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Practical approach for designing flexible pavements using recycled roadway materials as base course

Ali Ebrahimi*, Brian R. Kootstra, Tuncer B. Edil and Craig H. Benson

University of Wisconsin-Madison, Madison, Wisconsin, USA

Resilient modulus and plastic deformation of two recycled base course materials, recycled pavement material (RPM) and road surface gravel (RSG) and natural aggregate (Class 5), were investigated using a large-scale model experiment (LSME) and laboratory bench-scale resilient modulus (BSRM) tests. The RPM and RSG were tested alone and with 10% by weight Class C fly ash. The LSME tests indicate that the summary resilient modulus (SRM) of the unbound base course materials increases with increasing thickness of the base course and RPM and RSG exhibit significantly higher rate of plastic deformation (i.e. three to four times) than Class 5 aggregate. Stabilisation of the recycled materials by fly ash reduces the required thickness of the base course up to 30% when designed in accordance with the AASHTO 1993 design guide. The SRM and plastic deformation from LSME tests were used in the Mechanistic-Empirical Pavement Design Guide (MEPDG) to predict the lifetime expectancy of a pavement with a base course consisting of recycled materials alone and with fly ash stabilisation. Stabilisation of recycled materials used as base course can reduce the required thickness of the base course up to 30% or increase the service life of pavements by more than 20%.

Keywords: pavement design; MEPDG; recycled roadway materials; base course

1. Introduction

Recycling existing pavement materials during rehabilitation and reconstruction of roads provides a more sustainable alternative to conventional methods such as full removal and replacement of the pavement materials. Existing deteriorated asphalt surface can be pulverised and mixed with the underlying materials to form a new recycled base layer known as recycled pavement material (RPM). The depth of pulverisation typically ranges from 100 to 300 mm and includes some or all of the existing base course and even part of the underlying subgrade (Epps, 1990). Similarly, when upgrading unpaved gravel roads to a roadway with a paved surface, the existing road surface gravel (RSG) can be recycled to form a base or sub-base.

In situ recycling of roadway materials is cost effective and environmentally friendly, resulting in reduced energy consumption, greenhouse gas emissions and waste material disposal (Lee, Edil, Tinjum, & Benson, 2010; Wen & Edil, 2009). However, the asphalt binder in RPM and fines in RSG may adversely affect the strength, stiffness, and plastic deformation of recycled materials used as base course (Cooley, 2005; Kim, Labuz, & Dai, 2007; Kootstra, Ebrahimi, Edil, & Benson, 2010; Mohammad, Herath, Rasoulian, & Zhongjie, 2006; Taha, 2003). One method to enhance the performance of these recycled roadway materials is chemical stabilisation with binders like cement, asphalt emulsion, lime, cement kiln dust or fly ash.

*Corresponding author. Email: aebrahimi@geosyntec.com

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The behaviour of pavement materials stabilised with fly ash has been receiving increasing attention in recent years (Edil, Benson, Bin-Shafique, Tanyu, Kim, & Senol, 2002; Mohammad, Herath, Rasoulian, & Zhongjie 2006). Fly ash is a by-product of coal combustion at electric power plants and often has self-cementing properties. Adding Class-C or self-cementing high-carbon fly ash to RPM and RSG increases the California bearing ratio (CBR) and resilient modulus (Camargo, Edil, & Benson, 2009; Hatipoglu, Edil, & Benson, 2008; Wen & Edil, 2009). Field studies have also shown significant and persistent increases in the resilient modulus of fly ash-stabilised layers over several years of service (Bin-Shafigue, Edil, Benson, & Senol, 2004; Wen, Tharaniyil, Ramme, & Krebs, 2004).

Determining the appropriate thickness of the pavement layers based on engineering properties is a critical task in the design of pavements, and can be particularly challenging when alternative materials are used. The objective of this study was to develop a methodology to incorporate RPM and RSG as base course (alone and with fly ash stabilisation) in pavement design. Mechanical behaviour of the materials was characterised through a large-scale model experiment (LSME) as well as laboratory bench-scale resilient modulus (BSRM) tests in accordance with NCHRP 1-28a. Data from the BSRM test were compared to those from the LSME to account for the effects of the test conditions and scale on resilient modulus. Resilient moduli and plastic deformations obtained from the LSME were used to develop a methodology for designing pavements with these materials. Two design methods using the AASHTO 1993 and NCHRP (2006) (Mechanistic-Empirical Pavement Design Guide; MEPDG) were considered.

2. Materials

Recycled RPM and RSG used alone and with fly ash stabilisation were evaluated as alternatives to conventional crushed aggregate base. Limestone aggregate, meeting the Minnesota Department of Transportation's (MnDOT) gradation specification for Class 5 base course, was selected as a reference base course aggregate (referred to herein as Class 5 aggregate). The RPM was an equal mixture of pulverised hot mix asphalt and limestone base course from a roadway reconstruction project in Madison, Wisconsin. Asphalt-coated aggregates in the RPM were mostly limestone and dolomite (based on X-ray diffraction) and were coated with 0.1 to 3 mm of asphalt binder (Figure 1). The RPM had an asphalt content of 4.7% (ASTM D6307). Actual RSG was not available. Thus, a synthetic RSG was created by combining Class 5 base with clay fines to meet

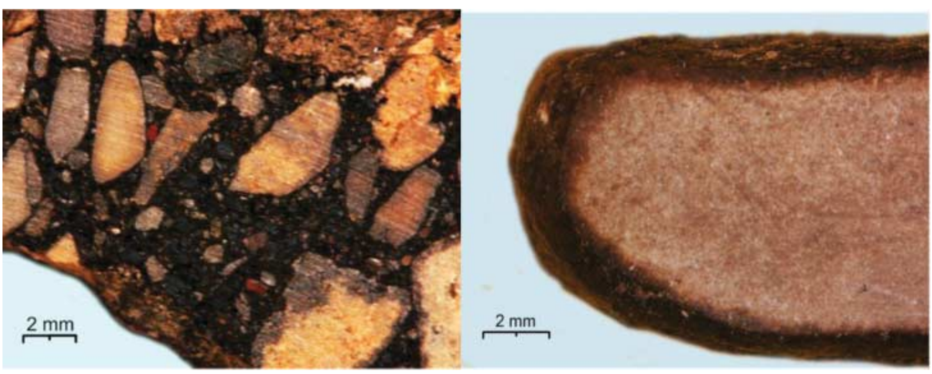


Figure 1. Cross section of RPM particles coated with asphalt binder.

the gradation and plasticity requirements for surface course materials as described in AASHTO M 147. Particle size distributions of the Class 5, RPM, and RSG are given in Figure 2.

Fly ash was obtained from Unit 2 of Columbia Power Station (Alliant Energy) in Portage, Wisconsin. Columbia fly ash has self-cementing properties and classifies as Class C according to ASTM C 618 (Table 1). RPM and RSG alone and with 10% by weight fly ash (called 'SRPM' and 'SRSG', respectively) were tested as stabilised recycled base course materials. Index properties, classification, and compaction characteristics of these base materials are in Table 2.

