FINAL SUMMARY REPORT FOR THE PROJECT:

USE OF FLY ASH FOR RECONSTRUCTION OF BITUMINOUS ROADS

by

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1. INTRODUCTION

Recycling part or all of the pavement materials in an existing road during rehabilitation and reconstruction is an attractive construction alternative. For roads with a hot mix asphalt (HMA) surface, the HMA, underlying base, and a portion of the existing subgrade often are pulverized to form a new base material referred to as recycled pavement material (RPM). Compacted RPM is overlain with a new HMA layer to create a reconstructed or rehabilitated pavement. This process is often referred to full-depth reclamation (FDR). Similarly, when an unpaved road with a gravel surface is upgraded to a paved road, the existing road surface gravel (RSG) is blended and compacted to form a new base layer that is overlain with an~~]~~ HMA surface. Recycling pavement and road materials in this manner is both cost effective and environmentally friendly.

Recycled base materials may contain asphalt binder, fines, and/or other deleterious materials that can adversely affect strength and stiffness. To address this issue, chemical stabilizing agents such as cement, asphalt emulsions, lime, cement kiln dust (CKD), or cementitous fly ash can be blended with RPM or RSG to increase the strength and stiffness. This “stabilized” material is ~~often~~ referred to as SRPM or SRSG. Use of industrial material resources for stabilization, such as CKD or fly ash, is particularly attractive in the context of sustainability.

The purpose of this study was to develop a practical method to design local roadways using SRPM or SRSG as the base layer and Class C fly ash as the stabilizing agent in the context of the “gravel equivalency” (GE) design methodology employed for local roads in Minnesota. The project consisted of four major elements: (i) laboratory testing (), (ii) prototype pavement evaluation (), field assessment of two existing roadways constructed with SRPM and SRSG (), and (iv) assessment of potential impacts to ground water (). This summary report was created as a design guide and includes step-by-step design procedures along with practical implications relevant to implementation. Detailed reports describing each of the four major elements in the study can be obtained by clicking on the PDF icons cited above. A summary of a similar study conducted by MnDOT and the Waseca County Highway Department at CSAH 8 in Waseca, MN is included in the appendix to this report.

2. METHODOLOGY

The design methodology presented in this report was developed using a three-pronged approach:

- Laboratory tests were conducted on conventional test specimens to evaluate how fly ash content, curing time, and freeze-thaw cycling affect the strength and stiffness of RPM, RSG, SRPM, and SRSG.

- Prototype-scale tests were conducted to understand the stiffness of RPM, RSG, SRPM, and SRSG operative in full-scale pavement profiles under cyclic loading representative of field conditions. Results of these prototype-scale tests were used to develop the design procedure.

- Pavement monitoring was conducted at two field sites employing SRPM and SRSG to confirm that the pavements were performing satisfactorily when subjected to full-scale loading under realistic conditions, including exposure to severe weather conditions imposed by winter in Minnesota. These field sites were also instrumented to evaluate potential impacts to ground water.

The testing program was conducted with three different base course materials: (i) a granular base comparable to Class 5 base used in Minnesota, (ii) RPM from a FDR project in Madison, WI, and (iii) a simulated RSG created by blending commercially available soil and aggregates to form a test material having characteristics of RSG meeting the criteria in AASHTO M 147. SRPM and SRSG were created by blending the RPM and RSG with Class C fly ash from Columbia Power Station in Portage, Wisconsin. The fly ash content was maintained at 10% in the prototype evaluation due to the high level of effort associated with LSME testing. However, 10% is the common fly ash content used in practice.

Properties of the materials are summarized in Tables 1 and 2. Their particle size distribution curves are shown in Fig. 1. These materials have characteristics similar to materials employed in actual projects in Minnesota. Thus, the findings and procedures reported in this study are believed to have general applicability for design of local roads in Minnesota.

The prototype-scale tests were conducted in the large-scale model experiment (LSME) at the University of Wisconsin-Madison. The LSME is a testing facility where full-scale pavement profiles can be evaluated under full-scale cyclic loading conditions (Fig. 2). Previous studies have shown that pavement moduli obtained by analyzing LSME data are representative of full-scale conditions. Background on the LSME and detailed information on the LSME tests conducted in this study are available in the aforementioned project reports linked electronically to this document.

3. DESIGN PROCEDURE

3.1 Background on Gravel Equivalency

The GE procedure for design of local roads employs GE factors that are similar conceptually to the layer coefficients employed when designing flexible pavements using the AASHTO *Guide for Design of Pavement Structures*. The GE method provides a means of equating the structural performance of all bituminous and aggregate layers constituting a pavement structure with respect to the structural performance of MnDOT's Class 5 and 6 aggregate bases. GE of a pavement structure is computed as:

$$GE = a_1D_1 + a_2D_2 + a_3D_3 \tag{1}$$

where D_1, D_2, and D_3 are thicknesses of the HMA surface, the granular base course, and a granular subbase course (if present) and a_1, a_2, and a_3 are corresponding GE factors. Type of pavement material is used to define each of the GE factors using tables published by MnDOT. The effect of subgrade is not considered in the GE thickness.

3.2 Equivalency-Based Design

The design procedure developed in this study is based on the premise that the pavement constructed with the alternative base material has equivalent structural capacity as the pavement constructed with conventional base course. The conventional pavement is assumed to consist of a HMA layer and a MnDOT Class 5 base course layer (no subbase). Thickness of the alternative base course is selected to ensure that the pavement with alternative materials has equivalent structural capacity.

