Characterization of Emulsion Aggregate Mixtures

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Emulsion aggregate mixtures (EAMs) are one type of low-quality cold-mixed asphalt concrete mixture. In Illinois, road-mixed and plant-mixed EAMs are used frequently as base or surface courses on low-volume rural roads (less than 1,000 average daily traffic). This research characterizes the EAM engineering properties that contribute to the observed performance of EAM pavements using laboratory testing. Dense-graded and open-graded aggregates treated with HFE-300 and CSS-1 emulsions are considered. Shear strength, repeated loading response (resilient modulus, permanent deformation), dynamic cone penetrometer, and curing effects are evaluated. The study concludes that EAMs are characterized as an improved granular material.

Emulsion aggregate mixtures (EAMs) are one type of low-quality cold-mixed asphalt concrete mixture. In Illinois, road-mixed and plant-mixed EAMs are used frequently as base or surface courses on low-volume rural roads (less than 1,000 average daily traffic). EAMs made with dense-graded and open-graded aggregates are placed frequently in 100-mm (4-in.) layers over existing pavements. Motor graders or conventional pavers are used to place the EAMs. At least a single surface treatment is generally applied before the first winter.

This study was conducted to characterize EAM engineering properties. The literature reports studies on EAM Marshall stabilities, tensile strengths, curing effects, and EAM pavement fatigue performance. This laboratory study focuses on the pertinent EAM engineering properties of shear strength, repeated load response, and dynamic cone penetrometer (DCP) indications.

LITERATURE

Research in the 1970s reported that EAM material properties improve with moisture loss. Darter et al. (*1*) reported Marshall stabilities ranging from 2 to 13 kN (500 to 3,000 lbf), depending on the emulsion type, and tensile strengths at generally less than 165 kPa (24 psi). Diametral modulus values are reported by Schmidt et al. (*2*) on the order of 1034 to 3445 MPa (150 to 500 ksi) at 23°C (73°F) after 30 days of curing.

Some researchers evaluated EAMs in a fully cured condition (*2–4*). They reported that EAMs have the "ultimate" potential to obtain diametral moduli equivalent to those of hot-mix asphalt concrete (HMAC). Research for The Asphalt Institute's thickness design manual concluded that the EAM modulus was equivalent to that of HMAC, and therefore that EAM fatigue life was equivalent to that of HMAC (*5–7*).

Walter (*8*) and Hicks et al. (*9*) reviewed the performance of 237 km (148 mi) of open-graded EAMs in the Pacific Northwest; they concluded that open-graded EAMs are more resistant to fatigue cracking than HMAC. Santucci (*6*) reported that EAM pavements with greater than 20 percent voids seldom exhibit fatigue cracking. The San Diego Road Test (*10*) reported fatigue cracking of HMAC surfaces on EAM bases. However, it was not reported whether the EAM bases developed fatigue cracks. The field performance indicates that EAMs are not characterized with a fatigue life.

PRELIMINARY FIELD STUDY

A preliminary field study was conducted to review the performance of EAM pavements in Illinois. Approximately 195 km (121 mi) of EAM pavements with ages ranging from construction day to 20 years were reviewed (*11*). Structural distresses (rutting, fatigue cracking) were rare, thus precluding a formal pavement distress survey. The lack of fatigue cracking was confirmed through interviews with experienced users (R. Beyers, Emulsicoat, Inc., Urbana, Illinois; J. Renner, Louis Marsch, Inc., Morrisonville, Illinois; B. Miteff, Koch Materials Company, Chicago, Illinois; F. Cramer, Marshall County Highway Engineer, retired, personal communication, May 28, 1993; R. Nelson, President, Advanced Asphalt, personal communication, Princeton, Illinois, May 18, 1993; D. Johnson, Crawford County Highway Superintendent; P. Koberlein, Assistant County Highway Superintendent, Sangamon County; and S. Armon, Advanced Asphalt.)

DCP tests were conducted on representative sections. The DCP provides a rapid indication of in situ shear strength. The DCP values give the penetration rate in millimeters per blow (in./blow) for each drop of an 8-kg (17.6-lb) mass.

Typical EAM pavement penetration rates were approximately 3 mm/blow (0.12 in./blow). For comparison, typical high-quality HMAC DCP penetration rates were considerably less, approximately 0.5 mm/blow (0.02 in./blow) (*11*). The lower the penetration rate, the greater the shear strength.

