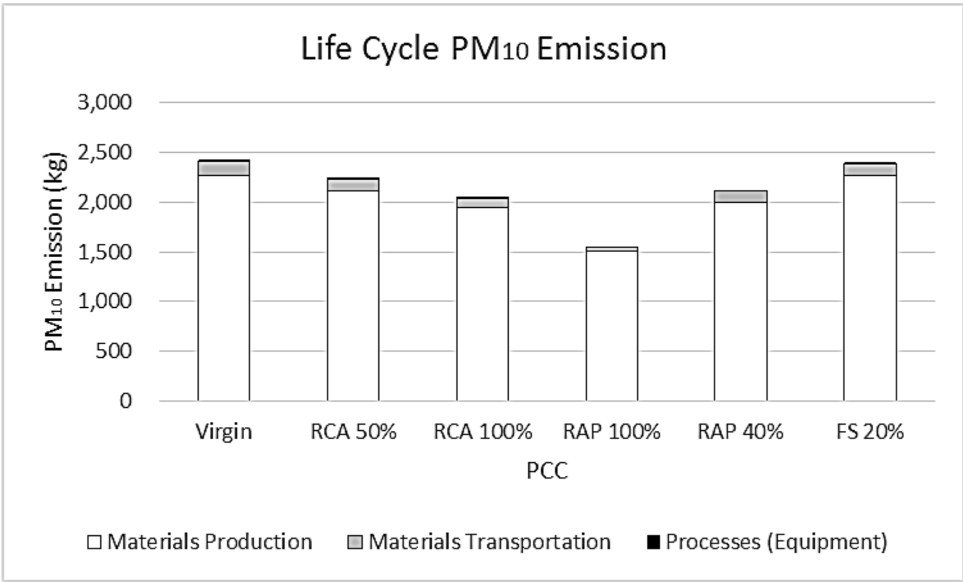


Figure 5.10. Life-cycle PM_{10} emission for PCC made with recycled materials.

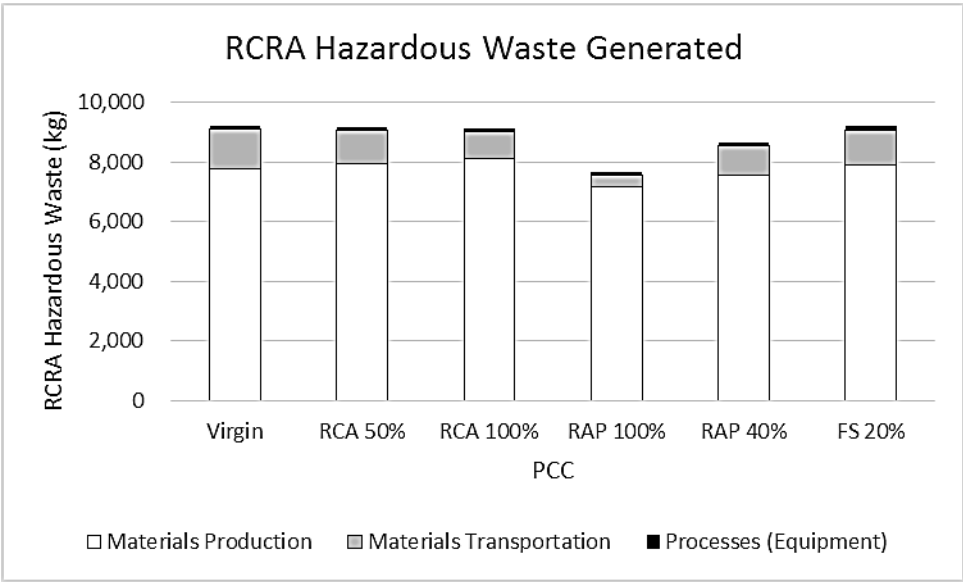


Figure 5.11. Life-cycle RCRA hazardous waste generated for PCC made with recycled materials.

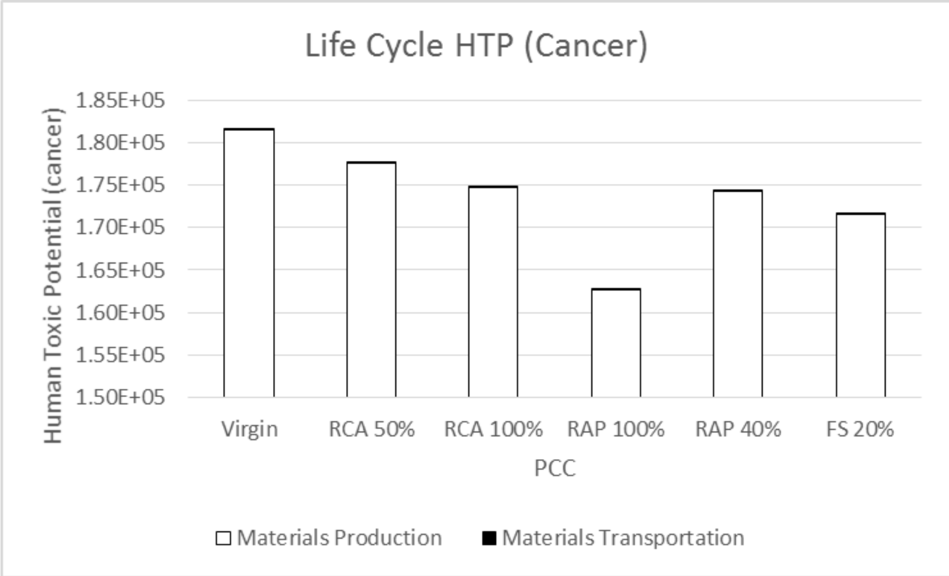


Figure 5.12. Life-cycle human toxicity potential (cancer) for PCC made with recycled materials.

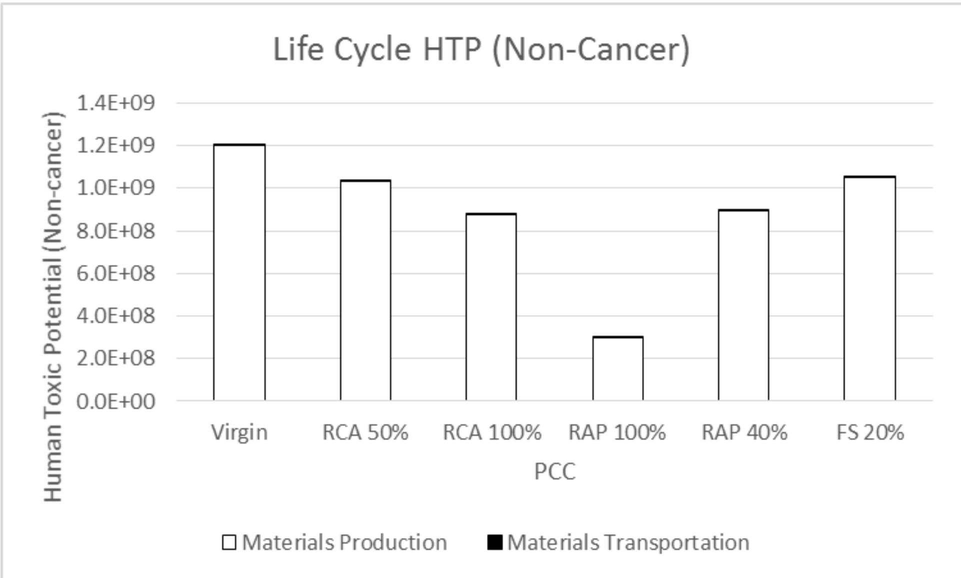


Figure 5.13. Life-cycle human toxicity potential (non-cancer) for PCC made with recycled materials.

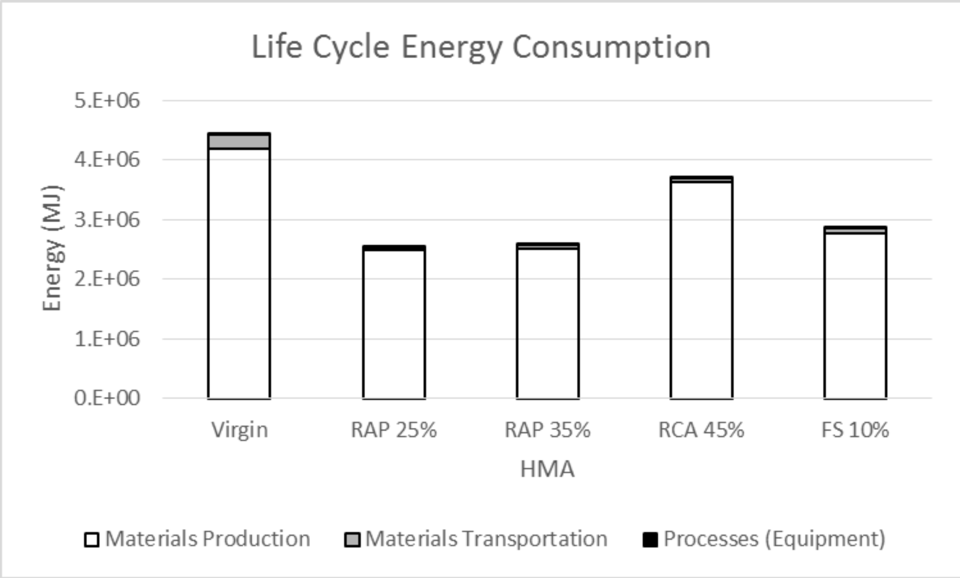


Figure 5.14. Life-cycle energy consumption for HMA made with recycled materials.

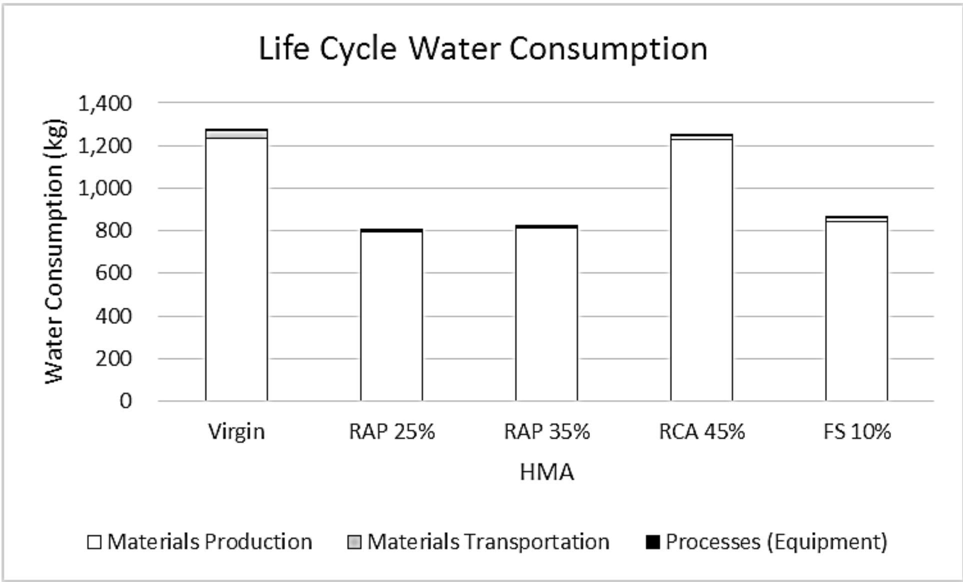


Figure 5.15. Life-cycle water consumption for HMA made with recycled materials.

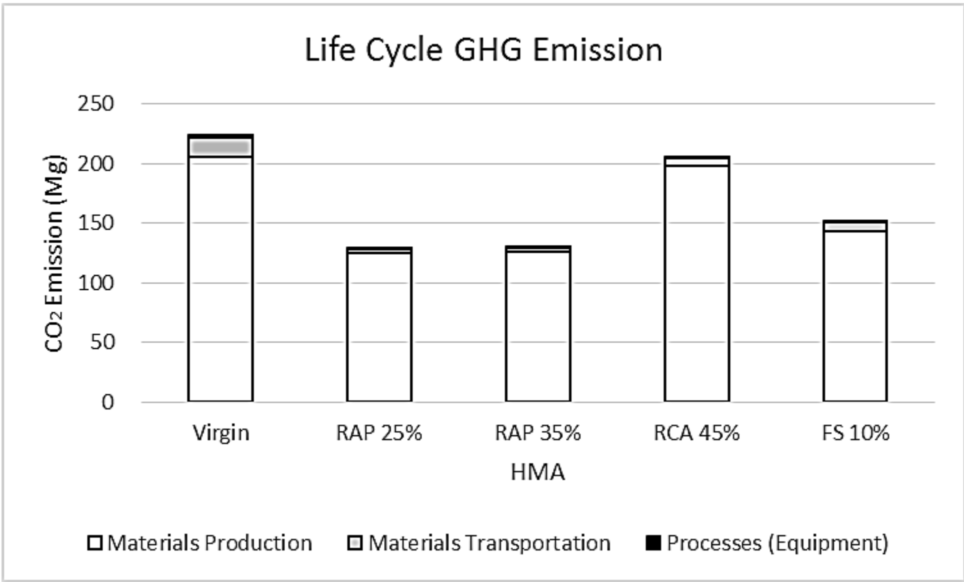


Figure 5.16. Life-cycle greenhouse gas emission for HMA made with recycled materials.

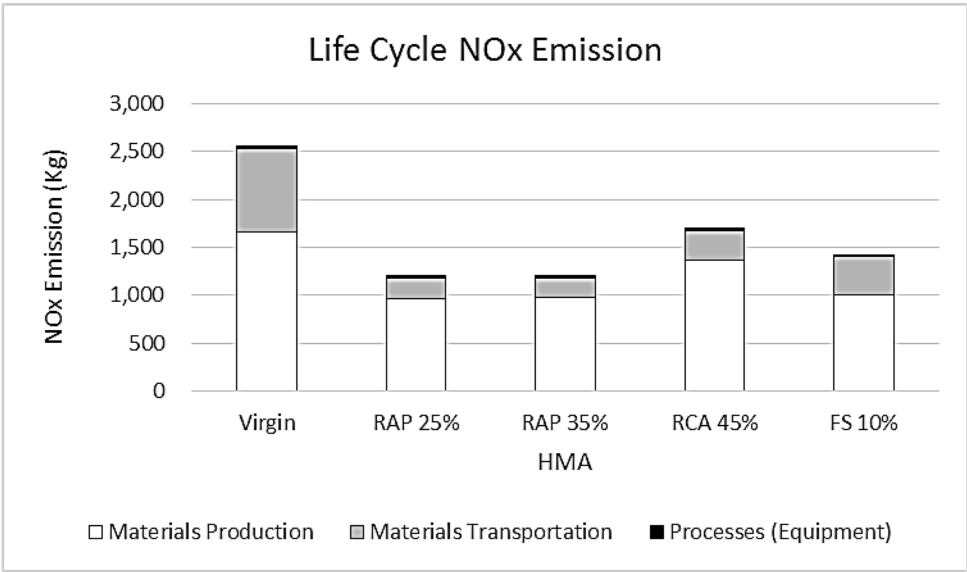


Figure 5.17. Life-cycle NO_x emission for HMA made with recycled materials.

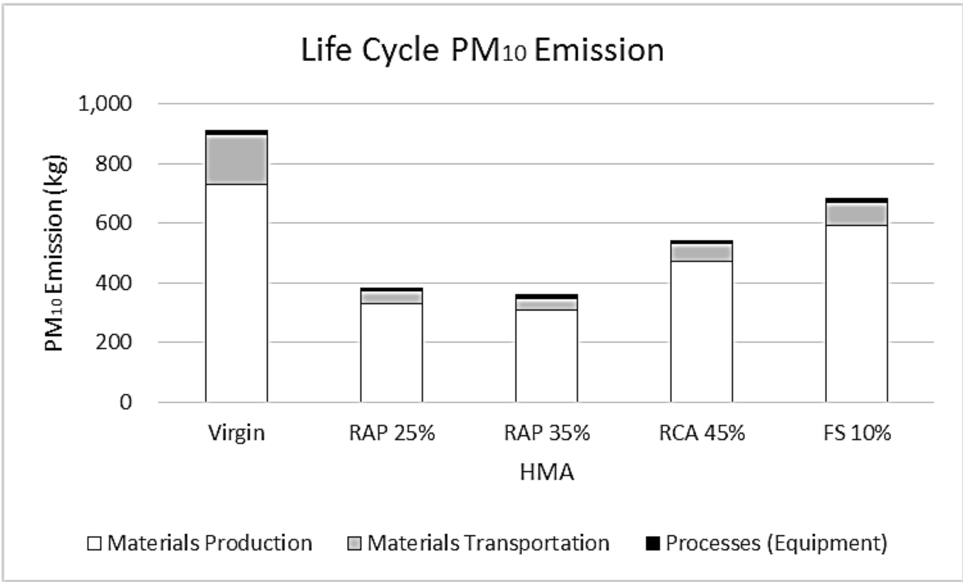


Figure 5.18. Life-cycle PM_{10} emission for HMA made with recycled materials.

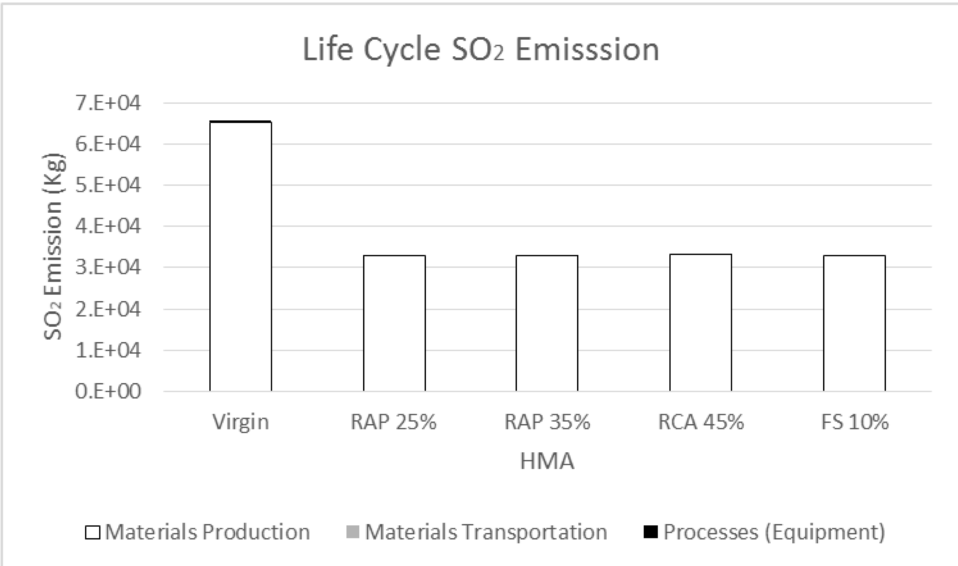


Figure 5.19. Life-cycle SO_2 emission for HMA made with recycled materials.

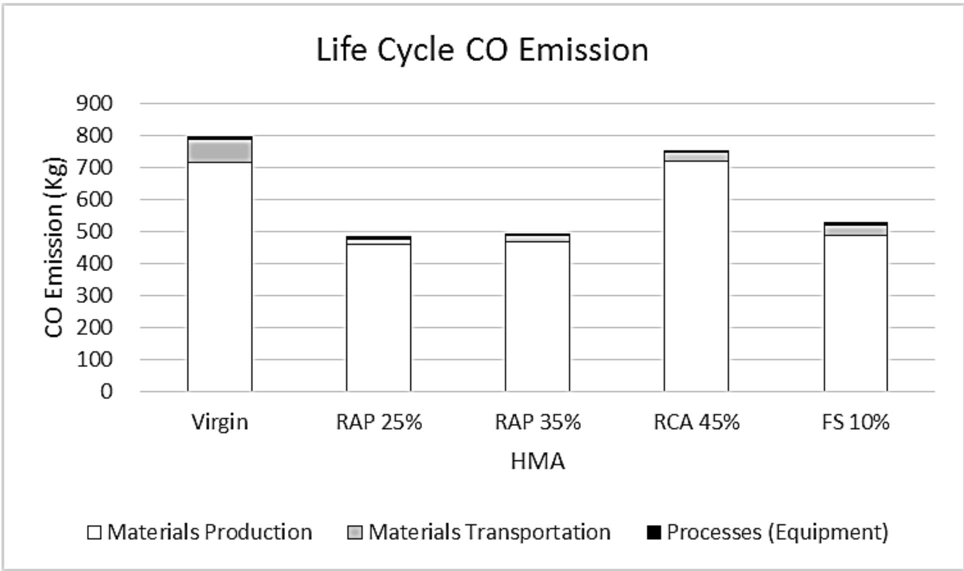


Figure 5.20. Life-cycle CO emission for HMA made with recycled materials.

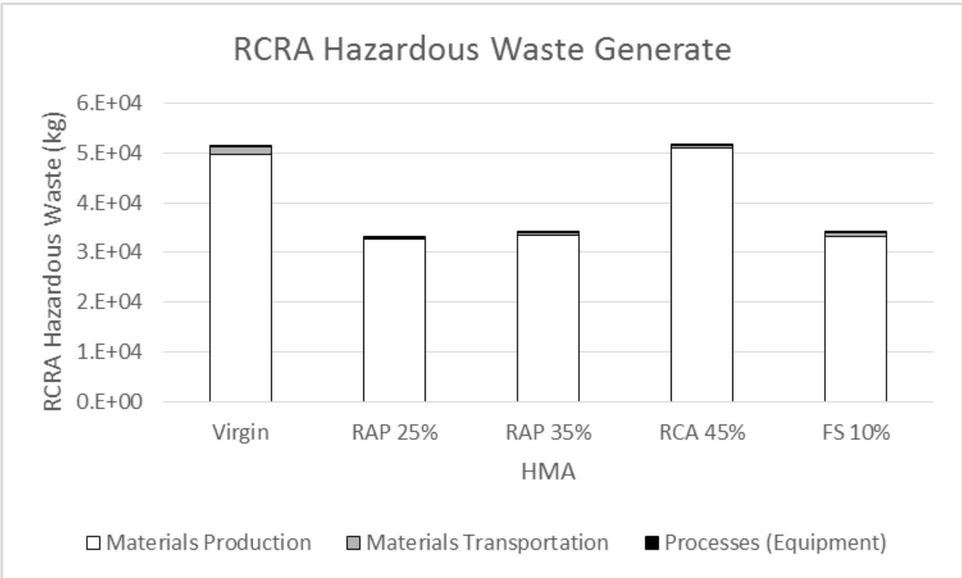


Figure 5.21. Life-cycle RCRA hazardous waste generated for HMA made with recycled materials.

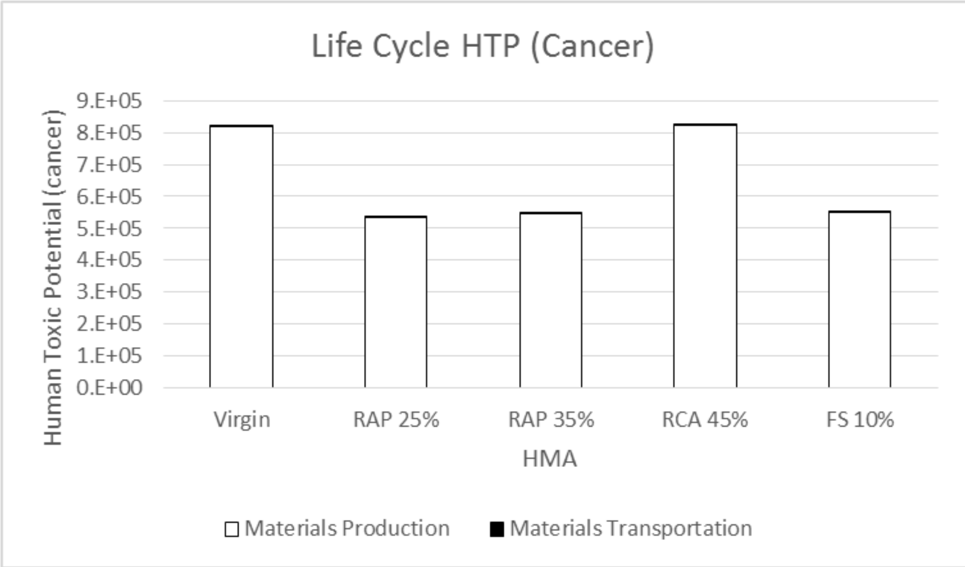


Figure 5.22. Life-cycle human toxicity potential (cancer) for HMA made with recycled materials.

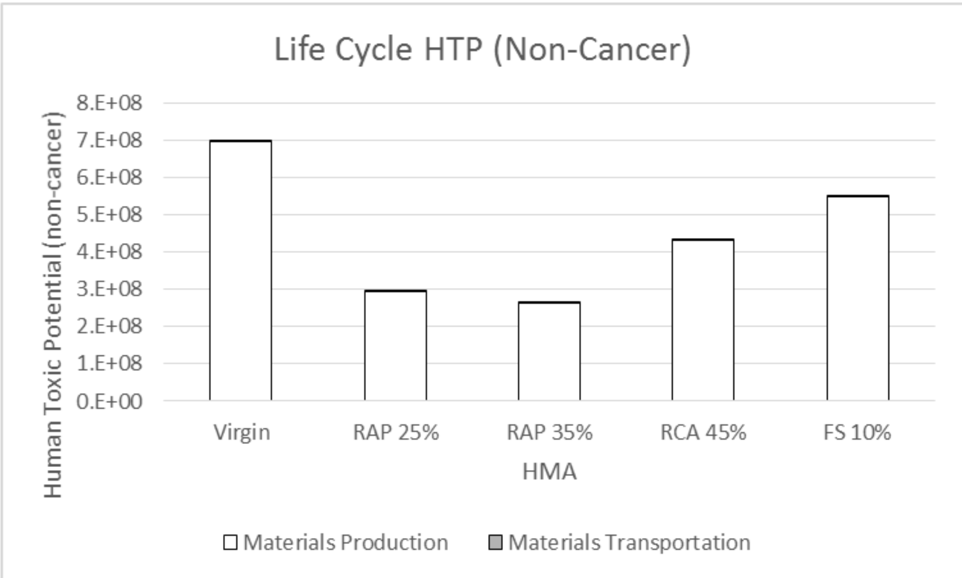


Figure 5.23. Life-cycle human toxicity potential (non-cancer) for HMA made with recycled materials.

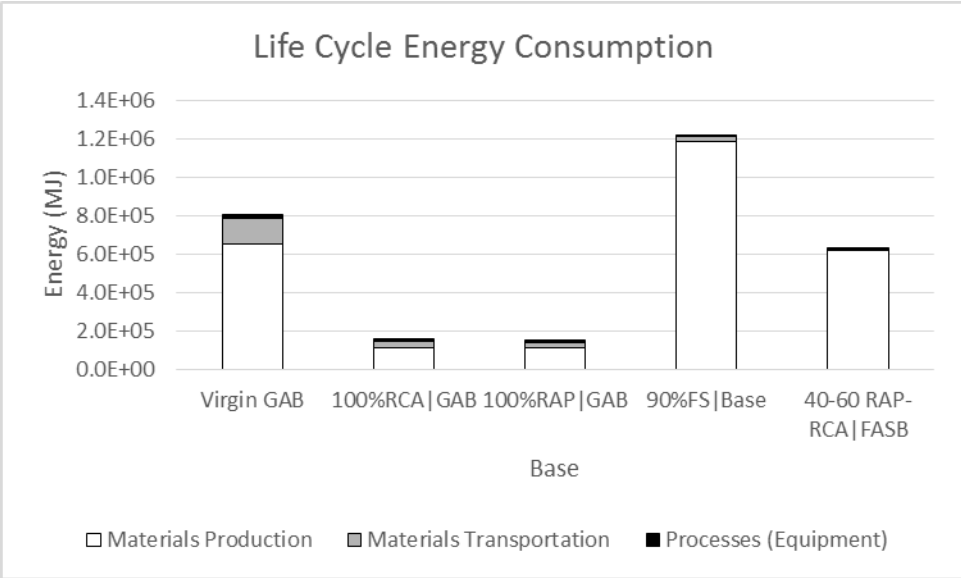


Figure 5.24. Life-cycle energy consumption for base made with recycled materials.