The GE of the pavement structure using the conventional Class 5 base is:

$$GE_c = a_1\, D_1 + a_c\, D_c \tag{2}$$

where the subscript ‘c’ denotes the conventional Class 5 base (Fig. 3). Similarly, for the alternative recycled base material:

$$GE_a = a_1\, D_1 + a_a\, D_a \tag{3}$$

where the subscript ‘a’ denotes the alternative recycled base course (Fig. 3). For an equivalent pavement structure, $GE_a = GE_c$. If the HMA thickness is assumed to be the same for both pavements, the relationship between thicknesses and GE factors for the conventional and recycled base materials is:

$$\frac{a_a}{a_c} = \frac{D_c}{D_a} \tag{4}$$

A similar procedure can be carried out with the AASHTO design method based on structural number. For the AASHTO method, the ratio of the thicknesses is:

$$\frac{D_c}{D_a} = \frac{0.249 \log Mr_a - 0.977}{0.249 \log Mr_c - 0.977} \tag{5}$$

where Mr_a (units?) is the summary resilient modulus of the alternative recycled base course and Mr_c (units?) is the summary resilient modulus of the conventional Class 5 base course. Eq. 5 can be used to determine the thickness of an alternative base course of recycled material using the resilient modulus of the alternative and conventional base course materials:

$$D_a = D_c \frac{0.249 \log Mr_c - 0.977}{0.249 \log Mr_a - 0.977} \tag{6}$$

Alternatively, the GE factor for an alternative recycled base material can be obtained by combining Eqs. 4 and 5:

$$a_a = \frac{0.249 \log Mr_a - 0.977}{0.249 \log Mr_c - 0.977} \tag{7}$$

In Eq. 7, $a_c = 1.0$ as stipulated in the GE design method.

Eqs. 6 and 7 require that the summary resilient modulus of the Class 5 base course and the alternative recycled material as input. LSME testing was conducted to obtain these summary resilient moduli for conditions operative at field scale. These moduli vary with thickness for the granular materials (Class 5 base, RPM, and RSG), but are independent of thickness for the stabilized materials (SRPM and SRSG) (Fig. 4).

These relationships can be used with Eq. 7 to define the GE factor for each alternative recycled material (Fig. 5).

As shown in Fig. 5, the GE for RSG (a_{RSG}) is less than that of Class 5 base (a_{RSG} < a_c = 1.0), the GE factor for RPM (a_{RSG} =1.07) is essentially the same as the GE factor for Class 5 base, and GE factor for SRPM and SRSG is greater than that of Class 5 base. In addition, the GE factors for SRPM and SRSG are nearly identical, and can be described by a single equation. RPM is the only alternative material that has a constant GE factor. This occurs because the resilient modulus of RPM and Class 5 gravel vary with layer thickness in a similar manner (Fig. 4).

Given the lack of field experience with this method, the following recommendations are made when applying the equations shown on Fig. 5:

- Maintain a_{SRPM} and a_{SRSG} within the range of 1.0 to 1.5.
- Use a_{RPM} = 1.0.

3.3 Alternative Base Course Selection Procedure

The following procedure is recommend for selecting the thickness of an alternative base course:

1. Create a conventional pavement design with Class 5 base material (or comparable aggregate base) using methods published by MnDOT or using local experience.

2. Determine the gravel equivalency factor for the recycled base material using the thickness of Class 5 base material from the conventional design (D_c) and the equations in Fig. 5. If a_{SPRM} or a_{SRSG} exceeds 1.5, set it at 1.5. Similarly, if a_{SPRM} or a_{SRSG} is less than 1.0, set it at 1.0.

3. Compute the thickness of the alternative base course (D_c) using

$$D_a = \frac{1}{a_a} D_c \tag{8}$$

where $a_a = a_{SPRM}$, a_{SRSG}, a_{RPM}, or a_{RSG} (depending on the material selected).

Ali/Brian – please create a numerical example problem using the procedure in Section 3. Describe this example problem in a step-by-step format and paste it here. Use SI units, but include English equivalents (inches for length units).

4. PRACTICAL IMPLICATIONS

4.1 Fly Ash Content

Bench-scale testing conducted in this study on conventional test specimens showed that the summary resilient modulus of SRPM and SRSG increases significantly as the fly ash content is increased (Fig. 6). This behavior is significantly different from that observed in stabilized subgrades, where little increase in modulus is obtained for fly contents > 10%.

Although 10% fly ash is most common in practice, designers may wish to increase the fly ash content to increase the modulus of SRPM and SRSG. The following procedure can be used to account for this increase in modulus due to higher fly ash content:

1. Conduct resilient modulus tests on specimens of SRPM and SRSG at 10% fly ash content and the desired fly ash content using AASHTO TP46-94 or the locally adopted method.

2. Determine the summary resilient modulus at 10% fly ash content (SMr_{10}) and at the desired fly ash content (SMr_X at X%). If resilient modulus testing is impractical, conduct unconfined compression tests and estimate the summary resilient modulus using:

$$SM_{rx} = 3280 \text{ UCS} \tag{9}$$

where SM_{rx} is in MPa and UCS is the unconfined compressive strength (MPa). Eq. 9 was obtained from bench-scale tests on conventional specimens of SRPM and SRSG, as shown in Fig. 7.

3. Compute the layer coefficient for X% fly ash (a_X) using:

$$a_x = a_a \frac{0.249 \log SMr_x - 0.977}{0.249 \log SMr_{10} - 0.977} \tag{10}$$

where a_a is the layer coefficent for 10% fly ash. If a_x computed with Eq. 9 exceeds 1.5, set $a_X = 1.5$.