Further testing was conducted using the falling weight deflectometer (FWD). FWD testing was limited to sections with generally 203 mm (8 in.) or more of EAM thickness. The FWD applied a 40-kN (9,000-lbf) load to a 305-mm (12-in.)-diameter plate. Deflection sensors were placed at 305-mm (12-in.) intervals to 914 mm (36 in.).

Various algorithms are available to backcalculate subgrade modulus and approximate EAM modulus using the FWD data (*12*). However, for EAM comparisons, the actual deflections were considered more appropriate. In particular, the deflection at the center of the load *D*0 and the AREA parameter describing the deflection basin were used.

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EAM *D0*'s of 0.6 mm (22 mil) were typical as compared with HMAC values of 0.3 mm (12 mil) and 0.9 mm (36 mil) for untreated aggregate bases (*13,14*). The EAM AREA was 457 mm (18 in.). The EAM AREAs are between the HMAC AREA of 559 mm (22 in.) and the untreated aggregate AREA of 356 mm (14 in.) (*13,14*). EAM FWD responses are bracketed between HMAC and untreated aggregate base responses.

LABORATORY STUDY

This laboratory investigation characterized the engineering properties that contribute to the favorable FWD and DCP responses and the general lack of EAM pavement structural distresses. To adequately characterize the responses, testing was conducted on several mixes prepared from five aggregates, two emulsions, and an asphalt cement. The laboratory tests included rapid shear strength, repeated loading, and DCP.

Materials

Five aggregates were used, as indicated in Table 1. Aggregates A, B, and C were for laboratory-mixed specimens. Aggregates D and E were from plant-mixed EAM stockpiles. Dense-graded aggregates were A, B, and D; open-graded aggregates were C and E.

Aggregate A was a partially crushed gravel with nonplastic fines. Aggregates B and C were crushed dolomitic limestone with a plasticity index of 4. Aggregate D was a natural river gravel, and Aggregate E was a crushed dolomitic limestone containing chert.

The two most commonly used emulsions in Illinois, HFE-300 and CSS-1, were selected for this research. The HFE-300 contained 67.5 percent residual asphalt cement, with a penetration in excess of 300. The CSS-1 contained 62.3 percent residual asphalt cement, with a penetration of 136.

Mix Designs

Mix designs are imperative in the laboratory but are used as general guidelines in the field. NCHRP Report 259 (*15*) reviewed the Illinois and The Asphalt Institute Hveem procedures and concluded:

> [A] precise laboratory design (even if obtainable) is not critical for achieving a successful pavement. At best it can only serve as a general guideline for an initial job-mix formula with adjustments being made following evaluation . . . [of] such factors as workability, coating, plasticity, and ease of compaction.

Dense-graded aggregate EAM mix designs were conducted using the Illinois method (*16*). The coating test described in Asphalt Institute Publication MS-19 (*17*) was used for the open-graded aggregates; combinations that produced 90 to 100 percent coating were used for this research. The residual asphalt cement (ResAC) contents of the mixtures are indicated in Table 2.

To bracket the EAM properties, testing was repeated on the untreated aggregates and a low-quality hot-mixed bituminous aggregate mix (BAM). The untreated dense-graded aggregates were compacted at 100 percent of AASHTO T-99 moisture and density, which is different from the EAM moisture and density.

The BAM mixes used Aggregates A, B, and C treated with AC-5 asphalt cement, with a penetration of 141. The Marshall method of mix design, using 50 blows per side, was conducted on the BAM mixes (*18*.) Table 2 indicates the asphalt cement contents.

EAM Compaction Moisture

The literature reported that EAM properties improve with moisture loss. There are three sources of water in an EAM at mixing: hygroscopic water in the aggregate, water added to improve coating, and water in the emulsion. The sum of these three sources is the total moisture content of the mix.

TABLE 1 Aggregate Gradations

	Percent Passing				
	Lab-Mixed			Plant-Mixed	
Sieve mm	A Dense	B Dense	C Open	D Dense	E Open
38	100			100	
25	96	100	100	97	100
19		97	91		
12	75	86	45	88	40
5	51	48	4	68	2
1	35	19	3	46	
0.43	22	14	2	25	
0.15	11	10		6	
0.075	9	9	1	4	

1 mm = 0.039 in.

