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Durability of Recycled Concrete Aggregate as a Pavement Base Material Including Drainage: A Laboratory and Simulation Study

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Abstract

Recycled concrete aggregates (RCAs) have the potential to be used as a sustainable, cost-effective, and environmentally friendly material in pavement base construction. However, there is a lack of information on the durability, strength, and hydraulic properties of RCA. The primary purpose of this study was to evaluate the properties and performances of commonly available RCAs in Oklahoma as pavement bases through laboratory testing and AASHTOWare Pavement ME simulations. For this purpose, three RCAs (RCA-1, RCA-2, and RCA-3) and a virgin limestone aggregate (VLA-1) were collected from local sources. RCA-1 and RCA-3 were produced in the field by crushing the existing concrete pavement of Interstate 40 and US 69 sections, respectively. RCA-2 was produced by crushing concrete and rubble collected in a local recycling plant. Laboratory testing for this study included particle size distribution, wash loss, optimum moisture content and maximum dry density (OMC-MDD), Los Angeles (LA) abrasion, durability indices (D_c and D_f), permeability (k), and resilient modulus (M_r). The properties of aggregates were compared and the service life (performance) of aggregate bases was studied through mechanistic analysis using the AASHTOWare Pavement ME design software (version 2.6, AASHTO, USA). The results indicated that the properties of RCAs can differ greatly based on the origin of the source materials and the methods used in their processing. Recycled aggregates from concrete pavements of interstate and state highways exhibited similar or improved performance as virgin aggregates. RCA produced in a recycling plant was found to show durability and strength issues due to the presence of inferior quality materials and contaminants. Also, the results indicated that the fine aggregate durability test is a useful tool for screening recycled aggregates to ensure quality during production and construction. Bottom-up fatigue cracking was identified as the most affected performance criterion for flexible pavements when using RCA as the base layer. The findings will help increase the use of RCA as pavement base to promote environmental sustainability.

Keywords: recycled concrete aggregate; durability; resilient modulus; permeability; performance; sustainability

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1. Introduction

In recent years, the use of recycled materials in highway construction has increased significantly due to emphasis on the conservation of natural resources and increased environmental stewardship and sustainability. The United States (U.S.) construction industry generates over twice the amount of municipal solid waste and is a significant contributor to the waste stream in the country. Construction and demolition (C&D) waste, produced from building, pavement, bridge, and other activities, is estimated to have reached 600 million tons in 2018, according to the U.S. Environmental Protection Agency (USEPA) [1]. Some waste management strategies involve placing C&D waste in landfills, causing considerable negative environmental impacts. As concrete constitutes a significant portion of waste generated by the construction industry in the U.S., reusing waste concrete in new construction or reconstruction can reduce the amount of C&D waste placed in landfills and promote sustainable construction practices.

Based on the experience of transportation agencies [2,3], the use of recycled concrete aggregate (RCA) has potential to produce durable and sustainable pavements with improved performance and service life. Several researchers have found that using RCA as a base and subbase material for new pavements is a suitable alternative to virgin aggregates [4–6]. Similar or improved strength of RCAs, in contrast to virgin aggregates, was reported in several studies [4,7–11]. Bozyurt et al. [9] studied the resilient modulus (M_r) of recycled asphalt pavement (RAP) and RCA as unbound base layers in Minnesota. It was reported that the RCA showed the lowest plastic strain during M_r testing. Gabr et al. [10] studied the resilient modulus of RCAs from two different sources and a virgin aggregate. Based on the results of this study, it was recommended that RCAs could be used as base materials when compacted at 98% of the maximum dry density. In terms of M_r and permanent strain, the RCA aggregates were found to have improved M_r values compared to those of virgin aggregate. Nataatmadja et al. [12] observed that RCA could have higher M_r values than virgin aggregates under low deviatoric stress. Similarly, several other studies reported higher M_r values of RCAs than those of virgin aggregates [13,14].

In addition, rigid and flexible pavements with an RCA base were found to exhibit similar or better performance in the field than the bases with virgin aggregates [3,15–20]. Zhang et al. [3] studied the long-term performance of pavement bases constructed with RCA. Aggregate blends of recycled materials and virgin aggregates were studied for an eight-year period with seasonal freezing in Minnesota. It was found that the climatic variations affected the performance more significantly than traffic loading in terms of long-term performance. Also, replacing virgin aggregate bases with 100% or 50% of RCA improved the stiffness of the base layer in the long run. However, the international roughness index (IRI) and rutting showed that the ride quality did not experience any significant changes. Kim et al. [11] compared the performance of concrete pavements with RCA and virgin aggregate bases in Iowa. The results indicated comparable performance of RCA bases and subbases constructed with virgin aggregates. However, the RCA test sections were found to have a few longitudinal and transverse cracks. These distresses likely resulted from lane-to-shoulder separation and lane-to-shoulder drop-off. Also, RCA has been used in blends with other recycled materials, including RAP, recycled bricks, glass, and fly-ash, as pavement base and subbase material [8,14,18,21].

In spite of the results reported in previous studies, there are concerns among state DOTs and private companies about the durability, permeability, and service life of pavement bases and subbases constructed with RCA. The quality of RCA can vary widely depending upon the source and quality of the original aggregates, age, environmental conditions, recycling process, specifications, and contaminants present in RCAs. Also, the specifications for acceptance or rejection of RCAs are currently lacking, particularly relative to drainage.

Furthermore, very limited data are available on RCAs, making it difficult for state DOTs to develop objective specifications for acceptance of such materials in new construction and reconstruction of roadway pavements. The present study sought to fill this gap through laboratory testing, addressing the stiffness, durability, and permeability of selected RCAs that are commonly used in constructing aggregate bases of roadway pavements in Oklahoma.

2. Objectives

The primary purpose of this study was to evaluate the properties and performances of commonly available RCAs in Oklahoma as pavement bases through laboratory testing and AASHTOWare Pavement ME simulations. For this purpose, RCAs from different sources were collected for testing. The properties of RCAs were determined and compared with those of a commonly used virgin limestone aggregate. The properties were then used for AASHTOWare Pavement ME simulations. The specific objectives of this study were to

- Assess the stiffness, durability and drainage-related properties of RCAs commonly available in Oklahoma to be used as pavement base.
- Evaluate the effect of source and quality of RCA on the performance of flexible and rigid pavements.
- Compare the performance and service life of pavement bases constructed with recycled and virgin aggregates using the AASHTOWare Pavement ME simulations.