4. Compute the thickness of the alternative base course with X% fly ash using:

$$D_x = \frac{1}{a_x} D_c \tag{11}$$

4.2 Curing Time

The LSME tests used to develop the design method described in this report were conducted after 7 d of curing. However, the hydration reactions associated with fly ash in SRPM or SRSG continue for many weeks after initial hydration, resulting in greater cementation and increasing modulus. This effect is shown in Fig. 8, which shows data from bench-scale tests on conventional test specimens of SRPM and SRSG cured for various periods of time.

At this time, insufficient information to confirm that increases in modulus occurring in the field are of comparable magnitude as those observed in the laboratory. Thus, no correction for curing time is recommended. Neglecting the temporal increase in modulus due to curing also makes the design method described in Section 3 conservative.

4.3 Freeze-Thaw Deterioration

Freeze-thaw cycling causes volume change and movement of particles in base courses and subgrades, and has the potential to cause a reduction in modulus due breaking of cement bonds between particles. The effect of freeze-thaw cycling on modulus of SRPM and SRSG was evaluated by conducting bench-scale tests on conventional test specimens that were subjected to 5 cycles of freeze-thaw cycling. This testing regime was selected based on prior studies, which showed that reductions in modulus due to freeze-thaw cycling occur within 5 cycles.

Results of the freeze-thaw tests are summarized in Table 3. Reductions in modulus due to freeze-that cycling for SRPM and SRSG ranged between 5 and 15%. These reductions likely are offset by gains in modulus due to additional hydration. Thus, no correction for the effect of freeze-thaw cycling is recommended.

4.4 Field Performance

Mechanical and environmental monitoring data were collected and evaluated at field sites in Waseca, MN and Chisago County, MN where fly ash was used to stabilize recycled alternative base materials. The field site in Waseca employed SRPM as part of a reconstruction project for a city street with an HMA surface. At Chisago County, SRSG was used as base course for an HMA pavement when upgrading a gravel road.

Falling weight deflectometer (FWD) tests were conducted at both field sites to assess the modulus of the SRPM and SRSG over time.

For the Waseca site, data from the FWD surveys indicated that the field moduli remained stable over 4 yr, despite several seasons of freezing and thawing. For the Chisago site, FWD testing indicated that the modulus of the SRSG decreased slightly during the first year, but remained stable thereafter at about 350 MPa. These findings indicate that the properties of SRPM and SRSG generally are maintained in the field, even under the severe winter conditions in Minnesota. Periodic monitoring of these field sites with a FWD is recommended to assess the long-term performance of the stabilized recycled base materials.

5. ENVIRONMENTAL CONSIDERATIONS

5.1 Field Observations

Pan lysimeters were installed beneath the pavement at the field sites in Waseca and Chisago County, MN to measure the rate at which liquid is transmitted by pavement structures and to determine chemical constituents in the liquid that is transmitted (referred to as leachate). Leachate from both sites was analyzed for 20 MPCA soil leaching value (SLV) elements. Column tests were also conducted in the laboratory on samples of the SRPM and SRSG from the field sites. Data from these column tests were used as input when modeling potential ground water impacts at the field sites.

Data were collected from the Waseca lysimeter from 2004 to 2008~~7~~, during which the pavement transmitted approximately 20 mm/yr of leachate. The lysimeter at Chisago County was periodically flooded by perched ground water during snowmelt events. This unanticipated condition rendered data from the Chisago County lysimeter unreliable. Consequently, data collection from the Chisago lysimeter was terminated within one year after installation.

Chemical analysis of leachate from the Waseca lysimeter showed that concentrations of many trace elements were reasonably steady towards the end of the monitoring period, or were decreasing (Fig. 9). During the monitoring period, concentrations of most elements were below USEPA maximum contaminant levels (MCLs) and Minnesota health risk levels (HRLs) established by the Minnesota Dept. of Public Health. Concentrations exceeding MCLs and/or HRLs at least one time included As (MCL exceeded), Pb (MCL exceeded), Sb (MCL and HRL exceeded), Se (MCL and HRL exceeded), and Tl (MCL and HRL exceeded). These exceedances were infrequent, only modestly above the MCL or HRL, and were measured at the bottom of the SRPM layer (not in ground water). Thus, these exceedances do not reflect ground water conditions or impacts to ground water. In fact, modeling showed that exceeding MCLs or HRLs in ground water concentration at the edge of the right of way is highly unlikely under most conditions (see Sec 5.2).

5.2 Potential Ground Water Impacts

Potential impacts to ground water were evaluated by conducting simulations with two different programs: WiscLEACH and STUWMPP. Both of these programs are used in Midwestern states to evaluate potential impacts to ground water from leaching associated with industrial material resources used in roadway construction, including fly ash used to stabilize recycled base materials.

Simulations were conducted with WiscLEACH in two steps: calibration and assessment. Calibration consisted of simulations of the Waseca site where the seepage velocity was adjusted until reasonable agreement was obtained between concentrations predicted by WiscLEACH and concentrations measured in the lysimeter. Leaching data from column tests conducted on samples of SRPM collected during construction were used as input.

Calibration showed that good agreement between predicted and measured concentrations was obtained using the 75th percentile seepage velocity measured in the field. The calibration was then checked by comparing predictions made for As and Sb concentrations observed in the lysimeter. Good agreement was obtained between these predicted and measured concentrations as well.

Assessment consisted of making predictions of maximum ground water concentrations at the right of way for the Waseca site over a 100-yr period. These simulations showed that concentrations above the MCL at the point of compliance were obtained only for Sb, and these concentrations were only slightly above the MCL. Thus, the potential for ground water impacts at the Waseca site is very small.

STUWMPP findings – Paul … do you want to add some summary info here?