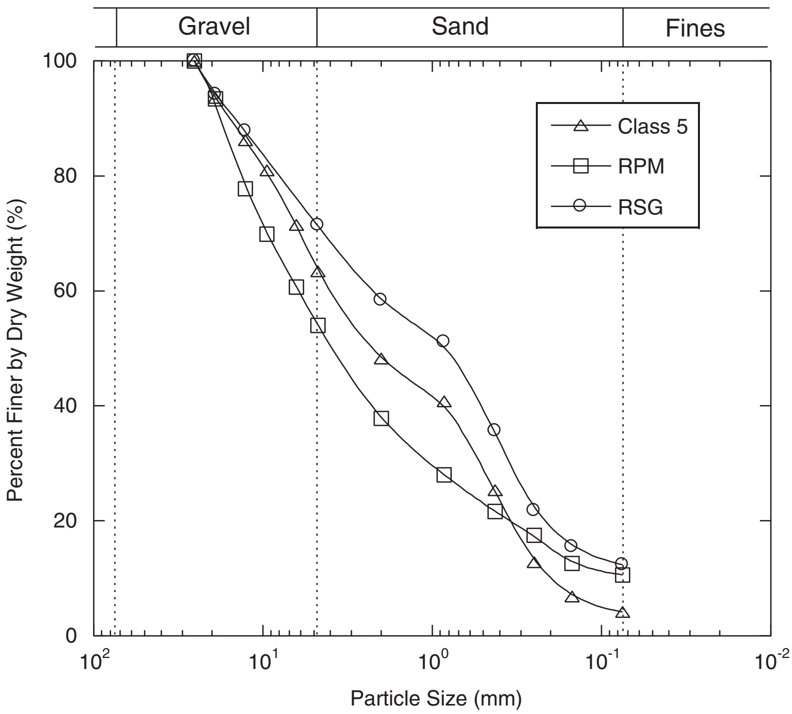


Figure 2. Particle size distributions of Class 5 base, RPM, and RSG

Table 1. Chemical composition of Columbia fly ash and typical Class-C fly ash.

Parameter	Columbia fly ash	Typical Class-C fly ash ASTM C618
SiO_2, %	31.1	40
Al_2O_3, %	18.3	17
Fe_2O_3, %	6.1	6
$SiO_2 + Al_2O_3 + Fe_2O_3$, %	55.5	63
CaO, %	23.3	24
MgO, %	3.7	2
SO_3, %	–	3
CaO/SiO_2	0.8	0.6
$CaO/(SiO_2 + Al_2O_3)$	0.4	0.4
Loss on ignition, LOI, %	0.7	6

Table 2. Index properties of base course materials used in study.

Material	Optimum water content w_{opt} (%)	Max unit weight $\gamma_{d\ max}$ (kN/m^3)	Liquid limit (LL) (%)	Plastic limit (PL) (%)	Fines content (%)	AASHTO (USCS)	Poisson ratio (ν)
Class 5	5.0	20.9	NP	NP	4	A-1-a (SP)	0.35*
RPM	7.5	21.2	NP	NP	11	A-1-a (GW-GM)	0.35*
RSG	7.5	22.6	21	14	12	A-2-4 (SC-SM)	0.32*
SRPM	8.5	20.4	–	–	–	–	0.2
SRSG	6.6	22.0	–	–	–	–	0.2

Notes: Particle size analysis by ASTM D422, $\gamma_{d\ max}$ and w_{opt} by ASTM D698, AASHTO classification by ASTM D3282, asphalt content by ASTM D6307 and Atterberg limits by ASTM D4318. *Data from Schuettpelz et al. (2010).

3. Methods

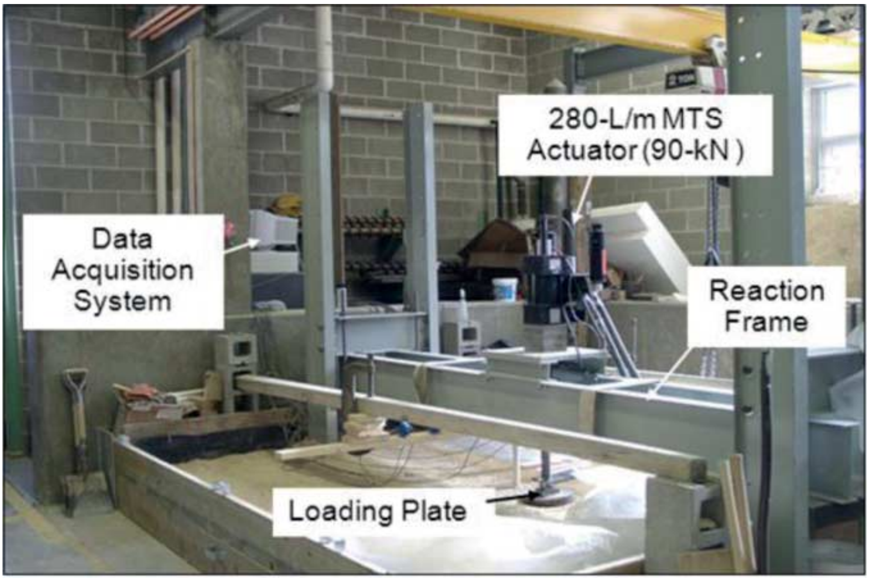
3.1. *Large-scale model experiment*

Elastic and plastic deformations of the alternative recycled base course materials were measured in the large-scale model experiment (LSME) (Figure 3). The LSME applies cyclic loading simulating truck traffic to a prototype pavement structure. The loads and deformations are used to determine the resilient modulus and plastic strain of the base course materials under conditions similar to the field. The LSME accounts for scale effects and strain amplitude due to varying layer thickness and accumulated plastic deformation (Tanyu, Kim, Edil, & Benson, 2003).

The LSME consists of a pavement profile in a 3 × 3 × 3 m test pit (Figure 3). The pavement profile consists of 2.5 m of uniform sand corresponding to a deep, relatively strong sub-grade and a base course layer. The LSME apparatus is located inside a building in an air-conditioned environment where temperatures averaged around 24°C. The impact of temperature on material behaviour was not considered in this study. The RPM, RSG and Class 5 aggregate were tested in two base course thicknesses (0.2 and 0.3 m) to account for the effect of strain amplitude on the resilient modulus and plastic deformations. Each material was compacted to 100% of standard Proctor maximum dry unit weight at optimum moisture content (Table 1) in 0.1 m lifts using a plate vibratory compactor. A nuclear density gauge was used to control the as-compacted dry unit weight to reach maximum dry unit weight. Fly ash-stabilised materials were tested only with 0.3 m depth corresponding to typical field conditions (Wen et al., 2004). For fly ash stabilisation, air-dried base material was mixed with 10% by weight of Class-C fly ash and then water was added to bring the mixture to optimum moisture content. The stabilised layers were placed in 0.15 m lifts and compacted to standard Proctor maximum dry unit weight within 1 h of adding water. It was allowed to cure in ambient conditions, in this case the laboratory conditions of room temperature and humidity similar to the field practice.