3. Materials and Methods

Materials

For the purpose of this study, recycled aggregates were collected from three different sources in Oklahoma. The sources of the RCAs were decided upon based on the recommendations of the Oklahoma DOT's aggregate branch manager. Attempts were made to collect RCAs from both plant-based and recycled in-place sources, as previous studies indicated that the quality of RCAs can vary with source and production practices [22,23]. Therefore, among the three RCAs, one was collected from a plant-based source (RCA-2), and the other two (RCA-1 and RCA-3) from recycled in-place construction sites in Oklahoma. The collected aggregates, as received, can be seen in Figure 1. The RCA-1 and RCA-3 were produced in the field by crushing the existing concrete pavement of Interstate 40 and US 69 sections, respectively. Figure 2 provides a pictorial view of the recycling operation used on Interstate 40 and US 69. After taking the concrete rubble out of the existing pavement, the construction company placed this at an off-site plant. The rubble was crushed at that off-site plant to produce RCA of the desired gradation. The recycled aggregates were then transported and stockpiled near the construction site. During construction, aggregates were collected from the stockpile and placed on the pavement as a base. Working closely with the Oklahoma DOT and construction company, the research team collected RCA-1 and RCA-3 aggregates. RCA-2 was produced in a recycling plant by crushing concrete and rubble. Generally, this concrete and rubble may come from different sources, such as the demolition of buildings, driveways, and other sources. Therefore, RCAs are expected to differ in properties. Also, a commonly used virgin limestone aggregate (VLA-1) was collected from Davis, Oklahoma. The aggregates used in this study represent possible material candidates that could be used for designing and constructing pavement bases in Oklahoma. Oklahoma DOT standard specifications [24] recommend using one of four types of gradation (ODOT Types A, B, C, and D) to construct an unbound aggregate base. All the aggregates were processed by the contractors to satisfy the requirements of Oklahoma DOT specified Type A aggregate [24]. Table 1 presents the ODOT Type A gradation. As

this gradation contains a range of percent passing for each sieve size, the upper (UL) and lower limits (LL) are mentioned in Table 1.

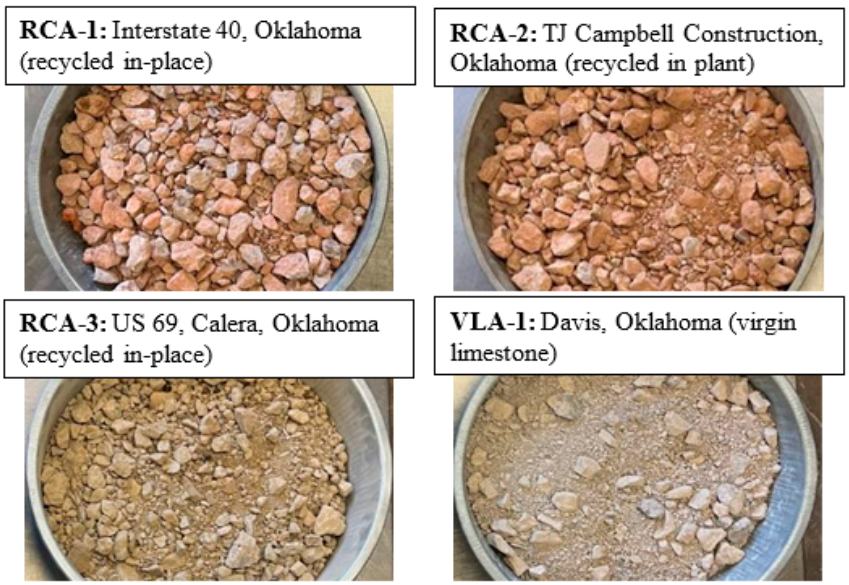


Figure 1. Recycled and virgin aggregates used in this study.



Figure 2. Overview of operation used to produce RCA from highway concrete pavement.

Table 1. Oklahoma DOT gradation Type A upper and lower limits.

Sieve Sizes (mm)	Upper Limit Type A (%Passing)	Lower Limit Type A (%Passing)
19	100	40
9.5	75	30
4.75	60	25
2	43	20
0.425	26	8
0.075	12	4

Figure 3 presents the initial as-received gradations of RCA-1, RCA-2, RCA-3, and VLA-1 aggregates. Also, the upper and lower limits of the ODOT Type A aggregate are shown in Figure 3. It was observed that the VLA-1 aggregate had an initial gradation that fell around the middle of the upper and lower limits of Type A aggregate base. Comparatively, the RCA-1 aggregate went beyond or outside of the lower limit gradation for sieve sizes 4.75 mm and 2 mm, and RCA-3 followed the lower limit gradation for sieve sizes 2 mm, 0.425 mm, and 0.075 mm. These differences in the aggregate gradations likely resulted from the breaking mechanisms used to produce these aggregates, since the type of crusher used can affect the gradation and quality of the RCA produced [25]. The literature indicated significant variations in the strength of aggregates due to differences in gradation [26]. In order to reduce the variability from aggregate gradations, aggregate blends

similar to the upper limit (UL) and lower limit (LL) of ODOT Type A gradations were prepared for evaluation.

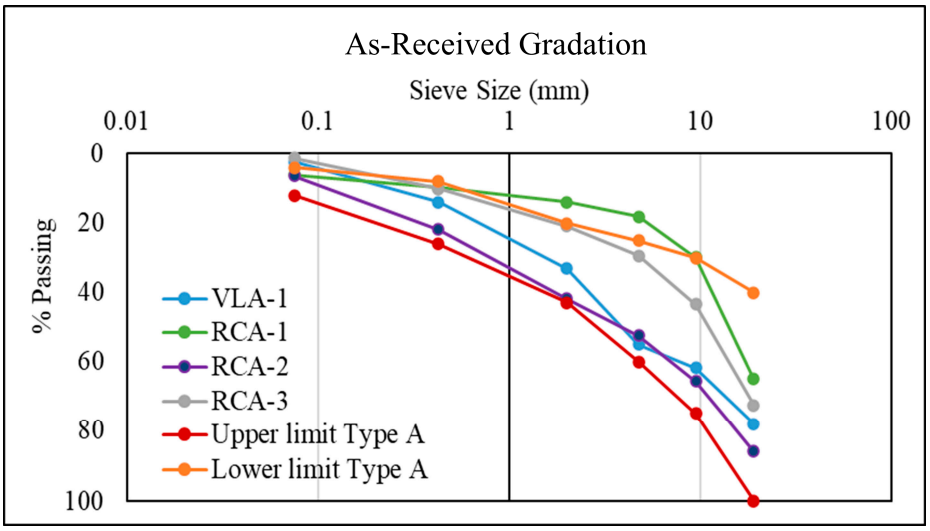


Figure 3. As-received gradations of RCA-1, RCA-2, RCA-3, and VLA-1 aggregates.

4. Methods

Laboratory tests in this study included wash loss, optimum moisture content (OMC) and maximum dry density (MDD), Los Angeles (LA) abrasion test, coarse and fine durability index (D_c and D_f), permeability (k), and resilient modulus (M_r). Additionally, the performances of the aggregate bases (RCA-1, RCA-2, RCA-3, and VLA-1) were studied through mechanistic analysis using the AASHTOWare Pavement ME Design software (version 2.6, AASHTO, USA). The workflow diagram for this study is presented in Figure 4. A brief description of the tests used in this study is presented in the following section.