5.3 Effect of Site Conditions

Parametric simulations were conducted with WiscLEACH to evaluate how site specific factors affect trace element concentrations in ground water caused by leaching from recycled base materials stabilized with fly ash. Independent variables were varied one at a time in a systematic manner, with all other variables held constant. Input data for the Waseca site were used to define the variables held constant.

Results of these simulations were used to identify conditions that result in lower peak concentrations at the edge of a right of way. The following conditions were identified:

- lower peak concentrations are expected at sites with greater depth to ground water,
- presence of a less permeable layer within the pavement profile (e.g., HMA with low air voids content, fine-grained subgrade, etc.) will reduce peak concentrations in ground water,

- use of a thinner layer of SRPM, when practical, will result in lower peak concentrations, and
- application to narrower roadways, such as city streets and secondary highways, has less impact on ground water than applications on wide highway pavements.

Lower concentrations are also expected at sites where ground water flows more rapidly due to increased dilution. Given the number of factors that may affect peak concentrations at the right of way, site-specific assessments are recommended.

Paul – would you like to add anything from STUWMPP here?

TABLES

Table 1. Index properties for Class 5 base, RPM, and RSG.

Material	D_{50} (mm)	C_u	C_c	G_s	w_{opt} (%)	$\gamma_{d\ max}$ (kN/m^3)	Asphalt Content (%)	LL (%)	PL (%)	Gravel Content (%)	Sand Content (%)	Fines Content (%)	USCS Symbol	AASHTO Symbol
Class 5 Base	2.25	33.3	0.7	2.72	5.0	20.9	-	NP	NP	36.6	59.3	4.1	SP	A-1-a
RPM	3.89	89.5	2.5	2.64	7.5	21.2	4.6	NP	NP	46.0	43.0	10.6	GW-GM	A-1-a
RSG	0.80	40.0	1.0	2.73	7.5	22.6	-	21	14	28.6	59.0	12.4	SC-SM	A-2-4
SRPM	-	-	-	-	8.5	20.4	-	-	-	-	-	-	-	-
SRSG	-	-	-	-	6.6	22	-	-	-	-	-	-	-	-

D_{50} = median particle size, Cu = coefficient of uniformity, Cc = coefficient of curvature, Gs = specific gravity, w_{opt} = optimum water content, $\gamma_{d\ max}$ = maximum dry density, LL = liquid limit, PL = plastic limit, NP = non-plastic.

Note: Particle size analysis conducted following ASTM D 422, Gs determined by ASTM D 854, $\gamma_{d\ max}$ and w_{opt} determined by ASTM D 698, USCS classification determined by ASTM D 2487, AASHTO classification determined by ASTM D 3282, asphalt content determined by ASTM D 6307, and Atterberg limits determined by ASTM D 4318.

Table 2. Physical properties and chemical composition of Columbia fly ash.

Parameter	Columbia	Typical Class C
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, %	0.7	6
Fineness (retained on #325 sieve) %	12	-

Table 3. Change in SRM due to freeze-thaw cycling.

Material	Fly Ash Content (%)	Change in SRM (%)
Class 5 base	0	-7.0
RPM	0	14
RSG	0	1.0
RPM	10	-15
RSG	10	-5.0

FIGURES

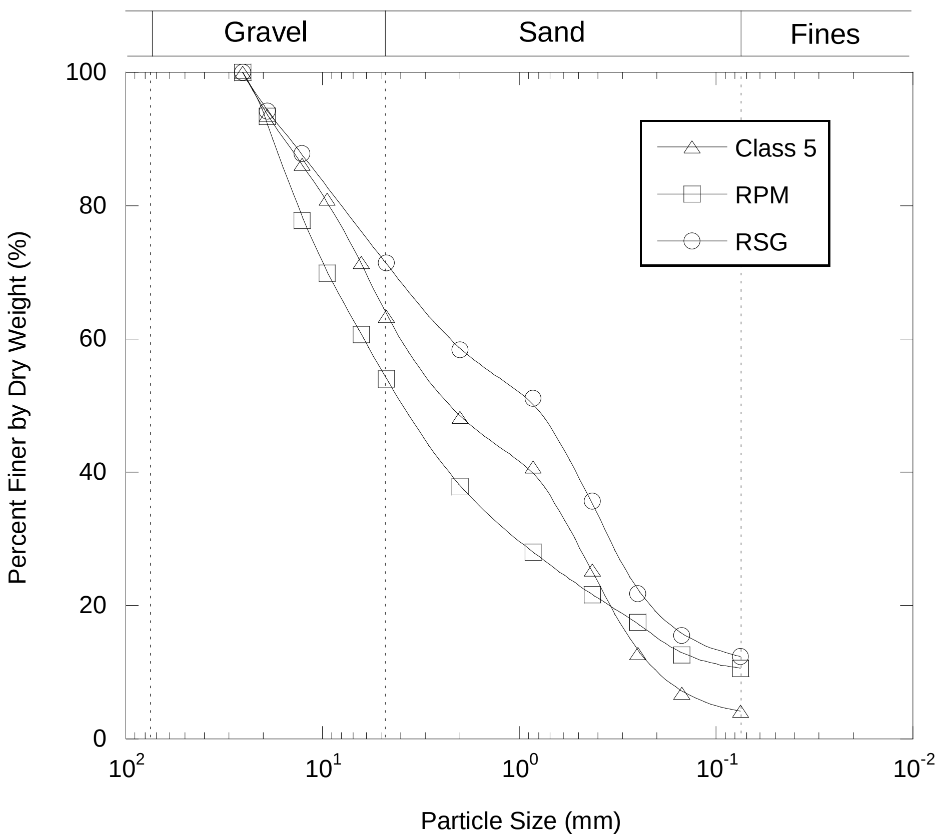

Fig. 1. Particle size distributions of Class 5 base, RPM, and RSG used in study.