A loading frame (100 kN actuator with 165 mm stroke) and a steel loading plate (125 mm radius and 25 mm thickness) were used to apply cyclic loading to the surface of the base layer. The stress applied to the surface of the base course was obtained by conducting non-linear finite element simulations of a pavement profile similar to the one in the LSME, but with a 0.1 m thick HMA layer. The program MICHPAVE (Harichandran, Baladi, & Yeh, 1989) was used to simulate stress dependency of the base course modulus. The simulated pavement was subjected to traffic wheel loads corresponding to 4-axle trucks (70 kN per axle and 35 kN per wheel set) with a tyre pressure of 700 kPa.

The MICHPAVE analysis showed that the stress at the surface of the base course decreased to 144 kPa and was relatively uniform within the 125 mm radius of the loading plate. Thus, a load



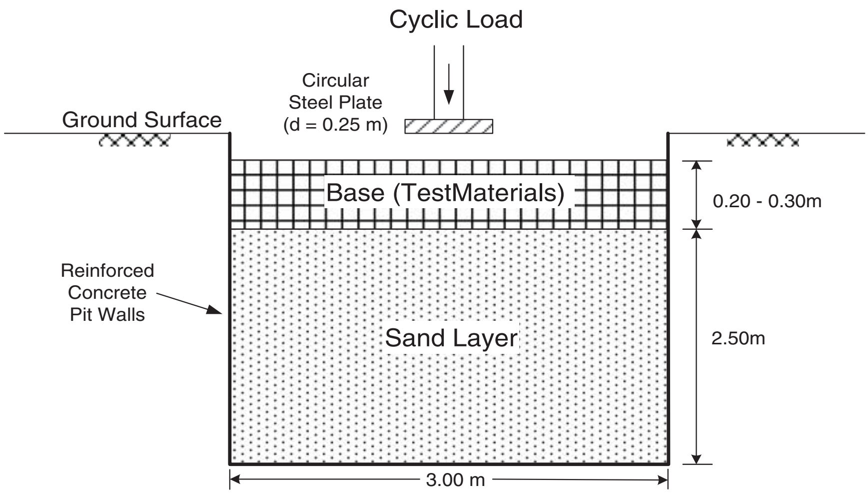


Figure 3. Photo of large-scale model experiment (LSME, top) and schematic of the LSME (bottom) used for prototype pavement testing (after Kim et al., 2005).

of 7 kN was applied to the steel loading plate so that the average stress for the plate was 144 kPa. The steel plate was to simulate the distributed pressure of a wheel on asphalt layer at the surface of the base course layer. This load was applied as a haversine pulse shape with a loading period of 0.1 s followed by a rest period of 0.9 s (NCHRP 1-28a).

Deflections at the surface of the base course layer and sub-grade were measured using six linear variable differential transformers (LVDTs) with 5 mm stroke. Four of the LVDTs measured the deflection at the surface of the base course, and two of them at the surface of the sub-grade. Total, elastic, and plastic deflections at top of the loading plate and the sub-grade were determined, and the difference designated as the deformation in the base course. The recoverable portion of the deflection during a loading pulse was designated as the elastic deflection. The difference between the total deflection and elastic deflection was designated as the plastic deflection. More details on the testing conditions can be found in Benson, Edil, Ebrahimi, Kootstra, Li, & Bloom (2009).

The state of stress in the base course was back-calculated using MICHPAVE. Resilient modulus of the base layer (M_r) was assumed to follow the nonlinear elastic power function model

$$M_r = k_1 \left(\frac{\sigma_b}{p_r}\right)^{k_2} \tag{1}$$

where σ_b is the bulk stress, p_r is a reference stress (1 kPa in this study), and k_1 and k_2 are empirical parameters. The parameter k_2 is dimensionless and represents the stress dependency of modulus. Typically k_2 falls in the range of 0.45 to 0.62 for granular base course materials (Huang, 2003). MICHPAVE was used to back-calculate the resilient modulus parameter k_1 of each base course material while keeping k_2 constant using the elastic deflection data for the base course recorded in the LSME. The parameter k_2 of each base course material was assumed to be constant in the LSME and set at the value obtained for the same material from the BSRM test. The parameter k_1 was varied until the deflection predicted by MICHPAVE matched the measured elastic deflection in the LSME following the procedure described by Tanyu et al. (2003). The underlying sand layer was assumed to be linear elastic with a modulus of 70 MPa. This inversion yields the resilient modulus as a function of bulk stress, σ_b, as well as the distribution of stress and strain within the pavement system. A summary resilient modulus (SRM) was computed, as suggested in NCHRP 1-28a, corresponding to a bulk stress of 208 kPa.

The average plastic strain (ε_p) in the base layer was defined as:

$$\varepsilon_p = \frac{d_p}{t} \times 100 \tag{2}$$

where d_p is the plastic deflection of base course (= plastic deflection at the surface of base course layer − plastic deformation at the surface of sub-grade) and t is the thickness of the base layer.

Before the LSME testing programme started, several control tests were conducted to calibrate and determine the repeatability of the test results. The viability of LSME as a prototype of a pavement section in terms of mechanical considerations has been also demonstrated in other research projects by comparing the back-calculated field modulus from falling-weight deflectometer tests (Kim, Edil, Benson, & Tanyu, 2005; Tanyu et al., 2003).

3.2. *Bench-scale resilient modulus tests*

BSRM tests were conducted on compacted specimens of the base course materials in accordance with NCHRP 1-28a (Procedure Ia). Specimens were compacted in six lifts of equal mass and thickness in a split mold (152 mm diameter, 305 mm height). All materials were compacted to 100% of standard Proctor maximum dry unit weight at optimum water content (Table 1). Equation (1) was fitted to the resilient moduli obtained from the BSRM test and a summary resilient modulus (SRM) was calculated corresponding to σ_b of 208 kPa as recommended in the procedure.

4. Results and discussion

4.1. *Resilient modulus and plastic deformation from LSME*

Plastic strain (ε_p) and summary resilient modulus (SRM) of the RPM, RSG, and Class 5 base as a function of number of load cycles (N) in the LSME are shown in Figure 4 for two base thicknesses (0.20 and 0.30 m). In all cases, the SRM and plastic strain increase monotonically with number of loading cycles.