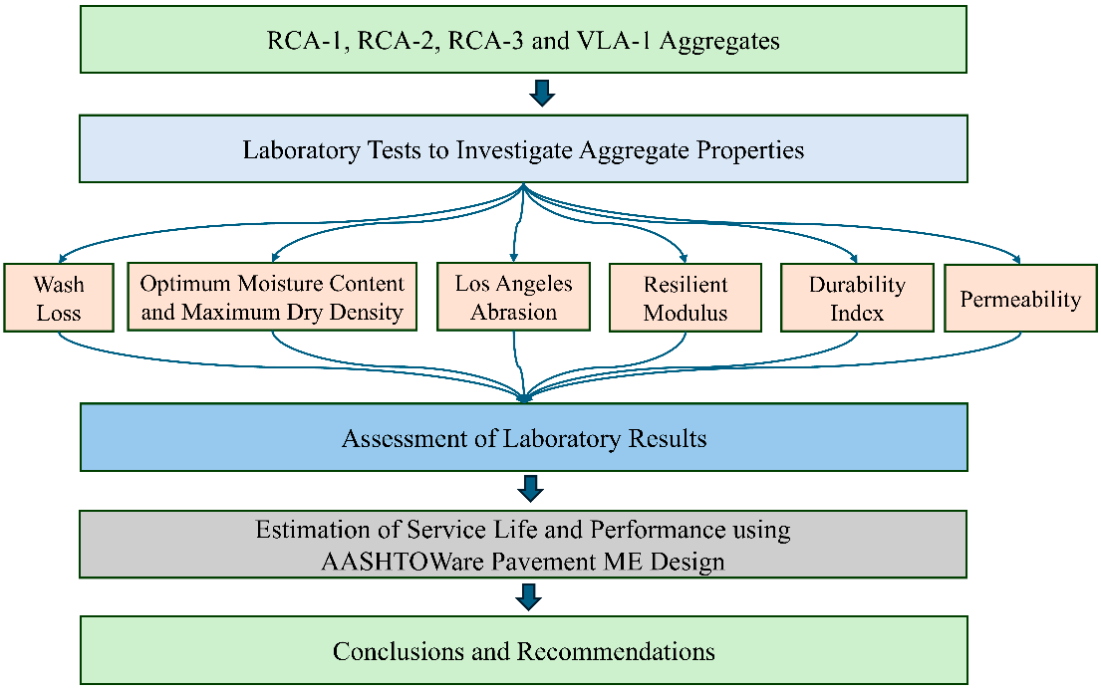


Figure 4. Workflow diagram of the current study.

4.1. Los Angeles (LA) Abrasion Test

The LA abrasion test is widely used to measure resistance to abrasion of virgin and recycled aggregates, providing an indicator of the relative quality of the aggregate. The test was conducted using the AASHTO T 96 [27] test method. The test involved placing a specific number of steel balls inside a rotating drum that rotates at a speed of 30–33 revolutions per minute. For the ODOT Type A gradation, the test required 11 steel spheres. The samples consisted of 5000 g of material, with 2500 g retained between sieve sizes 19 mm and 12.5 mm, and another 2500 g retained between sieve sizes 12.5 mm and 9.5 mm. The drum rotated for 500 cycles, and the material retained on the 1.70 mm sieve was washed, weighed, and compared to the initial weight. The percentage loss was then calculated as a measure of durability loss for the aggregate. In this study, in order to evaluate different levels of durability loss, the LA abrasion tests were conducted at 100, 300, and 500 cycles. According to Oklahoma DOT specifications [24], a maximum loss in the LA abrasion test of 50% is required after 500 cycles.

4.2. Durability Index

The AASHTO T 210 [28] test method was used to measure the durability of the fine (D_f) and coarse (D_c) fractions of aggregates. This test provides a rapid evaluation of aggregate quality by determining the resistance to generating claylike particles in the presence of water through mechanical degradation [29]. For the coarser fraction, the test was conducted on the portion of the material larger than the 4.75 mm sieve following the AASHTO T 210 [28] test method. The necessary equipment included a mechanical washing vessel, collection pan, mechanical agitator, calcium chloride solution, and distilled water (Figure 5). For the initial preparation, the coarser part of the aggregate, containing a set quantity of materials retained in sieve sizes of 12.5 mm, 9.5 mm, and 4.75 mm, was washed after two minutes in the agitator and dried for testing. The mechanical agitator was then used to degrade the aggregate particles to generate claylike fines (smaller than the 0.075 mm sieve) with a lateral reciprocating motion for 10 min with distilled water. The water with fines was then collected in a collection pan. A representative portion of the collected water was then mixed with a stock calcium chloride solution and placed in a graduated cylinder. After a 20 min sedimentation time, the level of the sediment column was measured, which was then used to calculate D_c.



(**a**) (**b**) (**c**)

Figure 5. Durability index test for coarse aggregate: (**a**) degradation using mechanical agitator; (**b**) initial setup; (**c**) final stage of the reading.

For the durability of the finer fraction of the aggregate (D_f), sample preparation was carried out in accordance with the AASHTO T 210 [29] test method. For this purpose, the aggregates passing a Number 4 sieve were thoroughly washed. After the sample was prepared, a mechanical sand shaker (Figure 6) was used, following the AASHTO T 176 [30] test method to generate claylike fines. The sample was then placed in a cylindrical tube and working solution was added. The heights of the clay and sand were measured to calculate D_f using Equation (1). According to ODOT specifications [24], a minimum of 40 for D_c and D_f is required to ensure that the material meets the required standard for use as an aggregate base while protecting its structural integrity.

$$D_f = \frac{\text{sand reading}}{\text{clay reading}} * 100 \quad (1)$$



(**a**) (**b**) (**c**)

Figure 6. Durability index test for fine aggregate: (**a**) sand equivalent shaker; (**b**) working solution; (**c**) cylindrical tube with sample.

4.3. Optimum Moisture Content (OMC) and Maximum Dry Density (MDD)

The stability of an aggregate base depends significantly on its density. Density relates to the permeability, specific gravity, unit weight, degree of compaction, and water content of the layer [31]. The moisture–density relationship determines the optimum moisture content (OMC) and maximum dry density (MDD) of aggregates. In this study, the modified Proctor test, following the AASHTO T 180 [32] test method, was used to determine the moisture–density relationship of aggregates for the upper and lower limit gradations using a manual compactor.

4.4. Permeability Test

The permeability (k) of the aggregates was determined in the laboratory using the falling head permeability test with a permeameter as described in ASTM D5084 [33]. The permeameter apparatus primarily included a steel cylinder measuring 159 mm in diameter and 165 mm in height, a base mold, a standpipe with a graduated scale, a pressure gauge, air and water connections, and a reservoir tank (Figure 7). To complete the assembly, the setup also required two porous cylindrical stones (150 mm in diameter and 12.5 mm thick), rubber gaskets, hose clamps, and a stopwatch. For the permeability test, specimens were compacted using the modified Proctor method at OMC. The compacted specimen was placed on a porous stone and the permeability mold was positioned around it. The mold contained a thick 0.6 mm rubber membrane held in place by o-rings, providing an air-tight seal between the membrane and the mold. A rubber gasket was used on the bottom mold to eliminate the possibility of water leakage. A second porous stone was placed on the top of the specimen. Then, a standpipe was connected to the specimen to monitor water flow

and head loss. The test used a flexible wall permeameter with a confining air pressure of 69 kPa to avoid short-circuiting the drainage path. This pressure pressed the membrane firmly against the aggregate particles, promoting full contact and limiting short-circuiting flow around the specimen. After sealing the entire apparatus and making sure that there was no leakage, the aggregate specimen was fully saturated with water. The standpipe was filled with water up to a predefined level. The time required to drop the water levels was recorded along with the temperature. The permeability of the sample was then calculated using the equation developed by Fwa et al. [34] (Equation (2)). Temperature correction was performed to calculate the permeability at a temperature of 20 °C using Equation (3).

$$v = k * i^n \tag{2}$$

where

v = specific discharge velocity in ft/day (cm/s);
k = coefficient of permeability in ft/day (cm/s);
n = experimental coefficient (unitless).

$$k_{20\ ^\circ C} = \left(\frac{\eta_{T^\circ C}}{\eta_{20^\circ C}}\right) k_{T^\circ C} \tag{3}$$

where

$k_{T^\circ C}$ and $k_{20^\circ C}$ = coefficient of permeability at T °C and 20 °C, respectively;
$\eta_{T^\circ C}$ and $\eta_{20^\circ C}$ = viscosity (N·m^{-2}·s) of water at temperatures T °C and 20 °C, respectively.