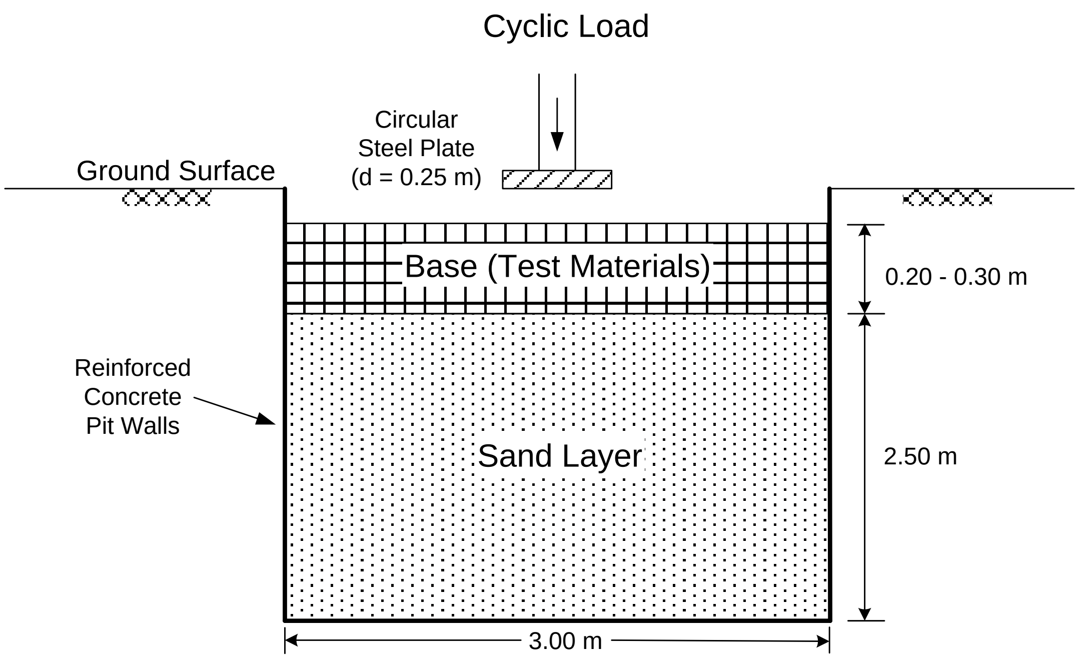

Fig. 2. Schematic of LSME used for prototype testing and evaluation.

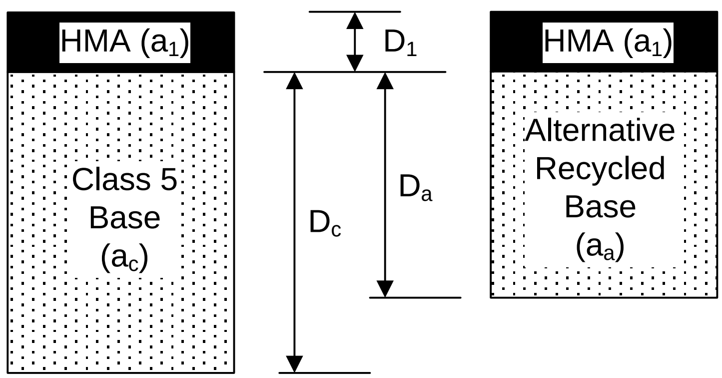

Fig. 3. Schematic of profiles for conventional pavement and alternative with recycled base material.

Need revised figure from Ali/Brian

Fig. 4. Summary resilient modulus of Class 5 base, RPM, RSG, SRPM, and SRSG as a function of base course thickness.

Need revised figure from Ali/Brian

Fig. 5. Gravel equivalency factor for RPM, RSG, SRPM, and SRSG as a function of thickness of Class 5 base.

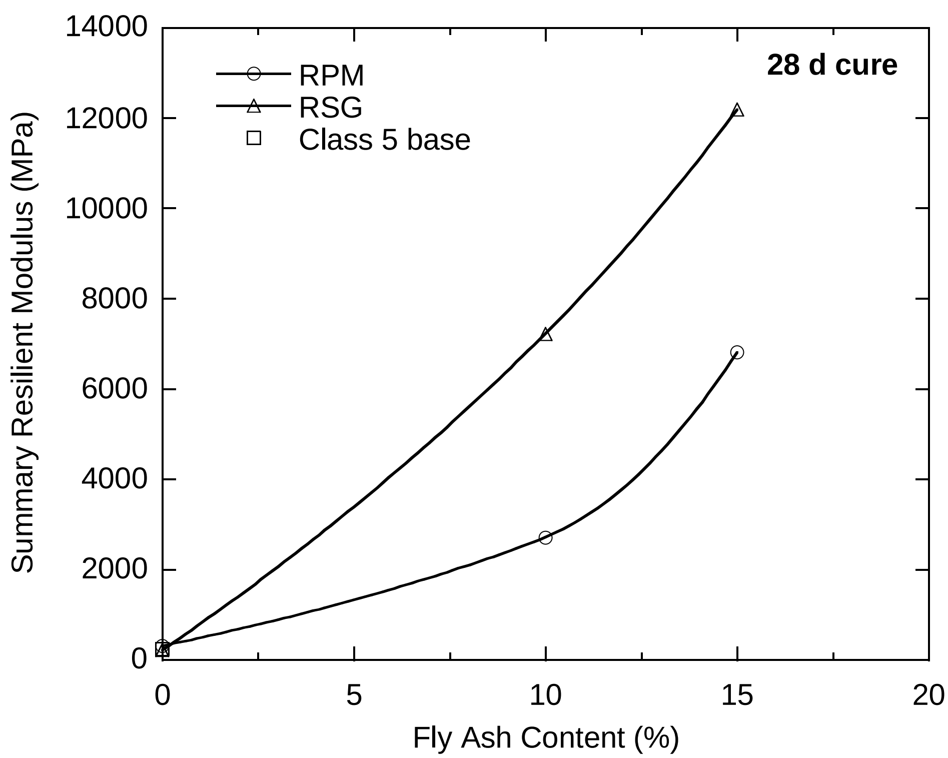

Fig. 6. Summary resilient modulus as function of fly ash content for SRPM and SRSG