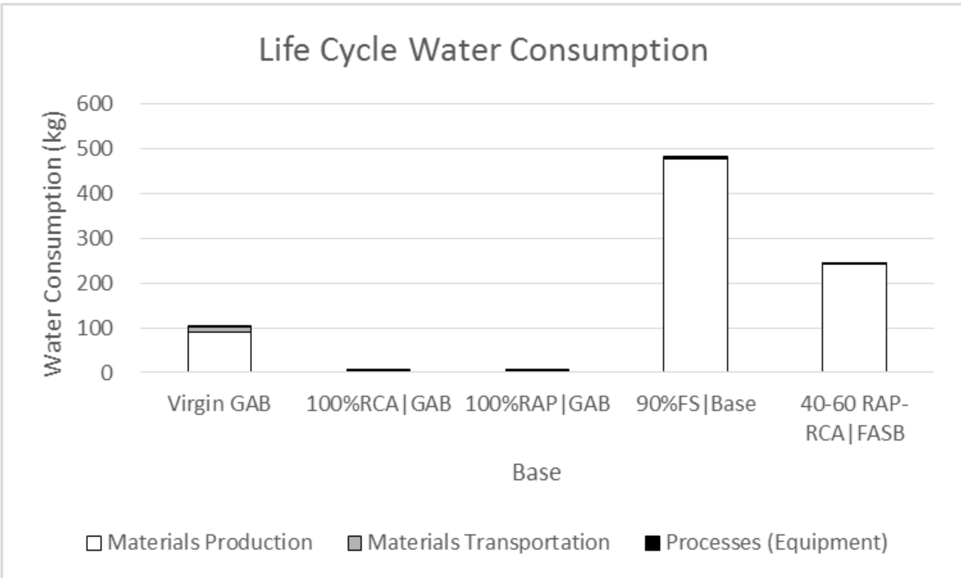


Figure 5.25. Life-cycle water consumption for base made with recycled materials.

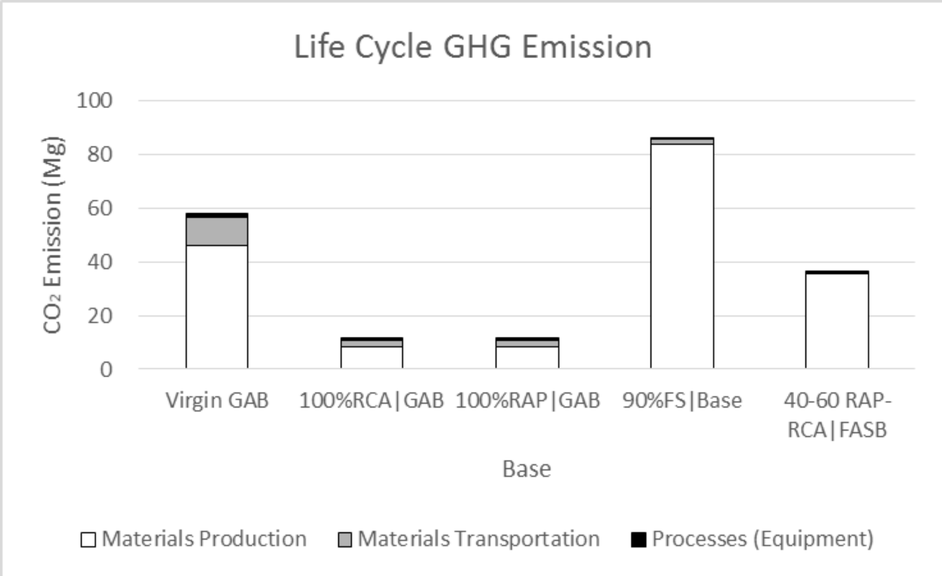


Figure 5.26. Life-cycle greenhouse gas emission for base made with recycled materials.

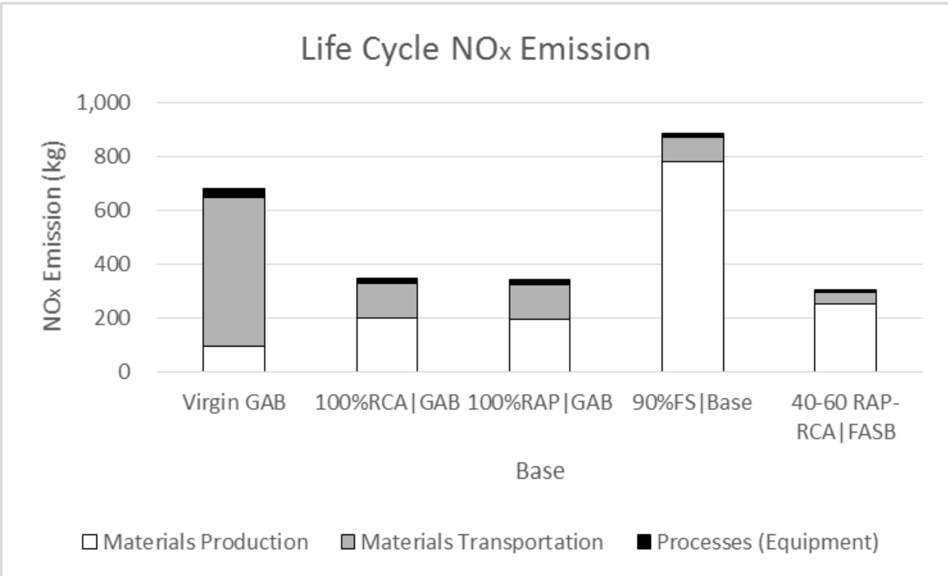


Figure 5.27. Life-cycle NOx emission for base made with recycled materials.

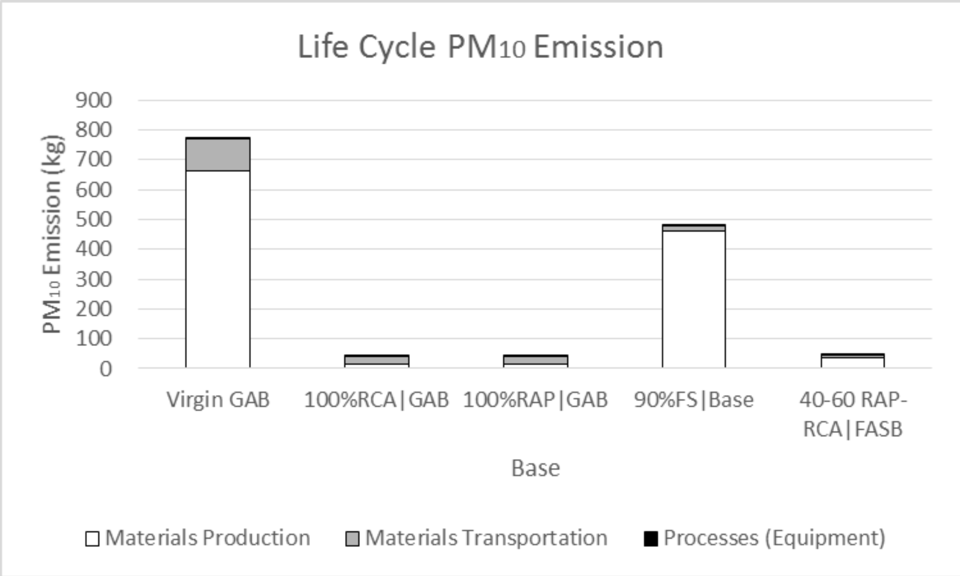


Figure 5.28. Life-cycle PM_{10} emission for base made with recycled materials.

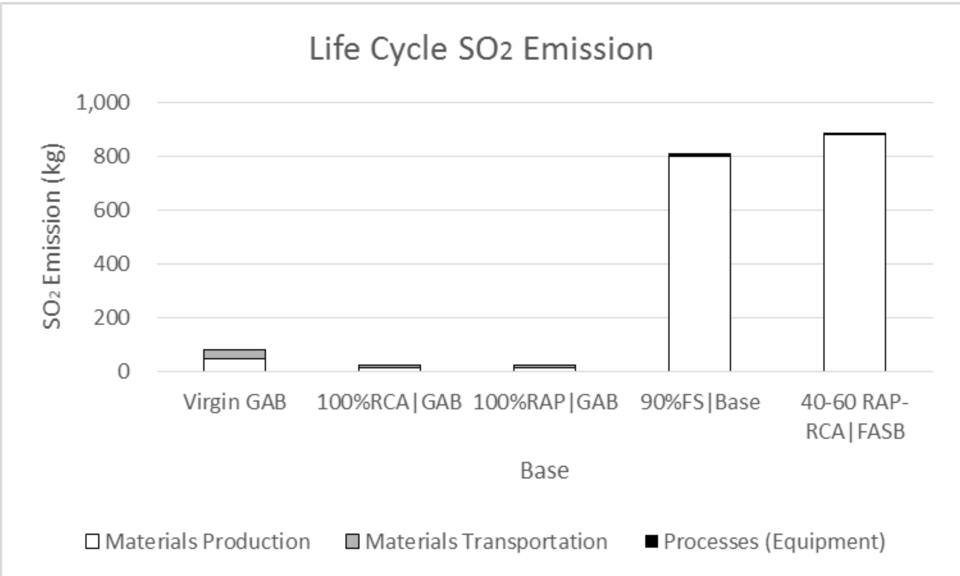


Figure 5.29. Life-cycle SO_2 emission for base made with recycled materials.

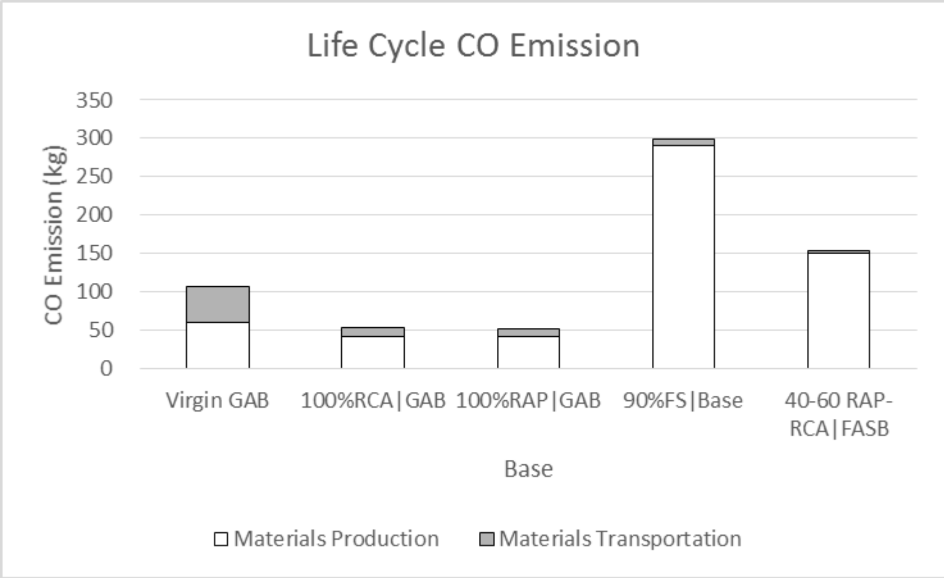


Figure 5.30. Life-cycle CO emission for base made with recycled materials.

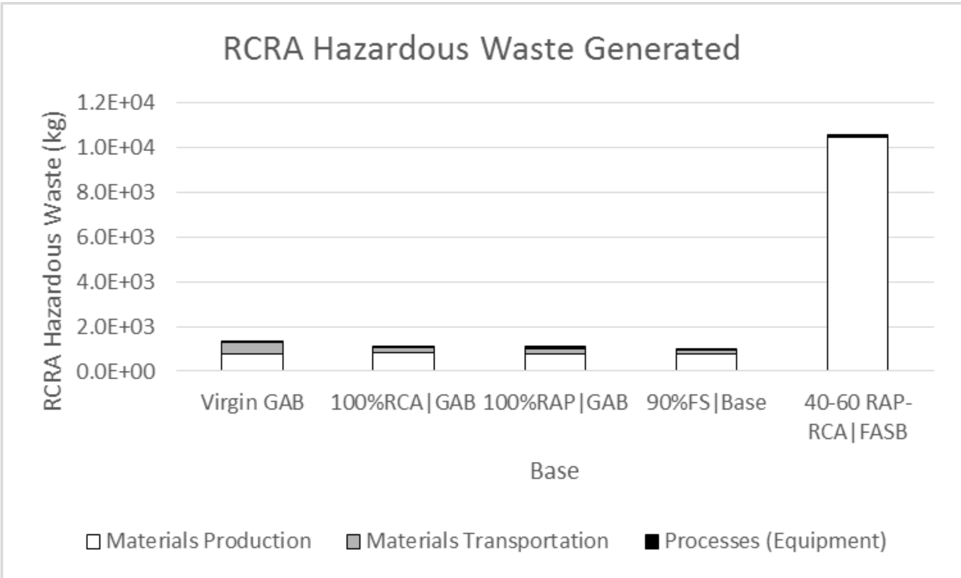


Figure 5.31. Life-cycle RCRA hazardous waste generated for base made with recycled materials.

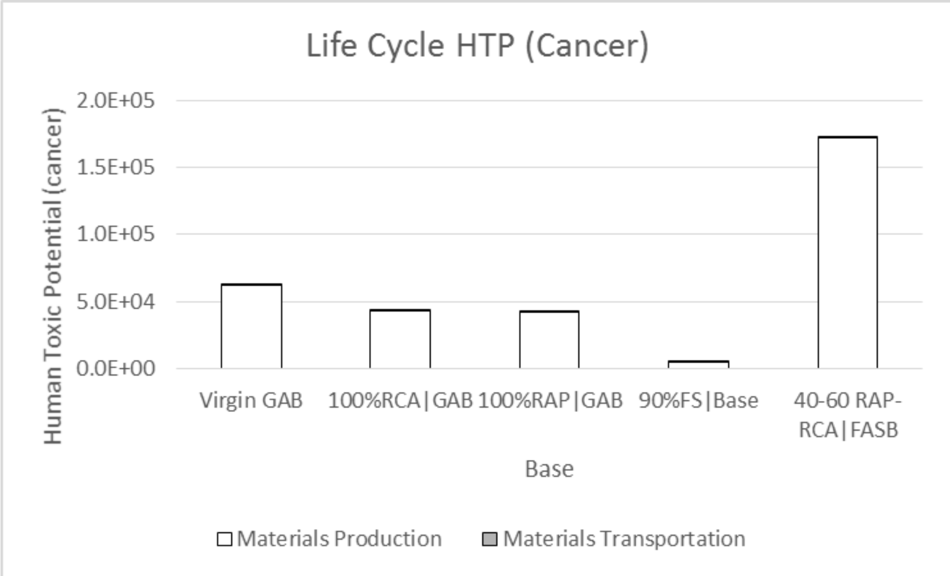


Figure 5.32. Life-cycle human toxicity potential (cancer) for HMA made with recycled materials.

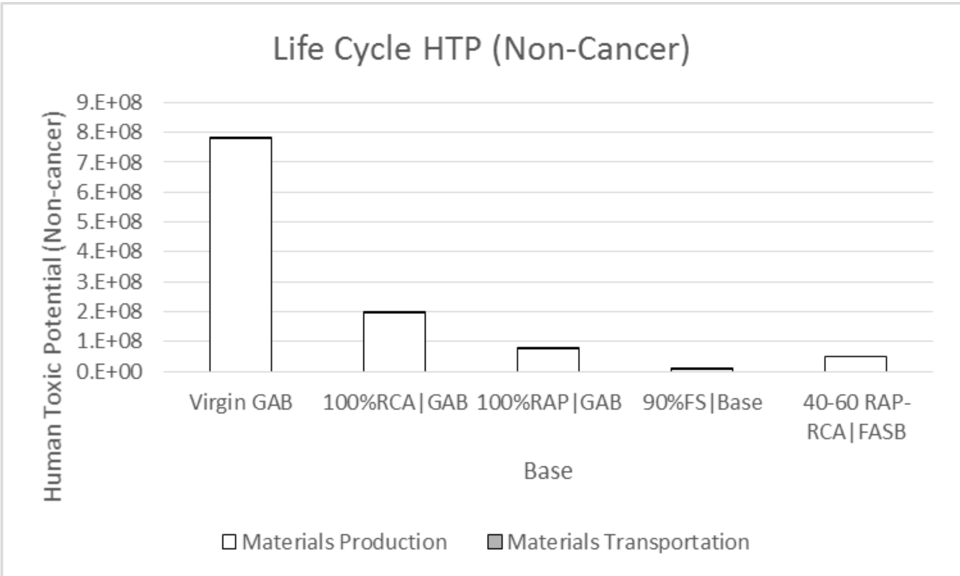


Figure 5.33. Life-cycle human toxicity potential (non-cancer) for base made with recycled materials.

5.1.3.3 Data Deficiency and Uncertainty

LCCA may be affected by various factors. First, the unit price of materials used in LCCA are collected from different sources and in different years. Prices vary signifcantly year to year, and are different from one contractor to another, and perhaps unit prices for some materials are over or underestimated. Second, the expense for construction activities (i.e., milling, crushing, demolition, rubblization, transportation, etc.) should be included in the unit price of materials for material-based LCCA. However, the data may cover only materials production, processing and transportation. Third, the discount rate (1%-8%) used to measure the future interest rate and inflation increases the uncertainty of LCCA. For example, in asphalt pavements, HMA produced with 25% RAP has a cost reduced by 47%-34% as discount rate ranges between 3% and 6%.

LCCA may also be affected by several factors. First, equipement chosen (i.e., engine capacity, productivity, fuel consumption, etc.) in the initial construction and maintenance phase can affect the environmental effects. Second, the activities in initial construction and maintenance are simplified. Mroueh et al. (2001) indicated that it is difficult to determine the most common working and implementation methods of the work stages for recycled materials. As a result, experience-based or measurement-based data on the working stages and their environmental loadings are rarely available.

5.1.4. Conclusions

The use of recycled materials in highway applications may yield cost savings and considerable environmental benefits, compared to highway applications with only virgin materials. In LCCA, PCCs made with recycled materials have comparable or higher cost (-6%-23%) than conventional PCC as a result of the higher amount of cement and water required in producing recycled PCC. HMAs made with recycled materials significatly reduce cost by 14%-47% due to the FDR technique used in recycled HMA. Bases made with recycled materials also significantly reduce cost (30%-50%) compared to conventional GAB base, in part because of the low price of recycled materials. Embankment also shows reduced cost with FS in use (60%). In LCCA, material production generally has the highest environmental loads and transportation inferiors, while process has the least effect, consistent with the study of Apyal (2008). With respect to materials that have the most environmental loads, cement and asphalt bitumen have the highest energy consumption, water consumption and gas emission; cement, asphalt bitumen and virgin aggregates have the highest hazardous waste generation and toxic chemicals discharge; cement and FS have the highest fume emission (PM_{10}); and recycled materials generally have the least environmental loads. Though there are many uncertainties within the life cycle analysis, the results from PaLATE can be helpful for decision makers in identifying the optimum scheme for pavements.

5.2 Evaluation of Recycled Materials in Highway Application by BE²ST-in-Highways™

5.2.1 Introduction

BE²ST-in-Highways™, based on MS Excel, is a highway rating system that utilizes life-cycle analysis of pavements constructed with various materials. BE²ST-in-Highways™ consists of five subprograms: M-EPDG for service life design, RealCost for life-cycle cost analysis, PaLATE for environmental analysis, a noise evaluation subprogram and a storm-water evaluation subprogram. The environmental effects accessed in BE²ST-in-Highways™ includes energy consumption, greenhouse gas emission, social carbon cost, water consumption, in-situ recycling, ex-situ recycling, traffic noise and hazardous waste. Since noise and storm-water involve the design of surroundings and facilities, not only the pavement itself, the default values were used in this study.

The structure of the BE²ST-in-Highways™ system is presented in Figure 5.34 (RMRC 2010). The judgement layer is dependent on the mandatory screening layer, which is dependent on regulations of local, state and national organizations, as well as the specific requirements from the project. There are three classes (gold, silver, and bronze) for rating the overall performance of pavements.

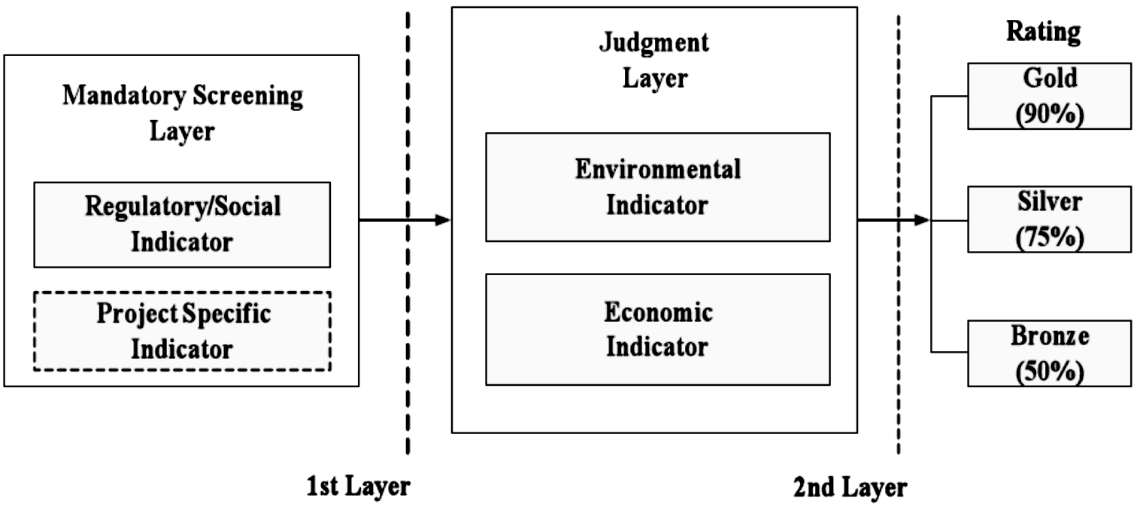


Figure 5.34. Structure of the BE²ST-in-Highways™ system (RMRC 2010).

5.2.2 Project Description and Model Creation

A two-lane roadway that is 1mile long and 24 feet wide was assumed in the analysis. The thickness of base layer is designed in accordance with AASHTO (1993). Service life is designed to be 20 years for each case, implying that after 20 years performance of pavement degrades to the degree that it is unable to meet normal usage. The design period is 40 years, which covers initial construction and one rehabilitation within the 20 year performance. The equivalent single axle load (ESAL) calculations for a low-volume traffic road is listed in Table 5.18.

.

Table 5.18. ESAL Calculations per AASHTO (1993).

Vehicle Description	Traffic Volume			Analysis Period (years)	Axle Load/Type			Gross Weight (pounds)	Equivalency Factors			ESAL's
	Quantity in the Design Lane	Days per Week	Weeks per Year		Axle 1 (kips)	Axle 2 (kips)	Axle 3 (kips)		Axle 1	Axle 2	Axle 3	
Passenger car	400	7	52	20	2/S	2/S	-	4,000	0.0002	0.0002	0	1,160
School bus	50	7	52	20	2	4	-	6,000	0.0002	0.0002	0	800
Package delivery truck	10	7	52	20	4	14	-	18,000	0.002	0.354	0	25,920
Beverage delivery truck	10	7	52	20	6	12	12/S	30,000	0.011	0.189	0.189	28,320
Garbage/dumpster truck	5	7	52	20	20	35/T		55,000	1.56	1.23	0	101,560
Semi-tractor trailer	25	7	52	20	12	34/T	34/T	80,000	0.189	1.08	1.08	427,520
Total	-	-	-	-	-	-	-	-	-	-	-	585,280

Note: S=single axle; T=tandem axle.

Parameters for structural design of flexible pavement and rigid pavement are presented in Table 5.19 and Table 5.21, respectively. The schematic of flexible pavement designs and rigid pavement designs are listed in Table 5.20 and Table 5.22, respectively. The replacement ratio of recycled materials keeps consistent with the previous work in PaLATE. Thickness of the surface and base of pavements is determined by structural requirements, different from the previous work in PaLATE. The details for thickness design is explained in the later sections.

Table 5.19. Flexible/Asphalt Pavement Design

Total AASHTO ESALs	585,280
Suggested mixture class	ESAL 2
Suggested binder grade	PG 64-22
Initial serviceability	4.5
Terminal serviceability	2.0
ΔPSI	2.5
Zr	-1.28
Combined Standard Deviation (Sd)	0.45
Reliability	90%
Resilient modulus of subgrade	10,389 psi
Required SN	2.700

Table 5.20. Schematic of Seven Alternative Flexible/Asphalt Pavement Designs

Design#	Surface type	Material in surface	Thickness of surface (in.)	Base type	Material in base	Thickness of base (in.)
1	HMA	Virgin materials	3	GAB	Virgin aggregate	4
2		35% RAP	3		100% RCA	3
3		35% RAP	3		100% RAP	3
4		45% RCA	3		100% RCA	3
5		45% RCA	3		100% RAP	3
6		10% FS	3.5	Cement-stabilized Base	90% FS + 10% cement	4
7		35% RAP	3	FASB	40% RAP + 60% RCA	1.4

Note: All the subgrade are made of virgin material, and the thickness of subgrade is 12 in.

Table 5.21. Rigid/Concrete Pavement Design

Roadway Classification	Local
Total Design ESALs	585,280
Suggested Mixture Class	ESAL 2
Terminal Serviceability	2.0
Combined Standard Error S_d	0.4
Change in Serviceability ΔPSI	2.5
Reliability Level	90%
Z_R	-1.282
Efficient modulus of subgrade reaction (k)	250 psi/in.
Joint Spacing	170 in.
Load Transfer Coefficient	3
Edge Support	1
Slab/Base Friction Coefficient	1.1
Drainage coefficient of base	1.2

Table 5.22. Schematic of Six Alternative Rigid/Concrete Pavement Designs

Design#	Surface type	Material in surface	Thickness of surface (in.)	Base type	Materials in base	Thickness of base (in.)
1	PCC	Virgin materials	8.5	GAB	Aggregate	7
2		50% RCA	8		100% RCA	7
3		100% RCA	8.5		100% RCA	7
4		40% RAP	8		100% RAP	7
5		100% RAP	6.5		100% RAP	7
6		20% FS	8.5		100% RCA	7

5.2.3. Assessment Results

5.2.3.1 Flexible/Asphalt Pavement

- Structural design

In conventional pavements, both the initial construction and the first rehabilitation use virgin materials. The existing materials from a conventional pavement are landfilled during the rehabilitation stage, while existing materials in recycled pavements are full-depth reclaimed in the rehabilitation stage. In this study, subbase and subgrade materials and properties were kept the same. Thus, the variables in life cycle analysis are HMA surface and base layer. The structural design for conventional asphalt pavement is summarized in Table 5.23. The total structural number (SN) was 2.72, greater than the minimum requirement of 2.70 from the pavement design analysis. The conventional pavement is considered the reference strategy for comparison purposes.

Table 5.23. Conventional asphalt pavement with virgin HMA & virgin GAB

Layer	New/Existing	Thickness (in)	Layer coefficient	Drainage coefficient	SN
HMA surface	N+N	3	0.44	1	1.32
GAB	N+N	4	0.12	1	0.48
Subgrade	-	12	0.08	1	0.96
Total	-	19	-	-	2.76

Strategy 1 is a recycled pavement, in which HMA surface consists of 35% RAP by weight (Shirodkar et al. 2011) and GAB base consists of 100% RCA (Aydilek et al. 2015). The structural design is summarized in Table 5.24. The total structural number (SN) is 2.738, greater than the minimum requirement of 2.700.

Table 5.24. Recycled asphalt pavement with 35% RAP in HMA & 100% RAP in GAB

Layer	New/Existing	Thickness (in)	Layer coefficient	Drainage coefficient	SN
HMA surface with 35% RAP	N+E	3	0.44	1	1.32
GAB with 100% RCA	N+E	3	0.166	1	0.498
Subgrade	-	12	0.08	1	0.96
Total	-	18	-	-	2.778

Note: HMA produced with RAP generally has higher stiffness and strength; thus, layer coefficient of 0.44 is also applied to HMA made with RAP.

Strategy 2 is a recycled pavement, in which HMA surface consists of 35% RAP by weight (Shirodkar et al. 2011) and GAB base consists of 100% RAP (Bennett and Maher 2005). The structural design is summarized in Table 5.25. The total structural number (SN) is 2.735, greater than the minimum requirement of 2.700.