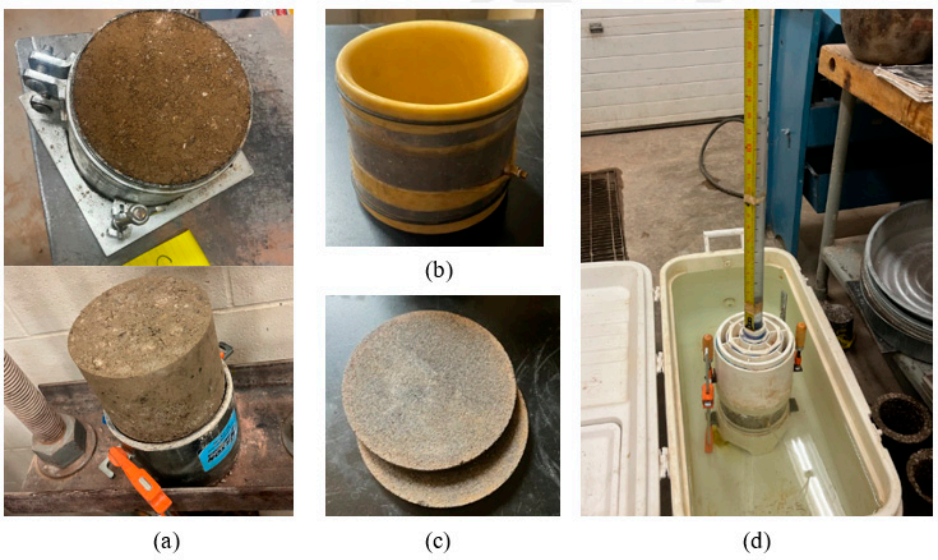


Figure 7. Permeability test: (**a**) compacted samples and extrusion; (**b**) rubber membrane with o-rings; (**c**) porous stones; and (**d**) permeability setup.

4.5. Resilient Modulus Test

The resilient modulus tests were conducted following the AASHTO T 307 [35] test method. For this purpose, specimens with a diameter of 150 mm and a height of 300 mm were compacted using an automatic mechanical compactor with cylindrical split steel mold. After compaction, the cylindrical specimen was placed on a loading platform and the mold was carefully extruded. A rubber membrane was placed over the sample to avoid the collapse of the specimen. Filter papers and 12.5 mm porous stones were placed on the top and bottom of the specimen and sealed with o-rings. The triaxial cell was placed over the specimen and sealed. In this test, compacted specimens were subjected to cyclic

loads (haversine-shaped) and static confining pressure in the triaxial chamber. A loading duration of 0.1 s followed by a resting period of 0.9 s was used for all resilient modulus testing using a servo-controlled hydraulic actuator (MTS). Two Linear Variable Differential Transducers (LVDTs) were mounted externally to measure the recoverable deformation, as seen in Figure 8. The test was performed following the loading sequences mentioned in the AASHTO T 307 [35] test method for base and subbase materials.



Figure 8. Setup for resilient modulus test.

Previous studies have used various models to correlate the resilient modulus of aggregate bases with stress levels. These models have employed different independent variables, such as confining pressure (σ_3), deviatoric stress (σ_d), bulk stress (θ), and octahedral stress (τ_{oct}) [26]. One of the widely used models for unbound aggregate bases is the bulk stress (θ) model, presented in Equation (2). The bulk stress model represents the resilient modulus (M_r) of aggregate bases as a function of θ.

$$M_r = k_1\theta^{k_2} \quad (4)$$

where

M_r = resilient modulus;
k_1, k_2 = parameters for the model;
θ = bulk stress = $\sigma_1 + \sigma_2 + \sigma_3$;
σ_1 = major principal stress;
σ_2 = intermediate principal stress = confining stress;
σ_3 = minor principal stress = confining stress.

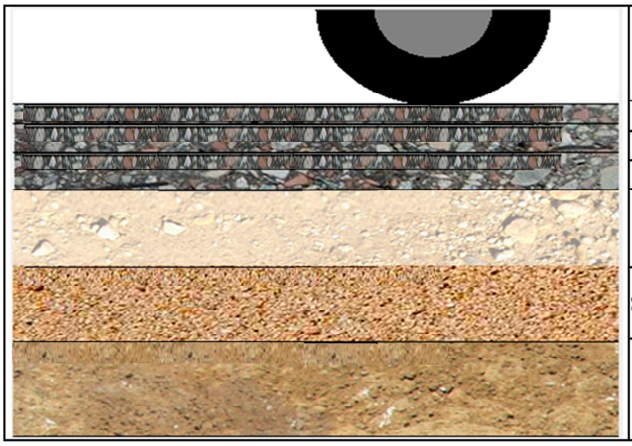
This study utilized the model to represent the resilient modulus of recycled and virgin aggregates. In this study, a θ value of 103.5 kPa (σ_2 = σ_3 = 20.7 kPa, and σ_1 = 41.4 kPa) was used to calculate the design M_r values for aggregate bases, as recommended in the AASHTO 1993 design [36].

4.6. AASHTOWare Pavement ME Simulations

AASHTOWare Pavement ME software (version 2.6, AASHTO, USA) is used to design pavement using a mechanistic-empirical design approach. In this study, the performances of a flexible (SH-48) and a rigid (SH-33) pavement in terms of pavement distresses were evaluated using AASHTOWare Pavement ME software (version 2.6, AASHTO, USA) based on traffic, climate, and material properties determined through laboratory testing.

The pavement section of SH-48 consists of three asphalt layers totaling 225 mm. Below the asphalt layer is a 200 mm granular aggregate base followed by a 200 mm thick stabilized

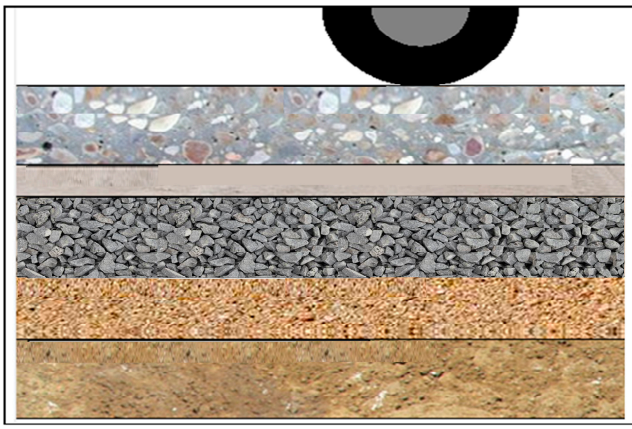
subgrade. A cross-section of this pavement section and associated properties are shown in Figure 9. The following distress thresholds were considered in the simulation: total rutting (12.5 mm), top-down fatigue cracking (20% lane area), and bottom-up fatigue cracking (20% lane area) [37].



	Thickness (mm)	Resilient Modulus (kPa)	Gradation
Layer 1 Asphalt	50	Default	S4
Layer 2 Asphalt	75	Default	S3
Layer 3 Asphalt	100	Default	S3
Aggregate Base	200	Varied	ODOT Type A
Stabilized Subgrade	200	68,950	A-2-7
Subgrade	Semi-infinite	55,160	A-7-6

Figure 9. Pavement section of SH-48 (flexible pavement).