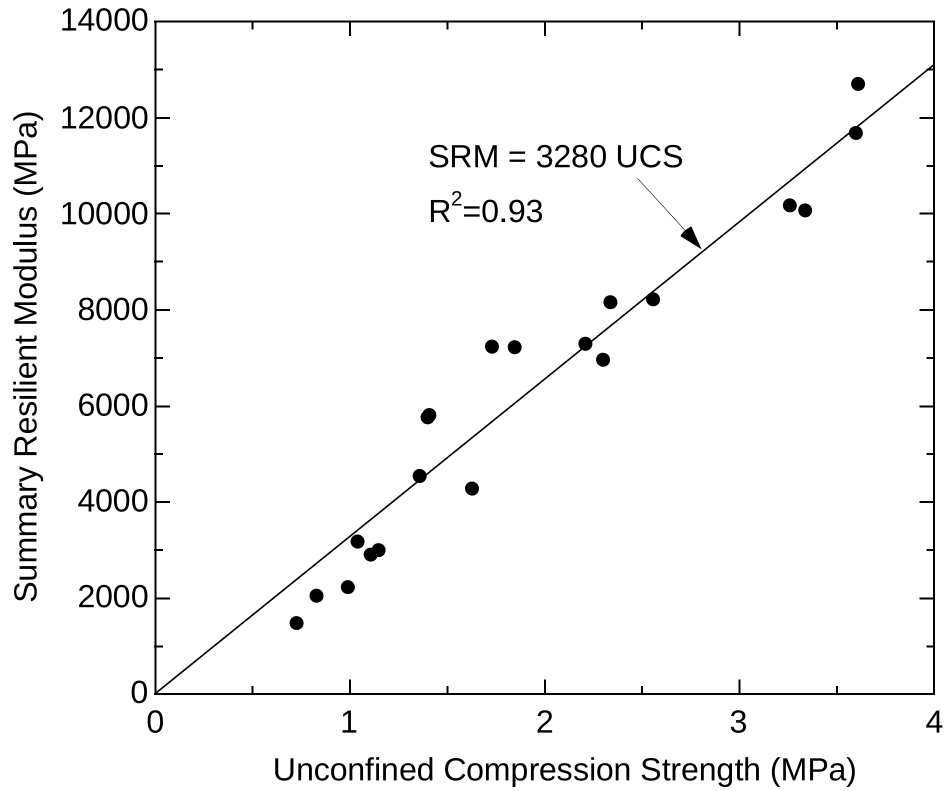

Fig. 7. Summary resilient modulus as function of SRPM and SRSG as a function of unconfined compressive strength.