The Class 5 base reaches a steady-state condition (negligible rate of plastic strain $\mathrm{d}\varepsilon_p/\mathrm{d}\ln N =$ 0.01, or 'plastic shakedown') in 2000 cycles. Werkmeister, Dawson, & Wellner (2001) showed

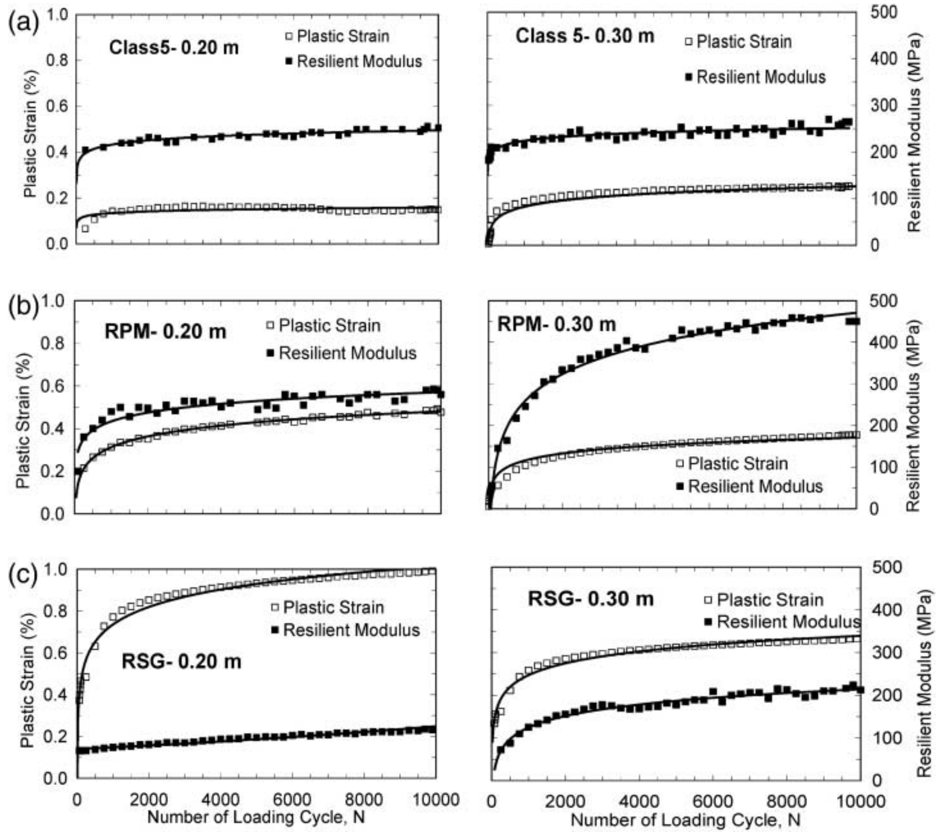


Figure 4. Plastic strain and resilient modulus vs. number of load cycles for (a) Class 5, (b) RPM, and (c) RSG with thickness of 0.2 (left) and 0.3 m (right) – (left axis, plastic strain; right axis, resilient modulus). Full lines are the fitted lines to the deformation data.

similar behaviour for conventional base course materials and natural aggregates. In contrast, RSG exhibits a high initial rate of permanent deformation for both layer thicknesses, which diminishes to a lower and near constant rate of deformation after 2000 cycles. This behaviour is attributed to the plastic fines (12%) in RSG (also noted by Yang, Huang, & Liao, 2008) and suggests that RSG exhibits creep-shakedown behaviour. The RPM exhibits a similar behaviour as the RSG. However, for RPM, the initial rate of plastic strain is lower, although the transition to a constant rate of plastic strain also occurs after 2000 cycles. The rate of plastic strain accumulation of RPM ($d\varepsilon_p/d\ln N = 0.07$) is lower than that of RSG ($d\varepsilon_p/d\ln N = 0.12$) when the rate of plastic strain becomes constant. Similar findings have been reported by Mohammad et al. (2006) for base courses constructed with recycled asphalt pavement subjected to cyclic loading. The longer transition to a constant rate of plastic deformation for RPM is attributed to the viscous characteristic of the asphalt coatings on the aggregates in RPM (Figure 1).

Plastic strain and resilient modulus of RPM and RSG stabilised with 10% fly ash (i.e. SRPM and SRSG) are shown in Figure 5 as a function of number of loading cycles. Four LSME test sequences of 10,000 cycles each were conducted on each material, with 7 days of curing between tests. The resilient modulus increases with increasing curing time for SRPM and SRSG. The SRPM exhibits a small and near constant rate of plastic strain after approximately 4000 cycles, and the SRSG exhibits a constant and very small plastic strain (<0.1%). Mohammad et al. (2006) also report

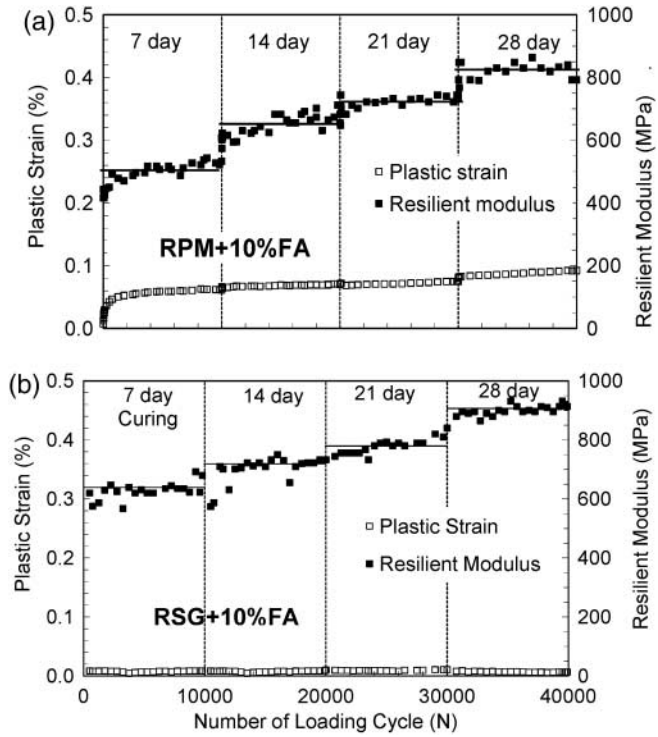


Figure 5. Permanent deformation and resilient modulus versus the number of load cycles for (a) RPM and (b) RSG stabilised with fly ash (0.3 m thicknesses)

small plastic strains for recycled foamed asphalt and blended calcium sulphate stabilised with fly ash and slag.

Lower plastic strains generally are associated with materials having higher resilient modulus (Kootstra et al., 2010). However, in this study, the largest plastic strains are associated with the RPM and RSG, even though these materials have similar or higher resilient modulus than the Class 5 base (see subsequent discussion). The higher plastic strain is attributed to viscous creep of the asphalt in the RPM and the fines in the RSG. In contrast, the SRPM and SRSG have the lowest plastic strains and the highest resilient moduli. Binding by the self-cementing fly ash reduces plastic creep in the RPM and RSG appreciably, which is also evident when Figures 4 and 5 are compared.