Table 5.25. Recycled asphalt pavement with 35% RAP in HMA & 100% RAP in GAB

Layer	New/Existing	Thickness (in)	Layer coefficient	Drainage coefficient	SN
HMA surface with 35% RAP	N+E	3	0.44	1	1.32
GAB with 100% RAP	N+E	3	0.165	1	0.495
Subgrade	-	12	0.08	1	0.96
Total	-	18	-	-	2.775

Strategy 5.26 is a recycled pavement, in which HMA surface consists of 45% RCA by weight (Wong et al. 2007) and GAB base consists of 100% RCA (Aydilek et al. 2015). The structural design is summarized in Table 5.26. The total structural number (SN) is 2.735, greater than the minimum requirement of 2.700.

Table 5.26. Recycled asphalt pavement with 45% RCA in HMA & 100% RCA in GAB

Layer	New/Existing	Thickness (in)	Layer coefficient	Drainage coefficient	SN
HMA surface with 45% RCA	N+E	3	0.435	1	1.305
GAB with 100% RCA	N+E	3	0.166	1	0.498
Subgrade	-	12	0.08	1	0.96
Total	-	18	-	-	2.763

Strategy 4 is a recycled pavement, in which HMA surface consists of 45% RCA by weight (Wong et al. 2007) and GAB base consists of 100% RAP (Bennett and Maher 2005). The structural design is summarized in Table 5.27. The total structural number (SN) is 2.720, greater than the minimum requirement of 2.700.

Table 5.27. Recycled asphalt pavement with 45% RCA in HMA & 100% RAP in GAB

Layer	New/Existing	Thickness (in)	Layer coefficient	Drainage coefficient	SN
HMA surface with 45% RCA	N+E	3	0.435	1	1.305
GAB with 100% RAP	N+E	3	0.165	1	0.495
Subgrade	-	12	0.08	1	0.96
Total	-	18	-	-	2.760

Strategy 5 is a recycled pavement, in which HMA surface consists of 10% FS by weight (Bakis et al. 2006 and Braham 2002) and base is made of 90% FS and 10% cement additive (Gedik 2008). The structural design is summarized in Table 5.28. The total structural number (SN) is 2.716, greater than the minimum requirement of 2.700.

Table 5.28. Recycled asphalt pavement with 10% FS in HMA & 90% FS in Base

Layer	New/Existing	Thickness (in)	Layer coefficient	Drainage coefficient	SN
HMA surface with 10% FS	N+E	3.5	0.435	1	1.54
Base with 90% FS	N+E	4	0.064	1	0.256
Subgrade	-	12	0.08	1	0.96
Total	-	19.5	-	-	2.756

Strategy 6 is a recycled pavement, in which HMA surface consists of 35% RAP by weight (Shirodkar et al. 2011) and FASB consists of 40% RAP plus 60% RCA (Schwartz and Khosravifar 2013). The structural design is summarized in Table 5.29. The total structural number (SN) is 2.730, greater than the minimum requirement of 2.700.

Table 5.29. Recycled asphalt pavement with 30% RAP in HMA & 40% RAP + 60% RCA in FASB

Layer	New/Existing	Thickness (in)	Layer coefficient	Drainage coefficient	SN
HMA surface with 35% RAP	N+E	3	0.44	1	1.32
FASB with 40% RAP + 60% RCA	N+E	1.4	0.35	1	0.490
Subgrade	-	12	0.08	1	0.96
Total	-	16.4	-	-	2.770

◆ Weighting system for BE²ST-in-Highways™

The weighting system of BE²ST-in-Highways™ comprises eight environmental indicators and one economic indicator. The weights (level of importance) of these indicators are dependent on the requirement of specific projects. In this study, storm-water design and noise reduction methods are assumed to be the same for each case, but different pavement materials result in different levels of traffic noise. For example, the default score of asphalt pavement is 1, while the default score of concrete pavement is 0. As a result, cost for storm-water management has not been included in the total cost. Traffic noise is granted a light weight (2%) in the weighting system. Other indicators take up 10%-15% of the total weight, respectively, which are nearly equal. The weighting system is listed in Table 5.30.

Table 5.30. Weighting System

Indicators	Weighting (%)	Weight
Energy	10.00	0.10
Global Warming	10.00	0.10
In situ Recycle	15.00	0.15
Ex situ Recycle	15.00	0.15
Water Consumption	10.00	0.10
LCC	15.00	0.15
SCC	10.00	0.10
Traffic Noise	2.00	0.02
Hazardous Waste	13.00	0.13
Total	100.00	1.00

Note: LCC = life cycle cost; SCC = social cost of carbon.

◆ Results and discussions

Tables 5.31-5.36 compare the performance of recycled pavements with conventional pavements. In these tables, life cycle analyses (life cycle cost and life cycle environmental effect) were conducted by using PaLATE. The social cost of carbon (SCC) is the cost of reducing global warming potential, often used by agencies (e.g., a state DOT) to enforce sustainable construction. Average SCC are $5, $21 and $35 per Mg estimated in 2010 (in 2007 dollars) at the 5, 3, and 2.5 percent discount rates, respectively (RMRC 2010). RMRC (2010) suggested using $65 per Mg in calculating the SCC to consider the worst-case scenario. The default targets were used in this study, which can be modified with the requirements of a specific project.

Accomplished scores and awarded labels are listed in Table 5.37. An accomplished score is the sum of scores gained by indicators multiplied by their weight. “Gold” label is granted for a score between 100 and 90, “silver” for a score between 90 and 75, and “bronze” for a score between 75 and 50. An accomplished score less than 50 implies the recycled pavements are not as “green” as the conventional pavements. The results may vary with varied weighting system and/or varied targets.

As shown in Table 5.37, recycled asphalt pavements receive either “gold” or “silver” labels, implying excellent performance of these recycled pavements. This performance should be attributed to the FDR technology, by which in-situ recycling can be achieved. The conventional asphalt pavement requires landfilling the old materials and hauling the new materials to site in the rehabilitation stage, resulting in higher consumption in resources, higher gas emission and higher generation of hazardous waste.

Strategy 1 has a high recycled rate of 75.6%, shows a 57% reduction in CO_2, a 55% reduction in energy, a reduction of 54% in life cycle cost and a $9750 saving per mile in SCC (Table 5.31). Strategy 2 is similar to Strategy 1, exhibiting a 56% reduction in CO2, a 57% reduction in energy, a reduction of 54% in life cycle cost and a $9880 saving per mile in SCC (Table 5.32). These results are consistent with the study of Lee et al. (2011). Lee et al. (2011) indicated that asphalt pavement, in which the HCA surface contains 15% RAP and a base made of recycled pavement materials, showed a 43% reduction in CO_2, a 43% reduction in energy consumption, a 54% reduction in life cycle cost and a $16,967 saving per km in SCC. The divergence in savings of SCC is due to different dimensions of pavement and distance of transportation assumed.

Strategy 5 (10% FS in HMA and 90% FS in base) received a “silver” label due to high greenhouse gas emission and high water consumption (Table 5.35). Strategy 6 (35% RAP in HMA and 40% RAP+60% RCA in FASB) also receives “silver” due to a low ex-situ recycling rate (Table 5.36). FASB base has a high ex-situ recycling rate, in which aggregates are made of RCA and RAP. However, the volume of FASB (1.4 in. thickness) is far less than that of HMA layer (3 in. thickness), hence the HMA layer controls the awarded label (Tables 5.20 and 5.29).

Comparing Strategies 1 and 2, or Strategies 3 (45% RCA in HMA and 100% RCA in GAB) and 4 (45% RCA in HMA and 100% RAP in GAB), GAB made with 100% RAP and 100% RCA have nearly the same accomplished score (Table 5.37), though the score for single indicator is different (Tables 5.31-5.34). Comparing Strategies 1 and 6, recycled GAB shows a higher accomplished score than recycled FASB (Table 5.37), though FASB is much thinner than GAB. Strategy 5 has two variables; a different overlay and a different base. Since the thickness of cement-stabilized base is greater than that of other bases (Table 5.28), we can infer that the cement-stabilized base should be not as “green” as other bases.

Figures 5.35-5.40 present the AMOEBA graphs for different strategies. The AMOEBA graphs allow a quantitative comparison between the target score (two scores) and the score gained in the project. Using these graphs, the pros and cons of each strategy can be identified easily, which can help designers advance their design schemes and achieve their goals for a green highway design. For examples, Strategies1-5 all have a deficiency in SCC (Figure 5.35-5.40), implying that the cost reduced in managing carbon dioxide is not satisfactory. SCC is related to the emission of greenhouse gas, the cost to prevent global warming, as well as the yearly salary of one job (the base of the target set). Decision-makers can choose other schemes or strategies to reduce the greenhouse emission. It can be seen in Figure 5.39 that Strategy 5 (10% FS in HMA and 90% FS in base) has a deficiency in greenhouse gas reduction and water saving, and in Figure 5.40 that Strategy 6 (35% RAP in HMA & 40% RAP+60% RCA in FASB) has a deficiency in ex-situ recycling rate.

Table 5.31. Results of BE^{2}ST-in-Highway for Strategy 1 (asphalt pavement).

Criteria	Unit	Target	Reference	Strategy 1	Perfor-mance	Score
Energy Use	**MJ**	**≥ 10% Reduction (1 pt)** **≥ 20% Reduction (2 ptss)**	4,708,671	2,113,977	55.10%	2.00
GWP	**Mg**	**≥ 10% Reduction (1 pt)** **≥ 20% Reduction (2 ptss)**	263	113	57.03%	2.00
In Situ Recycling	**CY**	**≥ 10% Recycling Rate (1 pt)** **≥ 20% Recycling Rate (2 ptss)**	0.00	0.500	50.00%	2.00
Ex situ Recycling	**CY**	**≥ 10% Recycled Content (1 pt)** **≥ 20% Recycled Content (2 ptss)**	0.00	0.256	25.60%	2.00
Water Consumption	**kg**	**≥ 5% Reduction (1 pt)** **≥ 10% Reduction (2 ptss)**	1,230	617	49.84%	2.00
Life Cycle Cost	**$**	**≥ 10% Reduction (1 pt)** **≥20% Reduction (2 ptss)**	347,975	161,152	53.69%	2.00
Social Carbon Cost	**$**	**≥ $19,750/mi Saving (1 pt)** **≥ $39,500/mi Saving (2 ptss)**	$17,095.00	$7,345.00	$9,750	0.49
Traffic Noise	**no unit**	**HMA (1 pt)** **SMA or OGFC (2 ptss)**	1	1	1	1.00
Hazardous Waste	**kg**	**≥5% Reduction (1 pt)** **≥10% Reduction (2 ptss)**	45,539	25,541	43.91%	2.00

Note: The discount rate is 4%. Performance is the degree of achievement in reducing the consumption of resources, reducing the generation of gas and hazardous waste, cutting down the costs, and increasing recycling rate. Positive performance means more environmental benefits gained, negative performance means more environmental loads caused.

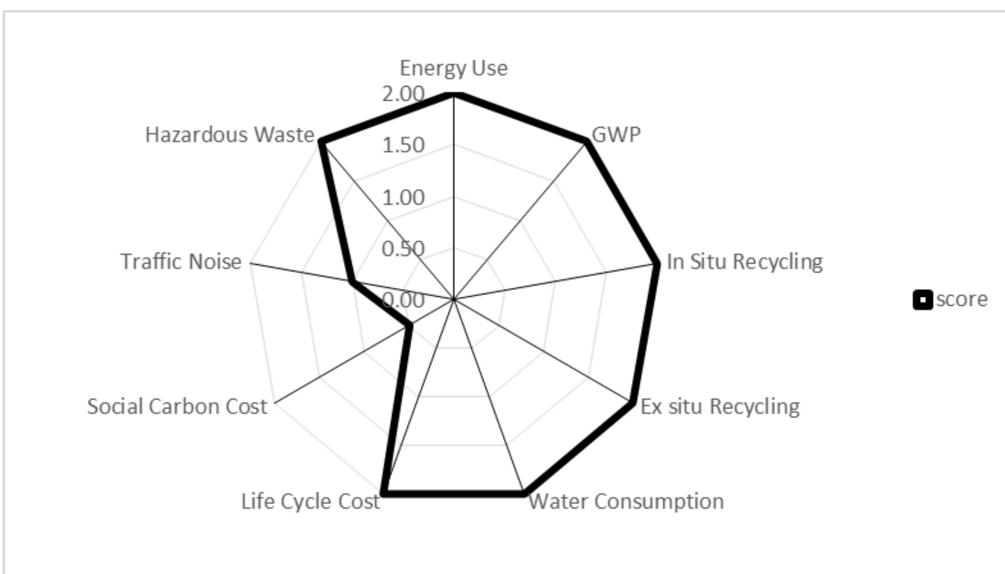


Figure 5.35. AMOEBA graph for recycled asphalt pavement of Strategy 1.

Table 5.32. Results of BE^{2}ST-in-Highway for Strategy 2 (asphalt pavement).

Criteria	Unit	Target	Reference	Strategy 2	Performance	Score
Energy Use	**MJ**	**≥ 10% Reduction (1 pt)** **≥ 20% Reduction (2 pts)**	4,708,671	2,032,270	56.84%	2.00
GWP	**Mg**	**≥ 10% Reduction (1 pt)** **≥ 20% Reduction (2 pts)**	263	111	57.79%	2.00
In Situ Recycling	**CY**	**≥ 10% Recycling Rate (1 pt)** **≥ 20% Recycling Rate (2 pts)**	0.00	0.500	50.00%	2.00
Ex situ Recycling	**CY**	**≥ 10% Recycled Content (1 pt)** **≥ 20% Recycled Content (2 pts)**	0.00	0.256	25.60%	2.00
Water Consumption	**kg**	**≥ 5% Reduction (1 pt)** **≥ 10% Reduction (2 pts)**	1,230	616	49.92%	2.00
Life Cycle Cost	**$**	**≥ 10% Reduction (1 pt)** **≥20% Reduction (2 pts)**	347,975	157,927	54.62%	2.00
Social Carbon Cost	**$**	**≥ $19,750/mi Saving (1 pt)** **≥ $39,500/mi Saving (2 pts)**	$17,095.00	$7,215.00	$9,880	0.50
Traffic Noise	**no unit**	**HMA (1 pt)** **SMA or OGFC (2 pts)**	1	1	1	1.00
Hazardous Waste	**kg**	**≥5% Reduction (1 pt)** **≥10% Reduction (2 pts)**	45,539	25,462	44.09%	2.00

Note: The discount rate is 4%. Performance is the degree of achievement in reducing the consumption of resources, reducing the generation of gas and hazardous waste, cutting down the costs, and increasing recycling rate. Positive performance means more environmental benefits gained, negative performance means more environmental loads caused.

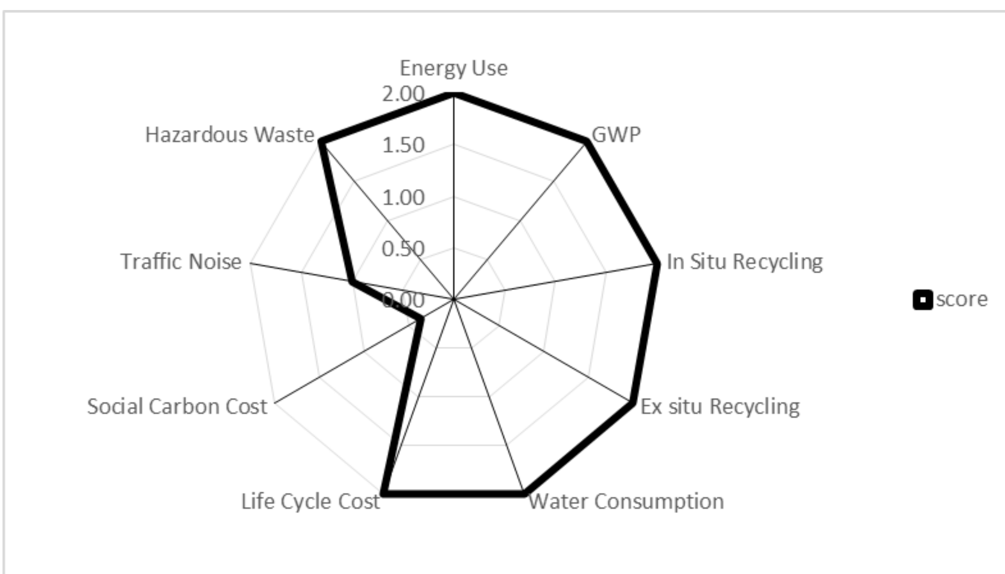


Figure 5.36. AMOEBA graph for recycled asphalt pavement of Strategy 2.

Table 5.33. Results of BE^{2}ST-in-Highway for Strategy 3 (asphalt pavement).

Criteria	Unit	Target	Reference	Strategy 3	Performance	Score
Energy Use	**MJ**	**≥ 10% Reduction (1 pt)** **≥ 20% Reduction (2 pts)**	4,708,671	2,901,582	38.38%	2.00
GWP	**Mg**	**≥ 10% Reduction (1 pt)** **≥ 20% Reduction (2 pts)**	263	163	38.02%	2.00
In Situ Recycling	**CY**	**≥ 10% Recycling Rate (1 pt)** **≥ 20% Recycling Rate (2 pts)**	0.00	0.500	50.00%	2.00
Ex situ Recycling	**CY**	**≥ 10% Recycled Content (1 pt)** **≥ 20% Recycled Content (2 pts)**	0.00	0.3285	32.85%	2.00
Water Consumption	**kg**	**≥ 5% Reduction (1 pt)** **≥ 10% Reduction (2 pts)**	1,230	939	23.66%	2.00
Life Cycle Cost	**$**	**≥ 10% Reduction (1 pt)** **≥20% Reduction (2 pts)**	347,975	177,494	48.99%	2.00
Social Carbon Cost	**$**	**≥ $19,750/mi Saving (1 pt)** **≥ $39,500/mi Saving (2 pts)**	$17,095.00	$10,595.00	$6,500	0.33
Traffic Noise	**no unit**	**HMA (1 pt)** **SMA or OGFC (2 pts)**	1	1	1	1.00
Hazardous Waste	**kg**	**≥5% Reduction (1 pt)** **≥10% Reduction (2 pts)**	45,539	39,525	13.21%	2.00

Note: The discount rate is 4%. Performance is the degree of achievement in reducing the consumption of resources, reducing the generation of gas and hazardous waste, cutting down the costs, and increasing recycling rate. Positive performance means more environmental benefits gained, negative performance means more environmental loads caused.

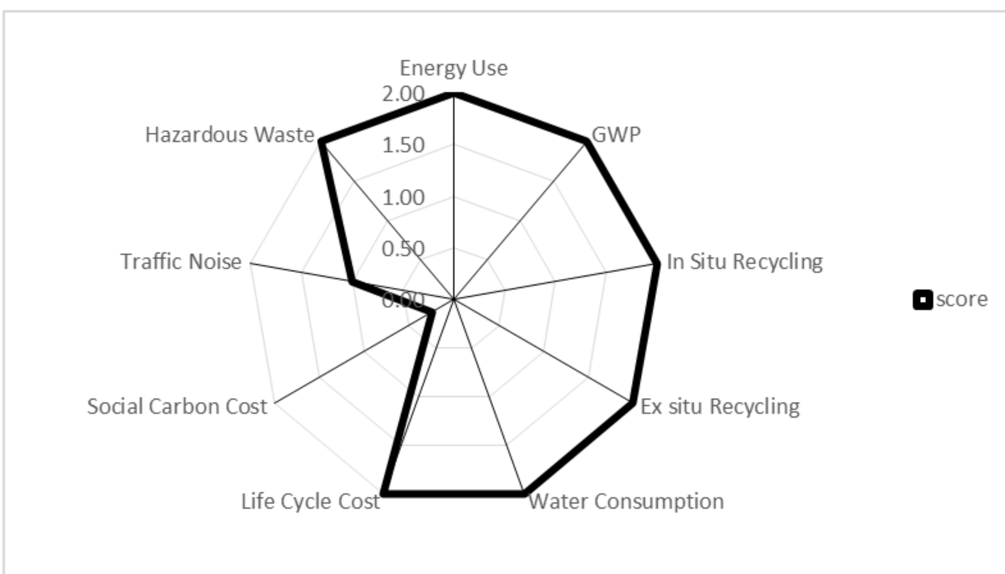


Figure 5.37. AMOEBA graph for recycled asphalt pavement of Strategy 3.

Table 5.34. Results of BE^{2}ST-in-Highway for Strategy 4 (asphalt pavement).

Criteria	Unit	Target	Reference	Strategy 4	Performance	Score
Energy Use	**MJ**	**≥ 10% Reduction (1 pt)** **≥ 20% Reduction (2 pts)**	4,708,671	2,899,697	38.42%	2.00
GWP	**Mg**	**≥ 10% Reduction (1 pt)** **≥ 20% Reduction (2 pts)**	263	163	38.02%	2.00
In Situ Recycling	**CY**	**≥ 10% Recycling Rate (1 pt)** **≥ 20% Recycling Rate (2 pts)**	0.00	0.500	50.00%	2.00
Ex situ Recycling	**CY**	**≥ 10% Recycled Content (1 pt)** **≥ 20% Recycled Content (2 pts)**	0.00	0.3285	32.85%	2.00
Water Consumption	**kg**	**≥ 5% Reduction (1 pt)** **≥ 10% Reduction (2 pts)**	1,230	939	23.66%	2.00
Life Cycle Cost	**$**	**≥ 10% Reduction (1 pt)** **≥20% Reduction (2 pts)**	347,975	177,167	49.09%	2.00
Social Carbon Cost	**$**	**≥ $19,750/mi Saving (1 pt)** **≥ $39,500/mi Saving (2 pts)**	$17,095.00	$10,595.00	$6,500	0.33
Traffic Noise	**no unit**	**HMA (1 pt)** **SMA or OGFC (2 pts)**	1	1	1	1.00
Hazardous Waste	**kg**	**≥5% Reduction (1 pt)** **≥10% Reduction (2 pts)**	45,539	39,511	13.24%	2.00

Note: The discount rate is 4%. Performance is the degree of achievement in reducing the consumption of resources, reducing the generation of gas and hazardous waste, cutting down the costs, and increasing recycling rate. Positive performance means more environmental benefits gained, negative performance means more environmental loads caused.

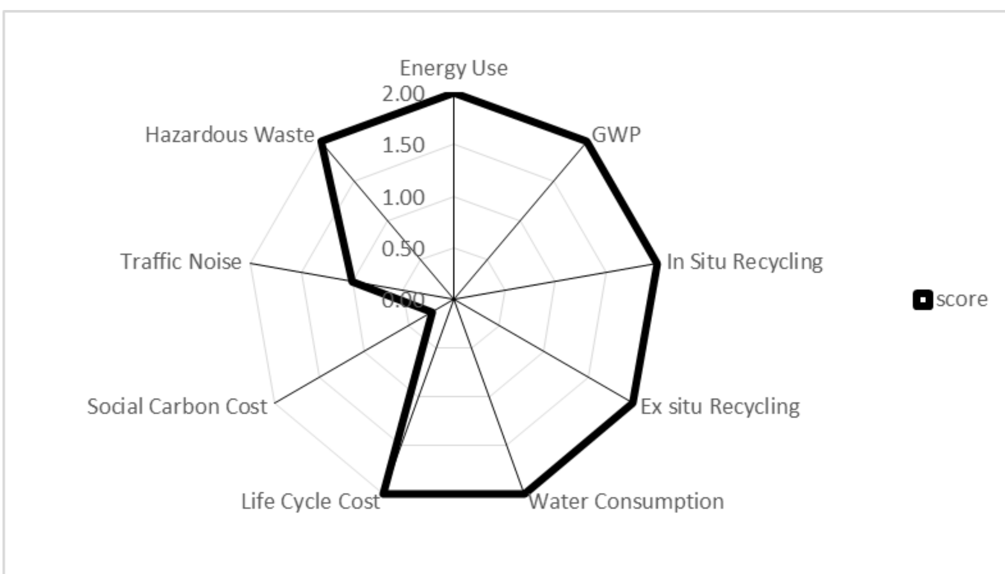


Figure 5.38. AMOEBA graph for recycled asphalt pavement of Strategy 4.

Table 5.35. Results of BE²ST-in-Highway for Strategy 5 (asphalt pavement).

Criteria	Unit	Target	Reference	Strategy 5	Performance	Score
Energy Use	**MJ**	**≥ 10% Reduction (1 pt)** **≥ 20% Reduction (2 pts)**	4,708,671	3,718,206	21.03%	2.00
GWP	**Mg**	**≥ 10% Reduction (1 pt)** **≥ 20% Reduction (2 pts)**	263	217	17.49%	1.75
In Situ Recycling	**CY**	**≥ 10% Recycling Rate (1 pt)** **≥ 20% Recycling Rate (2 pts)**	0.00	0.5000	50.00%	2.00
Ex situ Recycling	**CY**	**≥ 10% Recycled Content (1 pt)** **≥ 20% Recycled Content (2 pts)**	0.00	0.2540	25.40%	2.00
Water Consumption	**kg**	**≥ 5% Reduction (1 pt)** **≥ 10% Reduction (2 pts)**	1,230	1,229	0.08%	0.02
Life Cycle Cost	**$**	**≥ 10% Reduction (1 pt)** **≥20% Reduction (2 pts)**	347,975	178,198	48.79%	2.00
Social Carbon Cost	**$**	**≥ $19,750/mi Saving (1 pt)** **≥ $39,500/mi Saving (2 pts)**	$17,095.00	$14,105.00	$2,990	0.15
Traffic Noise	**no unit**	**HMA (1 pt)** **SMA or OGFC (2 pts)**	1	1	1	1.00
Hazardous Waste	**kg**	**≥5% Reduction (1 pt)** **≥10% Reduction (2 pts)**	45,539	30,797	32.37%	2.00

Note: The discount rate is 4%. Performance is the degree of achievement in reducing the consumption of resources, reducing the generation of gas and hazardous waste, cutting down the costs, and increasing recycling rate. Positive performance means more environmental benefits gained, negative performance means more environmental loads caused.

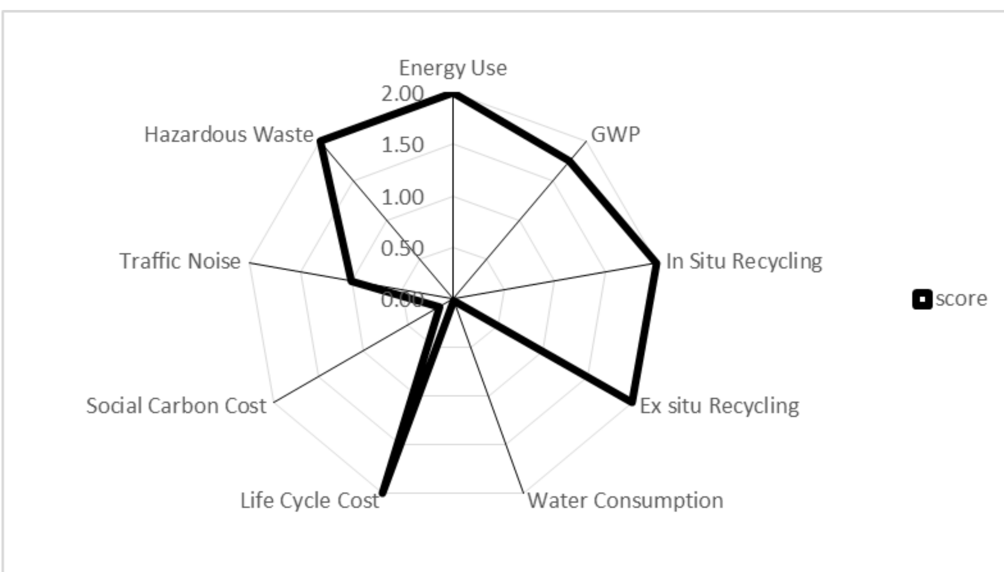


Figure 5.39. AMOEBA graph for recycled asphalt pavement of Strategy 5.

Table 5.36. Results of BE^{2}ST-in-Highway for Strategy 6 (asphalt pavement).