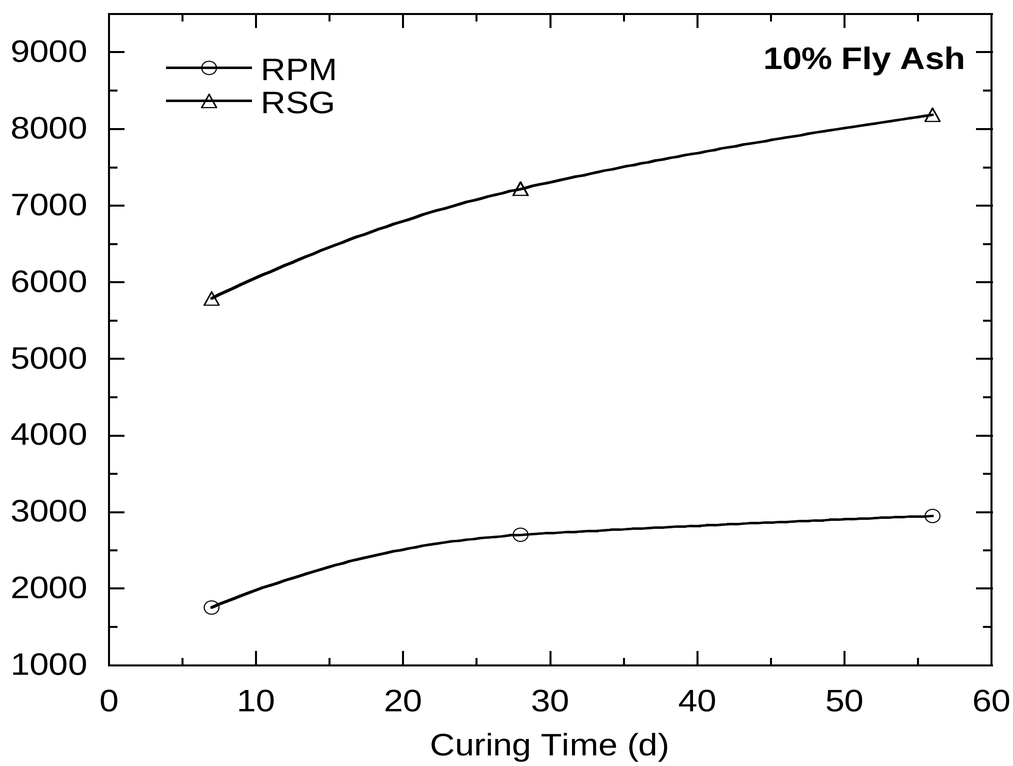

Fig. 8. Summary resilient modulus as function of curing time for SRPM and SRSG

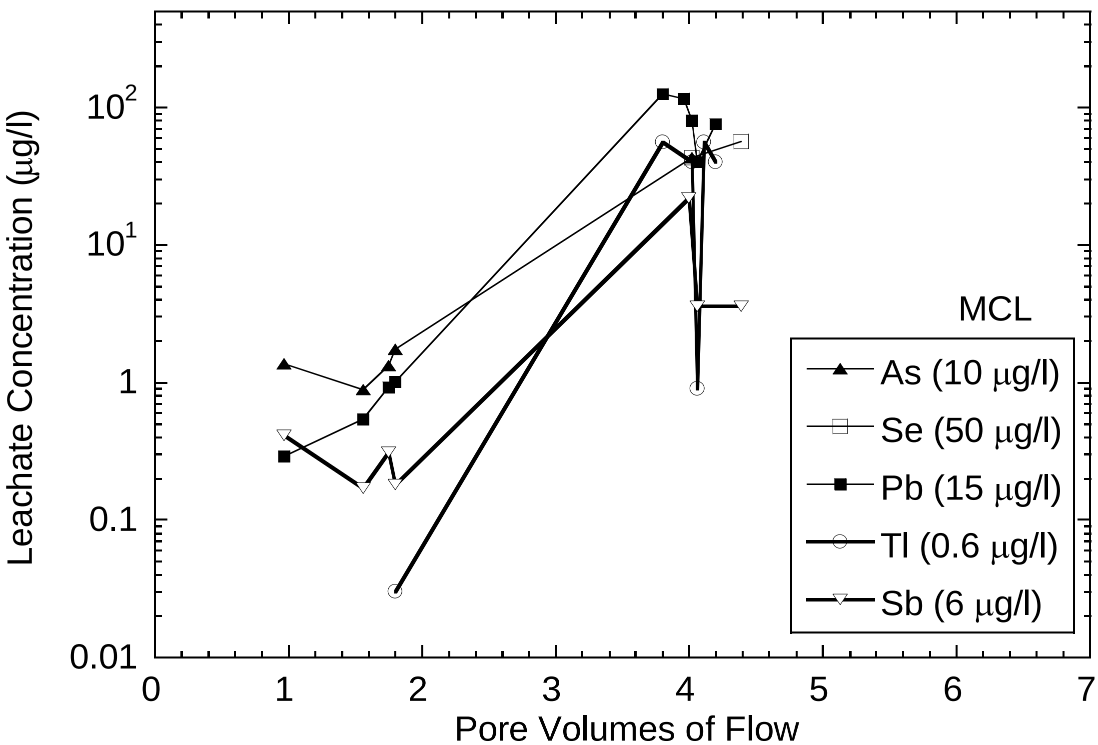

Fig. 9. Concentrations of select trace elements in lysimeter at Waseca site.

APPENDIX – SUMMARY OF CSAH 8 EXPERIENCE

FLY ASH SOIL STABILIZATION ON WASECA CSAH 8
LRRB INVESTIGATION 736
EXECUTIVE SUMMARY

17 October 2000

John Siekmeier, MnDOT Office of Materials and Road Research
Jeff Blue, Waseca County Highway Department

INTRODUCTION

The project is located on Waseca CSAH 8 south of Waseca, MN. The anticipated benefits of mixing coal fly ash into the silty-clay-loam subgrade were to dry and stabilize the subgrade and to increase subgrade stiffness and uniformity. Grading and ash stabilization occurred in the Fall 1999. Paving was conducted in Fall 2000. The ADT is estimated at about 750 vehicles per day and the soil type is a silty-clay-loam with a soil factor of 130. There are five one-half mile test sections that contain the following ash blends:

- 100% Class C fly ash
- 65% Sherco #3 ash, 35% Riverside #8 ash
- 100% Sherco #3 ash
- 65% Sherco #3 ash, 35% Class C fly ash
- Control section without ash

Fly ash is produced by burning pulverized coal in coal-fired boilers. The powdery ash is collected by electrostatic precipitators, baghouses, or mechanical devices such as cyclones. The various ash types were produced by NSP in Minnesota and supplied to the project by Mineral Solutions. The ash types and sources are Class C fly ash from NSP's Blackdog and High Bridge power plants, dry scrubber ash from the NSP's Sherco Unit 3 in Becker, MN, and cyclone ash from NSP's Riverside Unit 8 in Minneapolis, MN.

CONSTRUCTION

Construction was performed jointly by Midstate Reclamation and Trucking (Lakeville, MN) and the Waseca County Highway Department. The ash was blended into the soil at a rate of 14% by weight of dry soil to a depth of 8 inches. The total width was 40 feet (two 12-foot lanes plus 8-foot shoulders). The construction process included the following steps:

- Embankment construction
- Salvaged aggregate wind-rowed to sides
- Ash end-dumped on subgrade
- Ash spread with motor grader
- Dry-mixed with rotary mixer
- Wet-mixed as water was added at mixer
- Pad-foot vibratory compaction
- Pneumatic-tire compaction
- Smooth-drum compaction
- All compaction completed in two hours
- Subgrade shaped with motor grader
- Salvaged aggregate replaced
- 4 inches of new Mn/DOT Class 5
- Hot mix asphalt in 2000

The production rate at start up was about 0.33 mile per day and by completion had reached about 0.75 mile per day. A realistic estimate for future projects would be about one mile per day with an experienced construction team and few problems. The factors affecting production included delivery of ash and water, ability to measure mixing rates, weather, and equipment breakdowns.