The cumulative permanent deformation of unstabilised RPM and RSG under traffic loading likely will be higher than conventional natural aggregate. Thus, excessive rutting may be encountered in flexible pavements that employ RPM or RSG in lieu of conventional base course materials. In contrast, RPM or RSG stabilised with cementitious fly ash should result in less rutting than conventional base aggregate (Kootstra et al., 2010).

4.2. *Resilient modulus from BSRM test*

Table 3 shows resilient modulus and fitting parameters in Equation (1) for RPM, RSG and Class 5. The stress dependency of base course materials is reflected in k_2 parameter in Equation (1). Class 5 base has $k_2 = 0.53$, which is in the typical range for the granular materials (Huang, 2003). RPM has lower k_2 ($= 0.34$) indicating lower dependency on bulk stress. RSG has $k_2 = 0.44$,

Table 3. Summary resilient modulus (SRM) and power model fitting parameters from Equation (1) for base course materials.

Material	Test method	Thickness (mm)	Fly ash content (%)	Curing time (day)	Measured parameters		
					k_1	k_2	M_r (MPa)
Class 5 base	Lab*	–	0	–	13.6	0.53	236
	LSME	200		–	19.7	0.53	284
		300		–	29.5	0.53	426
RPM	Lab*	–	0	–	49.2	0.34	309
	LSME	200		–	50.0	0.34	307
		300		–	82.0	0.34	505
SRPM	Lab*	–	10	7	1753.0	0.00	1753
		–		28	2702.0	0.00	2702
	LSME	300		7	483.0	0.00	483
		300		28	845.0	0.00	845
RSG	Lab	–	0	–	21.6	0.44	226
	LSME	200		–	11.0	0.44	115
		300		–	20.6	0.44	216
SRSG	Lab	–	10	28	5150.0	0.00	5150
	LSME	300		7	673.0	0.00	673
		300		28	918.0	0.00	918

Notes: Summary resilient modulus (M_r) is calculated at a bulk stress of 208 kPa. *Reported by Camargo et al. (2009).

intermediate between RPM and Class 5 base, reflecting the effect of fines content compared to Class 5 aggregate (i.e. less sensitive to the bulk stress than Class 5), as described by Huang (2003). For the materials stabilised with fly ash, the resilient modulus is independent of bulk stress ($k_2 \approx 0$) due to the cementation of particles. Chemical bonds between particles prevail over the inter-particle friction, which precludes stress dependency of the resilient modulus. Therefore, the stress and strain levels do not affect the resilient modulus and plastic deformation of SRPM and SRSG in the pavement (fatigue cracking would be an exception).

4.3. *Comparison between SRM from LSME and BSRM tests*

SRM of base course materials from the LSME and BSRM tests are shown in Figure 6. SRM of the Class 5 base, RPM, and RSG from the LSME are up to 1.5 times larger than those from the BSRM test. This difference in the resilient moduli of the unstabilised materials is attributed to the interplay of the strain amplitude in the two test methods. Tanyu et al. (2003) indicate that the resilient moduli of granular materials from the LSME are higher than those from BSRM tests because of the lower strains in the thicker layers at prototype scale.

To illustrate this effect, strain dependency of the modulus of the unstabilised materials was characterised using the backbone curve developed by Hardin and Drnevich (1972). Resilient moduli from the LSME and BSRM tests were normalised with respect to the low strain Young's modulus from seismic tests (E_s), as shown in Figure 7. More details are in Benson et al. (2009). The seismic tests were conducted using micro-electromechanical systems (MEMS) buried at various depths in the compacted base course materials in the LSME. They were used to measure the travel time of seismic waves transmitted from the surface by a hammer impact. The shear strain for the low strain Young's modulus (E_s) was $<10^{-5}$.

The E_s was calculated as described by Schuttpelz, Fratta, & Edil (2010):

$$E_s = V_P^2 \rho \frac{(1+\nu)(1-2\nu)}{(1-\nu)} \tag{3}$$

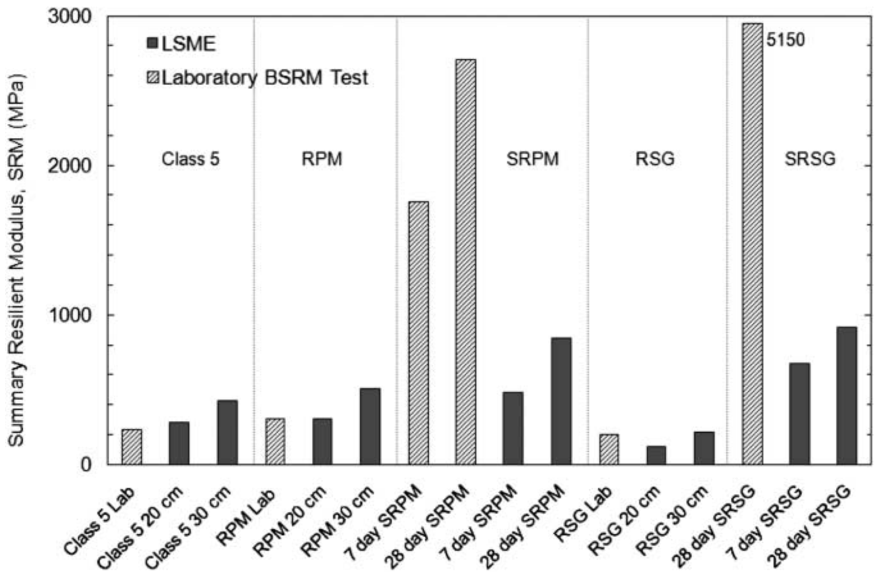


Figure 6. Summary resilient modulus (SRM) from the LSME and laboratory BSRM test.

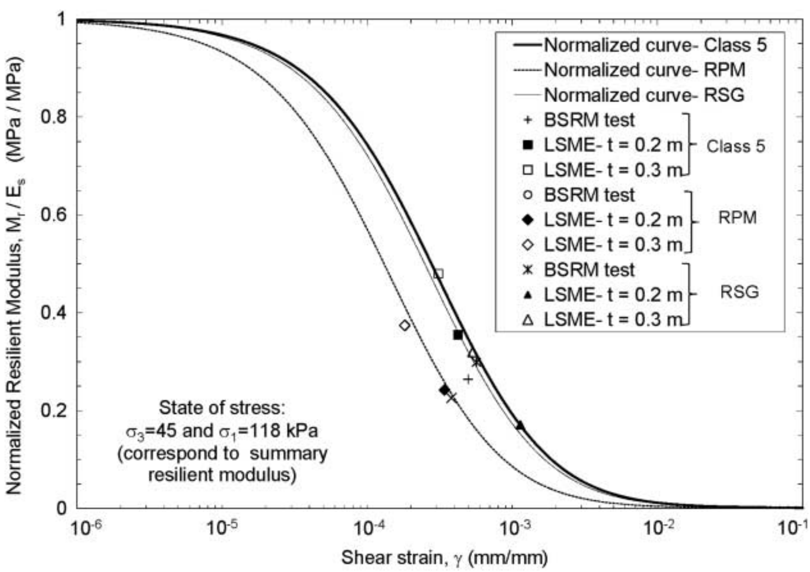


Figure 7. Strain dependency of the resilient modulus of recycled materials from the BSRM test and LSME.

where $V_P = P$-wave velocity calculated from seismic testing, $\rho =$ mass density, and $\nu =$ Poisson's ratio (see Table 2). As shown in Figure 7, the same stress level in the LSME and BSRM test (confining stress = 45 kPa) resulted in different strain levels due to the scale effect.