The pavement section on SH-33 was a rigid pavement having a thickness of 250 mm (top layer). This layer of Portland cement concrete (PCC) was supported by a 100 mm thick cement stabilized base, a 200 mm thick granular base, and a 200 mm thick stabilized subgrade. A cross-section of this pavement section and associated properties are shown in Figure 10. The following distress thresholds were considered in the evaluation of the pavement: terminal IRI (3100 mm/km), mean joint faulting (5 mm), and transverse cracking (15% slabs) [37].



	Thickness (mm)	Resilient Modulus (kPa)	Gradation
PCC	250	2.9E7	-
Cement Stabilized Base	100	1.4E7	-
Aggregate Base	200	Varied	ODOT Type A
Stabilized Subgrade	200	68,950	A-2-7
Subgrade	Semi-infinite	55,160	A-7-6

Figure 10. Pavement section of SH-33 (rigid pavement).

The aggregate bases were modeled with different aggregate base options (RCA-1, RCA-2, RCA-3, and VLA-1) for both flexible and rigid pavement sections. The properties of the upper limit gradation of the aggregates were used for this evaluation. The analyses were conducted by varying the M_r values to assess the performance of the aggregate bases. The M_r values used for the simulation were obtained from laboratory testing. The design M_r values based on a θ value of 103.5 kPa ($\sigma_2 = \sigma_3 = 20.7$ kPa, and $\sigma_1 = 41.4$ kPa) were calculated and incorporated in the Pavement ME software (version 2.6, AASHTO, USA) as an input. A design life of 20 years, annual average daily traffic (AADT) of 4000, and the same climate conditions were considered in the simulations. Also, the

resilient moduli of stabilized subgrade and natural subgrade were assumed to be 68,950 and 55,160 kPa, respectively.

5. Results and Discussion

5.1. Wash Loss and Contaminants of Different Aggregates

Recycled concrete aggregates often contain foreign materials or contaminants. Fine particles can adhere to the surface of the aggregates. Wash loss was performed to determine the quantities of fines present in the different-size fractions of the recycled and virgin aggregates. The results are summarized in Table 2. It is seen that the VLA-1 and RCA-3 aggregates show relatively less wash losses, indicating good quality aggregates regarding wash loss. However, the RCA-1 and RCA-2 aggregates had significantly higher wash losses. The RCA-1 aggregate had clay clumps, indicating possible contamination with the underlying soil.

Table 2. Percent wash loss of different aggregates.

Size	%Loss Due to Washing with Water			
	VLA-1	RCA-1	RCA-2	RCA-3
¾″	0.64	1.45	1.63	0.69
3/8″	0.50	1.29	1.51	0.15
No. 4	0.94	3.12	3.56	0.93
No. 10	1.23	4.86	7.30	1.68

In addition, contaminants or foreign particles were found in all RCAs at different levels. RCA-1 had some twigs and mostly clay clumps (Figure 11). The RCA-2 aggregate produced in a recycling plant had numerous clay clumps, bricks, tiles, and other contaminants that could not be removed through washing. Comparatively, VLA-1 and RCA-3 showed significantly less contamination. Therefore, RCAs produced in recycling plants, like RCA-2, may negatively affect the quality of the material. However, high-quality aggregate is expected from known sources like highways and airfield pavements [23].

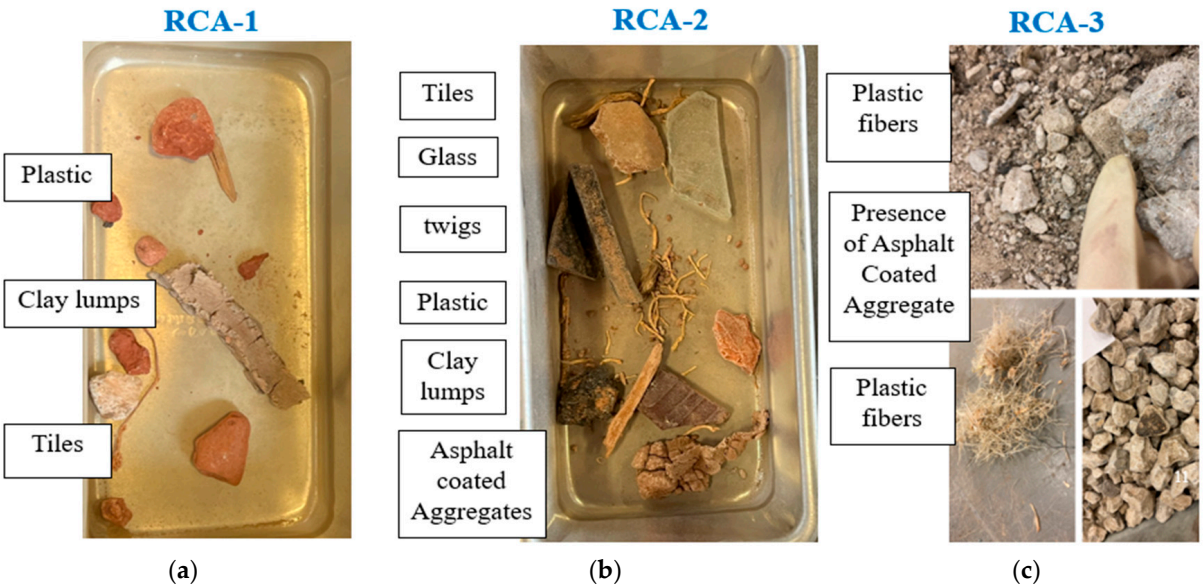


Figure 11. Contaminants present in (**a**) RCA-1, (**b**) RCA-2, and (**c**) RCA-3 aggregates.

5.2. Los Angeles (LA) Abrasion Test

The losses of aggregates in the LA abrasion machine subjected to 100, 300, and 500 cycles are presented in Figure 12. From Figure 12, RCA-3 and VLA-1 exhibited similar

losses in the LA abrasion for all the aforementioned cycles. At 300 and 500 cycles, RCA-1 and RCA-2 had significantly higher losses than RCA-3 and VLA-1. The presence of clay clumps and contaminants found in RCA-1 and RCA-2 are thought to be responsible for the higher values of abrasion losses. The RCA-3 aggregate is expected to have improved behavior because of the high quality (fewer foreign particles) of RCA obtained from state highway pavements. The low abrasion loss of VLA-1 and RCA-3 indicates that these aggregates will have higher resistance to degradation caused by abrasion. The study conducted by Arulrajah et al. [8] also found that RCA aggregates satisfied the allowable limits of the LA abrasion test set by the state agency. Therefore, aggregates produced in-place exhibiting qualities similar to RCA-3 are expected to perform well as a pavement base.