TABLE 2 At-Test Specimen Conditions

Aggregate	Binder Type	ResAC%[a]	% T-99 Density[b]	Moisture Content, %
A Dense Lab-Mixed	none	none	100.0	6.3
	HFE-300	4.0	92.2	2.8
	CSS-1	4.0	92.2	2.3
	AC-5	4.0	100.7	none
B Dense Lab-Mixed	none	none	100.0	7.2
	HFE-300	4.0	90.6	1.5
	CSS-1	4.0	89.2	1.8
	AC-5	4.0	102.2	none
C Open Lab-Mixed	none	none	100.0[c]	0.1
	HFE-300	3.0	100.0	1.0
	CSS-1	3.0	100.0	1.0
	AC-5	3.0	99.1	none
D Dense Field-Mix	none	none	100.0	7.7
	HFE-300	3.4	90.0	0.6
E Open Field-Mix	none	none	100.0[c]	0.1
	HFE-300	2.9	100.0	1.8

[a] Residual Asphalt Cement content, percent of dry aggregate weight.

[b] AASHTO T-99 was conducted on the untreated dense-graded aggregates; EAM and BAM densities are by weight of dry aggregate, as a percentage of the T-99 density.

[c] Open-graded untreated aggregates were compacted to a maximum density without fracturing particles. EAM and BAM densities are by weight of dry aggregate, as a percentage of the maximum density.

The Asphalt Institute recommends compaction when 50 percent of the total moisture (at mixing) has evaporated (*19*). EAMs for this research were compacted after 50 percent of the total moisture had evaporated; testing was conducted immediately following compaction. Advantages of using the "50 percent drying" criterion for EAM properties are as follows:

1. Adequate performance is assured on construction day,
2. Estimates of time to "ultimate strength" are not required for structural design, and
3. EAM pavement properties improve with curing and traffic.

Density and Voids Analysis

Density and voids analysis for EAMs requires consideration of the moisture content. The density and voids analysis equations in NCHRP 259 (*15*) using the aggregate apparent specific gravity are recommended. The NCHRP 259 equations used are as follows:

$$\text{MC\%} = \frac{\text{mass of water}}{\text{mass of dry mixture}} \times (100 + \text{AC\%}) \qquad (1a)$$

$$\text{Dry agg. density} = \frac{\text{wet mix density}}{1 + \dfrac{\text{AC\%}}{100} + \dfrac{\text{MC\%}}{100}} \qquad (1b)$$

$$\text{VMA\%} = \left(1 - \frac{\gamma_d}{G_{sa}\,\gamma_w}\right) 100 \qquad (1c)$$

$$V_{amc}\% = \left(1 - \frac{\gamma_d}{G_{sa}\,\gamma_w} - \frac{\gamma_d\,\text{AC\%}}{100\,B\,\gamma_w}\right) 100 \qquad (1d)$$

$$V_{air}\% = V_{amc}\% - \frac{\gamma_d\,\text{MC\%}}{\gamma_w} \qquad (1e)$$

where

$AC\%$ = residual asphalt cement (% of dry aggregate),
$MC\%$ = moisture content (% of dry aggregate),
$VMA\%$ = voids in mineral aggregate,
$V_{amc}\%$ = air and moisture volume (%),
B = specific gravity of asphalt cement,
G_{sa} = apparent specific gravity of aggregate,
γ_w = density of water (62.4 pcf), and
γ_d = EAM dry aggregate density.

TEST PROCEDURES

Rapid shear strength and repeated loading tests were conducted on 152-mm (6-in.)-diameter by 305-mm (12-in.) cylindrical specimens under triaxial stress states. DCP tests were conducted on materials compacted in 203-mm (8-in.)-diameter by 381-mm (15-in.) cylindrical steel molds. All tests were conducted at 23°C (73°F).

Specimen Preparation

Compaction was with a full-faced pneumatic vibratory compactor, applying 134 N (30 lbf) of force per blow. The 305-mm (12-in.) triaxial specimens were compacted in five lifts; the 381-mm (15-in.) DCP specimens were compacted in six lifts.

The compacted triaxial specimens were encased in two membranes: an inner 0.8-mm (31-mil) neoprene membrane, and an outer 0.6-mm (25-mil) latex membrane. Aluminum loading platens were sealed to the specimen with the membranes and clamps. Confinement during handling was provided by a vacuum until the triaxial chamber was assembled. The triaxial chamber consisted of an aluminum base plate, lid, and an acrylic cell. After assembly, confinement was provided by compressed air.