SRM from the BSRM test on the SRPM and SRSG consistently was 3 to 5 times higher than the SRM from the LSME, i.e. opposite the behaviour for granular materials. This difference is attributed to the mixing and curing conditions associated with fly ash stabilised materials, as reported in several investigations (Bin Shafique et al., 2004; Wen & Edil, 2009). More thorough

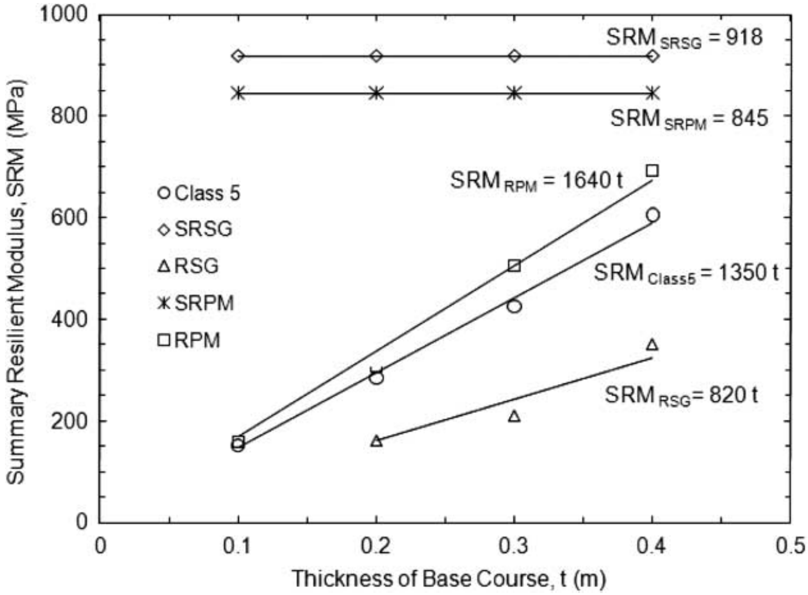


Figure 8. Summary resilient modulus (SRM) of Class 5 base, RPM, RSG, SRPM, and SRSG as a function of base course thickness.

mixing and controlled curing occurs when preparing small specimens for a BSRM test compared with the LSME or the field. For example, the temperature was about 24°C and the humidity was between 70 to 80% during the LSME test, whereas the BSRM specimens were cured at 25°C and 100% humidity. Thus, the BSRM specimens probably cured more uniformly than the fly ash stabilised materials in the LSME.

The relationship between SRM and layer thickness of base course from the LSME is shown in Figure 8. Resilient moduli corresponding to typical base course thicknesses other than the 0.20 m and 0.30 m thicknesses tested in the LSME were predicted using the backbone curve calibrated with the LSME. More details are presented in Benson et al. (2009). For the unstabilised base materials, the SRM is consistently higher for thicker base course layers due to the lower shear strain amplitude in thicker layers for the same surface load.

5. Design approaches for recycled materials as base course

Two design approaches were developed for flexible pavements using unstabilised and stabilised RPM and RSG in the base: (1) an equivalency-based design using AASHTO 1993 design guide and (2) lifetime expectancy-based design using the Mechanistic-Empirical Pavement Design Guide, (NCHRP 2006). To simulate field conditions, SRM from the LSME were used to develop the method.

5.1. *Equivalency-based design using AASHTO 1993*

Equivalency-based design was developed based on the premise of generating a pavement structure constructed with recycled base course materials that have equivalent structural capacity as the pavement constructed with conventional base course materials. AASHTO 1993 *Guide for Design of Pavement Structures* uses the structural number (SN) to describe the structural capacity and contribution of each pavement layer. Two main factors control the SN of the base course according to the AASHTO 1993: layer thickness and layer coefficient, the latter reflecting the stiffness of the layer (function of the SRM). The SN of the entire pavement is defined as the summation of the SN of the pavement layers (AASHTO 1993)

$$\mathrm{SN} = [\mathrm{SN}_1 + \mathrm{SN}_2\mathrm{m}_2 + \mathrm{SN}_3\mathrm{m}_3]/25 = [b_1t_1 + b_2t_2m_2 + b_3t_3m_3]/25 \tag{4}$$

where m_i is the drainage modification factor (assumed equal to 1 in this study), b_i is the layer coefficient, and t_i is the thickness (mm) of the layer i ($i = 1$ asphalt, $i = 2$ base course, $i = 3$ sub-base). The layer coefficient (b_2) of a granular base course is empirically related to resilient modulus (AASHTO, 1993) by

$$b_2 = 0.249 \log \mathrm{SRM} - 0.44 \tag{5}$$

where SRM is the summary resilient modulus of the granular base material (in MPa). Base course material stabilised with fly ash was also assumed to follow Equation (5).

Layer coefficients (b_2) for the recycled materials used in this study were calculated using Equation (5) by employing the SRM from the LSME (Figure 8), and are shown in Figure 9. The layer coefficients are within the typical range of layer coefficients presented in AASHTO 1993 for base course layers. For the materials without fly ash, the layer coefficient varies with thickness because the lower strain amplitude in thicker layers results in higher SRM (Figure 7). For example, the layer coefficient of 0.3 m thick RPM is 0.20, whereas the layer coefficient for a 0.2 m thick RPM is 0.17 because of the higher strains in a thinner layer of RPM. In contrast, the layer coefficient for the materials stabilised with fly ash (SRPM and SRSG) does not vary with base course thickness because the SRM of stabilised materials is not stress or strain dependent. The layer coefficient for the stabilised materials is also higher than the layer coefficients for unstabilised materials, indicating that base courses constructed with stabilised materials have higher structural capacity.