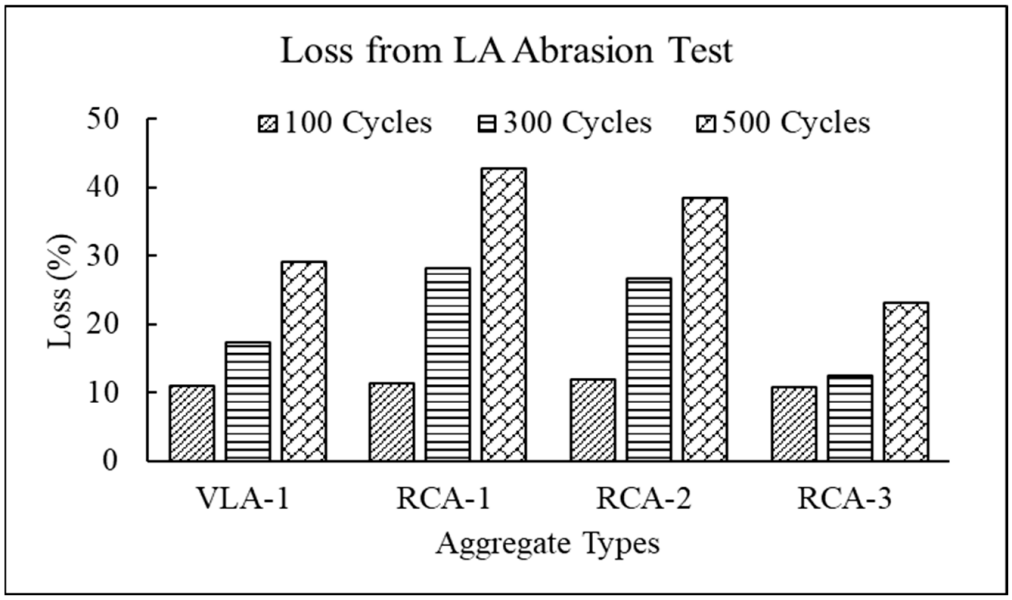


Figure 12. Abrasion loss of different aggregates.

5.3. Durability Index Test

Figure 13a presents the durability index of the coarser part (D_c) of the aggregates. It was observed that the D_c values of all aggregates met the minimum Oklahoma DOT requirement of 40. It was observed that the coarser part mostly contained crushed concrete clumps, which were expected to perform well. Furthermore, the D_c values of RCA-2 and RCA-3 were comparable to that of VLA-1. Also, the D_c value of RCA-3 was higher than that of VLA-1, suggesting a greater resistance to durability issues. In contrast, RCA-1 had the lowest value of D_c, suggesting a lower resistance to durability issues than the virgin aggregate.

Figure 13b,c show the durability indices for upper and lower limits of fine aggregate. From Figure 13b,c, it was observed that the durability indices of RCA-1 (34 for the UL and 29 for the LL) and RCA-2 (26 for the UL and 23 for the LL) did not satisfy the ODOT minimum requirement of 40. However, VLA-1 and RCA-3 met the requirements. During testing, it was observed that both RCA-1 and RCA-2 contained clay clumps and other contaminants. Controlling the amount of these contaminants is essential to maintaining the quality of RCA bases. From the aforementioned results, it is evident that the fine aggregate durability index test could be used as a screening tool to ensure the quality of recycled aggregate.

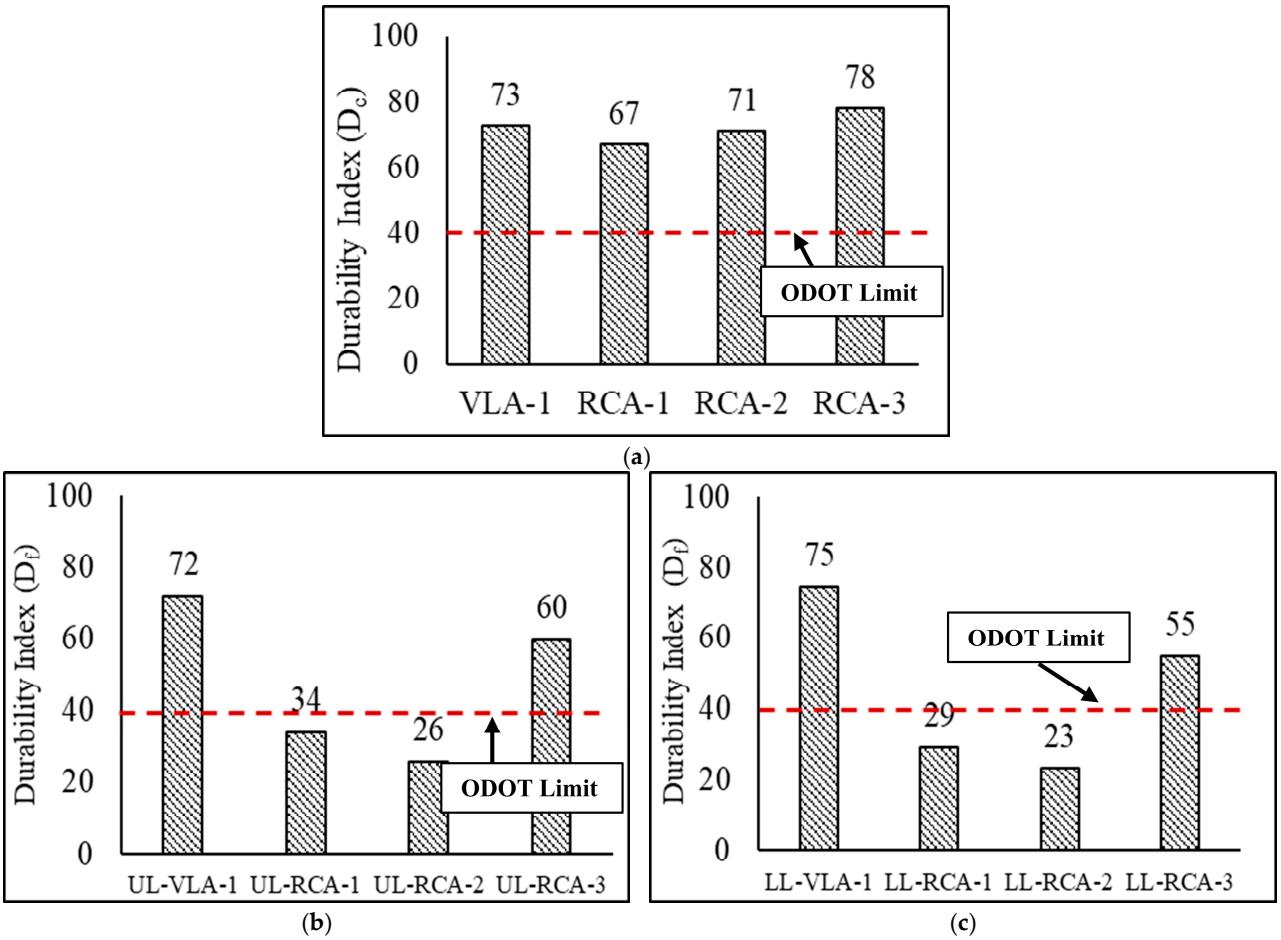


Figure 13. Durability indices for (**a**) coarse aggregate, (**b**) upper limit of fine aggregate, and (**c**) lower limit of fine aggregate.