Test Equipment

The rapid shear strength and repeated load tests (resilient modulus, permanent deformation potential) were conducted with an electrohydraulic loading apparatus manufactured by MTS Systems, Inc. (MTS). The MTS apparatus was fitted with a 44.5-kN (10-kip) ram and load cell. A function generator controlled the load pulse: either constant strain for the rapid shear test or haversine for repeated loading. A personal computer fitted with an 8-channel data translation analog:digital board triggered the MTS and recorded data. Total specimen deflections were measured using linear variable differential transducers (LVDTs).

RAPID SHEAR TEST

The rapid shear test defined the shear strength and the Mohr-Coulomb failure envelope. Three different confining pressures (σ_3) were used: 34, 103, and 207 kPa (5, 15, and 30 psi). Deviator stress ($\sigma_d = \sigma_1 - \sigma_3$) was applied axially at a constant strain rate of 38 mm (1.5 in.) per second. This rate corresponded to 5 percent strain in 400 msec. Total specimen deflections were measured using the LVDT in the loading ram.

Shear strength was defined as the maximum deviator stress. If there was no clearly defined peak stress, then the shear strength was defined as the deviator stress at 5 percent axial strain.

Using the failure deviator stresses and confining pressures, the Mohr-Coulomb failure envelope was determined by evaluating the best-fit regression line according to

$$\sigma_1 = a + b\sigma_3 \tag{2}$$

where

σ_1 = major principal stress, $\sigma_d + \sigma_3$,
σ_3 = minor principal stress (confining pressure), and
a, b = regression coefficients.

Then the cohesion (C) and angle of internal friction (ϕ) were evaluated by

$$C = \frac{a}{2\sqrt{b}} \tag{3}$$

$$\phi = \arcsin\frac{(b-1)}{(b+1)} \tag{4}$$

The relation between σ_1 and σ_3 can be expressed as

$$\sigma_1 = \sigma_3 N_\phi + 2C\sqrt{N_\phi} \tag{5}$$

where

$$N_\phi = \tan^2(45 + \frac{\phi}{2}) \tag{6}$$

This procedure has been used successfully in previous studies *(20,21)*.

REPEATED LOADING TEST

Cylindrical specimens 152 mm (6 in.) diameter by 305 mm (12 in.) were subjected to various triaxial stress states that were less than the failure stress states. Haversine load pulses were applied by the MTS with a load pulse duration of 0.1 sec (10 Hz). A 0.9-sec rest period between load pulses was used. Deflections were measured using external LVDTs mounted on the loading piston.

Permanent Deformation

An indication of permanent deformation (rutting) potential was determined from analysis of the conditioning sequence of the triaxial repeated load specimens. The repeated load specimens were conditioned with 1,000 applications of 310-kPa (45-psi) deviator stress at 103-kPa (15-psi) confining pressure before resilient modulus testing. Data recorded were permanent deformation, resilient deformation, and applied deviator stress. Measurements were made at 1, 10, 50, 100, 500, and 1,000 load applications.

The model used to evaluate permanent deformation, $\epsilon_p\%$, was

$$\epsilon_p\% = AN^b \tag{7}$$

where A is the antilog of a in $\log \epsilon_p\% = a + b \log N$ and N is the number of cycles.

TABLE 3 Resilient Modulus Stress States

Deviator Stress, σ_d kPa	Confining Pressure, σ_3 kPa	Bulk Stress, θ kPa	Principal Stress Ratio, σ_1/σ_3
68.9	34.5	172.3	3
103.4	34.5	206.7	4
137.8	68.9	344.5	3
206.7	68.9	413.4	4
206.7	103.4	516.8	3
310.1	103.4	620.1	4
310.1	206.7	930.2	2.5
413.4	206.7	1033.5	3

1 kPa = 0.145 psi

Resilient Modulus

After the conditioning sequence, resilient modulus testing was conducted at various stress states. One hundred load pulses at each of the stress states in Table 3 were applied. Principal stress ratios (σ_1/σ_3) varied from 2.5 to 4.

The resilient modulus was calculated after the 100th load application at each stress state. Resilient modulus is defined as

$$E_R = \frac{\sigma_d}{\epsilon_r} \tag{8}$$

where

E_R = resilient modulus,
σ_d = applied deviator stress, and
ϵ_r = recoverable axial strain.