The layer coefficients presented in Figure 9 can be applied directly to the design of flexible pavements in accordance with AASHTO 1993. However, an equivalency design approach was

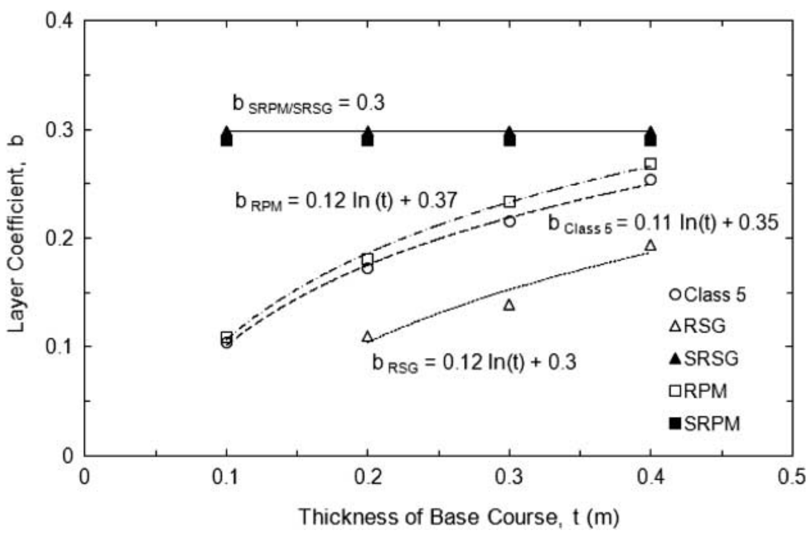


Figure 9. Layer coefficients for Class 5 base, RPM, RSG, SRPM, and SRSG as a function of base course thickness.

developed as a design tool to relate the required thickness of recycled materials relative to the thickness of conventional base course aggregate. Designers have experience with common thicknesses of natural aggregate base in many applications. Thus, this approach gives them a simple tool for selecting a base course comprised of recycled materials.

The equivalency-based design equates the SN of a pavement having an alternative recycled base course to that of a pavement constructed with quality aggregate base. MnDOT's Class 5 base (MnDOT, 2005) was used as the standard base material. The structural number of base course consisting of recycled material (SN_r) was set equal to the structural number of conventional base course material (SN_c). These SN are computed as

$$\mathrm{SN}_r = b_1 t_1 + b_r t_r \tag{6}$$

$$\mathrm{SN}_c = b_1 t_1 + b_c t_c \tag{7}$$

where the subscripts c and r denote the conventional and the alternative recycled base course materials (Figure 10).

If the HMA thickness and properties are assumed to be the same for the two pavement configurations the relationship between thicknesses and layer coefficients for the conventional and recycled base materials is

$$\frac{t_c}{t_r} = \frac{b_r}{b_c} \tag{8}$$

Substituting Equation (5) into Equation (8) yields

$$\frac{t_r}{t_c} = \frac{0.249 \log \mathrm{SRM}_c - 0.44}{0.249 \log \mathrm{SRM}_r - 0.44} \tag{9}$$

where SRM is in MPa.

The relationship between SRM and thickness in Figure 8 was used in Equation (9) to create a design graph for recycled materials as a base course (Figure 11). RPM has nearly an equivalent thickness to Class 5 base in a pavement structure because both materials have similar moduli. A thicker layer of RSG is required to obtain a base equivalent to the Class 5 base because RSG has lower SRM than Class 5 aggregate. Similarly, fly ash stabilisation improves the structural capacity of the RPM and RSG and results in a thinner equivalent base course layer (e.g. a 0.22-m thick SRPM or SRSG layer is equivalent to a 0.3 m thick Class 5 base).

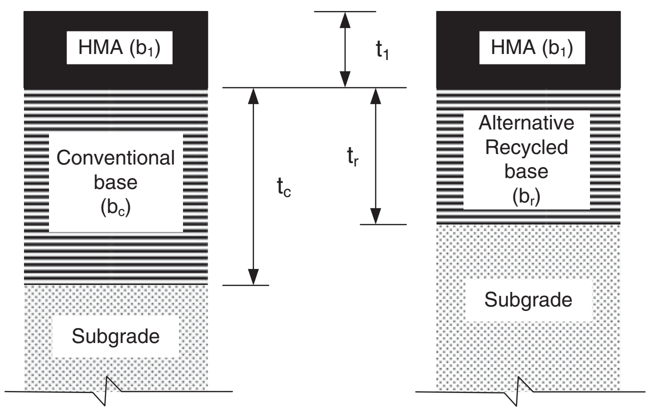


Figure 10. Schematic pavement profiles for equivalency-based design between conventional and alternative recycled base course materials.

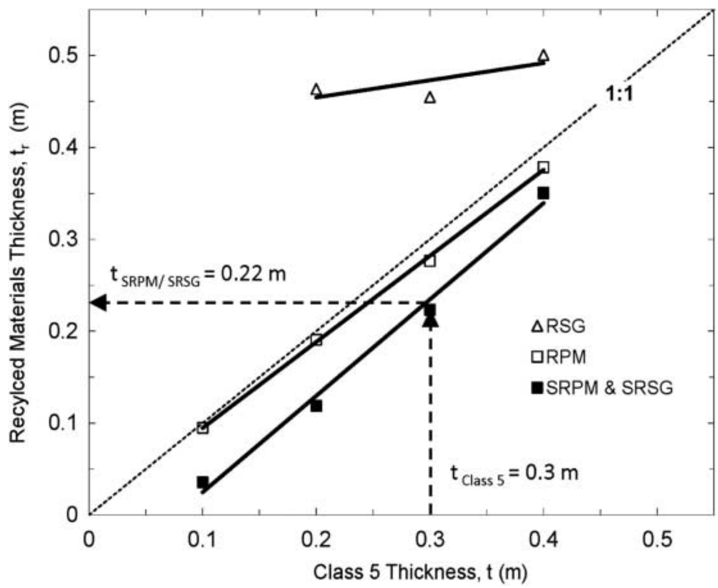


Figure 11. Alternative recycled material thickness as a function of Class 5 thickness.

5.2. *Equivalency-based design using MEPDG*

An equivalency-based design approach was developed using the Mechanistic-Empirical Pavement Design Guide (NCHRP 2006) so that plastic deformation of base course could be accounted for explicitly in the design (plastic deformation is not implicit in the AASHTO 1993 method). MEPDG uses mechanistic-empirical models to predict damage accumulation over the predicted service life of a pavement data on traffic, climate, materials, and the pavement structure as input. The data input to MEPDG are shown in Table 4.

SRM and plastic deformation obtained from the LSME were used in MEPDG to predict the rut depth and international roughness index (IRI) of a pavement. The calibration factor (B_{s1}) in Table 4 was determined by inversion of the LSME data (plastic deformations from the LSME

Table 4. Input parameters for MEPDG programme.