TESTING

An extensive testing program was implemented to measure and document the results of this project. The tests included laboratory tests to develop target water content and density for maximum strength, field tests before, during, and after construction including long term tests, laboratory tests before and after ash addition for classification and mechanical properties, and environmental tests both field and laboratory. The results will be used to quantify both moisture and stiffness before and after ash addition and to assess environmental impacts. Preliminary results show an increase in stiffness and uniformity, but raise questions about environmental issues.

PAVEMENT DESIGN

The potential benefit of using ash for soil stabilization can be summarized by comparing a pre-ash pavement design, which is based the soil properties prior to ash addition, with a post-ash pavement design, based on modified soil-ash properties. To quantify the structural benefit of ash stabilization, the increase in the subgrade modulus must be measured. Also, an empirical relationship between the modulus and the R-value must be determined to use the existing design procedure recommended in the Mn/DOT State Aid Manual. Therefore the following mechanical properties were measured using both in situ and laboratory tests and related to one another using an empirical correlation.

The pre-ash properties were a soil factor of about 130 with an assumed R-value of 10. The actual mean R-value was 18.6 (standard deviation of 1.9). This results in a design R-value (mean minus one standard deviation) of 16.7, which was rounded to 15. Given this soil type (R-value 10 to 15) and the estimated traffic, the HMA thickness is 8 to 8.75 inches based on the design procedure recommended in the Mn/DOT State Aid Manual.

The pre-ash subgrade modulus was estimated using the empirical relationship to be 30.4 MPa (4400 psi) based on an R-value of 18.6. The actual measured modulus, based on falling weight deflectometer (FWD) measurements, was 31 MPa (4500 psi). The design modulus used for the pre-ash design (8 to 8 3/4 inches HMA) was 25.2 MPa (3700 psi) based on the design R-value of 15. An additional note of interest is that the mean measured moisture content at the time of in situ testing was 17.8% for the silty-clay-loam soil, which has a standard Proctor optimum moisture content of 18.0%.

The post-ash design used a design modulus of 42.8 MPa (6200 psi) based on the mechanical properties measured about one month after ash stabilization. The FWD modulus increased by a factor of about 1.7 for the composite soil-ash over soil structure and the dynamic cone penetrometer (DCP) modulus increased by a factor of about 3.7 for the soil-ash layer. In order to use the same design procedure recommended in the State Aid Manual, this post-ash composite modulus was converted to an estimated R-value of 27.5, which was rounded to 25 for design. The resulting pavement design has a HMA thickness of 7 inches. Again an interesting note is that the mean measured moisture content post-ash averaged 17.7% compared to the 17.8% pre-ash.

In summary, the possible affect on design would be a HMA thickness reduction of 1 to 1.75 inches. The equivalent structural sections would be a no ash section (subgrade R-value of 10 to 15) with 8 to 8-3/4 inches of HMA or an ash modified section (subgrade R-value of 25) with 7 inches of HMA. The reduction in HMA thickness would be based on the assumption that the subgrade modulus has been permanently modified by the ash addition. Retrieving additional undisturbed samples for laboratory testing or conducting additional in situ testing has not been purposed at this time. Only pavement surface quality testing is anticipated.

COSTS

The additional costs related to the ash modification portion of the project were $50,000 for ash material and delivery, $25,000 for the rotary mixer and operator, and an additional estimated amount for grading and compaction by county staff. Therefore the additional cost for ash modification was estimated to be about $1.60 per square yard given the 14% addition rate and 8-inch depth. This can be compared to the estimated cost of HMA. Given a 1 to 1.75 inch reduction in thickness and a cost of $1.25 per square yard per inch of HMA, it is estimated that $1.25 to $2.20 per square yard would be saved in HMA costs.

CONCLUSIONS AND RECOMMENDATIONS

For this Waseca CSAH 8 project, it was decided not to use a thin HMA layer, but rather to expect a longer design life and lower maintenance. Additional factors in this decision were that the paving and grading were part of a single contract, which complicated thinning the HMA after the contract had begun. Also, the construction transitions between ash-stabilized and non-stabilized sections were a consideration.

In summary, the ash improved stability and uniformity and was able to be used with minimal contracted equipment at a reasonable cost. However, several modifications are needed that would improve the construction process. Some areas for improvement include: controlling dust while dumping, spreading the ash uniformly, controlling water at the rotary mixer, and monitoring the moisture content before and after ash addition.

For future projects it is recommended that the grading and stabilization be included in the first contract with the intent of using ash to improve the worst locations. A bid price per square yard of ash stabilization is recommended. Following the grading and stabilization work, in situ testing and final pavement design would occur. The resulting pavement design would require less HMA since the design would be based on a more uniform subgrade where the worst-case locations had been improved. The final paving contract would then be less costly.

In conclusion, ash stabilization has been proven to be beneficial in providing a durable construction platform that can carry construction equipment and local traffic prior to and during HMA paving. Ash stabilization appears to be capable of providing relatively long-term pavement support, however additional observation and testing of pavement condition is required. Performance, in terms of ride (PSR) and pavement distress, should be monitored for five to ten years. Yet to be determined is resolution of several environmental issues concerning the possible toxicity of the ash. It is likely that future specifications will place strict limits on metal and chemical concentrations in the ash.