Linear regression of the stress-sensitive response was used to obtain a best-fit equation:

$$\log E_R = a + n \log \theta \tag{9}$$

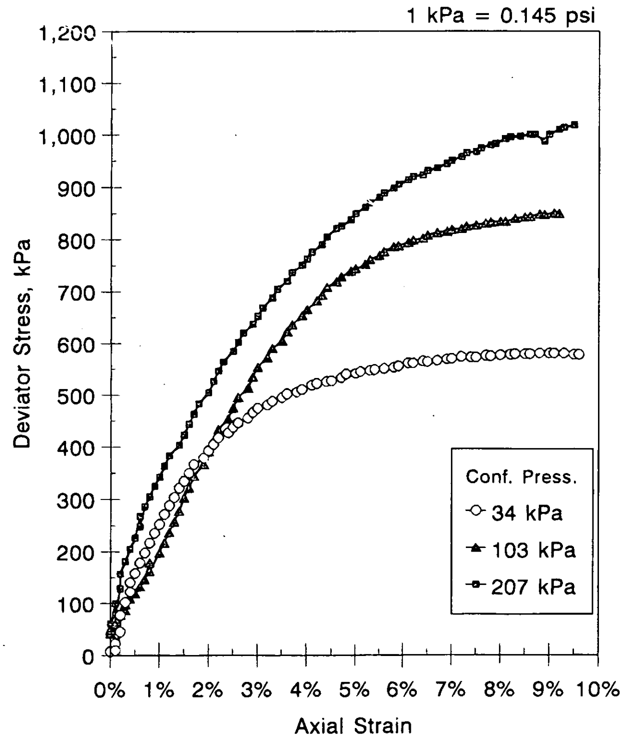

FIGURE 1 Aggregate A untreated rapid shear.

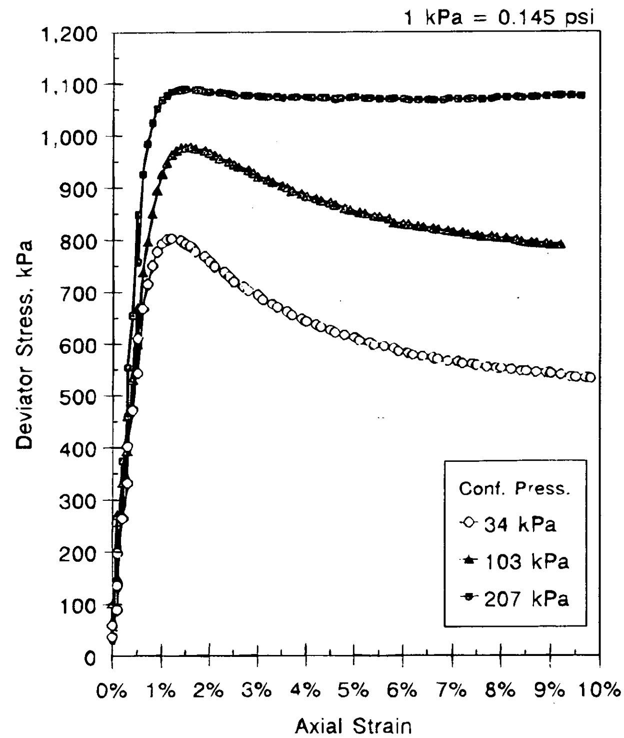

FIGURE 2 Aggregate A EAM rapid shear.

where θ = bulk stress, $\sigma_1 + \sigma_2 + \sigma_3 = \sigma_d + 3\sigma_3$, and a, n = regression coefficients.

The response was transformed into the θ model:

$$E_R = K\theta^n \qquad (10)$$

where K is the antilog of a in Equation 9. The stress sensitivity is depicted by n; an n of 0 represents a constant modulus material.

DYNAMIC CONE PENETROMETER TESTS

The DCP was used to obtain a rapid indication of in situ shear strength. Ayers (*20*) presented an extensive study concerning development and prediction of shear strength of granular materials using the DCP.

DCP testing in the laboratory was performed on the untreated aggregates and EAMs in a 203-mm (8-in.)-diameter by 381-mm (15-in.) cylindrical steel mold. The rod penetration after each anvil blow is the penetration rate, which was recorded in millimeters per blow.

SHEAR STRENGTH RESULTS

Figure 1 presents the response for dense-graded untreated Aggregate A for the three confining pressures. This response is a ductile failure, a typical one for untreated aggregates. There is no defined peak; therefore, failure stress is taken at 5 percent axial strain.