Traffic	Initial two-way		4000 AADTT	
	Number of lanes		2	
	Operation speed		110 km/h	
	Dual tyre spacing		0.3 m	
	Tyre pressure		800 kPa	
Environment		I–94 Minnesota, USA		
Asphalt binder	Thickness		0.1 m	
Superpave binder	A		10.98	
Grading	VTS		−3.6	
Base course A-1-a	Thickness		0.3 m	
	Modulus		From LSME, presented in Figure 8	
Subgrade	Thickness		0.5 m	
	Modulus		70 MPa	
Rutting for granular materials				
Rutting calibration factor	RSG	RPM	Class 5	SRPM/SRSG
B_{s1}	1.7	1.4	1.0	0.1

were matched with predictions from MEPDG). The rut depth and international roughness index (IRI) were then determined in pavement structures consisting of various base course materials (i.e. Class 5, RPM, RSG, and SRPM/SRSG). Material properties and geometry of HMA and sub-grade layers were assumed to be the same for all cases. The sub-grade modulus was assumed to be 70 MPa and the average annual daily truck traffic (AADTT) was assumed to be 4000.

Service lives of pavement structures constructed with RSG, RPM, Class 5 aggregate, and SRPM/SRSG as base course were determined based on two criteria: a limiting rut depth of 12.7 mm (Huang, 2003) and a limiting IRI of 2.7 m/km (NCHRP 1-37A). Service lives based on limiting rutting depth are shown in Figure 12(a) and based on IRI in Figure 12(b) for pavements with different base course materials and thicknesses. The rutting calibration factors (B_{s1}) and SRM for base course thicknesses other than 0.2 and 0.3 m were extrapolated from the LSME data.

The service life of a pavement constructed with RSG is shorter than for Class 5 aggregate due to the lower resilient modulus (Figure 6) and more rapid rutting (Figure 12) of RSG. The service life of a pavement constructed with RPM is similar to the service life for a pavement with Class 5

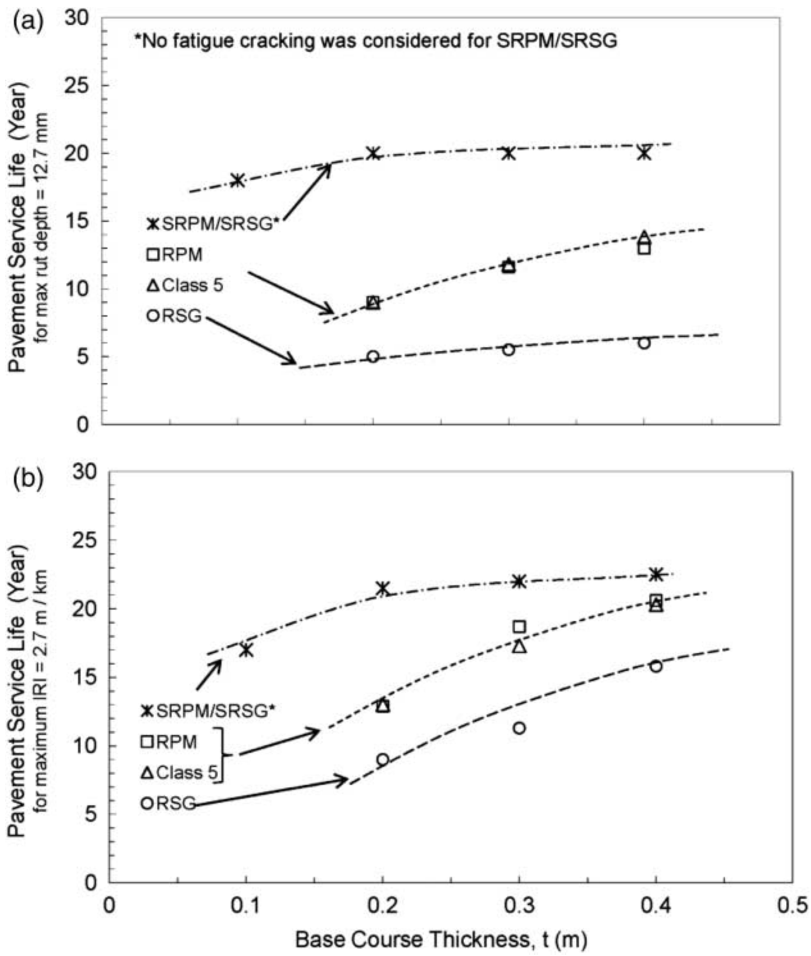


Figure 12. Life time expectancy of pavement structure with conventional and recycled base course materials for limiting rut depth (a) and IRI (b) from MEPDG.

aggregate base, even though RPM has higher rutting potential compared to Class 5 aggregate (i.e. rutting calibration factor = 1.4 vs. 1.0 in Table 4). RPM has higher resilient modulus (500 MPa for RPM vs. 400 MPa for Class 5 aggregate for 0.3 m thickness, Figure 8), which results in different stress distribution and consequently different contributions to rutting from the base course layer. Consequently, rutting is comparable for RPM and Class 5 aggregate.

Fly ash stabilisation of RPM or RSG increases the service life appreciably. Using 0.3 m thick SRPM or SRSG base instead of 0.3 m thick Class 5 or RPM base increases the service life of the pavement structure from 17 to 21 years.

6. Conclusions

Large-scale model experiments (LSME) and standard bench-scale resilient modulus (BSRM) tests were conducted on recycled pavement material (RPM), reclaimed road surface gravel (RSG) and conventional Class 5 aggregate base from Minnesota. The RPM and RSG were tested alone and with fly ash stabilisation.

(1) Stabilisation of RPM or RSG with self-cementing fly ash (10% by weight) increased the summary resilient modulus (SRM) of the base layer significantly.
(2) SRM of unstabilised base course materials back-calculated from the LSME is higher than SRM from the BSRM tests and varies with layer thickness. These differences in SRM are attributed to the differences in strain amplitudes induced in the two methods. In contrast, SRM back-calculated from the LSME for RPM and RSG stabilised with fly ash is independent of layer thickness and also smaller than SRM from the BSRM tests. These differences are attributed, respectively, to the more competent material generated and to the thorough mixing and curing procedure used to prepare specimens for BSRM tests compared to the LSME. The mixing and curing in the LSME was considered to be more representative of field conditions.
(3) Deflection data from the LSME indicated that RPM and RSG have higher potential for accumulating plastic deformation during the service life of a pavement compared to natural aggregate. RPM and RSG stabilised with fly ash (SRPM and SRSG) exhibit negligible plastic deformation.
(4) Since resilient modulus and plastic strain obtained from the LSME are considered to be more representative of field conditions than those from the BSRM tests, they were adopted for developing equivalency-based design methodologies incorporating recycled base materials based both on the AASHTO 1993 and Mechanistic-Empirical Pavement Design Guide methodologies. Design considerations incorporate layer coefficients of unstabilised granular base materials increasing with thickness of the base course layer. Fly ash-stabilised materials have a layer coefficient that is independent of layer thickness. Pavements constructed with RPM base have a similar service life as those with Class 5 aggregate base, while pavements constructed with RSG base have shorter service life due to the lower resilient modulus and more rapid rutting of RSG. Stabilisation of recycled materials used as base course can reduce the required thickness of the base course up to 30% or increase the service life of pavements by more than 20%.

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