Criteria	Unit	Target	Reference	Strategy 6	Performance	Score
Energy Use	**MJ**	**≥ 10% Reduction (1 pt)** **≥ 20% Reduction (2 pts)**	4,708,671	2,013,328	57.24%	2.00
GWP	**Mg**	**≥ 10% Reduction (1 pt)** **≥ 20% Reduction (2 pts)**	263	114	56.65%	2.00
In Situ Recycling	**CY**	**≥ 10% Recycling Rate (1 pt)** **≥ 20% Recycling Rate (2 pts)**	0.00	0.5000	50.00%	2.00
Ex situ Recycling	**CY**	**≥ 10% Recycled Content (1 pt)** **≥ 20% Recycled Content (2 pts)**	0.00	0.1575	15.75%	1.58
Water Consumption	**kg**	**≥ 5% Reduction (1 pt)** **≥ 10% Reduction (2 pts)**	1,230	807	34.39%	2.00
Life Cycle Cost	**$**	**≥ 10% Reduction (1 pt)** **≥20% Reduction (2 pts)**	347,975	114,868	66.99%	2.00
Social Carbon Cost	**$**	**≥ $19,750/mi Saving (1 pt)** **≥ $39,500/mi Saving (2 pts)**	$17,095.00	$7,410.00	$9,685	0.49
Traffic Noise	**no unit**	**HMA (1 pt)** **SMA or OGFC (2 pts)**	1	1	1	1.00
Hazardous Waste	**kg**	**≥5% Reduction (1 pt)** **≥10% Reduction (2 pts)**	45,539	34,422	24.41%	2.00

Note: The discount rate is 4%. Performance is the degree of achievement in reducing the consumption of resources, reducing the generation of gas and hazardous waste, cutting down the costs, and increasing recycling rate. Positive performance means more environmental benefits gained, negative performance means more environmental loads caused.

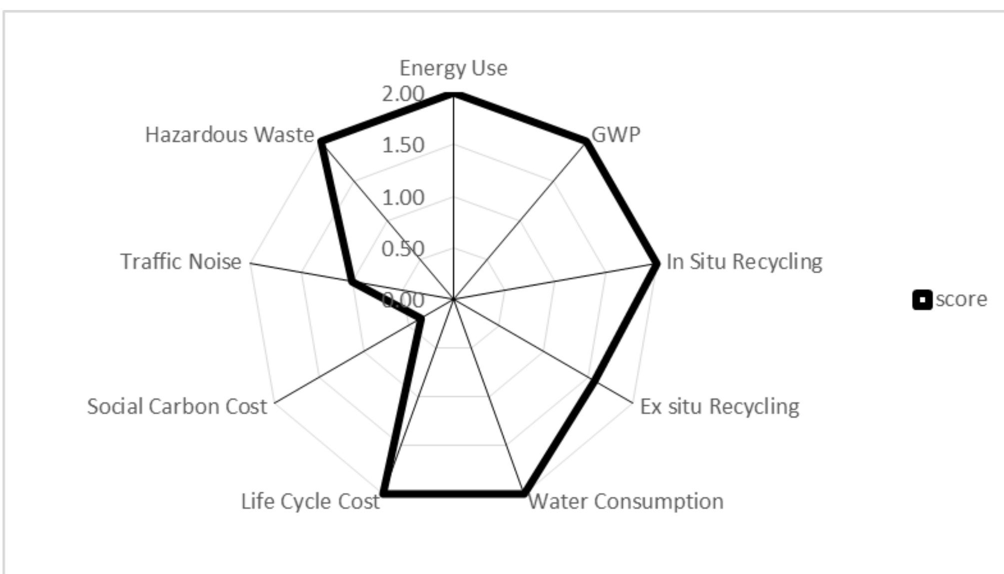


Figure 5.40. AMOEBA graph for recycled asphalt pavement of Strategy 6.

Table 5.37. Rating of BE^{2}ST-in-Highway for asphalt pavement.

Strategy #	Scenarios	Accomplished Score	Awarded Label
1	35% RAP in HMA & 100% RCA in GAB	91.47%	Gold
2	35% RAP in HMA & 100% RAP in GAB	91.50%	Gold
3	45% RCA in HMA & 100% RCA in GAB	90.65%	Gold
4	45% RCA in HMA & 100% RAP in GAB	90.65%	Gold
5	10% FS in HMA & 90% FS in Base	78.58%	Silver
6	35% RAP in HMA & 40% RAP+60% RCA in FASB	88.26%	Silver

5.2.3.2 Rigid/Concrete Pavement

- Structural design

In conventional pavements, virgin materials are used during both the initial construction and the first rehabilitation stage. The existing materials from conventional pavement are landfilled in rehabilitation stage. In recycled pavements, PCC surface is reclaimed and used in GAB base, and GAB base is recycled and used in PCC surface. In this study, subgrade properties are kept the same in each case. The variables in life cycle analysis are PCC surface and base layer. The structural design for conventional concrete pavement is summarized in Table 5.38. The conventional pavement is considered as the reference strategy for comparison purposes.

Table 5.38. Conventional concrete pavement with virgin PCC & virgin GAB

Layer	Thickness (in)	Elastic Modulus (ksi)	Modulus of Rupture (psi)	Poisson's Ratio
Conventional PCC Layer	8.5	4091	590	0.2
Aggregate Base	7	15,000	-	-
Subgrade	-	-	-	-

Note: Poisson's ratio is between 0.15 and 0.2 for PCC layer.

Strategy 1 is a recycled pavement, in which PCC surface consists of 50% RCA (Volz et al. 2014) and GAB base consists of 100% RCA (Aydilek et al. 2015). The structural design is summarized in Table 5.39.

Table 5.39. Recycled concrete pavement with 50% RCA in PCC &100% RCA in GAB.

Layer	Thickness (in)	Elastic Modulus (ksi)	Modulus of Rupture (psi)	Poisson's Ratio
PCC Layer with 50% RCA	8	3811	610	0.2
GAB with 100% RCA	7	20,000	-	-
Subgrade	-	-	-	-

Note: Poisson's ratio is between 0.15 and 0.2 for PCC layer.

Strategy 2 is a recycled pavement, in which PCC surface consists of 100% RCA (Volz et al. 2014) and GAB base consists of 100% RCA (Aydilek et al. 2015). The structural design is summarized in Table 5.40.

Table 5.40. Recycled concrete pavement with 100% RCA in PCC & 100% RCA in GAB.

Layer	Thickness (in)	Elastic Modulus (ksi)	Modulus of Rupture (psi)	Poisson's Ratio
PCC Layer with 100% RCA	8.5	4,243	605	0.2
GAB with 100% RCA	7	20,000	-	-
Subgrade	-	-	-	-

Note: Poisson's ratio is between 0.15 and 0.2 for PCC layer.

Strategy 3 is a recycled pavement, in which PCC surface consists of 40% RAP (Hossiney 2012) and GAB base consists of 100% RAP (Bennett and Maher 2005). The structural design is summarized in Table 5.41.

Table 5.41. Recycled concrete pavement with 40% RAP in PCC & 100% RAP in GAB.

Layer	Thickness (in)	Elastic Modulus (ksi)	Modulus of Rupture (psi)	Poisson's Ratio
PCC Layer with 40% RAP	8	2,800	517	0.2
GAB with 100% RAP	7	20,000	-	-
Subgrade	-	-	-	-

Note: Poisson's ratio is between 0.15 and 0.2 for PCC layer.

Strategy 4 is a recycled pavement, in which PCC surface consists of 100% RAP (Hossiney 2012) and GAB base consists of 100% RAP (Bennett and Maher 2005). The structural design is summarized in Table 5.42.

Table 5.42. Recycled concrete pavement with 100% RAP in PCC & 100% RAP in GAB.

Layer	Thickness (in)	Elastic Modulus (ksi)	Modulus of Rupture (psi)	Poisson's Ratio
PCC Layer with 100% RAP	6.5	1,250	370	0.2
GAB with 100% RAP	7	20,000	-	-
Subgrade	-	-	-	-

Note: Poisson's ratio is between 0.15 and 0.2 for PCC layer.

Strategy 5 is a recycled pavement, in which PCC surface consists of 20% RAP (Singh and Siddique 2012, Siddique et al. 2009) and GAB base consists of 100% RCA (Bennett and Maher 2005). The structural design is summarized in Table 5.43.

Table 5.43. Recycled concrete pavement with 20% FS in PCC & 100% RCA in GAB.

Layer	Thickness (in)	Elastic Modulus (ksi)	Modulus of Rupture (psi)	Poisson's Ratio
PCC Layer with 20% FS	8.5	4,525	594	0.2
GAB with 100% RCA	7	20,000	-	-
Subgrade	-	-	-	-

Note: Poisson's ratio is between 0.15 and 0.2 for PCC layer.

◆ Results and discussions

Life cycle cost and environmental analysis were conducted using PaLATE. Tables 5.44-5.48 compare the performance of recycled pavements with conventional pavements. Accomplished scores and awarded labels are listed in Table 5.49. The results may be different in a different weighting system. Figures 5.41-5.45 present the AMOEBA graphs for different strategies. Using these graphs, the pros and cons of each strategy can be identified easily, which can help decision makers identify the optimum scheme and allow designers to modify their design schemes for a greener highway.

Comparing Strategies 1 and 2, when RCA content increases from 50% to 100% in PCC surface, the accomplished score falls by about 20% (Table 5.49). This is because incorporating RCA leads to higher energy consumption, higher water usage, higher greenhouse gas emission and a higher production of hazardous waste (Tables 5.44 and 5.45). PCC made with 100% RCA is 0.5 in. thicker than PCC made with 50% RCA (Table 5.22), which is a reason for the increment in consumption and emission. The thickness of layer is a structural requirement. RCA improves elastic modulus and reduces modulus of rupture of PCC (Tables 5.39 and 5.40); therefore, PCC with 100% RCA should become thicker to meet required stiffness.

Strategy 2 (100% RCA in PCC, 100% RCA in GAB) received the lowest score dropping approximately to 50% (Table 5.49), indicating the recycled pavement is as "green" as conventional pavement. A score of 50% is the threshold for whether recycled pavement is more "green" than conventional pavement. As seen in Table 5.45, Strategy 2 has a high recycling rate (50%+31.55%) and cost savings (41.6%), while water consumption and hazardous waste generation are higher than conventional concrete pavement. Based on this result, one can conclude that concrete pavements with 100% RCA in GAB and RCA replacement of coarse aggregates at any percentage in PCC should be more "green" than conventional concrete pavements. The accomplished score increases with increasing RCA replacement ratio until the optimum replacement ratio (between 35% and 100%), after which the score will decrease to 50%.

Comparing Strategies 3 and 4, when RAP content increases from 40% to 100% in PCC layer, the accomplished score rises by 20% and label upgrades from "silver" to "gold" (Table 5.49). The reason is that RAP replacing virgin aggregates reduces energy consumption, water usage, greenhouse gas emission and the production of hazardous waste (Table 5.46 and Table 5.47). Table 5.22 shows that PCC with 100% RAP is 1.5 in. thinner than PCC with 40% RAP, which is a reason for the reduction in consumption and emission. The reduced thickness is due to the reduced elastic modulus when RAP is incorporated into PCC (Table 5.41 and Table 5.42), though the modulus of rupture for PCC made with RAP reduces as well.

Strategy 4 (100% RAP in PCC, 100% RAP in GAB) is labeled "gold," implying excellent performance of the recycled pavement (Table 5.49). In addition, PCC made with 100% RAP has the lowest thickness (Table 5.22), which is a reason that Strategy 4 has the most reduction in consumption and emission. Through the above analysis, one can conclude that RAP replacement of both coarse and fine aggregates at any percentage in PCC should improve the performance (accomplished score) of a recycled highway, and the score increases as replacement ratio increases.

Strategy 5 (20% FS in PCC, 100% RCA in GAB) is labeled "bronze" for its higher energy consumption, higher water usage and higher greenhouse gas emission (Table 5.48). However, the reduction of hazardous waste is higher than the other strategies (Table 5.48). PCC with 20% FS has a higher thickness compared to other recycled PCC (Tables 5.22 and 5.43), which is a reason for the higher consumption and emission. Since the score for a single indicator cannot be negative, water consumption and greenhouse gas emission that is too high cannot be reflected in the rating system. Otherwise, the total score of Strategy 5 may be reduced slightly. This does not mean that FS replacement of fine aggregates in PCC is not recommended, however, since additives can be used to modify their properties.Table 5.44. Results of BE^{2}ST-in-Highway for Strategy 1 (concrete pavement).

Criteria	Unit	Target	Reference	Strategy 1	Performance	Score
Energy Use	**MJ**	**≥ 10% Reduction (1 pt)** **≥ 20% Reduction (2 pts)**	15,213,544	13,382,180	12.04%	1.20
GWP	**Mg**	**≥ 10% Reduction (1 pt)** **≥ 20% Reduction (2 pts)**	1,066	961	9.85%	0.98
In Situ Recycling	**CY**	**≥ 10% Recycling Rate (1 pt)** **≥ 20% Recycling Rate (2 pts)**	0.00	0.5000	50.00%	2.00
Ex situ Recycling	**CY**	**≥ 10% Recycled Content (1 pt)** **≥ 20% Recycled Content (2 pts)**	0.00	0.2754	27.54%	2.00
Water Consumption	**kg**	**≥ 5% Reduction (1 pt)** **≥ 10% Reduction (2 pts)**	5,381	5,076	5.67%	1.13
Life Cycle Cost	**$**	**≥ 10% Reduction (1 pt)** **≥20% Reduction (2 pts)**	1,097,804	652,312	40.58%	2.00
Social Carbon Cost	**$**	**≥ $19,750/mi Saving (1 pt)** **≥ $39,500/mi Saving (2 pts)**	$69,290.00	$62,465.00	$6,825	0.35
Traffic Noise	**no unit**	**HMA (1 pt)** **SMA or OGFC (2 pts)**	0	0	0	0.00
Hazardous Waste	**kg**	**≥5% Reduction (1 pt)** **≥10% Reduction (2 pts)**	21,811	20,682	5.18%	1.04

Note: The discount rate is 4%. Performance is the degree of achievement in reducing the consumption of resources, reducing the generation of gas and hazardous waste, cutting down the costs, and increasing recycling rate. Positive performance means more environmental benefits gained, negative performance means more environmental loads caused.

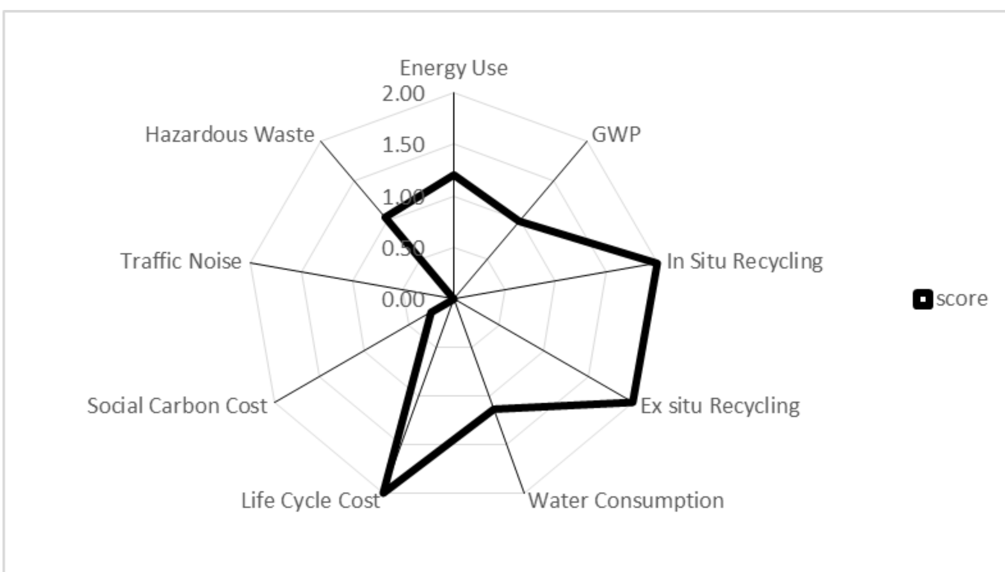


Figure 5.41. AMOEBA graph for recycled concrete pavement of Strategy 1.

Table 5.45. Results of BE^{2}ST-in-Highway for Strategy 2 (concrete pavement).

Criteria	Unit	Target	Reference	Strategy 2	Performance	Score
Energy Use	**MJ**	**≥ 10% Reduction (1 pt)** **≥ 20% Reduction (2 pts)**	15,213,544	14,279,082	6.14%	0.61
GWP	**Mg**	**≥ 10% Reduction (1 pt)** **≥ 20% Reduction (2 pts)**	1,066	1,035	2.91%	0.29
In Situ Recycling	**CY**	**≥ 10% Recycling Rate (1 pt)** **≥ 20% Recycling Rate (2 pts)**	0.00	0.5000	50.00%	2.00
Ex situ Recycling	**CY**	**≥ 10% Recycled Content (1 pt)** **≥ 20% Recycled Content (2 pts)**	0.00	0.3155	31.55%	2.00
Water Consumption	**kg**	**≥ 5% Reduction (1 pt)** **≥ 10% Reduction (2 pts)**	5,381	5,446	-1.21%	0.00
Life Cycle Cost	**$**	**≥ 10% Reduction (1 pt)** **≥20% Reduction (2 pts)**	1,097,804	641,130	41.60%	2.00
Social Carbon Cost	**$**	**≥ $19,750/mi Saving (1 pt)** **≥ $39,500/mi Saving (2 pts)**	$69,290.00	$67,275.00	$2,015	0.10
Traffic Noise	**no unit**	**HMA (1 pt)** **SMA or OGFC (2 pts)**	0	0	0	0.00
Hazardous Waste	**kg**	**≥5% Reduction (1 pt)** **≥10% Reduction (2 pts)**	21,811	22,573	-3.49%	0.00

Note: The discount rate is 4%. Performance is the degree of achievement in reducing the consumption of resources, reducing the generation of gas and hazardous waste, cutting down the costs, and increasing recycling rate. Positive performance means more environmental benefits gained, negative performance means more environmental loads caused.

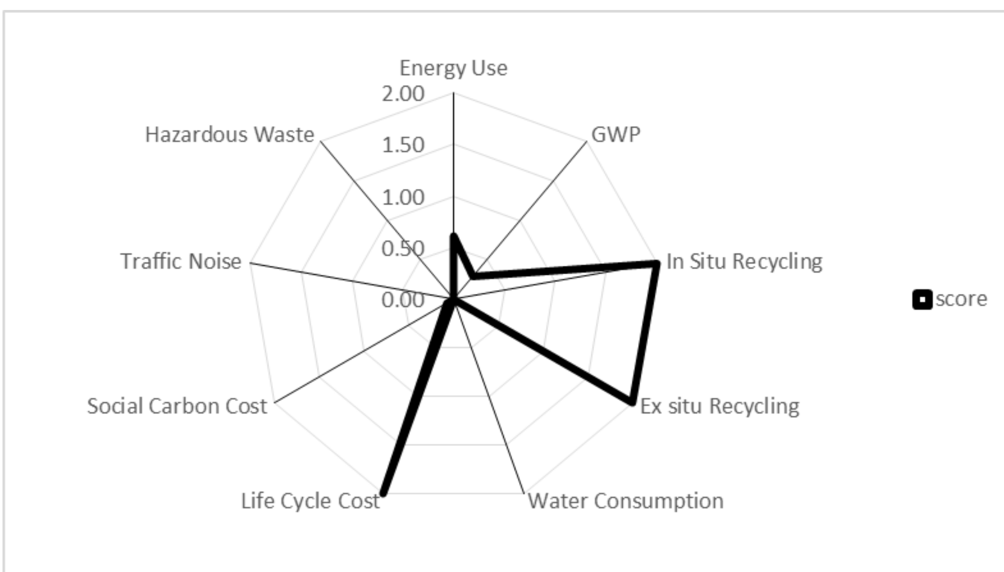


Figure 5.42. AMOEBA graph for recycled concrete pavement of Strategy 2.

Table 5.46. Results of BE^{2}ST-in-Highway for Strategy 3 (concrete pavement).

Criteria	Unit	Target	Reference	Strategy 3	Performance	Score
Energy Use	**MJ**	**≥ 10% Reduction (1 pt)**	15,213,544	13,151,670	13.55%	1.36
		≥ 20% Reduction (2 pts)				
GWP	**Mg**	**≥ 10% Reduction (1 pt)**	1,066	924	13.32%	1.33
		≥ 20% Reduction (2 pts)				
In Situ Recycling	**CY**	**≥ 10% Recycling Rate (1 pt)**	0.00	0.5000	50.00%	2.00
		≥ 20% Recycling Rate (2 pts)				
Ex situ Recycling	**CY**	**≥ 10% Recycled Content (1 pt)**	0.00	0.2938	29.38%	2.00
		≥ 20% Recycled Content (2 pts)				
Water Consumption	**kg**	**≥ 5% Reduction (1 pt)**	5,381	5,093	5.35%	1.07
		≥ 10% Reduction (2 pts)				
Life Cycle Cost	**$**	**≥ 10% Reduction (1 pt)**	1,097,804	718,405	34.56%	2.00
		≥20% Reduction (2 pts)				
Social Carbon Cost	**$**	**≥ $19,750/mi Saving (1 pt)**	$69,290.00	$60,060.00	9,230	0.47
		≥ $39,500/mi Saving (2 pts)				
Traffic Noise	**no unit**	**HMA (1 pt)**	0	0	0	0.00
		SMA or OGFC (2 pts)				
Hazardous Waste	**kg**	**≥5% Reduction (1 pt)**	21,811	20,580	5.64%	1.13
		≥10% Reduction (2 pts)				

Note: The discount rate is 4%. Performance is the degree of achievement in reducing the consumption of resources, reducing the generation of gas and hazardous waste, cutting down the costs, and increasing recycling rate. Positive performance means more environmental benefits gained, negative performance means more environmental loads caused.

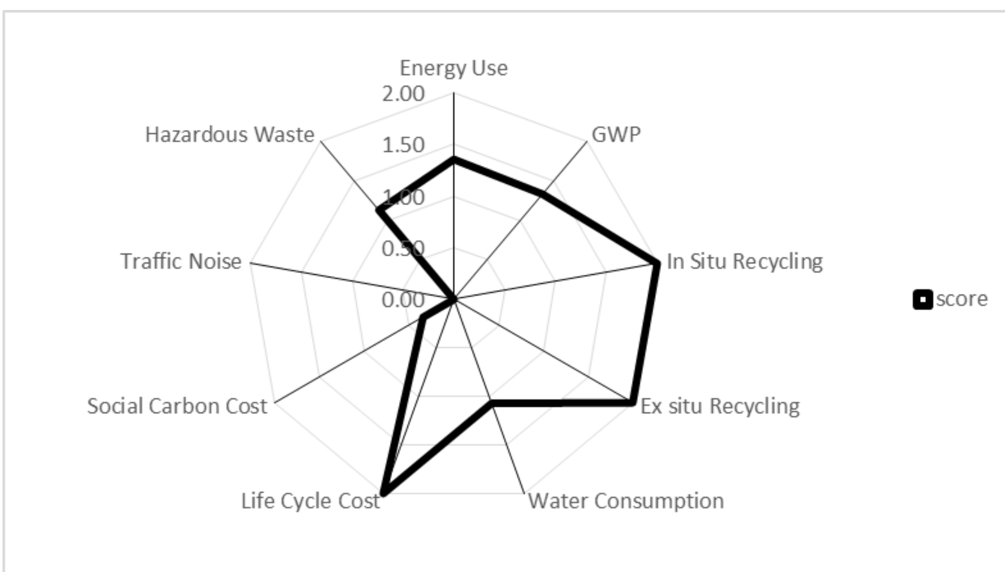


Figure 5.43. AMOEBA graph for recycled concrete pavement of Strategy 3.

Table 5.47. Results of BE^{2}ST-in-Highway for Strategy 4 (concrete pavement).

Criteria	Unit	Target	Reference	Strategy 4	Performance	Score
Energy Use	**MJ**	**≥ 10% Reduction (1 pt)** **≥ 20% Reduction (2 pts)**	15,213,544	10,463,405	31.22%	2.00
GWP	**Mg**	**≥ 10% Reduction (1 pt)** **≥ 20% Reduction (2 pts)**	1,066	739	30.68%	2.00
In Situ Recycling	**CY**	**≥ 10% Recycling Rate (1 pt)** **≥ 20% Recycling Rate (2 pts)**	0.00	0.5000	50.00%	2.00
Ex situ Recycling	**CY**	**≥ 10% Recycled Content (1 pt)** **≥ 20% Recycled Content (2 pts)**	0.00	0.4268	42.68%	2.00
Water Consumption	**kg**	**≥ 5% Reduction (1 pt)** **≥ 10% Reduction (2 pts)**	5,381	4,434	17.60%	2.00
Life Cycle Cost	**$**	**≥ 10% Reduction (1 pt)** **≥20% Reduction (2 pts)**	1,097,804	609,609	44.47%	2.00
Social Carbon Cost	**$**	**≥ $19,750/mi Saving (1 pt)** **≥ $39,500/mi Saving (2 pts)**	$69,290.00	$48,035.00	$21,255	1.08
Traffic Noise	**no unit**	**HMA (1 pt)** **SMA or OGFC (2 pts)**	0	0	0	0.00
Hazardous Waste	**kg**	**≥5% Reduction (1 pt)** **≥10% Reduction (2 pts)**	21,811	15,722	27.92%	2.00

Note: The discount rate is 4%. Performance is the degree of achievement in reducing the consumption of resources, reducing the generation of gas and hazardous waste, cutting down the costs, and increasing recycling rate. Positive performance means more environmental benefits gained, negative performance means more environmental loads caused.

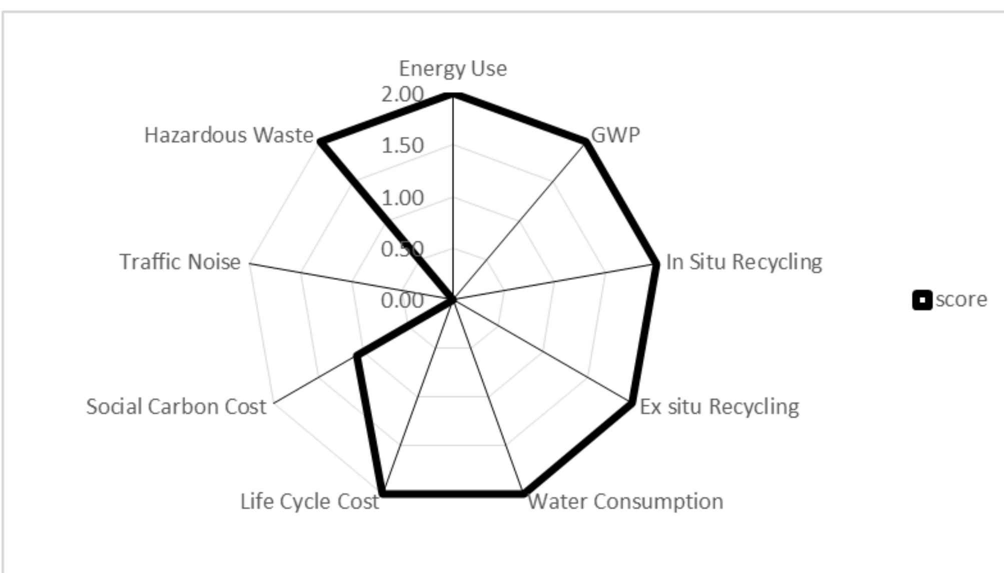


Figure 5.44. AMOEBA graph for recycled concrete pavement of Strategy 4.

Table 5.48. Results of BE²ST-in-Highway for Strategy 5 (concrete pavement).