Figure 2 is for the same aggregate treated with 4 percent ResAC HFE-300 emulsion. The response is initially brittle, with a ductile postpeak failure, typical for the EAMs. Comparison of Figures 1 and 2 indicates that there is no substantial shear strength difference.

To quantify the difference between ductile and brittle responses, the secant modulus (E_{sec}) was used. Secant modulus was defined as the ratio of 50 percent of the failure deviator stress to the corresponding strain. Secant modulus is generally insensitive to confining pressure (*11*); thus, the average secant modulus of the confining pressures was used to characterize the stress-strain response. Table 4 shows that all of the EAM secant moduli increase in excess of 50 percent over the untreated aggregates.

TABLE 4 Properties from Triaxial Testing

Agg.	Binder Type	Shear Strength: $\phi°$ [a]	Shear Strength: C [a] kPa	Shear Strength: E_{sec} [b] MPa	Shear Strength: Shear Str. [c] kPa	Repeated Load: K [d] MPa	Repeated Load: n [d]	Repeated Load: ϵ_p% [e]
A Dense	none	27.6	158	21	642	8.8	0.493	2.2
	HFE-300	26.6	241	152	893	25.7	0.414	0.4
	CSS-1	42.3	262	193	1468	73.5	0.258	0.6
	AC-5	---	---	1288	10728 [f]	97.2	0.394	0.1
B Dense	none	41.0	117	103	777	11.6	0.557	1.0
	HFE-300	35.1	214	179	1009	24.4	0.440	0.4
	CSS-1	31.5	310	165	1258	53.7	0.341	0.4
	AC-5	---	---	1192	10245 [f]	304.0	0.252	0.1
C Open	none	44.7	55	41	591	16.9	0.492	1.1
	HFE-300	35.4	83	131	510	19.4	0.466	3.3
	CSS-1	---	---	172	978	19.7	0.481	0.2
	AC-5	17.6	627	213	1773	168.5	0.242	0.5
D Dense	none	45.4	41	117	542	3.2	0.742	0.6
	HFE-300	38.0	172	131	927	43.5	0.365	0.4
E Open	none	41.6	69	48	579	7.4	0.597	0.5
	HFE-300	33.1	96	131	522	60.3	0.330	1.4

[a] Mohr-Coulomb friction angle, ϕ degrees, and cohesion, C.
[b] Secant modulus, E_{sec} at 50% of peak stress.
[c] Deviator stress at σ_3=69 kPa (10 psi).
[d] Resilient modulus constants, $E_R = K\theta^n$.
[e] Predicted permanent strain, $\epsilon_p\% = AN^b$, at N =1000.
[f] Unconfined compressive strength using 445kN ram.

--- Specimen tested unconfined

1 kPa = 0.145 psi

The results of the numerous shear strength tests are presented in Table 4. For comparative purposes, the predicted shear strength (deviator stress) at 69-kPa (10-psi) confining pressure was calculated using Equations 5 and 6. Several general trends are evident from a review of the shear strength responses:

1. The untreated materials generally indicate a ductile response, whereas the EAMs exhibit a brittle failure, as demonstrated by comparison of Figures 1 and 2.
2. EAMs generally have a higher cohesion than the untreated aggregates; the friction angle generally decreases by emulsion treatment.
3. The dense-graded aggregate shear strength (σ_d) is improved with emulsion addition.
4. The shear strength of the open-graded aggregate EAM with HFE-300 is slightly less than the untreated aggregates. This is attributed to the heavy volatile constituents of HFE-300 emulsion. The secant modulus is still substantially improved over the untreated aggregate.
5. EAM shear strength is substantially less than the BAM shear strength.

REPEATED LOADING RESULTS

The repeated loading sequence evaluated rutting potential and resilient modulus. EAM rutting potential is considerably less than the potential rutting of untreated aggregates (Table 4). The predicted permanent deformation after 1,000 cycles generally decreases by 60 percent or more.

The open-graded aggregate EAMs (Aggregates C and E) increased in rutting potential with addition of HFE-300 emulsion. This increase is attributed to the heavy volatile constituents of high-float emulsions. However, field performance does not suggest that significant rutting occurs in EAM pavements made with HFE-300 or CSS-1 emulsions.

Referring to Table 4, the EAMs are less stress sensitive (lower n value) than the untreated aggregates. This trend holds for the dense-graded and open-graded EAMs. In all cases, EAMs have a larger K and lower n, which contribute to an improved modulus response.