5.4. Optimum Moisture Content (OMC) and Maximum Dry Density (MDD)

The OMCs and MDDs of different aggregates are presented in Figure 14a,b, respectively. It was observed that RCAs had higher OMCs than the virgin aggregate. For instance, for the upper limit, RCA-1, RCA-2, and RCA-3 had OMC values of 6.3%, 8.7%, and 10%, respectively, while VLA-1 had an OMC of 5.1%. For the lower limit, RCA-1, RCA-2, and RCA-3 had OMC values of 5.6%, 7.2%, and 8.3%, respectively, while VLA-1 had an OMC of 2.7%. Also, the MDDs of all RCAs were lower than those of the virgin aggregate (VLA-1). The MDD of the VLA-1 aggregate was 2313 kg/m^3 for the upper limit, whereas that of RCA-3 was 1943 kg/m^3 for the same gradation. The RCAs used in this study were produced by crushing existing concrete structures. The coarser part of the aggregate is primarily composed of aggregate with attached cement mortar, which can absorb more water and increase the water demand. Also, these concrete coated aggregates are porous, allowing more water to penetrate while reducing their density. Other researchers have found a similar trend where the OMC of RCA is higher, and the MDD is lower, than that of virgin aggregates [6,7,14]. Gabr & Cameron [14] studied the characteristics of two RCAs obtained from southern Australia. Both RCAs exhibited significantly higher OMC than the control quartize aggregates. Also, the MDD values of the RCAs were found to be at least 10% lower than those of the control aggregate.

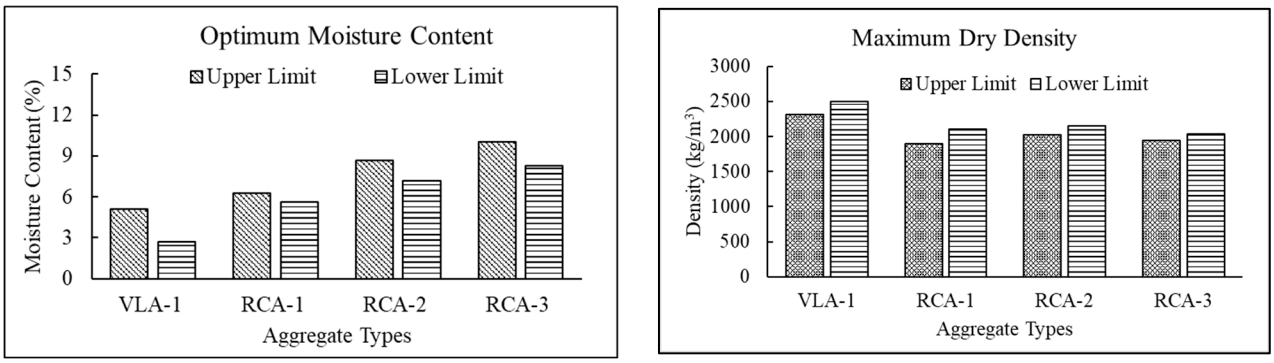


Figure 14. (**a**) Optimum moisture content and (**b**) maximum dry density of different aggregates.

5.5. Resilient Modulus (M_r) of Aggregates

Table 3 presents the design resilient modulus values of all aggregates based on Equation (4). From Table 3, the design resilient modulus values for the VLA-1 aggregate were found to vary between 126,519 kPa and 77,221 kPa for the upper and lower limit gradations, respectively, with an average of 101,870 kPa. The average resilient modulus for RCA-1 was 124,712 kPa (159,958 kPa for UL to 89,460 kPa for LL), for RCA-2 was 81,358 kPa (88,253 kPa for UL to 74,463 kPa for LL), and for RCA-3 was 98,719 kPa (104,856 kPa for UL to 92,576 kPa for LL). From Table 3, the RCA-1 aggregate exhibited higher M_r values than the corresponding values for the VLA-1 aggregate for the upper and lower limit gradation. Also, the RCA-2 aggregate exhibited a similar average M_r value as the VLA-1 aggregates. Based on these results, RCA-1 and RCA-3 are expected to perform similarly to or better than the VLA-1 aggregate as pavement bases. Regarding RCA-2, the M_r values were found to be lower than those of the other aggregates. The reduced strength of RCA-2 was believed to be caused by the inferior quality of the source materials. The high M_r values observed for RCA-1 and RCA-3 likely resulted from the proper control of quality during the construction of the highway pavement section. Both of these aggregates (RCA-1 and RCA-3) were produced on-site from existing highway pavements with high-quality materials, producing high-quality recycled aggregates. Gabr & Cameron [14] reported that the resilient modulus of RCA aggregates was greater than that of a local quartzite used as virgin aggregate for pavement base. Other researchers have reported M_r of recycled aggregates to be higher than or comparable to that of a virgin aggregate [8,13].

Table 3. Design resilient modulus of different aggregates.

Aggregate Type	Gradation	Limit	K_1	K_2	Design Resilient Modulus (kPa)	Average M_r
VLA-1	ODOT Type A	Upper	21,701	0.38	126,519	101,870
		Lower	10,027	0.44	77,221	
RCA-1		Upper	20,770	0.44	159,958	124,712
		Lower	12,745	0.42	89,460	
RCA-2		Upper	18,224	0.34	88,253	81,358
		Lower	11,640	0.40	74,463	
RCA-3		Upper	20,671	0.35	104,856	98,719
		Lower	13,189	0.42	92,576	

5.6. Permeability of Aggregates

Figure 15a,b present the permeability (k) results of all aggregates for the upper and lower limit gradations, respectively. The k value of the aggregates for the upper and lower limit gradations showed no specific trend. However, the values were similar in magnitude. The reason for this behavior can be attributed to the fact that aggregate gradation remained consistent (at the UL and LL) during the testing. It was reported that a less permeable but more stable base layer may have a permeability of 15,250 to 24,400 cm/day [25]. All the aggregate bases exhibited relatively low permeability values, indicating possible problems with pavement drainage.

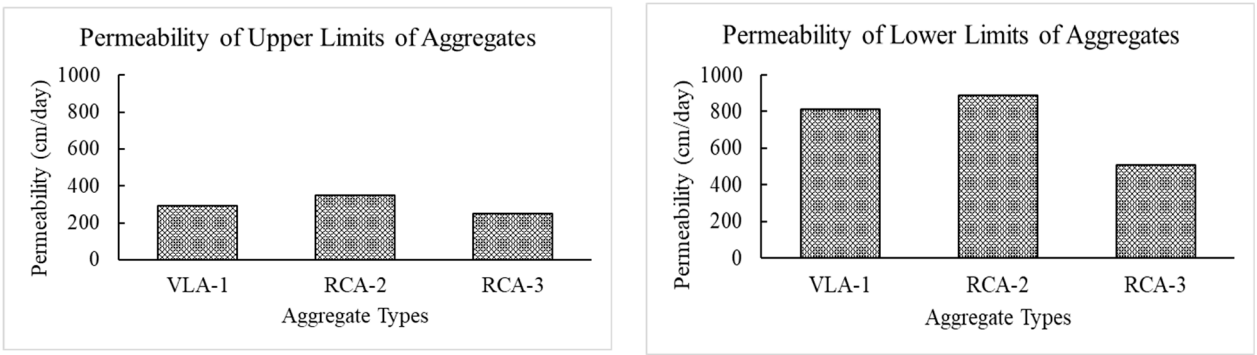


Figure 15. Permeability of (**a**) upper and (**b**) lower limits of different aggregates.

5.7. Assessment of Service Life Using AASHTOWare Pavement ME Simulations

As mentioned earlier, the performances of a flexible pavement (SH-48) and a rigid pavement (SH-33) with different aggregate bases were simulated using the AASHTOWare Pavement ME software (version 2.6, AASHTO, USA). Table 4 presents the performances of the flexible pavement section (SH-48) with different aggregate base alternatives. From Table 4, it was observed that the changes in aggregate types, as well as changes in resilient modulus value, did not affect pavement rutting. For all aggregate types, the total pavement rutting was found to vary between 10.7 and 10.9 mm. Therefore, for a pavement structure similar to SH-48, rutting performance may not be affected by changes in design resilient modulus from 88,253 kPa (RCA-2) to 159,958 kPa (RCA-1). The 225 mm thick asphalt layers on top of the aggregate base may be contributing to the improved rutting performance of SH-48.