Criteria	Unit	Target	Reference	Strategy 5	Performance	Score
Energy Use	**MJ**	**≥ 10% Reduction (1 pt)** **≥ 20% Reduction (2 pts)**	15,213,544	15,199,630	0.09%	0.01
GWP	**Mg**	**≥ 10% Reduction (1 pt)** **≥ 20% Reduction (2 pts)**	1,066	1,067	-0.09%	0.00
In Situ Recycling	**CY**	**≥ 10% Recycling Rate (1 pt)** **≥ 20% Recycling Rate (2 pts)**	0.00	0.5000	50.00%	2.00
Ex situ Recycling	**CY**	**≥ 10% Recycled Content (1 pt)** **≥ 20% Recycled Content (2 pts)**	0.00	0.2382	23.82%	2.00
Water Consumption	**kg**	**≥ 5% Reduction (1 pt)** **≥ 10% Reduction (2 pts)**	5,381	5,997	-11.45%	0.00
Life Cycle Cost	**$**	**≥ 10% Reduction (1 pt)** **≥20% Reduction (2 pts)**	1,097,804	830,304	24.37%	2.00
Social Carbon Cost	**$**	**≥ $19,750/mi Saving (1 pt)** **≥ $39,500/mi Saving (2 pts)**	$51,061.31	$51,109.21	$-65	0.00
Traffic Noise	**no unit**	**HMA (1 pt)** **SMA or OGFC (2 pts)**	0	0	0	0.00
Hazardous Waste	**kg**	**≥5% Reduction (1 pt)** **≥10% Reduction (2 pts)**	21,811	15,722	27.92%	2.00

Note: The discount rate is 4%. Performance is the degree of achievement in reducing the consumption of resources, reducing the generation of gas and hazardous waste, cutting down the costs, and increasing recycling rate. Positive performance means more environmental benefits gained, negative performance means more environmental loads caused.

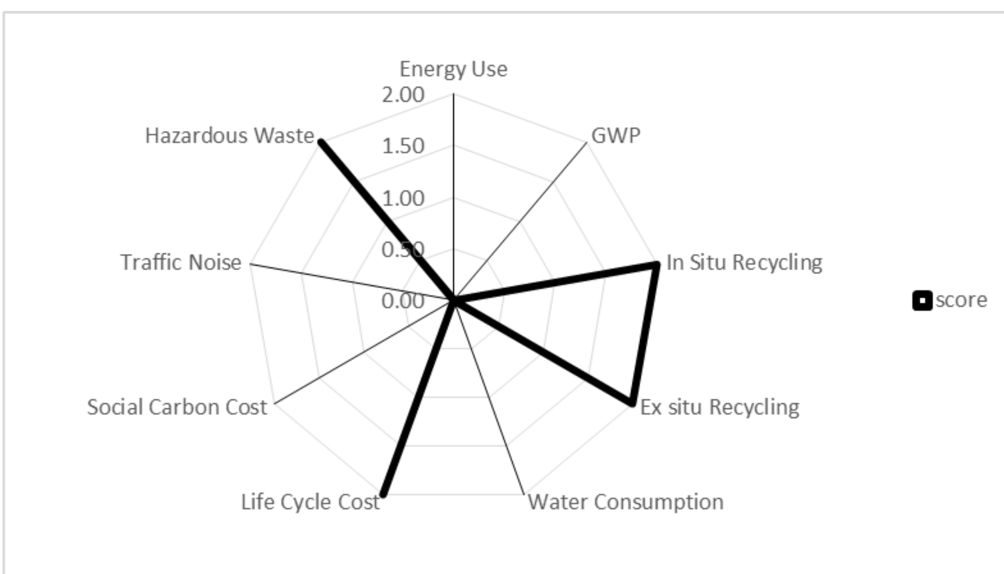


Figure 5.45. AMOEBA graph for recycled concrete pavement of Strategy 5.

Table 5.49. Rating of BE^{2}ST-in-Highway for rigid/concrete pavement.

Strategy #	Scenarios	Accomplished Score	Awarded Label
1	50% RCA in PCC, 100% RCA in GAB	70.07%	Bronze
2	100% RCA in PCC, 100% RCA in GAB	50.04%	Bronze
3	40% RAP in PCC, 100% RAP in GAB	73.46%	Bronze
4	100% RAP in PCC, 100% RAP in GAB	93.38%	Gold
5	20% FS in PCC, 100% RCA in GAB	58.05%	Bronze

5.2.4 Conclusions

BE^{2}ST-in-Highway™ provides a unique ranking system for recycled pavements' life cycle analysis. BE^{2}ST-in-Highway™ starts with the structural design of pavements to ensure the pavement has desirable bearing capacity and durability in service. The system takes advantage of PaLATE to conduct life cycle economic and environmental analysis, as well as other components to estimate service life, and assess traffic noise and storm water management. BE^{2}ST-in-Highway™ also quantifies the performance of pavements with a score and label, which helps decision makers identify the optimum strategies. From this study, the following conclusions can be drawn:

1. Replacing a portion of virgin materials with recycled materials in highway applications generally reduces life cycle cost and contributes to sustainable development of pavements, compared to using only virgin materials.
2. Recycled asphalt pavements generally meet the requirement of "green highway," while recycled concrete pavements may have difficulties in obtaining the "gold" label associated with "green highway." FDR used in recycled asphalt pavements is a main reason for the significant reduced in cost, consumption and emission.
3. Though some strategies for recycled concrete pavement (i.e., 100% RCA in PCC and 100% RCA in GAB) received low scores, these strategies can be advanced by using additives (i.e., fly ash) or using new technologies (i.e., CSOL).
4. GAB with 100% RAP and 100% RCA have nearly the same performance. Recycled GAB may be more "green" than FASB and cement-stabilized base, since cement, asphalt, or emulsified asphalt are not required in the production of GAB materials.
5. Since there is no negative point to reflect the worse-case performance of recycled pavements compared to conventional pavements, extra attention should be paid to avoid or modify the disadvantages of using recycled pavements.

Chapter 6: Summary & Conclusions

The recommendations for revising the current Maryland specifications were presented in Chapter 4. To develop such revised specifications, pilot studies are needed for developing the experimental data to assess impact on highway material properties, defining rational acceptance values and statistically based specification tolerances. The findings and conclusions of this synthesis study on the recycled materials and applications can be summarized as follows:

6.1 Recycled Concrete Aggregate (RCA)

Bulk specific gravity (S_G) of RCA ranges from 2.1 to 2.5, dependent on different sources and in general, is less than that of natural aggregates. CBR of RCA ranges from 90.0%- 148.0%, generally lower than that of natural aggregate, but may be higher than natural aggregates due to the present of residue cement in RCA aggregates. M_R of RCA is 2-2.6 times higher than natural aggregate; it increases with increasing bulk stress and decreases with enhancing capacity of water absorption. Water absorption capacity of RCA (3.7-8.7%) is greater than that of natural aggregate (0.8-3.7%). Sodium sulfate loss of RCA is greater compared to natural aggregates. Los Angeles abrasion loss of RCA (20%-45%) is higher than that of natural aggregates (15%-30%). Micro-deval degradation of RCA is lower than that of natural aggregates.

RCA in GAB

Raising dry density can elevate CBR of RCA-GAB mixtures. Fines (minus No. 200 sized materials) component reduces shear strength of RCA-GAB mixtures; degradation of RCA aggregates also weakens shear strength. M_R of RCA-GAB mixtures is higher for 100% RCA than different combinations of RCA and virgin GAB materials. RCA has good bearing strength and drainage properties, and meets all requirements for long-term performance of dense-graded aggregate base or subbase. Permanent deformation is less for 100% RCA, compared to natural aggregates. RCA addition increases permanent deformation of RCA-GAB mixtures; however, 100% GAB or 100% RCA has the least permanent deformation, compared to their mixtures.

Typically, effluent from drainage layers containing RCA are alkaline with a pH of 11-12. The pH value of effluent reaches a peak quickly and then decreases over time. Concentrations of Ca, Cr and Cu decrease over time, while concentrations of Fe increase at first and then decrease slightly. Typically, leached concentrations decrease with reduced fine aggregate content, and increasing liquid to solid ratio. In pH-dependent leaching tests, Ca shows increased concentrations with decreasing pH, while Cr, Cu, Fe, and Zn show minimum concentrations at neutral pH but increased concentrations at acidic or alkaline conditions.

For GAB material made with RCA, sufficient stability, shear strength, stiffness, permeability, and free drainage should be ensured in granular base, especially in flexible pavements. Large, angular, cubical and durable aggregates are preferred in producing GAB material. It is recommended that harmful impurities such as lead and asbestos be removed prior to reuse. Dust should be removed by washing RCA aggregates to prevent tufa formation. Prevent layer infiltrated by moisture is suggested to preclude mobilization and loss of stability.

RCA replacing part of natural aggregates in GAB has many advantages. For example, RCA can be easily and economically recycled by crushing concrete in place with a mobile plant, saving landfill space. RCA reduces water and energy consumption, as well as carbon dioxide emission during mining and transportation process.

RCA in PCC

Alkaline-silica reaction (ASR) is adverse to the durability of concrete, since ASR produces internal pressure and cracking in concrete. RCA experiencing ASR during primary service life has a high potential for expansion. Workability of fresh concrete reduces as RCA is used. Permeability of RCA PCC is about five times that of conventional PCC, which can be mitigated by reducing w/c from 0.05 to 0.1, or blending fly ash or slag cement into PCC mixtures. Concrete incorporating coarse RCA has the same or slightly lower compressive strength as conventional concrete. Coarse RCA reduces the modulus of rupture of PCC by up to 8%. Both coarse and fine RCA increase drying shrinkage of PCC. Using fine RCA increases shrinkage by 20%-50%; using coarse and fine RCA together increases shrinkage by 70%-100%. RCA generally reduces thermal expansion and contraction of concrete. Entrained air improves the resistance to degradation and cracking when concrete undergoes shrinkage and expansion. Medium-strength PCC contained RCA has slight permanent deformation, and high-strength PCC contained RCA has higher deformation, while low-strength PCC contained RCA shows the most deformation and the earliest failure.

The pH of RCA leachate ranges from 11.3 to 12.1. However, increased alkalinity in water passing RCA can be ignored, since PCC layer has low permeability even incorporating RCA. Stockpiling RCA contributes to lower leachate pH. Concentrations of Cu and Zn are found to be independent of the content of RCA; they exhibit peak concentrations at a pH of 2.0 and minimum concentrations at a pH of 7.5-13.0. As, Cr, Pb, and Se may exceed USEPA MCL (maximum contaminant level) in some States.

When PCC incorporates RCA as aggregates, RCA should be sieved and washed to remove fine particles (< No. 4) before use. Stockpiles of RCA should be maintained at saturated surface-dry condition. To prevent the occurrence of ASR in PCC contained RCA, fly ash, ground granulated blast-furnace slag, or silica fume can be used to mitigate ASR. Using blended cement or low-alkali Portland cement can be used as well. To minimize negative effects of RCA on fresh concrete workability, water-reducing additives and fly ash can be added. Blending RCA with conventional aggregates is also effective. Enough water should be ensured to meet the requirement of workability. European studies encourage recycling old concrete pavement with good strength, durability and condition, instead of existing pavements distressed for D-cracking or ASR.

RCA replacing natural aggregates can save about $4/ton for PCC paving, and up to $5 million on a single project. Using 30% RCA replacement in PCC can reduce environmental impact by 6.5%; using 50% RCA replacement can reduce environmental impact by 20%.

RCA in HMA

Optimum asphalt content (OAC) of HMA made with RCA is higher than that of conventional HMA. OAC increases linearly with increasing RCA content, especially fine RCA contents. HMA made with RCA has 3%-5% higher air voids, compared to conventional HMA. Air voids increase with increasing RCA content, especially with fine RCA contents. Some studies indicated that RCA reduces voids in mineral aggregate (VMA) of HMA, while other studies indicated that VMA increases with increasing RCA replacement, especially fine RCA replacement. RCA reduces voids filled with binder (VFB) of HMA. Marshall S/F (stability/flow) ratio typically decreases as RCA replacement increases. Slag-cement paste coat or heat treat on RCA also reduces Marshall stability. There is an argument whether RCA affects M_R of HMA. Some studies indicated that RCA weakens M_R of HMA, and M_R decreases with increasing RCA content and/or increasing binder content; other studies indicated that RCA improves M_R of HMA; yet another study indicated that RCA acting as filler has no effect on M_R. M_R of HMA with RCA is more temperature-dependent than conventional HMA, and M_R rises with dropped temperature. Higher compaction level improves M_R and load spreading capacity.

RCA replacement (100%) can improve fatigue life of HMA. Increment of fatigue life is greater with the addition of fine RCA than the same content of limestone powder. Though moisture resistance degrades with rising content of RCA, HMA made with fine RCA still has better moisture resistance compared to HMA made with limestone powder. Anti-stripping agents can improve moisture resistance. RCA coated with 5% bitumen emulsion has higher moisture resistance and fatigue resistance. RCA coated with liquid silicone resin has higher water absorption and fracture resistance, though coating RCA is difficult in mixing process. Some studies indicated that RCA exacerbates permanent deformation of HMA and the deformation increases as RCA content increases; other studies indicated that RCA improves deformation resistance; yet another study indicated that permanent deformation is independent of RCA content.

When RCA is used in HMA, air voids (by compaction) should be reduced to mitigate OAC and improve durability. Moisture resistance of HMA should be improved by penetrating RCA with different sealants (i.e., bitumen emulsion, slag cement paste, liquid silicone resin), heating RCA in the oven prior to compaction, or adding anti-stripping additive.

HMA with RCA has lower density; therefore, a lower mass of mixture is required. Use of RCA in HMA also reduces the need for quarrying and saves landfill sites.

RCA in Drainage/Fill

LA abrasion is 43.7% for RCA of No.4 gradation, but varies between 32% and 38% when particles smaller than 4 mm are removed by wet sieving. Mass loss of RCA exists in both acidic and alkali environments. An acidic environment degrades more RCA particles than an alkaline environment. Water flow has little effect on density of RCA. Water absorption remains constant in an alkaline environment, but drops greatly in acidic environment. Even though increasing fine RCA content degrades water flow, RCA of No. 4 gradation does not block water flow. Reducing fine particles can improve permeability, but reduce stability of the drainage layer.

Penetration resistance, compressive strength, and splitting tensile strength rises as cement content increases, but ductility reduces at the same time. High cement content of CCA mixtures results in a higher compressive strength and splitting tensile strength than concrete sand mixtures. CCA does not affect air-entrained flowable fill mixtures to develop enough penetration resistance. Splitting tensile strengths and compressive strengths of air-entrained flowable fill mixtures are consistently low over time and unaffected by the addition of CCA. Fly ash-flowable fill mixtures containing RCA take longer time to develop penetration resistance than mixtures containing concrete sand. Compressive strength and splitting tensile strength of the mixtures containing RCA are lower than that of mixtures containing concrete sand. RCA requires more water to meet given flow value than concrete sand.

RCA leachate has an initial pH of 12.5, slightly decreases to 12.1-12.3, but remains constant afterwards. Concentration of silicon and calcium in drainage water is relatively constant over time at both acidic and alkali levels. RCA precipitates more calcite than limestone, especially at a higher percentage of fine RCA particles, which can be reduced by washing RCA several times or reducing the usage of hydrated cement.

For drainage materials or flowable fill materials containing RCA, impurities included in RCA should be limited to gain high quality and consistence. Un-hydrated cement in RCA may alter its properties and complicate stockpiling; therefore, un-hydrated cement should be removed as much as possible. Stockpiles should be separated from water courses to avoid alkaline leachate. Material transporting, handling and storage, need additional care to avoid segregation of coarse and fine aggregates.

Recycling RCA only involves demolishing and removing old concrete, and crushing and processing demolition, saving cost and energy. Fuel consumption and transportation costs can be reduced if RCA is recycled on site. In addition, RCA usage reduces the consumption of natural aggregate and landfill places.

6.2 Reclaimed Asphalt Pavement Aggregate (RAP)

The S_G of RAP varies between 2.27 and 2.45, lower than natural aggregates. Unit weight of RAP is 120-140 pcf, slightly lower than virgin aggregate. Maximum dry density of compacted RAP varies between 115 pcf and 130 pcf, comparable to that of compacted sand. Water absorption of RAP is slightly lower than that of natural aggregate. Moisture content of RAP is 5%-8%, depending on the stockpiled conditions. CBR of RAP is lower than natural aggregates.

RAP in GAB

Optimum moisture content (OMC) varies between 5.3% and 7.1% for RAP-base blends, comparable to conventional GAB material. Increasing RAP content reduces OMC of RAP-base blends. Some studies indicated that permeability of RAP-base blends is higher than that of conventional GAB, and the permeability rises with rising content of RAP; yet other studies indicated that permeability of GAB made with 100% RAP is lower than that of conventional GAB. Permeability decreases as RAP content increases. Permeability is directly related to fines (particles passing the #200 sieve) content, and permeability decreases as fines content increases. Permeability also increases with freezing-thawing cycles due to disintegration of particles.

The M_R of RAP is higher than virgin aggregate base materials. M_R increases linearly with increasing bulk stress and RAP content. One hundred percent RAP achieves the largest M_R. M_R decreases as gradation becomes finer, which is also determined by coarse particle content, density and angularity. Higher compactive effort improves M_R by increasing the density of mixtures. M_R decreases with increasing moisture content, temperature and confining pressure. CBR of GAB also decreases with increasing RAP content, as well as finer gradation; however, another study indicated that CBR increases with increasing RAP content to a certain level and then decreases. Some studies showed that UCS decreases with increasing RAP content, yet other studies showed that UCS increases with increasing RAP content and that coarse RAP improves UCS more than fine RAP. RAP from pavements that have exhibited stripping has low strength. Coarse aggregates provide shear strength. One hundred percent RAP has the highest friction angle of 44° - 45°. For RAP-soil base materials, friction angle decreases with increasing content of fine sand. Cohesion of 100% RAP is 17-131 kPa. There are no durability concerns regarding the use of RAP in granular base, though permanent deformation of GAB increases with increasing RAP contents.

GAB with 100% RAP has the highest deformation and creep. Elevating moisture content leads to more permanent deformation. Rejuvenators help prevent premature fatigue and low temperature cracking failures.

Most leaching concentrations of RAP-soil base materials are below detection limit. RAP has higher leachate of hydrocarbons and some PAHs compared to natural aggregates, but these concentrations decrease rapidly and eventually are less than detection limits. Chemical oxygen demand (COD) concentrations are lower than USEPA limit of 120 mg/L.

When RAP is used in GAB material, the content of RAP should not exceed 50% by weight. RAP can be blended with virgin aggregate to improve its strength and to reduce its creep and permanent deformations. Un-stabilized RAP should include at least 75% GAB material and meet Limerock Bearing Ratio requirement. Asphalt binder content should not exceed 1.5% by weight. Using 20%-50% RAP can result in a cost savings of 14%-34% per ton. Natural resources and landfill places can be saved when RAP is used in GAB materials.

RAP in FASB

Maximum dry density of FASB material decreases with increasing RAP content. Optimum moisture content (OMC) of FASB material varies between 5.3% and 7.1%, decreasing with increasing RAP content. M_R ranges between 100 ksi and 800 ksi, dependent on type of aggregates and binders, mixing and curing conditions, and compaction methods. M_R increases with increasing percentage of cement or fly ash, and a longer curing period. As temperature is elevated from 50°F to 104°F, M_R reduces by 30%-44%. Loading rate, confining pressure and temperature affect M_R more than deviatoric stress. CBR increases linearly with increasing fly ash content. UCS increases with increasing stabilizing agent (i.e., cement, fly ash) content and curing period, but decreases with increasing RAP content.

Dry and soaked indirect tensile strength (ITS) reduces as RAP percentage rises. Increasing RAP percentage improves soaked ITS for mixtures containing GAB material, but degrades soaked ITS for mixtures containing RCA. Stockpiling reduces soaked and dry ITS by 27% and 16% on average. Cement significantly improves ITS and 1% cement improves dry and soaked ITS by 40% and 300%, respectively. Raising foamed asphalt content exacerbates permanent deformation. Higher aging RAP material facilitates permanent deformation in moist conditions, though improves resistance to permanent deformation under dry conditions. Adding cement or fly ash can largely reduce permanent deformation in dry and moist conditions.

The pH of groundwater leaching is 6.5-8.5 for RAP used as base material, within EPA limits. Adding cement raises the pH value. Elongating curing periods reduces the pH value. Concentration of As, Se and Sb may exceed USEPA groundwater maximum contaminant level (MCL) slightly, which are typically associated with the asphalt binder.

It is recommended that RAP should be blended with a minimum of 50% approved base course aggregate when RAP is used in FASB. Asphalt emulsion shall meet Limerock Bearing Ratio strength requirement and not exceed 3% by weight, in case of shear failure. Cement-stabilized RAP should include at least 50% approved base course material. Cement shall meet Limerock Bearing Ratio requirement and not exceed 2% by weight. Excessive fines (i.e., more than 12% passing No.200 sieve) should be prohibited in FASB.

FASB has the advantages to reduce the required thickness of pavement and hence saves cost. FASB also exhibits significantly better performance than bitumen asphalt in handling early traffic and resisting rain before placement of wearing course. Foamed asphalt mixes help to improve flexibility and reduce brittleness of pavement. When FASB incorporates RAP into paving projects, energy-saving can be up to 3% in MJ/tonne compared to FASB that incorporates fresh asphalt binder.

RAP in Drainage/Fill

The S_G of RAP is lower than that of conventional fill material. RAP has good drainage characteristics, and is regarded as a freely drainable material. RAP-soil mixture is a poorly drained material and hydraulic conductivity linearly decreases with increasing soil content. RAP has an effective friction angle of 37°, and effective cohesion of 8 psi. Creep rupture occurs in RAP fill materials before shear failure. Strength and stiffness of RAP are less susceptible to moisture, compared to limerock. One hundred percent RAP yields the highest M_R than other combinations of RAP-soil mixtures. Dry unit weight of RAP is not sensitive to moisture. The addition of fine aggregates (i.e. passing the #40 sieve size), rather than double compaction effort, contributes more to a high limerock bearing ratio. However, excessive fines can result in long-term total and differential settlement, leading to collapse.

Static compaction rather than dynamic compaction (vibratory or Proctor compaction) is more favorable to gain higher limerock bearing ratio. Compressibility of compacted RAP is greatly dependent on stress level and is highly sensitive to temperature. RAP compacted at high temperatures tends to gain higher stiffness and lower compressibility. RAP has higher potential of collapse than conventional fill material and RCA, and is comparable to the collapse potential of clay. At small confining pressure (i.e., 5 psi and 10 psi), significant and rapid creep deformations may occur. High asphalt content or high shear stress facilitates and accelerates creep. RAP generally ruptures more quickly than clay.

Field samples collected from surface waters and groundwater as well leachates collected from laboratory column leaching tests at different pHs all yield concentrations far below EPA limits for drinking water. Al, Cd, Cu, and Pb concentrations are generally within the chronic EPA water quality limit and chronic MD ALT (Maryland aquatic toxicity limits) for fresh water.

RAP used in drainage/ fill materials can reduce energy and natural resource consumption; reduce greenhouse gas emissions associated with mining and production of natural aggregates; and solve the problem of overproduced RAP that cannot be completely consumed by HMA.

RAP in HMA

RAP replacing 50% or more virgin aggregates has higher ITS, compared to conventional HMA mixtures. Rejuvenator additives degrade ITS, but improve fracture resistance. HMA mixtures with 100% RAP replacement provide the highest stiffness values compared to other replacement ratio, regardless of testing frequency, moisture condition and asphalt type. Moisture addition and elevating the temperature reduces mixture stiffness. Increasing RAP content improves stiffness (M_R and dynamic modulus), but variance of stiffness (for different RAP samples) also increases. Use of rejuvenators degrades M_R, while use of crumb rubber improves M_R. Rutting resistance rises as RAP content rises up to 50%. HMA with 100% RAP has a higher fatigue resistance compared to conventional HMA. Aged asphalt binder provides high resistance to low temperature cracking and fatigue cracking. However, at low temperature, increasing asphalt binder content results in lower ductility and lower fatigue resistance. Rejuvenators and crumb rubber additives help to improve fatigue resistance.

Leaching tests of HMA containing RAP show that concentrations of all heavy metals are below detection limits, except chromium. Still, Cd concentration is 50 times below the level considered hazardous per EPA Resource Conservation Recovery Act. Cr and Pb are below the maximum concentration of contamination for TCLP, but may exceed the limit of drinking water standards. Volatile organic compounds and semi-volatile organic compounds are below detection limits. Naphthalene is detected at 0.25 mg/L, but is still well below the regulatory guideline of 7.5 mg/L.

Since variability of mix properties increases with higher RAP content, it is recommended that a large number of samples be taken for quality control and quality assurance. Crushing and screening RAP help to gain consistent properties and meet the gradation and volumetric requirements. Attention should be paid to central plants recycling high RAP content and/or using improper virgin binder grade, which easily leads to accelerated fatigue and thermal cracking. Large and conical RAP stockpiles are preferred. A minimum stockpile frequency of testing is recommended, based either on the amount of RAP used or days of production. Additional tests are needed if mixture properties change during stockpiling.

According to statistics, using 10% RAP can save up to 6% fuel cost. Using 50% RAP in HMA applications reduces energy consumption to about the level to produce cold mix asphalt. Use of RAP can also eliminate disposal problems, save natural materials and good-quality aggregates.

RAP in PCC

Unit weight of PCC decreases with increasing RAP content. At the same w/c ratio, RAP concrete is less workable than conventional concrete. However, RAP concrete still has satisfied workability and can easily be mixed and consolidated. RAP reduces the compressive strength, tensile strength and flexural strength of concrete, and strengths decrease as RAP content increases. Strengths reduce more for RAP substituting both coarse and fine aggregate than for RAP substituting *only* coarse or fine aggregate. RAP substituting only fine aggregate is in between. Compressive strength increases over time during curing period. High w/c ratios reduce compressive strength, and the highest compressive strength is found at a w/c ratio of 0.50. A w/c ratio varying from 0.5 to 0.7 has little effect on flexural strength.

Elastic Modulus increase with curing time and decrease with increasing RAP content. Studies indicated that ACI method may underestimate elastic modulus for concrete without RAP and overestimate elastic modulus when RAP content is high. Concrete with higher RAP content generally experiences more creep and shrinkage over time, though one study indicates that shrinkage is independent of RAP content. High content of cement paste exacerbates creep. AASHTO method may underestimate creep of concrete containing RAP. Fly ash additive delays curing, and causes the prediction of the AASHTO method to be inaccurate. Addition of RAP enhances the toughness of concrete, especially coarse RAP. The toughness of concrete with fine RAP is comparable to conventional concrete. Air void content is generally independent of RAP content. Concrete with RAP has low chloride permeability, even though increasing RAP content slightly raises chloride ion penetrability. Increasing RAP content slightly degrades freeze-thaw resistance of concrete; however, concrete with 50% coarse RAP replacement maintains adequate durability.

Concrete made with RAP has similar leaching performance to concrete made with virgin materials. Concentrations of chloride and nitrate leached from concrete with RAP may be a little higher than that of conventional concrete.

It is recommended to use less than 35% coarse RAP replacement in concrete, in order to meet required fresh concrete properties, strength and durability. It is unnecessary to wash RAP to achieve required workability and strength. Strength loss due to incorporation of RAP can be mitigated by aging asphalt, which improves strength and modulus, reinforcing the bonding between asphalt and aggregates. Use of RAP in PCC addresses the problem of overproduced RAP. Virgin aggregate partly replaced by RAP is cost-saving and environmentally friendly.

6.3 Foundry Sand (FS)

FS is classified as a lightweight material. The specific gravity of FS ranges between 2.38 and 2.72. Variance is caused by different fines and additive contents. On average, the maximum dry unit weight of FS is 11 kN/m^3 and is not sensitive to variations in moisture content. FS has lower fineness modulus and bulk density than natural sand. The variation is caused by sand mineralogy, particle gradation, particle shape and fine content. Water absorption of FS is about 0.38%-4.15%, higher than that of natural sand.

FS in Crack Sealant & HMA

Density of HMA decreases with increasing FS content. As FS content increases from zero to 20%, density of HMA decreases from 2.4 g/cm^3 to 2.28 g/cm^3. ITS of HMA mixtures decrease with increasing FS content either in wet or dry condition, due to clay content in FS. In moist conditions, adding anti-stripping agent can improve ITS. ITS is hardly affected by absorption, angularity and fines content in FS. One study indicates that Marshall stability of HMA decreases (i.e., from 12.1 kN to 9.7 kN) as FS content increases (i.e., from zero to 20%), while another study indicates that FS improves stability of HMA mixtures. Overall, FS replacement less than 10% yields desirable stability. Flow value decreases (i.e., from 3.48 mm to 2.4 mm) as FS content increases (i.e., from zero to 20%), due to increased fine content. Sensitivity to moisture damage (i.e., stripping) increases with increasing FS replacement due to silica in FS; therefore, FS replacement should be less than 15%.