Table 4. Effect of aggregate types on the performance of flexible pavement (SH-48).

Aggregate Base	Base Thickness (mm)	Design Resilient Modulus (kPa)	Total Pavement Rutting (mm)	Top-Down Fatigue Cracking (% Lane area)	Bottom-Up Fatigue Cracking (% Lane Area)
VLA-1 (UL)	200	126,519	10.7	14.19	18.57
RCA-1 (UL)	200	159,958	10.7	14.14	10.08
RCA-2 (UL)	200	88,253	10.9	14.28	31.90
RCA-2 (UL)	250	88,253	10.9	14.22	27.66
RCA-2 (UL)	300	88,253	10.9	14.22	23.93
RCA-2 (UL)	350	88,253	10.7	14.26	20.14
RCA-3 (UL)	200	104,856	10.7	14.23	18.72

The top-down fatigue cracking of the SH-48 pavement with 200 mm of RCA-1 (159,958 kPa), RCA-2 (88,253 kPa), RCA-3 (104,856 kPa), and VLA-1 (126,519 kPa) as a pavement base was found to be 14.14%, 14.28%, 14.23%, and 14.19% of the lane area,

respectively. This indicates that the changes in aggregate types (RCAs and VLA), as well as a resilient modulus between 88,253 kPa and 159,958 kPa, will not affect the top-down fatigue cracking performance of SH-48.

However, a significant difference was found in bottom-up fatigue cracking among different aggregate options. For a 200 mm aggregate base, RCA-1 exhibited the lowest bottom-up fatigue cracking value at 10.08%. Comparatively, for the same layer thickness, VLA-1, RCA-2, and RCA-3 had 18.57%, 31.90%, and 18.72% bottom-up fatigue cracking. The pavement section with RCA-1, RCA-3, and VLA-1 showed significantly lower values of bottom-up fatigue cracking than that with RCA-2. The low design resilient modulus of RCA-2 resulted in a weaker base layer and eventually led to higher bottom-up fatigue cracking. In order to improve the bottom-up fatigue cracking of RCA-2, the thickness of the aggregate base was increased to 350 mm using increments of 50 mm. It was expected that the added thickness would add to the strength of the base layer and reduce bottom-up fatigue cracking. For a 350 mm aggregate base, the bottom-up fatigue cracking of RCA-2 improved to 20.14%. Both RCA-1 and RCA-3 showed similar or better performance for all pavement distresses compared to VLA-1 for the same layer thickness. The results suggest that recycled aggregates can be viable for pavement base layers, provided their properties are carefully evaluated and appropriate thicknesses are used. The aggregate bases produced with RCA from on-site crushing operations had improved pavement distress resistance because of the better M_r values obtained from laboratory testing.

The performance of the SH-33 section is presented in Table 5. From Table 5, pavement distresses were unaffected by the changes in aggregate base thickness. The section performed exceptionally well in terms of pavement distresses. The concrete layer, which is much stronger and stiffer than other layers, controls the overall performance. However, the use of recycled aggregates for the construction of pavement base of concrete pavement may prove to be economically feasible as it will reduce material and hauling cost without compromising performance.

Table 5. Effect of aggregate type on the performance of rigid pavement (SH-33).

Aggregate Base	Base Thickness (mm)	Resilient Modulus (kPa)	Terminal IRI (mm/km)	Mean Joint Faulting (mm)	JPCP Transverse Cracking (% slabs)
VLA-1 (UL)	200	126,519	1800	1.5	0.96
RCA-1 (UL)	200	159,958	1798	1.5	0.96
RCA-2 (UL)	200	88,253	1801	1.5	0.96
RCA-3 (UL)	200	104,856	1800	1.5	0.96

6. Conclusions and Recommendations

This study generated useful data of recycled aggregate for possible use as an unbound aggregate base or subbase of a new or reconstructed pavement in Oklahoma. For this purpose, the properties of the RCAs (RCA-1, RCA-2, and RCA-3) and virgin aggregate (VLA-1) were evaluated in terms of wash loss, moisture–density relationships, LA abrasion loss, durability indices, permeability, and resilient modulus. Additionally, the properties obtained through laboratory testing were used to assess the performance of aggregate bases for a flexible pavement section (SH-48) and a rigid pavement (SH-33) section using AASHTOWare Pavement ME simulations. The following conclusions could be drawn from the results presented in the preceding sections:

- The properties of RCAs can differ greatly based on the origin of the source materials and the methods used in their processing, which may influence their appropriateness for pavement applications. To ensure consistent quality, agencies may require

contractors to obtain RCAs from approved sources. Additionally, standardization of production processes, including crushing, blending, and transportation, is essential.

- Good-quality recycled aggregates from concrete pavements of interstate and state highways, similar to those used in this study, are expected to exhibit similar performance as virgin aggregates of comparable quality. Improved durability and strength properties are expected from these RCAs. Since the concrete used during the original pavement construction was of good quality and met specifications, the resulting RCAs were expected to meet the specification requirements, as demonstrated in this study.
- The results of this study showed that the RCAs obtained from a recycling plant exhibited the lowest durability and strength characteristics among all the aggregates tested. RCAs in a recycling plant are typically produced by crushing concrete and rubble from various sources, such as demolished buildings, driveways, and other structures. Due to the diverse origins of these materials, there is minimal control over their quality, which introduces significant variability in the properties of the resulting aggregate. As a result, the performance of pavement bases constructed with such materials can be unpredictable. Therefore, careful consideration should be given when using RCA from recycling plants, as it may contribute to long-term deterioration in pavement performance.
- A variety of foreign contaminants, such as twigs, plastic, and clay clumps, were found in the aggregates, particularly in RCAs sourced from recycling plants. These contaminants likely contributed to the durability and performance issues observed in RCAs. To address this, agencies might consider specifying allowable limits for contaminant content in RCAs. Also, it was evident from the current study that the fine aggregate durability test could be used as a screening tool to ensure the quality of recycled aggregate.
- Several methods have been identified in the literature to reduce or eliminate such impurities, including smart demolition and dismantling, screening, automated sensor-based sorting, advanced dry recovery, and the use of wind sifters [38]. Additionally, care must be taken during the hauling and transport of crushed concrete from demolished pavements to avoid contamination with subgrade soil.
- The use of RCA from different sources may have little impact on the rutting and top-down fatigue cracking performance of flexible pavement, provided that strong asphalt surface layers are present. However, if the aggregate has a low design resilient modulus, such as RCA-2 in this study, it can lead to significant bottom-up fatigue cracking. To mitigate this, a thicker base layer can be incorporated to compensate for the lower modulus and enhance resistance to bottom-up fatigue cracking failure.
- The influence of the aggregate type on the performance of rigid pavement was found to be insignificant due to the dominance of the thick concrete layer. In this case, RCA may be a valuable alternative for reducing costs.

Typically, RCAs are produced close to projects, reducing hauling costs, and are less costly than virgin aggregates. In cases where the virgin aggregates are far away from the project, the RCAs can provide a cost-effective and sustainable solution for pavement construction. Proper specifications should be developed and used by state DOTs to ensure the performance of pavement constructed with recycled aggregate.

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