HMA containing FS does not release hazardous substances into the environment. Ferrous and aluminum FS are safe substitutes for virgin sands in construction applications. The addition of ferrous or aluminum FS to HMA has not shown any harm to the environment.

Studies have suggested that AASHTO pavement design method can be used to design asphalt pavements incorporating FS as fine aggregate. The same field-testing procedures, methods and equipment used for conventional HMA mixes are suitable to pavements containing FS. Since properties of recycled FS are largely determined by the type of original FS (green or resin), identifying the type of FS and how the sand streams separate and comingle helps to predict the properties of FS well. FS containing excessive fines should be screened prior to blending or limiting FS usage. Bentonite should be processed to reduce fines contents. Clay content and organic-based additive should be quantified and limited in producing HMA. For most FS, the sand equivalent test is not applicable, but methylene blue test is encouraged for measuring clay content. Coal and organic binders should be combusted. FS should be free of thick coatings of burnt carbon, binders and mold additives.

The case of gray iron FS used in HMA shows that 10% FS replacement saves 75% in costs. Energy spent on handling and recycling foundry byproducts saves up to 50 million mBtu in the exploration of virgin materials, disposal of foundry products and construction of landfills. Reuse of FS is an effective way to reduce emissions (i.e., greenhouse gas) in the environment, conserve landfill capacity and save virgin sands.

FS in Drainage/Embankment & Base

FS is generally non-plastic or low-plastic sand. Plastic behavior of FS is associated with clay content. With 6%-10% clay, liquid limit is more than 20, and plastic index is more than 2. FS has low water absorption, varying with different binders and additive types. Hydraulic conductivity of FS is about $2.7x10^{-3}$ cm/s at a hydraulic gradient of 0.5, high enough to provide good drainage capacity for structural fill applications. Permeability value of FS is $6x10^{-4}$- $5x10^{-3}$ cm/sec. When FS contains bentonite clay more than 6% by weight, permeability value decreases significantly to $1x10^{-7}$- $3x10^{-6}$ cm/sec. Lime addition improves hydraulic conductivity more than three orders of magnitude.

FS has sufficient shear strength and compressibility to be an embankment material. CBR of FS is 11%-30%, higher than that of granular sands. CBR increases as water content increases up to optimum water content, and then drops further with additional water. Compacted FS has sufficient shear strength for embankment fills. The friction angle of FS is 30°-36°, comparable to that of natural sands. Typically, cohesion of FS is 3700 psf. UCS is susceptible to water content; therefore, intrusion of excess water should be prevented in the field and rain should be monitored at the time of compaction. Prolonging curing time helps to improve strengths of cement-amended or lime-amended FS-crushed rock mixtures. The effect of freeze-thaw on FS mixtures depends on cementitious reactions. Strength reduces/increases as freezing action retards/accelerates the cementitious reactions. FS is more compressible than natural sand and has sufficient strength to resist breakdown under compaction. Owing to the weaker binder compared to sand grains, stress concentrates at particle contacts tend to cause crush of binder. Swell is negligible in FS, even for those with a high bentonite content (4.7-10.5%). High cement ratio may cause fragile of cement stabilized FS, leading to premature cracks.

FS does not cause groundwater or surface water contamination. Concentrations of Zn, Pb, Cr, and Fe may exceed the EPA limits; however, the difference is only 10%, which may be considered acceptable. Metal concentration drops gradually over time (i.e., 48 hr. or 72 hr.). The PAHs in green sands are much higher than those in chemical binder FS. Phenolic/ester sands have higher PAHs than furan/acid and silicate sands.

FS containing clays should be compacted to optimum water content in structural fill, and consistent moisture content should be maintained in compaction. Green sands require moisture during transportation and placement in case of dusting. FS can be transported, placed and compacted with conventional construction equipment.

Recycling FS can reduce costs of HMA pavement for both producers and end users. Use of FS as a fine aggregate reduces carbon footprint. FS typically has more consistent composition and higher quality compared to natural sands used in construction.

FS in Flowable Fill/Self-Compacted Concrete

FS degrades workability of SCC. The higher the FS content, the lower the workability, and the amount of superplasticizer required to modify workability increases. FS is less likely to segregate and provides a favorable flow; FS substitution of sand enhances viscosity. Water helps to improve flowability, however, excessive water leads to bleedings and volume instability, prolongs setting time and lowers quality. Concrete mixtures with 30% FS replacement have comparable compressive strength to conventional concrete, though compressive strength decreases with increasing FS content. Temperature has little effect on compressive strength, but slightly weakens splitting tensile strength. Some studies indicated that concrete with 10%-15% FS replacement has the highest strength. Drying shrinkage of SCC mixtures increases as FS replaces sand and decreases significantly as fly ash replaces Portland cement. FS enhances the resistance to chloride penetration. Coulomb value decreases as FS content increases up to 15%. FS facilitates carbonation in concrete, and carbonation depth increases as FS replacement increases; therefore, the substitution rate of FS should be within 30% for structural concrete. FS weakens sulphate resistance of concrete and resistance decreases with increasing substitution rate of FS; therefore, 10% is the maximum substitution rate in resisting sulphate attack.

The pH increases as cement or lime is added into FS mixtures. Metal concentrations from flowable fill materials with FS are lower than EPA maximum limits. Leachate from FS used in producing iron, steel, and aluminum are below the regulatory limits for hazardous waste. Generally, organic remains contained in organic binders are already burned or shaken away in casting processes; because of this, organic matters will not cause environmental problems. According to studies, acetone and naphthalene are below USEPA TCLP toxicity criteria. The other organic compounds are not detectable, and are below USEPA TCLP toxicity criteria.

It is recommended that FS should be combined with natural sand (i.e., round sand) to achieve desirable performance. FS should be screened and crushed to obtain the desired gradation before usage. Properties of FS can affect the quality of concrete. Therefore, performance tests should be conducted on FS source prior to recycling. Cementitious materials can be a combination of Portland cement with fly ash, etc. Sodium silicate binder systems are not desirable in Portland cement.

FS can be obtained from foundries with lower material cost. Disposal cost is reduced through recycling FS. Other waste materials (i.e., fly ash) can also be used beneficially when FS is used in producing concrete.

FS in PCC

A study indicated that water absorption of concrete with 5% FS is higher than that of conventional concrete, and water absorption decreases when the substitution rate of FS exceeds 5%. Another study indicated that water absorption increases with increasing FS content in concrete. FS reduces workability of FS, and slump drops (i.e., from 200 mm to zero) as FS replacement increases (i.e., from zero to 80%-100%). Whether FS reduces or improves strengths of concrete is yet to be determined. There are studies indicated that when w/c is high enough, strengths of concrete made with FS can be higher than that of conventional concrete. For the maximum strengths and modulus of concrete containing FS, some studies stated 15% FS replacement provides the maximum values, while other studies agreed with a 10% FS replacement. Concrete with 10%-30% FS replacement shows higher compressive strength than the concrete (30 MPa) without FS, at all ages. Splitting tensile strength of concrete with 10% FS is slightly higher than that of conventional concrete, while 5% and 15% FS replacement degrade strength. Modulus of elasticity range from 5.2%- 12% depending on FS content and curing time.

Drying shrinkage increases as concrete incorporates FS. The increase or decrease of drying shrinkage is consistent with compressive strength and modulus of elasticity. Concrete incorporating FS exacerbates carbonation; the maximum carbonation depth may occur at 60% FS replacement. For every 10% increment of FS replacement ratio, an average increment of 0.17 mm and 0.33 mm in carbonation depth occurs at 90 days and 365 days, respectively. Concrete with 10% FS is less affected by freezing-thawing cycles compared to the other ratios of FS replacement.

Metal concentrations tested by TCLP are below the EPA limits for hazardous waste. Only As may exceed National Primary Drinking Water Standard tested by SPLP. Fungal-treated concrete with FS shows a significant reduction in metal concentration. Significant concentrations of organic compounds have not been found in FS.

Since using alkyd urethane binder elevates Co and Pb concentrations, foundries are encouraged to use alternative binder systems with lower metal concentrations. To avoid excessive waste residues, screening systems and magnetic separators are needed to segregate usable sand from other wastes and to separate particles of varying sizes prior to recycling. Clean sand replaced by FS can reduce cost by 25% or \$6.44/ton. Using FS can prevent over-exploitation of river sand and the introduction of salinity into rivers.

6.4 Dredged Material (DM)

DM in Fill

DM itself is not suitable for construction and needs to be amended with other materials (e.g., bottom ash, air foam, rubber, cement) for improved properties. Unit weight of fill materials containing DM is hardly affected by cement and water content, significantly reduced by the addition of air foam, and elevated by the addition of bottom ash. In comparison, rubber-stabilized DM mixture has the minimum unit weight. Flowability of fill materials increases slightly with increasing air foam content, increases dramatically with increasing water content, decreases slightly with increasing cement and/or bottom ash contents, and decreases with increasing rubber content. Hydraulic conductivity decreases as bentonite content (from DM) rises and/or pressure on DM mixtures rises, and increase as fly ash or steel slag fines is added.

The addition of cement improves strengths, modulus (elastic modulus) and ductility. A little cement is enough to solidify large amounts of soils, though a high dosage of fly ash is better for strength enhancement. The strength of air-foam stabilized DM increases with higher cement content and/or decreasing air foam content, but air foam improves stiffness of DM mixture. The addition of bottom ash improves strengths and stiffness. The addition of rubber degrades strengths and stiffness. Stiffness of rubber-added DM is less than that of bottom ash-added DM. The addition of steel slag fines and crushed glass improves strength, and steel slag fines is more effective than crushed glass in improving strength and CPT tip resistance. Steel slag is approximately twice as effective in solidifying DM, compared to cement-fly ash blend. However, increasing steel slag fines content reduces compressibility and requires greater consolidation to obtain enough compressibility.

Arsenic leached from aged DM-steel slag fines blends is less than the regulatory limits. Field arsenic concentration is less than the detection limit and TCLP limit. Less than 25% Cr is leached from 100% DM, meeting the Maryland State requirements.

When selecting additives for DM fill material, consider effectiveness in reduction of water content, regulatory requirements and restrictions, processing facility configuration, applicability to a wide range of sediments and chemical contaminants, availability, and cost. Contaminated dredged sediments can be treated with a combination of chemical additives and separation technologies.

Recycling DM can solve the problems of storage, space, management and disposal of DM. Virgin materials can be saved when DM is used as fill materials. Other waste materials (i.e., fly ash, cement dust, lime dust) can be beneficially used as additives or modifiers to DM.

DM in Lightweight Aggregate/Brick (LWA)

Specific gravity of LWA made of water treatment residue ranges from 1.12 to 1.78. Specific gravity and bulk density increases with increasing sintering temperature due to densification. Crushing strength of LWA made of reservoir sediment is higher than commercial LWA that serves as structural aggregates and the crushing strength increases with increasing density. LWA made of reservoir sediment reduces the density of concrete mixtures by about 29%-35%, and provides satisfactory workability. The 28-day compressive strength of concrete made with LWA ranges from 19.8 to 34.7 MPa, higher than ASTM C330 requirement of 17 MPa. The 28-day flexural strength of concrete ranges from 5.3 to 7.2 MPa, depending on aggregate density and w/c ratio. Bricks made of reservoir sediment yield a maximum density of 2.5 g/cm^3 at 1100^{0}C (without clay). At 1150^{0}C, density decreases significantly as clay content decreases.

Novosol® river sediment offer a patented process for sediment stabilization. The bricks exhibit lower water absorption than standard bricks. Water absorption coefficients of Novosol® river sediment bricks are all within regulatory limits and increase with increasing sediment addition. Water absorption of bricks decreases as sintering temperature rises. Clay addition helps to reduce water absorption. Novosol® river sediment bricks are less permeable than standard bricks, have low plasticity and poor bonding ability. The compressive strength of brick made with water treatment residual increases with the increasing sintering temperature until 1150^{0}C (clay$\leq$20%), with maximum compressive strength occurring at 1100^{0}C without clay. Compressive strength of Novosol® river sediment bricks is higher than standard bricks, even though compressive strength decreases with increasing sediment content. Shrinkage increases with increasing sintering temperature. Shrinkage of water treatment residue brick significantly increases at 950^{0}C, and volume is reduced by 45% at 1100^{0}C. Shrinkage of Novosol® river sediment bricks is higher than standard bricks. Novosol® river sediment bricks have qualified freeze-thaw resistance, and the percentage of weight loss under freeze-thaw cycles is independent of sediment content.

Leachability of heavy metals from sediment brick is generally higher, compared to commercial bricks. However, most sediment bricks still meet the requirement of non-hazardous material. Sediment bricks (i.e., harbor sediment bricks in Bremen, Germany) may exhibit high concentrations at acidic condition but low concentrations at neutral and alkaline condition. Quantities of metals leached out of bricks are less than their treated or untreated sediments, since thermal treatment (i.e., 1050^{0}C) can destroy organic contaminants in sediments and transform remaining heavy metals into new minerals, with the exception of Cr, V, As and Mo.

Incorporating DM into the production of LWA can be cost-saving, since the unit price of LWA (i.e., HarborRock® LWA) are generally less than the average price of commercial LWA. Producing bricks or LWA with DM can save sparse resources and landfill spaces used to dispose DM.

DM in PCC/Cement

When DM acts as fine aggregates, density of concrete decreases significantly with the increase of DM content. When DM acts as fillers (either treated or untreated), density of concrete increases slightly with the increase of DM content. DM acting as either fine aggregate or filler in PCC dramatically reduces workability. The addition of superplasticizers can improve workability and reduce w/c ratio, while maintaining acceptable flow for concrete with DM as filler. However, when DM acts as fine aggregate, adding superplasticizer cannot lower w/c ratio while achieving acceptable flow.

The addition of untreated DM slows setting and hydration of concrete. Even though superplasticizer can accelerate hardening of concrete at early age, long-term hardening is determined by releasing initially absorbed water, independent of superplasticizer. As the w/c ratio rises, compressive strength of concrete keeps almost constant when replacement ratio of DM is less than 15%, but strength increases considerably at 20% DM replacement.

The effect of DM on the compressive strength of concrete is still uncertain. Tensile strength of concrete increases with increasing DM content. DM improves toughness and reduces shrinkage of concrete. A small amount (0.5%-1.0%) of salt or chloride content in DM accelerates heat evolution and strength gain of concrete at early ages. Clay (from DM) may lead to swelling and poor durability of concrete due to high water absorption. DM replacing cement reduces flowability of pastes. Raising the w/c ratio or adding plasticizer additives can help pastes made with DM to achieve similar flowability of cement paste. DM replacing natural sand improves compressive strength of mortars. The 28-day flexural strength of mortars increases slightly with increasing DM replacement to 15%, and the maximum flexural strength (at 15% DM replacement) is 18% higher than that of normal mortars. Weight loss is greater for mortar immersed in HCL than in H_2SO_4 solution, and increases with increasing DM content. Chloride concentrations slightly decrease with increasing DM content, but are below the water soluble chloride limit for Portland cement used in concrete.

TCLP test for New York/New Jersey harbor DM revealed that metal concentrations from untreated sediments are below U.S. limits for classification as hazardous materials. Treatment such as phosphate addition and thermal processing can reduce leachate of metals up to 89%.

Studies suggest treating DM from different sources separately, since properties of DM vary greatly from place to place. Corrosion protection measures should be adopted where DM is added into cement or concrete. Recycling DM can save considerable space consumed by disposal and placement of DM. Environmental concerns such as loss of open water and excessive sedimentation can be mitigated by using DM to replace natural sands.

Appendix

Maryland State Highway Authority- Recycled Material Availability Synthesis Study

Survey on the State of Practice of Recycled Materials in Highway Applications

Currently the use of recycled materials in highway applications in the US is expanding. However, their use is often limited due to regulatory, environmental and technical restrictions. The Maryland State Highway Authority is currently sponsoring this research study to document the state-of-the-art practice of employing selected recycled materials, and develop the technical requirements for their safe use in alternative highway applications.

The following four recycled materials are the focus of this survey in order to document the state of practice by your agency and within your region:

- Recycled Concrete Aggregate (RCA);
- Reclaimed Asphalt Pavement (RAP);
- Dredged Materials (DM);
- Foundry Sand (FS).

As our thanks for your participation, Maryland State Highway Administration will make the summary results of the survey available to all participants.
Please e-mail your responses, and any follow-up questions and clarifications to:
DSajedi@sha.state.md.us

Dan Sajedi
Soils and Aggregate Technology Division Chief
Phone: 443-572-5162; 866-926-8501

Contact Information

Name & Position: ....
Agency: ....
Address: ....
....
Telephone: ....
Email: ....

Maryland State Highway Authority- Recycled Material Availability Synthesis Study

1. Recycled Materials used by your agency in highway construction (check all that apply)

□ RCA □ RAP □ FS □ DM.

2. What was the source?

□From Bridge/ Highway structures □Demolished buildings/other structures
□From plants within your state □From plants outside your state
□Other (please specify): ____

3. In which applications was the recycled material used? Please check all that apply.

□GAB (Granular aggregate base) □FASB (Foam asphalt stabilized base)
□Drainage/Fill materials □Select Borrow
□HMA (Hot mix asphalt) □PCC (Portland cement concrete)
□Other ________________________________

4. Please identify technical challenges you experienced with such materials.

____
____
____

5. What are the environmental concerns in regards to the use of recycled materials? Please check all that apply.

□Elevated concentrations of metal/organic contaminants
□High/low pH levels;
□Other____ _________________

We would appreciate it if you can provide additional information for any of these four recycled materials in your state and including:

i) Key references & studies

____
____
____
____

ii) Technical data & specifications.

____
____
____
____
____

References

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

Abdelrahman, M., Alam, T.B., and Zollars, J. (2010). Performance of High Recycled Asphalt Pavement (RAP) Content as Base Layer in Flexible Pavement. *Journal of Solid Waste Technology and Management*, *Vol. 36*, No.3, pp. 131-142.

ACPA. (1998). *Concrete Pavement Engineering and Research*. Technical Information, American Concrete Pavement Association. Online Available: http://overlays.acpa.org/Downloads/Rubblization.pdf

ACPA. (2008). *Recycled Concrete in Subbases: A Sustainable Choice.* Technical Series, American Concrete Pavement Association, Washington, D.C.

AFNOR. (1983). *Facing Clay Bricks.* Standards and Collections, NF P13-304, France.

AFS. (2010). *Structural Fill & Embankment.* American Foundry Society. Available: http://www.afsinc.org/content.cfm?ItemNumber =7114

AFS. *Asphalt.* American Foundry Society. Available: http://www.afsinc.org/content.cfm?ItemNumber =7091

Aggarwal, Y., and Siddique, R. (2014). Microstructure and Properties of Concrete Using Bottom Ash and Waste Foundry Sand as Partial Replacement of Fine Aggregates. *Construction and Building Materials*, *Vol. 54*, pp. 210-223.

Agostini, F., Skoczylas, F., and Lafhaj, Z. (2007). About A Possible Valorisation in Cementicious Materials of Polluted Sediments after Treatment. *Cement and Concrete Composites*, *Vol. 29*, pp. 270-278.

Amorim, P., de Brito, J., and Evangelista, L. (2012). Concrete Made with Coarse Concrete Aggregate: Influence of Curing on Durability. *Materials Journal*, *Vol. 109* (2), pp. 195-204.

Aoual-Benslafa, F.K., Kerdal, D., Ameur, M., Mekerta, B., and Semcha, A. (2015). Durability of Mortars Made with Dredged Sediments. *Procedia Engineering*, *Vol. 118*, pp. 240-250.

APA. (2010). *Pavement Type Selection. IM-45*. Asphalt Pavement Alliance. Lanham, MD. Online Available: http://www.asphaltroads.org/assets/_control/content/files/pavement_type_selection_july_2010.pdf

APA. (2011). *Life-Cycle Cost Analysis: A Position Paper.* Asphalt Pavement Alliance, Lanham, MD. Online Available: http://www.asphaltroads.org/assets/_control/content/files/life_cycle_costs.pdf

Apyal, N. (2008). *Environmental, Performance and Life Cycle Cost Analysis of Three Maintenance Techniques of Asphalt Pavements*. Thesis for Master Degree, The Oklahoma State University.

Arabani, M., and Azarhoosh, A.R. (2012). The Effect of Recycled Concrete Aggregate and Steel Slag on the Dynamic Properties of Asphalt Mixtures. *Construction and Building Materials*, *Vol. 35*, pp. 1-7.

Arabani, M., Moghadas, N.F, and Azarhoosh, A.R. (2012). Laboratory Evaluation of Recycled Waste Concrete into Asphalt Mixtures. *International Journal of Pavement Engineering*, *Vol. 14* (6), pp. 531-539.

ARRA. (2001). *Basic Asphalt Recycling Manual*. Asphalt Recycling and Reclaiming Association, Annapolis, MD.

Asphalt Academy. (2002). *The Design and Use of Foamed Bitumen Treated Materials*. Interim Technical Guideline, 2nd Edition. Pretoria, South Africa.

ASTM C326 (2009). *Standard Test Method for Drying and Firing Shrinkages of Ceramic Whiteware Clays*. American Society for Testing and Materials.

Aydilek, A. and Mijic, Z. (2015). *Hydraulic and Environmental Behavior of Roadway Millings in Highway Shoulder Applications*. Quarterly report, Maryland State Highway Administration. MD.

Aydilek, A.H. (2015). *Environmental Suitability of Recycled Concrete Aggregates in Highways*. Research report, Maryland State Highway Administration, MD.

Aydilek, A.H., Haider, I., Cetin, A., Kaya, Z., and Hatipoglu, M. (2015). *Development of Design Guidelines for Proper Selection of Graded Aggregate Base in Maryland State Highways*. Report, Maryland State Highway Administration, MD.

Bakis, R., Koyuncu, H., and Demirbas, A. (2006). An Investigation of Waste Foundry Sand in Asphalt Concrete Mixtures. *Waste Management and Research*, *Vol. 24*, pp. 269-274.

Bakis, R., Koyuncu, H., and Demirbas, A.(2006). An Investigation of Waste Foundry Sand in Asphalt Concrete Mixtures. *Waste Management & Research, vol. 24*, pp. 269-274.
Basta, N.T., Ryan, J.A., and Chaney, R.L. (2005). Trace Element Chemistry in Residual-Treated Soil: Key Concepts and Metal Bioavailability. *Journal of Environmental Quality*, *Vol. 34*, pp. 49-63.
Bennert, T., Papp, W.J., Maher, A., and Gucunski, N. (2000). Utilization of Construction and Demolition Debris under Traffic-Type Loading in Base and Subbase Applications. *Transportation Research Record*, *Vol. 1714*, pp. 33-39.
Bennett, T. and Maher, A. (2005). *The Development of Performance Specification for Granular Base and Subbase Material.* Final Report, New Jersey Department of Transportation, Trenton, NJ.
Bennett, T. and Maher, A. (2005). *The Development of Performance Specification for Granular Base and Subbase Material*. Final Report, New Jersey Department of Transportation, Trenton, NJ.
Benson, C.H., and Bradshaw, S. (2011). *User Guideline for Foundry Sand in Green Infrastructure Construction*. Report, Recycled Materials Resource Center, University of Wisconsin-Madison, WI.
Berry, M., Stephens, J., Bermel, B., Hagel, A., and Schroeder, D. (2013). *Feasibility of Reclaimed Asphalt Pavement as Aggregate in Portland Cement Concrete*. Final Report, Montana Department of Transportation, Helena, Montana.
Bhat, S., and Lovell, C. (1997). *Use of Coal Combustion Residues and Waste Foundry Sands in Flowable Fill*. Final Report, Indiana Department of Transportation.
Braham, A. (2002). *The Use of Blended Recycled Foundry Sand in Hot Mix Asphalt*. Thesis for Master Degree, Civil and Environmental Engineering, University of Wisconsin-Madision.
Brand, A.S., Roesler, J.R., Al-Qadi, I.L., and Shangguan, P. (2012). *Fractionated Reclaimed Asphalt Pavement (FRAP) as a Coarse Aggregate Replacement in a Ternary Blended Concrete Pavement*. Research Report, Illinois Center for Transportation, Department of Civil and Environmental Engineering, University of Illinois at Urbana-Champaign.
Burgstaller, W., and Schinner, F. (1993). Leaching of Metals with Fungi. *Journal of Biotechnology*, *Vol. 27*, pp. 91-116.
Bushal, S., and Wen, H. (2013). Evaluate Recycled Concrete as Hot Mix Asphalt Aggregate. *Advances in Civil Engineering Materials*, *Vol. 2* (1), pp. 252-265.
Bushal, S., Li, X., and Wen, H. (2011). Evaluation of Effects of Recycled Concrete Aggregate on Volumetrics of Hot-Mix Asphalt. *Transportation Research Record*, *Vol. 2205* (3), pp. 36-39.
Carpenter, A.C., Gardner, K.H., Fopiano, J., Benson, C.H., and Edil, T.B. (2007). Life Cycle Based Risk Assessment of Recycled Materials in Roadway Construction. *Waste Management*, *Vol. 27* (10), pp. 1458-1464.
Castroa, P., Sanjuan, M. A., and Ganesca, J. (2000). Carbonation of Concrete in Mexico Gulf. *Building and Environment Vol. 35*, pp. 145-149.
CDRA. *Reclaimed Concrete Aggregate*. Material Fact Sheet, Construction & Demolition Recycling Association. Available: http://www.cdrecycling.org
CEDEX. (1994). *Recommendations for the Management of Dredged Material in Spanish Ports*. Centre for Studies and Experimentation of Public Works, Spain.
Celauro, C., Bernardo, C., and Gabriele, B. (2010). Production of Innovative, Recycled and High-Performance Asphalt Road Pavements. *Conservation and Recycling*. *Vol. 54* (6), pp. 337-347.
Chan, C.M., (2012). Variation of Shear Wave Arrival Time in Unconfined Soil Specimens Measured with Bender Elements. *Journal of Geotechnical and Geological Engineering*. *Vol. 30* (2), pp. 419-430.
Changling, H., Osbeck, B., and Makovicky, E. (1995). Pozzolanic Reaction of Six Principal Clay Minerals: Activation, Reactivity Assessment and Technology Effects. *Cement and Concrete Research*, *Vol. 25* (8), pp. 1691-1702.
Chen, M.J., and Wong Y.D. (2013). Porous Asphalt Mixture with 100% Recycled Concrete Aggregate. *Road Materials and Pavement Design*, *Vol. 14* (4), pp. 921-932.
Chesner, W.H., Collins, R.J., and MacKay, M.H. (1998). *User Guidelines for Waste and By-Product Materials in Pavement Construction*. Report, FHWA, McLean, VA.

Chiang, K.Y., Chien, K.L., and Hwang, S.J. (2008). Study on the Characteristics of Building Bricks Produced from Reservoir Sediment. *Journal of Hazardous Materials*, *Vol. 159*, pp. 499-504.

Chittoori, B., Puppala, A.J., and Raavi, A. (2014). Strength and Stiffness Characterization of Controlled Low-Strength Material Using Native High-Plasticity Clay. *Journal of Materials in Civil Engineering*, *Vol. 26* (6), pp. 1-8.

Cho, Y.H., Hun, T., Kim, I.T., and Choi, N.R. (2011). The Application of Recycled Concrete Aggregate (RCA) for Hot Mix Asphalt (HMA) Base Layer Aggregate. *Journal of Civil Engineering, Vol. 15* (3), pp. 473-478.

CIPEC. (2005). *Road Rehabilitation Energy Reduction Guide for Canadian Road Builders*. Canadian Industry Program for Energy Conservation. Available: http:// oee.nrcan.gc.ca/industrial/technical info/benchmarking/ roadrehab/Roadhab_eng_web.pdf.

Clary, J.A., DeGroot, D.J., and Highter, W.H. (1997). Structural Numbers for Reclaimed Asphalt Pavement Base and Subbase Course Mixes. *Transportation Research Broad*, *Vol. 1687*. Washington, D.C.

Colbert, B., and You, Z. P. (2012). The Determination of Mechanical Performance of Laboratory Produced Hot Mix Asphalt Mixtures Using Controlled RAP and Virgin Aggregate Size Fractions. *Construction and Building Materials*, *Vol. 26* (1), pp. 655-662.

Cooley, L.A, and Hornsby, H. (2012). *Evaluation of Crushed Concrete Base Strength*. Report, Mississippi Department of Transportation, Jackson, MS.

Cooley, L.A., Brumfield, J.J., Easterling, J. and Prithvi, S.K. (2007). *Evaluation of Recycled Portland Cement Concrete Pavements for Base Course and Gravel Cushion Material*. Final report, South Dakota Department of Transportation.

Corinaldesi, V., and Moriconi, G. (2009). Influence of Mineral Additions on the Performance of 100% Recycled Aggregate Concrete. *Construction and Building Materials*, *Vol. 23*, pp. 2869-2876.

Cosentino, P. J., Kalajian, E. H., Shieh, C. S., Mathurin, W. J., Gomez, F. A., Cleary, E. D., Treeratrakoon, A. (2003). *Developing Specifications for Using Recycled Asphalt Pavement as Base, Subbase or General Fill Materials, Phase II*. Final Report, Florida Department of Transportation.

Cosentino, P.J., Kalajian, E.H., Bleakley, A.M., Diouf, B.S., Misilo, T.J., Petersen, A.J., and Sajjadi, A.M. (2012). *Improving the Properties of Reclaimed Asphalt Pavement for Roadway Base Applications*. Final Report, Florida Department of Transportation.

Coz, A., Andres, A., Soriano, S., and Irabien, A. (2004). Environmental Behavior of Stabilized Foundry Sludge. *Journal of Hazardous Materials*, *Vol. 109*, pp. 95-104.

Crawford, A. and Aydilek, A. (2004). Beneficial Re-Use of Bottom Sea Dredged Sediments as Vertical Cutoff Wall Backfill Material. Report, Maryland Department of Transportation, MD.

Dafalla, M.A. (2013). Effects of Clay and Moisture Content on Direct Shear Tests for Clay-Sand Mixtures. *Advances in Materials Science and Engineering*, *Vol. 2013*, pp. 1-8.

Dalton, J.L., Gardner, K.H., Seager, T.P., Weimer, M.L., Spear, J.C.M., Magee, B.J. (2004). Properties of Portland Cement Made from Contaminated Sediments. *Resources, Conservation and Recycling*, *Vol. 41*, pp. 227-241.

Dam, T.V., Smith, K., Truschke, C., and Vitton, S. (2011). *Using Recycled Concrete in MDOT's Transportation Infrastructure–Manual of Practice*. Report, Michigan Department of Transportation, Lansing, MI.

de Juan, M.S., and Gutierrez. P.A. (2009) Study on the Influence of Attached Mortar Content on the Properties of Recycled Concrete Aggregate. *Construction and Building Material*, *Vol. 23*, pp. 872-877.

Delange, K., Braham, A., Bahia, H., Widjaja, M., Romero, P., and Harman, T. (2001). *Performance Testing of Hot Mix Asphalt Produced With Recycled Foundry Sand*. Annual Meeting of Transportation Research Board.

Delwar, M., Fahmy, M., and Taha, R. (1997). Use of Reclaimed Asphalt Pavement as an Aggregate in Portland Cement Concrete. *Materials Journal, Vol. 94* (3), pp. 251-256.

Deng, A. (2009). Contaminants in Waste Foundry Sand and Its Leachate. *International Journal of Environment and Pollution, Vol. 38* (4), pp. 425-443.

Deng, A., and Tikalsky, P.J. (2008). Geotechnical and Leaching Properties of Flowable Fill Incorporating Waste Foundry Sand. *Waste Management*, *Vol. 28* (2008), pp. 2161-2170.

Diefenderfer, B.K. and Apeagyei, A.K. (2010). *Cost Considerations of In-Place Recycling as a Pavement Rehabilitation Alternative*. Virginia Transportation Research Council. Online Available: http://www.cpe.vt.edu/

pavementevaluation/presentations/ Diefenderfer01.pdf
Doig, B. (2000). *Influence of Storage Time and Temperature on the Triaxial Characterization of RAP*. Thesis for Master Degree, Florida Institute of Technology, Melbourne, Florida.
Dolage, D.A.R., Dias, M.G.S., and Ariyawansa, C.T. (2013). Offshore Sand as a Fine Aggregate for Concrete Production. *British Journal for Applied Science & Technology, Vol. 3* (4), pp. 813-825.
Don, B. J., and Richmond, J.L. (2007). *Milling and Recycling*. Technical Paper, Chattanooga, TN.
Douglas, S.K. (2003). Toxicity Characteristic Leaching Procedure and Iron Treatment of Brass Foundry Waste. *Environmental Science and Technology, Vol. 2003* (37), pp. 367-371.
Dungan, R.S., and Dees, N.H. (2009). The Characterization of Total and Leachable Metals in Foundry Molding Sands. *Journal of Environmental Management, Vol. 90*, pp. 539-548.
Dungan, R.S., Lee, B.D., Shouse, P., and de Koff, J.P. (2007). Saturated Hydraulic Conductivity of Soils Blended with Waste Foundry Sands. *Soil Science, Vol. 10*, pp. 751-758.
Edil, T.B., Tinjum, J.M., and Benson, C, H. (2012). *Recycled Unbound Materials*. Final Report, Minnesota Department of Transportation.
Emery, J. (1993). *Spent Foundry Sand: Alternative Uses Study*. Canadian Foundry Association & Ministry of the Environment and Energy, Ontario.
Erdema, S., and Blanksonb, M.A. (2014). Environmental Performance and Mechanical Analysis of Concrete Containing Recycled Asphalt Pavement (RAP) and Waste Precast concrete as Aggregate. *Journal of Hazardous Materials, Vol. 264* (2014), pp. 403-410.
Esping, O. (2007). Early Age Properties of Self-Compacting Concrete—Effects of Fine Aggregate and Limestone Filler. Thesis for PhD Degree, Chalmers University of Technology, Sweden.
Estevez, B., Aguado, A., and Jose, A. (2008) *Environmental Impact of Concrete Recycling Coming from Construction and Demolition Waste*. Universitat Politecnicea de Catalunya, School of Engineering, Jordi Girona, Spain.
Etxeberria, M., Pacheco, C., Meneses, J.M., and Beerridi, I. (2010). Properties of Concrete Using Metallurgical Industrial By-Product as Aggregate. *Construction and Building Material, Vol. 24*, pp. 1594-1600.
Evangelista, L. and Brito, J. (2007). *Environmental Life Cycle Assessment of Concrete Made with Fine Recycled Concrete Aggregates*. Sustainable Construction, Materials and Practices, Lisbon.
Evangelista, L. and Brito, J. (2007). *Environmental Life Cycle Assessment of Concrete Made with Fine Recycled Concrete Aggregates*. Sustainable Construction, Materials and Practices, Lisbon.
Federal Highway Administration. (2004). *Foundry Sand Facts for Civil Engineers*. Report, Federal Highway Administration.
Feng, T.W., Lee, J.Y. and Lee, Y.J. (2001). Consolidation Behavior of a Soft Mud Treated With Small Cement Content. *Engineering Geology, Vol. 59* (3), pp. 327-334.
FHWA. (1997). *User Guidelines for Waste and Byproduct Materials in Pavement Construction: Reclaimed Asphalt Pavement Material Description*. Federal Highway Administration. Available: http://www.fhwa.dot.gov/publications/ research/ infrastructure/structures/97148/rap131.cfm
FHWA. (2015). *Road Surfacing Option Photo Album (Supplementary Document)*. Federal Highway Administration. Online Available: http://www.cflhd.gov/programs/techDevelopment/pavement/context-roadway-surfacing/ documents/roadway-surfacing-photos/ch9-a-fdr.pdf
Fick, G.J. (2008). *Testing Guide for Implementing Concrete Paving Quality Control Procedures*. Report, DTFH 61-06-H-00011, Federal Highway Administration, Washington, DC.
Fiore, S., and Zanetti, M.C. (2007). Foundry Waste Reuse and Recycling in Concrete Production. *Journal of Environmental Sciences, Vol. 3* (3), pp. 135-142.
Foundry Industry Recycling Starts Today (FIRST). (2003). *Case Study: Foundry Sand as a Structural Fill and Road Base*. Federal Highway Administration & Environmental Protection Agency, Washington, DC.
Foundry Industry Recycling Starts Today (FIRST). (2004). *Foundry Sand Facts for Civil Engineers*. Federal Highway Administration & Environmental Protection Agency, Washington, DC.
Fox, P.J., and Mast, D.G. (1998). *Geotechanical Perforamnce of a Highway Embankment Constructed Using Wate Foundry Sand*. Final Project, Indiana Department of Transportation.

Francingues, N.R., Thompson, D.W., and Hamons, F.S. (2011). Exploring Innovative Reuse of Dredged Material from Baltimore Harbor. Proceeding of 2011 WEDA Technical Conference and Texas A&M Dredging Seminar.

Frangopol, D.M. and Tsompanakis, Y. (2014). *Maintenance and Safety of Aging Infrastructure: Structures and Infrastructures Book Series.* CRC press.

Fu, P., and Harvey, J.T. (2007). Temperature Sensitivity of Foamed Asphalt Mix Stiffness: Field and Lab Study. *International Journal of Pavement Engineering*, *Vol. 8* (2), pp. 137-145.

Fu, P., Jones, D., and Harvey, J. (2010a). Micromechanics of the Effects of Mixing Moisture on Foamed Asphalt Mix Properties. *Journal of Materials in Civil Engineering*, *Vol. 22* (10), pp. 985-995.

Fu, P., Jones, D., and Harvey, J.T. (2011). The Effects of Asphalt Binder and Granular Materials Characteristics on Foamed Asphalt Mix Strength. *Construction and Building Materials*, *Vol. 25* (2), pp. 1093-1101.

Fu, P., Jones, D., Harvey, J.T., and Bukhari, S.A. (2008). Dry and Soaked Laboratory Tests for Foamed Asphalt Mixes. *Journal of the Association of Asphalt Paving Technologists*, *Vol. 77*, pp. 71-106.

Fu, P., Jones, D., Harvey, J.T., and Bukhari, S.A. (2009). “Laboratory Testing Methods for Foamed Asphalt Mix Resilient Modulus”. *Road Materials and Pavement Design*, *Vol. 10* **(**1), pp. 187-212.

Fu, P., Jones, D., Harvey, J.T., and Halles F.A. (2010b). Investigation of the Curing Mechanism of Foamed Asphalt Mixes Based on Micromechanics Principles. *Journal of Materials in Civil Engineering*, *Vol. 22* (1), pp. 29-38.

Gabr, A. and Cameron, D. (2012). Properties of Recycled Concrete Aggregate for Unbound Pavement Construction. *Journal of Materials in Civil Engineering*. *Vol. 24* (6), pp. 754-764.

Gambatese, J. A., and Rajendran, S. (2005). Sustainable Roadway Construction: Energy Consumption and Material Waste Generation of Roadways. Proceeding of Construction Research Congress 2005, San Diego, CA.

Ganne, V.K. (2009). *Long-Term Durability Studies on Chemically Treated Reclaimed Asphalt Pavement (RAP) Materials*. Thesis for Master Degree, University of Texas at Arlington.

Garber, S., Rasmussen, R., Cackler, T., Taylor, P., Harrington, D., Fick, G., Snyder, T., and Van Dam, C.L. (2011). *A Technology Deployment Plan for the Use of Recycled Concrete Aggregate in Concrete Paving Mixtures*. Final report, Federal Highway Administration, Washington, DC.

GB/T2842. (1981). *Test Method for Lightweight Aggregates*. China National Standard.

Gedik, A. (2008). *Utilization of Waste Foundry Sand as Highway Material*. Thesis for Master Degree, Department of Transportation Engineering, Istanbul Technical University, Istanbul, Turkey.

Gervais, C., Garrabrants, A. C., Sanchez, F., Barna, R., Moszkowicz, P., and Kosson, D. S. (2004). The Effects of Carbonation and Drying During Intermittent Leaching on the Release of Inorganic Constituents from a Cement-Based Matrix. *Cement and Concrete Research*, *Vol. 34* (1), pp. 119-131.

Gonzalez, G.P. (2002). *Recycling and Reuse of Reclaimed Portland Cement Concrete Aggregate*. Thesis for Master Degree, Lehigh University.

Gregory, J.S. and Edil, T.B.(2009). *Literature Search and Report on Recycled Asphalt Pavement and Recycled Concrete Aggregate*. Report, Minnesota Department of Transportation.

Grubb, D.G., Davis, A., Sands, S.C., Carnivale, M., Wartman, J., and Gallagher, P.M., (2007). Errata for: Field Evaluation of Crushed Glass-Dredged Material Blends. *Geotechnical and Geoenvironmental Engineering*, *Vol. 133* (1), pp. 127-128.

Grubb, D.G., Gallagher, P.M., Wartman, J., Liu, Y. and Carnivale III, M. (2006). Laboratory Evaluation of Crushed Glass-Dredged Material Blends. *Geotechnical and Geoenvironmental Engineering Vol. 132* **(**5), pp. 562-576.

Grubb, D.G., Wartman, J., and Malasavage, N.E. (2008). Crushed Glass-Dredged Material (CG-DM) Blends: Role of Organic Matter Content and DM Variability on Field Compaction. *Geotechnical and Geoenvironmental Engineering*, *Vol. 134* (11), pp. 1665-1675.

Grubb, D.G., Wartman, J., and Malasavage, N.E. (2010). Characterization of Slag Fines for Use as a Dredged Material Amendment. *Advances in Analysis, Modeling & Design*, *Vol. 138* (8), pp. 2953-2962.

Grubb, D.G., Wazne, M., Jagupilla, S., Malasavage, N.E., and Bradfield, W.B. (2013). Aging Effects in Field-Compacted Dredged Material: Steel Slag Fines Blends. *Hazardous, Toxic, and Radioactive Waste*, *Vol. 17* (2), pp. 107-119.

Grubb, D.G., Wazne, M., Jagupilla, S.C., and Malasavage, N.E. (2011). Beneficial Use of Steel Slag Fines to Immobilize Arsenite and Arsenate: Slag Characterization and Metal Thresholding Studies. *Hazardous. Toxic, and Radioactive Waste*, *Vol. 15* (3), pp. 130-150.

Gul, W.A. (2008). *Effect of Recycled Cement Concrete Content on Rutting Behavior of Asphalt Concrete*. Thesis for PhD Degree, Middle East Technical University, Ankara.

Guney, Y., Aydilek, A.H., and Demirkan, M.M. (2006). Geoenvironmental Behavior of Foundry Sand Amended Mixtures for Highway Subbases. *Waste Management*, *Vol. 26* (9), pp. 932-945.

Guney, Y., Sari, Y.D., Yalcin, M., Tuncan, A., and Donmez, S. (2010). Re-Usage of Waste Foundry Sand in High-Strength Concrete. *Waste Management*, *Vol. 30*, pp. 1705-1713.

Guthrie, W.S., Cooley, D., and Eggett, D.L. (2007). Effects of Reclaimed Asphalt Pavement on Mechanical Properties of Base Materials. *Transportation Research Board*, *Vol. 2005*, pp. 44-52.

Hadjieva-Zaharieva, R., and Buyle-Bodin, F. (2003). Opportunity for Using Recycled Aggregates in Reinforced Concrete: Carbonation Study. *Cement and Concrete Composites*, *Vol. 25* **(**2), pp. 223-232.

Hamer, K., and Karius, V. (2002). Brick Production with Dredged Harbour Sediments: An Industrial-Scale Experiment. *Waste Management*, *Vol. 22*, pp. 521-530.

He, G., and Wong, W. (2006). Effects of Moisture on Strength and Permanent Deformation of Foamed Asphalt Mix Incorporating RAP Materials. *Construction and Building Materials*, *Vol. 22* (2008), pp.30-40.

Hertwich, E., Mateles, S., Pease, W., and McKones, T. (2001). Human toxicity potentials for life cycle assessment and toxics release inventory risk screening. *Environmental Toxicology and Chemistry, vol 20(4)*, pp. 928-939.

Horvath, A. (2004). *A Life-Cycle Analysis Model and Decision-Support Tool for Selecting Recycled Versus Virgin Materials for Highway Applications*. Final Report, University of California at Berkeley.

Hossiney, N.J. (2008). *Evaluation of Concrete Containing RAP for Use in Concrete Pavement*. Thesis for Master Degree, University of Florida.

Hossiney, N.J. (2012). *Evaluation of Concrete Mixtures Containing RAP for Use in Concrete Pavements*. Thesis for PhD Degree, University of Florida.

Houston, S. L., Houston, W. N., and Williams, N. D. (1985). Thermomechanical Behavior of Seafloor Sediments. *Journal of Geotechnical Engineering*, *Vol. 11* (1249), pp. 1249-1263.

Hoyos, L.R., Puppala, A.J., and Ordonez, C.A. (2011). Characterization of Cement-Fiber-Treated Reclaimed Asphalt Pavement Aggregates: Preliminary Investigation. *Journal of Materials in Civil Engineering*, *Vol. 23* (7), pp. 977-989.

Huang, B., Li, G., Vukosavljevic, D., Shu, X., and Egan, B.K. (2005a). Laboratory Investigation of Mixing Hot-Mix Asphalt with Reclaimed Asphalt Pavement. *Transportation Research Board*, *Vol. 1929*, pp. 37-45.

Huang, B., Shu, X., and Li, G. (2005b). Laboratory Investigation of Portland Cement Concrete Containing Recycled Asphalt Pavements. *Cement and Concrete Research*, *Vol. 35*, pp. 2008-2013.

Huang, C., Pan, J.R., and Liu, Y. (2005c). Mixing Water Treatment Residual with Excavation Waste Soil in Brick and Artificial Aggregate Making. *Environmental Engineering*, *Vol. 131* (2), pp. 272-277.

Huang, Y.H., Zhu, W., Qian, X., Zhang, N., and Zhou, X. (2011). Change of Mechanical Behavior Between Solidified and Remolded Solidified Dredged Materials. *Engineering Geology*, *Vol. 119* (2001), pp. 112-119.

Hughes, C., (2002). *Recycled Foundry Sand: Sources and Properties*. Hot Mix Asphalt Technology, Available: http://www.hotmix.org/images/stories/rfs1.pdf.

IADC. (2009). Facts about Dredged Material as a Resource. International Association of Dredging Companies.

Ideker, J. H., Adams, M. P., Tanner, J., and Jones, A. (2011). *Durability Assessment of Recycled Concrete Aggregates for Use in New Concrete*. Report, Oregon Transportation Research and Education Consortium.

Ideraldo, B.I., Teixeira, P.C., Possamai. O. (2003). Technical availability of the reusing of the Melting Sand Residue: MSR on the hot bitumen concrete. *Proceeding of International Congress of Environment & Transport*, Avignon, France.

Javed, S., and Lovell, C.W. (1994). *Use of Waste Foundry Sand in Highway Construction*. Report, Purdue School of Engineering, West Lafayette, IN.

Jenkins, K.J., Long, F.M., and Ebels, L.J. (2007). Foamed Bitumen Mixes' Shear Performance. *International Journal of Pavement Engineering, Vol. 8* (2), pp. 85-98.
Ji, S.,Wan, L., and Fan, Z. (2001). The Toxic Compounds and Leaching Characteristics of Spent Foundry Sands. *Water Air and Soil Pollution. Vol. 2001* (132), pp. 347-364.
Karius, V., and Hamer, K. (2001). pH and Grain-Size Variation in Leaching Tests with Bricks Made of Harbour Sediments Compared to Commercial Bricks. *The Science of the Total Environment*, *Vol. 278*, pp. 73-85.
Kaur, G., Siddique, R., and Rajor, A. (2013). Micro-Structural and Metal Leachate Analysis of Concrete Made with Fungal Treated Waste Foundry Sand. *Construction and Building Materials*. *Vol. 38*, pp. 94-100.
Kazmee, H., Tutumluer, E., and Mishra, D. (2012). Effects of Material Blending on Strength, Modulus and Deformation Characteristics of Recycled Concrete Aggregates. *Proceedings of the 91st Annual Meeting of the Transportation Research Board*, Washington D.C.
Khatib, J. M., Baig, S., Bougara, A., and Booth, C. (2012). Foundry Sand Utilisation in Concrete Production. *Proceeding of 2nd International Conference on Sustainable Construction Material and Technologies*, Ancona, Italty.
Khatib, J.M., and Clay, R. (2004). Absorption Characteristics of Metakaolin Concrete. Cement and Concrete Research, *Vol. 34*(1), pp. 19-29.
Khatib, J.M., Herki, B.A., and Kenai, S. (2013). Capillarity of Concrete Incorporating Waste Foundry Sand. *Construction and Building Materials*, *Vol. 47*, pp. 867-871.
Khosravifar, S. (2012). *Design and Mechanical Properties of foamed Asphalt Stabilized Base Material*. Thesis for Master degree, University of Maryland, College Park.
Khosravifar, S., Goulias, D.G., and Schwartz, C.W. (2012). Laboratory Evaluation of Foamed Asphalt Stabilized Base Materials. GeoCongress 2012, Oakland, California.
Kim, H. and Goulias, D.G. (2014). Shrinkage Behavior of Sustainable Concrete with Crushed Returned Concrete Aggregate. *Journal of Materials in Civil Engineering*, *Vol. 27* (7), pp. 1-7.
Kim, Y., and Lee, H. (2006). Development of Mix Design Procedure for Cold In-Place Recycling with Foamed Asphalt. *Journal of Materials in Civil Engineering*, *Vol. 18* (1), pp. 116-124.
Kim, Y., Lee, H., and Heitzman, M. (2009). Dynamic Modulus and Repeated Load Tests of Cold In-Place Recycling Mixtures Using Foamed Asphalt. *Journal of Materials in Civil Engineering*, *Vol. 21* (6), pp. 279-285.
Kim, Y.T., Ahn, J., Han, W.J., and Gabr, M.A. (2010). Experimental Evaluation of Strength Characteristics of Stabilized Dredged Soil. *Journal of Materials in Civil Engineering*, *Vol. 22* (5), pp. 539-544.
Kim, Y.T., and Kang, H.S. (2011). Engineering Characteristics of Rubber Added Lightweight Soil as a Flowable Backfill Material. *Journal of Materials in Civil Engineering*, *Vol. 10*, pp. 1289-1294.
Kleven, J. R., Edil, T. B., and Benson, C. H. (2000). Evaluation of Excess Foundry System Sands for Use as Subbase Material. *Transportation Research Board*, *Vol. 1714*, pp. 40-48.
Kolay, P. K., and Akentuna, M. (2014). Characterization and Utilization of Recycled Concrete Aggregate from Illinois as a Construction Material. Geo-Congress 2014, Atlanta, Georgia.
Kou, S.C., Poon, C.S. and Lam, L. (2002). Use of Recycled Aggregates in Molded Concrete Bricks and Blocks. *Construction and Building Materials*, *Vol. 16*, pp. 281-289.
Kumar, M.P., and Monteiro, P.J.M. (1993). *Concrete: Structure, Properties and Materials, 2nd Edition*. Prentice Hall, New Jersey, NJ.
Kuo, S.S., Mahgoub, H.S., and Nazef, A. (2001a). Investigation of Recycled Concrete Made with Limestone Aggregate for a Base Course in Flexible Pavement. *Transportation Research Record*, *Vol. 1787*, pp. 99-108.
Kuo, S.S., Mahgoub, H.S., and Ortega, J.E. (2001b). *Use of Recycled Concrete Made with Florida Limestone Aggregate for a Base Course in Flexible Pavement*. Report, Florida Department of Transportation.
Lafhaj, Z., Samara, M., Agostini, F., Boucard, L., Skoczylas, F., and Depelsenaire, G. (2008). Polluted River Sediments From the North Region of France: Treatment with Novosol® Process and Valorization in Clay Bricks. *Construction and Building Materials*, *Vol. 22*, pp. 755-762.
Lee, C.H., Du, J.C., and Shen, D.H. (2012). Evaluation of Pre-Coated Recycled Concrete Aggregate for Hot Mix Asphalt. *Construction and Building Materials*, *Vol. 28*, pp. 66-71.
Lee, J., Edil, T.B., Benson, C.H., and J.M. (2011). Evaluation of Variables Affecting Sustainable Highway Design

Using The BE^{2}ST-in-Highways^{TM} System. *Journal of the Transportation Research Board, vol. 2233*, pp. 178-186.

Lee, K.W., Shukla, A., Soupharath, N., and Wilson, J. (2002). Low Temperature Cracking Resistance Characteristics of Recycled Asphalt Pavement Binder. Report, Rhode Island Department of Transportation Research and Technology Development.

Lee, T., and Benson, C.H. (2002). Using Waste Foundry Sands as Reactive Media I n Permeable Reactive Barriers. Geotechnical Engineering Report, University of Wisconsin-Madison.

Lee, T., and Benson, C.H. (2006). Leaching Behavior of Green Sands From Gray-Iron Foundries Used For Reactive Barrier Applications. *Environmental Engineering Science*, *Vol. 23* (1), pp.153-167.

Legreta, M., Odieb, L., Demarea, D., and Julliena, A. (2005). Leaching of Heavy Metals and Polycyclic Aromatic Hydrocarbons from Reclaimed Asphalt Pavement. *Water Research, Vol. 39*, pp. 3675-3685.

Li, X., Marasteanu, M., Williams, R., and Clyne, T. (2008). Effect of RAP (Proportion and Type) and Binder Grade on the Properties of Asphalt Mixtures. *Transportation Research Board*: *Vol. 2051*, pp. 90-97.

Lim, S., and Zollinger, D. G. (2003). Estimation of the Compressive Strength and Modulus of Elasticity of Cement-Treated Aggregate Base Materials. *Transportation Research Board*, *Vol. 1837*, pp. 30-38.

Lim, S., Kestner, D., Zollinger, D.G., and Fowler, D.W. (2003). *Characterization of Crushed Concrete Materials for Paving and Non-paving Applications*. Final report, Research and Technology Implementation Office, Texas Department of Transportation, Austin, Texas.

Limeira, J., Agulló, L., and Etxeberria, M. (2012). Dredged Marine Sand as a New Source for Construction Materials. *Materials of Construction*, *Vol. 62* (305), pp. 7-24.

Maher, A. (2013). *Preparation of a Manual for Management of Processed Dredge Material at Upland Sites*. Report, New Jersey Department of Transportation, Trenton, NJ.

Maher, A., and Boile, M. (2007). *Implementation of Construction Quality Assurance (CQA) Measures for Harbor Sediments and Processed Dredged Material*. Report, New Jersey Department of Transportation, Trenton, NJ.

Maher, A., Douglas, W.S., Jafari, F. and Gucunski, N. (2006). Field Placement and Evaluation of Stabilized Dredged Material (SDM) from New York/New Jersey Harbor. *Journal of Marine Georesource and Geotechnology*, *Vol. 24* (12), pp. 251-263.

Malasavage, N.E., Jagupilla, S., Grubb, D.G., Wazne, M., and Coon, W.P. (2012). Geotechnical Performance of Dredged Material—Steel Slag Fines Blends: Laboratory and Field Evaluation. *Journal of Geotechnical and Geoenvironmental Engineering*, *Vol. 138* (8), pp. 981-991.

Mast, D.G., and Fox, P.J. (1998). *Geotechnical Performance of Highway Embankment Constructed Using Waste Foundry Sand*. Technical Report, Joint Transportation Research Program, Federal Highway Administration.

McGarrah, E.J. (2007). *Evaluation of Current Practices of Reclaimed Asphalt Pavement/Virgin Aggregate as Base Course Material*. Final Report, Washington State Department of Transportation.

Members of the Innovative Reuse Committee (MIRC). (2007). *Innovative Reuse of Dredged Material*. Report, Maryland's Dredged Material Management Program.

Miguel, R.E., Ippolito, J.A., Leytem, A.B., Porta, A.A., Noriega, R.B., and Dungan, R.S. (2012). Analysis of Total Metals in Waste Molding and Core Sands from Ferrous and Non-Ferrous Foundries. *Journal of Environmental Management*, *Vol. 110*, pp. 77-81.

Miller, E., Bahia, H.U., Benson, C., Khatri, A., and Braham, A. (2001). Utilization of waste foundry sand in hot mix asphalt mixtures. *American Foundry Society Transactions*, *Vol. 103* (1), pp. 1393-1407.

Millrath, K. (2003). *Modifying Concrete Matrices with Beneficiated Dredged Material or Other Clayey Constitutes*. Thesis for PhD Degree, Columbia University.

Millrath, K., Kozlova, S., Shimanovich, S., and Meyer, C. (2001). *Beneficial Use of Dredge Material*. Progress Report, Department of Civil Engineering and Engineering Mechanics, Columbia University.

Mills-Beale, J., and You, Z. (2010). The Mechanical Properties of Asphalt Mixtures with Recycled Concrete Aggregates. *Construction and Building Material*, *Vol. 24*, pp. 340-345.

Ministry of Development (MOD). (2015). *General Technical Specification for Construction of Road and Bridges (PG-3)*. Madrid, Spain. Available: http://www.fomento.gob.es/MFOM.CP.Web/handlers/ pdfhandler.ashx?idpub=ICW020

Mir, B.A., Wani, B.A., Ahmad, N., Ayoub, R., and Aziz, J. (2013). Physical and Compaction behavior of Dredged material from Dal Lake Srinagar. *International Journal of Civil Engineering and Applications, Vol. 3* (7), pp. 4-8.

Mohammad, L., Abu-Farsakh, M., Wu, Z., and Chris, A. (2003). Louisiana Experience with Foamed Recycled Asphalt Pavement Base Materials. *Transportation Research Board, Vol. 1832*, pp.17-24.

Mohammad, L.N. (2006). Laboratory Evaluation of Untreated and Treated Pavement Base Materials. *Transportation Research Board*, *Vol. 1967*, pp. 78-88.

Mokwa, R.L. and Peebles, C.S. (2005), *Evaluation of the Engineering Characteristics of RAP/Aggregate Blends*. Report, Montana Department of Transportation.

Molin, D., Larsson, K., and Arvidsson, H. (2004). Quality of Reused Crushed Concrete Strength, Contamination and Crushing Technology. *International RILEM Conference on the Use of Recycled Materials in Building and Structures*, Barcelona, Spain.

Monosi, S., Sani, D., and Tittarelli, F. (2010). Used Foundry Sand in Cement Mortars and Concrete Production. *The Open Waste Management Journal*, *Vol. 3*, pp. 18-25.

Montemayor, T.A. (1998). *Compaction and Strength-Deformation Characteristics of Reclaimed Asphalt Pavement*. Thesis for Master Degree, Florida Institute of Technology, Melbourne, Florida.

Moropoulou, A., Bakolas, A., and Aggelakopoulou, E. (2001). The Effects of Limestone Characteristics and Calcination Temperature to the Reactivity of the Quicklime. *Cement and Concrete Research*, *Vol. 31* (4), pp. 633-639.

Morris, D.V. and Delphia, J.G. (1999). *Specifications for Backfill of Reinforced-Earth Retaining Walls*. Report, Texas Transportation Institute.

Mostafa, A.I., Hackmet, A.J., Wee, H.S, and Mark, F.R. (2002). Effect of Cement Type on Shear Behavior of Cemented Calcareous Soil. *Journal of Geotechnical and Geoenvironmental Engineering*, *Vol. 128*, pp. 520-529.

Moukwa, M. (1993). Effect of Clays on Fracture Properties of Cement-Bases Materials. *Cement and Concrete Research*, *Vol. 23*, pp. 711-723.

MPCA. (2000). *Protecting Water Quality in Urban Areas*. Guidance, Minnesota Pollution Control Agency, Saint Paul, MN.

Mroueh, U., Eskola, P., and Laine-Ylijoki, J. (2001). Life-cycle impacts of the use of industrial by-products in road and earth construction. *Waste Management, vol. 21*, pp. 271-277.

Naik, T.R., and Singh, S.S. (2001). Performance and Leaching Assessment of Flowable Slurry. *Journal of Environmental Engineering*, *Vol. 127* (4), pp. 359-368.

Nam, B.H, Behring, Z., Kim, J., and chopra, M. (2014). *Evaluate the Use of Reclaimed Concrete Aggregate in French Drain Applications*. Final report, Florida Department of Transportation, Tallahassee, FL.

NAPA. (2014). *Rubilization and Perpetual Pavement Save Money and Resource*. National Symposium on HMA Energy & Recycling, National Asphalt Pavement Association. Online Available: http://www.asphaltpavement.org/images/stories/ Rubblization_and_Perpetual_Pavements.pdf

Nataatmadja, A. (2002). Foamed Bitumen Mix: Soil or Asphalt. Proceedings of 9th International Conference on Asphalt Pavements, Copenhagen, Denmark.

National Cooperative Highway Research Program (NCHRP) 435. (2013). Recycled Materials and Byproducts in Highway Applications. *Transportation Research Broad*, *Vol. 8*, pp. 50-65.

NCHRP 435. (2013). Recycled Materials and Byproducts in Highway Applications. *Transportation Research Board*, *Vol. 6.*

NCHRP 452. (2001). Recommended Use of Reclaimed Asphalt Pavement in the Superpave Mix Design Method: Technician's Manual. *Transportation Research Board*, *Vol. 28.*

Ndiba, P.K., and Axe, L. (2009). Sequential Extraction of Phosphate- and Thermal-Treated New York/New Jersey Harbor Dredged Sediments. *Environmental Engineering Science*, *Vol. 26* (12), pp. 1755-1764.

Nejad, F.M., Azarhoosh, A.R., and Hamedi, G.H. (2013). The Effects of Using Recycled Concrete on Fatigue Behavior of Hot-Mix Asphalt. *Journal of Civil Engineering and Management*, *Vol. 19* (1), pp. 561-568.

Neville, A.M. (1997). *Properties of Concrete, 4th Edition*. John Wiley & Sons, New York, NY.

OC Waste&Recycling (2015). *Landfill Fees*. Online Available: http://oclandfills.com/landfill/landfill_fees

Ocean and Coastal Consultants (OCC) (2010). *Dredged Material Classification Project*. Final Report, New Jersey Department of Transportation, Trenton, NJ.
Oh, H., Lee, J., Banthia, N., and Talukdar, S. (2011). An Experimental Study of the Physicochemical Properties of a Cement Matrix Containing Dredged Materials. *Materials Sciences and Application*, *Vol. 2*, pp. 848-858.
Okafor, F.O. (2010). Performance of Recycled Asphalt Pavements as Coarse Aggregates in Concrete. *Leonardo Electronic Journal of Practices and Technologie, Vol. 17*, pp. 47-58.
Okuno, N., and Takahashi, S. (1997). Full Scale Application of Manufacturing Bricks from Sewage. *Water Science and Technology*, *Vol. 36*, pp. 243-250.
Olard, F., Noan, C., Bonneau, D., Dupriet, S., and Alvarez, C. (2008). Very High Recycling Rate (>50%) in Hot Mix and Warm Mix Asphalts for Sustainable Road Construction. Proceedings of the 4th Eurasphalt and Eurobitume Congress, Copenhagen, Denmark.
Otsuki. N., Miyazato. S., and Yodsudjai. W. (2003). Influence of Recycled Aggregate on Interfacial Transition Zone, Strength, Chloride Penetration and Carbonation of Concrete. *Journal of Materials in Civil Engineering*, *Vol. 15*, pp. 443-451.
Oweis, I.S. 1998. Geotechnics of Utilization of Dredged Sediments as Structural Fill. National Symposium on Contaminated Sediments: Coupling Risk Reduction with Sustainable Management and Reuse, Washington, DC.
Paranavithana, S., and Mohajerani, A. (2006). Effects of Recycled Concrete Aggregates on Properties of Asphalt Concrete. *Resource, Conservation and Recycling*, *Vol. 48*, pp. 1-12.
Park, D.W., Vo, H.V., Lim, Y. (2014). Recycling of Dredged Soil Waste Using Air-foam Stabilization Method as Highway Construction Material. Geo-Hubei 2014 International Conference on Sustainable Civil Infrastructure, Yichang, Hubei, China.
Pasandín, A.R., and Pérez, I. (2013). Laboratory Evaluation of Hot-Mix Asphalt Containing Construction and Demolition Waste. *Construction and Building Materials, Vol. 43*, pp. 497-505.
Pasandín, A.R., and Pérez, I. (2014). Mechanical Properties of Hot-Mix Asphalt Made with Recycled Concrete Aggregates Coated with Bitumen Emulsion. *Construction and Building Materials*, *Vol. 55*, pp. 350-358.
Pasandín, A.R., and Pérez, I. (2015). Overview of Bituminous Mixtures Made with Recycled Concrete Aggregates. *Construction and Building Materials, Vol. 74*, pp. 151-161.
Pathak, N., and Siddique, R. (2012). Effects of Elevated Temperatures on Properties of Self-Compacting Concrete Containing Fly Ash and Spent Foundry Sand. *Construction and Building Materials*, *Vol. 34*, pp. 512-521.
Pereira, P., Oliveira, J, and Picado-Santos, L. (2004). Mechanical Characterization of Hot Mix Recycled Materials. *International Journal of Pavement Engineering*, *Vol. 5* (4), pp. 211-220.
Pérez, I., Pasandín, A.R., and Medina, L. (2012). Hot Mix Asphalt Using C&D Waste as Coarse Aggregates. *Materials and Design*, *Vol. 36*, pp. 840-846.
Pérez, I., Toledano, M., Galego, J., and Taibo, J. (2007). Mechanical Properties of Hot Mix Asphalt Made with Recycled Aggregates from Reclaimed Construction and Demolition Debris. *Material and Construction*, *Vol. 57* (285), pp. 17-29.
Petrarca, R.W. and Galdiero, V.A. (1984). Summary of Testing of Recycled Crushed Concrete. *Transportation Research Record*, *Vol. 989*, pp. 19-26.
Plesser, S. W., Engelsen, J., Mehus, J., and Petkovic, G. (2006). Deterioration of RCA under Influence of Steady Water Drainage. International Conference on the Environmental and Technical Implication of Construction with Alternative Materials, Belgrade, Serbia and Montenegro.
Poindexter, M.E., Walker, J.E. (1988). Dredged Fill Performance: South Blakeley Island. *Hydraulic Fill Structures, Geotechnical special Publication*, *Vol. 21*, pp. 676-692.
Portland Cement Association (PCA). (2010). *Concrete Technology: Materials: Recycled Aggregates*. Available: http:// www.cement.org/tech/cct_aggregates_recycled.asp.
Potturi, A.K. (2006). *Evaluation of Resilient Modulus of Cement and Cement-Fiber Treated Reclaimed Asphalt Pavement (RAP) Aggregates Using Repeated Load Triaxial Test*. Thesis for Master Degree, The University of Texas at Arlington.

Prabhu, G.G., Bang, J.W., Lee, B.J., Hyun, J.H., and Kim, Y.Y. (2015). *Mechanical and Durability Properties of Concrete Made with Used Foundry Sand as Fine Aggregate*. Advances in Materials Science and Engineering, Hindawi Publishing Corporation.

Prabhu, G.G., Hyun, J.H., and Kim, Y.Y. (2014). Effects of Foundry Sand as a Fine Aggregate in Concrete Production, *Construction and Building Materials, Vol. 70*, pp. 514-521.

Rafi, M.M., Qadir, A., Siddiqui, S.H. (2011). Experimental Testing of Hot Mix Asphalt Mixture Made of Recycled Aggregates. *Waste Management and Research, Vol. 29* (12), pp. 1316-1326.

Ramanathan, R., Aydilek, A.H., and Tanyu, B. (2015). Development of a GIS-Based Failure Investigation System for Highway Soil Slopes. *Frontiers in Earth Science, vol. 9(2)*, pp. 165-178.

Ramanujam, J., and Jones, J. (2007). Characterization of Foamed-Bitumen Stabilization. *International Journal of Pavement Engineering, Vol. 8* (2), pp. 111-122.

Randall, R., Edge, B., Basilotto, J., Cobb, D., Graalum, S., He, Q., and Miertschin, M. (2000). *Texas Gulf Intracoastal Waterway (GIWW) Dredged Material: Beneficial Uses, Estimating Costs, Disposal Analysis Alternatives, and Separation Techniques*. Report, Texas Department of Transportation.

Rathje, E.M., Rauch, A.F., Folliard, K.J., Trejo, D., Little, D., Viyanant, C… Esfeller, M. (2001). *Recycled Asphalt Pavement and Crushed Concrete Backfill: State-of-the-Art Review and Material Characterization*. Report, Texas Department of Transportation.

Ravindrarajah, R. and Tam, C. T. (2005). Properties of Concrete made with Crushed Concrete as Coarse Aggregate. *Magazine of Concrete Research, Vol. 37* (130), pp. 29-38.

Reyes-Ortiz, O., Berardinelli, E., Alvarez, A. E., Carvajal-Muñoz, J. S., and Fuentes, L. G. (2012). Evaluation of Hot Mix Asphalt Mixtures with Replacement of Aggregates by Reclaimed Asphalt Pavement (RAP) Material. *Social and Behavioral Sciences, Vol. 53*, (3), pp. 379-388.

RMRC. (2010). *BE^2ST-IN-HIGHWAYS™: Building Environmentally and Economically Sustainable Transportation-Infrastructure-Highways*. User's Manual, Recycled Materials Resource Center, University of Wisconsin-Madison.

Rossetti, V.A., and Medici, F. (1995). Inertization of Toxic Metals in Cement Metrics: Effect of Hydration, Setting and Hardening. *Cement and Concrete Research, Vol. 25* (6), pp. 1147-1152.

Ruckel, P. J., Acott, S. M., and Bowering, R. H. (1983). Foamed-Asphalt Paving Mixtures: Preparation of Design Mixes and Treatment of Test Specimens. *Transportation Research Board, Vol. 911*, pp. 88-95.

Sadat Associates. (2000). *Use of Dredged Material for Construction of Roadway Embankments*. Final report, New Jersey Department of Transportation.

Saeed, A. (2008), Performance-Related Tests of Recycled Aggregates for Use in Unbound Pavement Layers. *Transportation Research Board*, NCHRP REPORT 598.

Sahmaran, M., Lachemi, M., Erdem, T., and Yucel, H.E. (2011). Use of Spent Foundry Sand and Fly Ash for the Development of Green Self-Consolidating Concrete. *Materials and Structures, Vol. 44*, pp. 1193-1204.

Sahmaran, M., Lachemi, M., Hossain, K., Ranade, R., and Li, V. (2009). "Influence of Aggregate Type and Size on Ductility and Mechanical Properties of Engineered Cementitious Composites." *Materials Journal, Vol. 106* (3), pp. 308-316.

Samara, M., Lafhaj, Z., and Chapiseau, C. (2009). Valorization of Stabilized River Sediments in Fired Clay Bricks: Factory Scale Experiment. *Journal of Hazardous Materials, Vol. 163*, pp. 701-710.

Samtani, N.C., Tanal,V., Wang, J., and Lancellotti, A.R. (1994). Effect of Lime Admixtures on Contaminated Dredged Sediments. 1st International Congress on Environmental Geotechnics, Edmonton, Alberta, Canada.

Sanchez, F., Langley White, M. K., and Hoang, A. (2009). Leaching from Granular Cement-Based Materials During Infiltration/Wetting Coupled with Freezing and Thawing. *Journal of Environmental Management, Vol. 90* (2), pp. 983-993.

Sayer, J.A., Kierans, M., and Gadd, G.M. (1997). Solubilisation of Some Naturally Occurring Metal Bearing Minerals, Limescale and Lead Phosphate by Aspergillus Niger. *FEMS Microbiology Letters, Vol. 154*, pp. 29-35.

Schwartz, C.W., Khosravifar, S., and Goulias, D.G. (2013). *Design and Evaluation of Foamed Asphalt Base Materials*. Final Report, Maryland State Highway Administration.

Senior, S.A., Szoke, S.I. and Rogers, C.A. (1994). Ontario's Experience with Reclaimed Materials for Use in Aggregates. International Road Federation Conference, Calgary, Alberta, Canada.
Shayan, A., and Xu, A. (2003). Performance and Properties of Structural Concrete Made with Recycled Concrete Aggregate. *Materials Journal*, *Vol. 100* (5), pp. 371-380.
Sheehan, C., Harrington, J., Murphy, J. P., and Riordan, J. (2010). An Investigation into Potential Beneficial Uses of Dredge Material in Ireland. *Journal of Resources*, *Vol. 55* (2), pp. 209-220.
Shehata, M.H., Thomas, M., and Bleszynski, R.F. (1999). The Effect of Fly Ash Composition on the Chemistry of Pore Solution. *Cement and Concrete Research*, *Vol. 29* (12), pp. 1915-1920.
Shen, J., Amirkhanian, S., Tang, B. (2007). Effects of Rejuvenator on Performance-Based Properties of Rejuvenated Asphalt Binder and Mixtures. *Construction and Building Materials*, *Vol. 21* (5), pp. 958-964.
Shetty, K.K., Nayak, G., and Shetty, R. (2014). Self-Compacting Concrete Using Red Mud and Used Foundry Sand. *International Journal of Research in Engineering and Technology*, *Vol. 3* (3), pp. 708-711.
Shirodkar, P., Mehta, Y., Nolan, A., Sonpal, K., Norton, A., Tomlinson, C., Dubois, E., Sullivan, P., and Sauber, R. (2011). A Study To Determine The Degree Of Partial Blending Of Reclaimed Asphalt Pavement (RAP) Binder For High RAP Hot Mix Asphalt. *Construction and Building Materials, vol. 25*, pp. 150-155.
Siddique, R., Aggarwal, Y., Aggarwal, P., Kadri, E., and Bennacer, R. (2011). Strength, Durability, and Micro-Structural Properties of Concrete Made with Used-Foundry Sand (UFS). *Construction and Building Materials, Vol. 25*, pp. 1916-1925.
Siddique, R., and Kaur, R. (2013). Properties of Self-Compacting Concrete Incorporating Waste Foundry Sand. *Leonardo Journal of Sciences*, *Vol. 23*, pp. 105-124.
Siddique, R., and Noumowe, A. (2008). Utilization of Spent Foundry Sand in Controlled Low-Strength Materials and Concrete. *Resources, Conservation and Recycling*, *Vol. 53*, pp. 27-35.
Siddique, R., de Schutter, G., and Noumowec, A. (2009). Effect of Used-Foundry Sand on the Mechanical Properties of Concrete. *Construction and Building Materials*, *Vol. 23*, pp. 976-980.
Siddique, R., Singh, G., Belarbi, R., Ait-Mokhtar, K., and Kunal. (2015). Comparative Investigation on the Influence of Spent Foundry Sand as Partial Replacement of Fine Aggregates on the Properties of Two Grades of Concrete. *Construction and Building Materials*, *Vol. 83*, pp. 216-222.
Silva, H. (2005). Study on the Use of Bituminous Mixtures Modified with Acrylic Fibres in Road Pavement Overlays. First International Conference on Transport Infrastructure, Beijing, China.
Silva, H., Oliveira, J., and Jesus, C. (2012). Are Totally Recycled Hot Mix Asphalts a Sustainable Alternative for Road Paving? *Resources, Conservation and Recycling*, *Vol. 60*, pp. 38-48.
Singh, G. and Siddique, R. (2012). Abrasion resistance and strength properties of concrete containing waste foundry sand (WFS). *Construction and Building Materials, vol. 28*, pp. 421-426.
Singh, G., and Siddique, R. (2011). Effect of Waste Foundry Sand (WFS) As Partial Replacement of Sand on the Strength, Ultrasonic Pulse Velocity and Permeability of Concrete. *Resources Manufacturing Conservation and Recycling*, *Vol. 26*, pp. 416-422.
Singh, G., and Siddique, R. (2012). Effect of Waste Foundry Sand (WFS) as Partial Replacement of Sand on the Strength, Ultrasonic Pulse Velocity and Permeability of Concrete. *Construction and Building Materials*, *Vol. 26*, pp. 416-422.
Smith, J. T., and Tighe, S. L. (2009). Recycled Concrete Aggregate Coefficient of Thermal Expansion. *Transportation Research Board*, *Vol. 2113*, pp. 53-61.
Snyder, B. and Bruinsma, J.E. (1996). Review of Studies Concerning Effects of Unbound Crushed Concrete Bases on PCC Pavement Drainage. *Transportation Research Record*, *Vol. 1519*, pp. 51-58.
Snyder, M. (2006). *Recycled Concrete Aggregate (RCA) for Infrastructure Elements*. Technical Report, Washington State Department of Transportation.
Soleimanbeigi, A. and Edil, T.B. (2015). Compressibility of Recycled Materials for Use as Highway Embankment Fill. *Journal of Geotechnical and Geoenvironmental Engineering*, *Vol. 141* (5), pp. 1-14.
Soleimanbeigi, A., Edil, T.B., and Benson, C.H. (2014). Engineering Properties of Recycled Materials for Use as Embankment Fill. Proceeding of Geo-Congress 2014, Atlanta, Georgia.
Springenschmid, R., and Sodeikat, C. (1998). High-Quality Concrete with Recycled Aggregates. Proceeding of 8th International Symposium on Concrete Roads, Lisbon, Portugal.

Stark, T.D., Arellano, D., Horvath, J.S., and Leshchinsky, D. (2004). Guidelines and recommended standards for geofoam applications in highway embankments. *Transportation Research Board*, NCHRP Report 529.

Steffes, R. (1999). *Laboratory Study of the Leachate from Crushed Portland Cement Concrete Base Material*. Research Report, Iowa Departrnent of Transportation, Ames, IA.

Tam, V.W., Tam, C.M., and Le, K.N. (2007). Removal of Cement Mortar Remains from Recycled Concrete Aggregate Using Pre-Soaking Approaches. *Resource, Conservation and Recycling, Vol. 50* (2007), pp. 82-101.

Tang, C.W., Chen, H.J., Wang, S.Y., and Spaulding, J. (2011). Production of Synthetic Lightweight Aggregate Using Reservoir Sediments for Concrete and Masonry. *Cement & Concrete Composites, Vol. 33*, pp. 292-300.

TFHRC. (2010). *Reclaimed Asphalt Pavement—Materials Description*. Turner–Fairbank Highway Research Center, McLean, VA. Available: http://www.tfhrc.gov/hnr20/recycle/waste-rap131.htm.

Thakur, J.K. (2011). *Geocell-Reinforced Unpaved and Paved Roads with Recycled Asphalt Pavement (RAP) Bases: Experimental Study and Damage Model Development*. Thesis for PhD Degree, University of Kansas.

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

Thakur, J.K., Han, J., Parsons, R.L. (2013). Creep Behavior of Geocell-Reinforced Recycled Asphalt Pavement (RAP) Bases. *Journal of Materials in Civil Engineering, Vol. 25* (10), pp. 1533-1543.

Thakur, J.K., Han, J., Pokharel, S.K., and Parsons, R.L. (2012). Performance of Geocell-Reinforced Recycled Asphalt Pavement (RAP) Bases over Weak Subgrade under Cyclic Plate Loading. *Geotextiles and Geomembranes, Vol. 35*, pp. 14-24.

Thevanayagam, S., Shenthan, T., Mohan, S., and Liang, J. (2002). Undrained Fragility of Clean Sands, Silty Sands, and Sandy Silts. *Journal of Geotechnical and Geoenvironmental Engineering, Vol. 10* (849), pp. 849-859.

Tikalsky, P., Bahia, H., Deng, A., and Snyder, T. (2004). *Excess Foundry Sand Characterization and Experimental Investigation in Controlled Low-Strength Material and Hot-Mixing Asphalt*. Report, U.S. Department of Energy.

Townsend, T. (1998). *Leaching Characteristics of Asphalt Road Waste*. Report, Florida Center for Solid and Hazardous Waste Management, Gainesville, FL.

Valdes, G., Perez-Jimenez, F., Miro, R., Martinez, A., and Botella, R. (2011). Experimental Study of Recycled Asphalt Mixtures with High Percentages of Reclaimed Asphalt Pavement (RAP). *Construction and Building Materials, Vol. 25* (3), pp. 1289-1297.

Volz, J.S., Khayat, K.H., Arezoumandi, M., Drury, J., Sadati, S., Smith, A., and Steele, A. (2014). *Recycled Concrete Aggregate (RCA) for Infrastructure Elements*. Report, U.S. Department of Transportation, Washington, D.C.

Wang, D.X., Abriakand, N.E., and Zentar, R. (2011). Durability Analysis of Fly Ash/Cement-Solidified Dredged Materials. Proceeding of Coastal and Maritime Mediterranean Conference.

Wasay, S.A., Barrington, S.F., and Tokunaga, S. (1998). Using Aspergillus Niger to Bioremediate Soils Contaminated by Heavy Metals. *Journal of Bioremediation & Biodegradation, Vol. 2*, pp. 183-190.

Weiland, C.D., and Muench, S.T. (2010). *Life Cycle Assessment of Portland Cement Concrete Interstate Highway Rehabilitation and Replacement*. Research report, Washington State Department of Transportation.

Weimann, K., and Muller, A. (2004). Properties of Building Materials Gained from Wet-processed Crushed Concrete Fines. Proceeding of International RILEM Conference on the Use of Recycled Materials in Buildings and Structures, Barcelona, Spain.

Wen, H., Mclean, D., Boyle, S., Spry, T., and Mjelde, D. (2014). *Evaluation of Recycled Concrete as Aggregate in New Concrete Pavements*. Research Report, Washington State Department of Transportation.

Wen, H., Warner, J., Edil, T., and Wang, G. (2010). Laboratory Comparison of Crushed Aggregate and Recycled Pavement Material With and Without High Carbon Fly Ash. *Geotechnical and Geological Engineering, Vol. 28*, pp. 405–411.

Winfield, L.E., and Lee, C.R. (1999). *Dredged Material Characterization Tests for Beneficial Use Suitability*. DOER Technical Notes Collection, U.S.

Winkler, E.S., and Bolshakov, A.A. (2000). Characterization of Foundry Sand Waste. Chelsea Center for Recycling and Economic Development, University of Massachusetts at Lowell.
Wirtgen. (2010). *Wirtgen Cold Recycling Manual, 3rd Edition*. Windhagen, Germany.
Wong, Y.D., Sun, D.D., and Lai, D. (2007). Value-added Utilization of Recycled Concrete in Hot Mix Asphalt. *Waste Management*, *Vol. 27*, pp. 294-301.
Wu, J. Y., and Tsai, M. (2009). Feasibility Study of a Soil-Based Rubberized CLSM. *Waste Management*, *Vol. 29*, (2), pp. 636-642.
Wu, M.Q. (2011). *Evaluation of High Percentage Recycled Asphalt Pavement as Base Course Materials*. Thesis for Master Degree, Washington State University.
Xiao, F., Amirkhanian, S.N., Shen, J., and Putman, B. (2009). Influences of Crumb Rubber Size and Type on Reclaimed Asphalt Pavement (RAP) Mixtures, *Construction and Building Materials*, *Vol. 23* (2), pp. 1028-1034.
Yazoghli-Marzouk, O., Vulcano-greullet, N., Cantegrit, L., Friteyre, L., and Jullien, A. (2014). Recycling Foundry Sand in Road Construction–Field Assessment. *Construction and Building Materials*, *Vol. 61*, pp. 69-78.
Yuan, D., Nazarian, S., Hoyos, L.R., and Puppala, A.J. (2010). *Cement Treated RAP Mixes for Roadway Bases*. Final Report, Research and Technology Implementation Office, Texas Department of Transportation.
Zhu, J., Wu, S., Zhong, J., and Wang, D. (2012). Investigation of Asphalt Mixture Containing Demolition Waste Obtained from Earthquake-Damaged Buildings. *Construction and Building Materials*, *Vol. 29*, pp. 466-475.
Zulkati, A., Wong, Y.D., and Sun, D.D. (2013). Mechanistic Performance of Asphalt-Concrete Mixture Incorporating Coarse Recycled Concrete Aggregate. *Journal of Materials in Civil Engineering*, *Vol. 2*5 (9), pp. 1299-1305.