Figure 3 compares the EAM modulus parameters for the θ-model from this research. Also demonstrated is the "typical" relationship for untreated aggregates from extensive research by Rada and Witczak (*22*). The EAMs indicate an improved modulus response resulting from the improvement in the K and n parameters.

The increase in K is related to an increase in shear strength, as illustrated in Figure 4. Figures 3 and 4 include results from testing in this research and additional testing, described further by Anderson (*11*), on untreated aggregates, EAMs, and BAMs.

DCP RESULTS

Table 5 summarizes the DCP penetration rates. The responses indicate that emulsion treatment decreases the penetration rate to approximately one-third of the untreated aggregate values. Figure 5 demonstrates the average penetration rate for the top 152 mm (6 in.) for Aggregate C. At depths greater than 152 mm (6 in.) in the laboratory mold, overburden effects reduce the differences between materials.

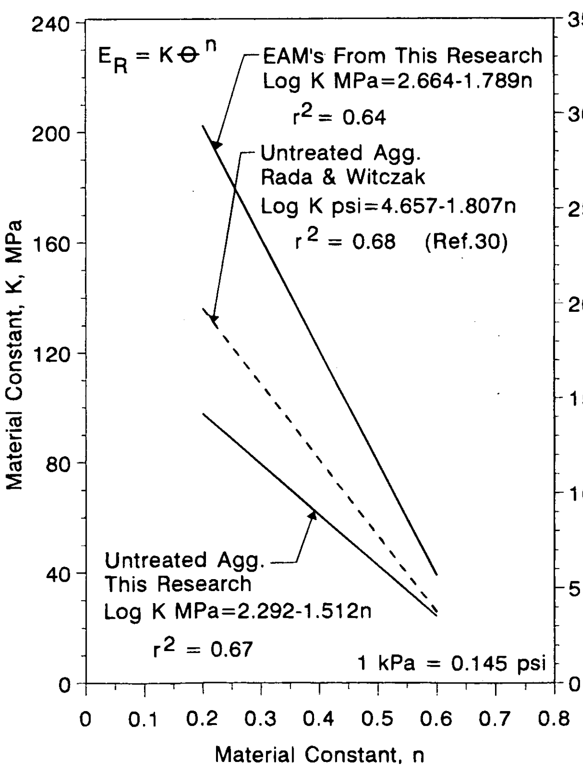

FIGURE 3 Comparison of modulus parameters.

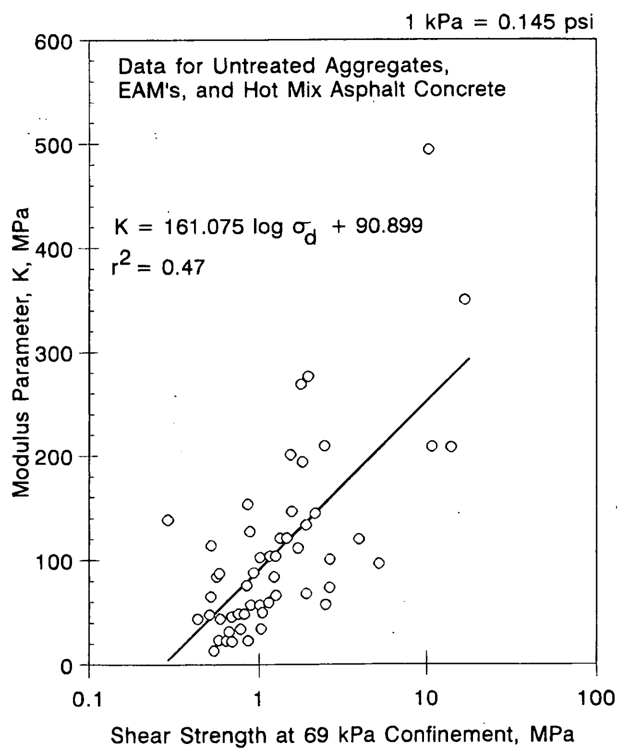

FIGURE 4 Modulus K and shear strength.

TABLE 5 Laboratory Dynamic Cone Penetrometer

Aggregate	DCP Penetration Rate, mm/blow	
	Untreated	EAM w/HFE-300
A	15	3
B	9	3
C	27	10
D	7	5
E	---	7

--- Test not conducted

1 mm = 0.039 inch

The DCP illustrates an important effect of the emulsion on a typical open-graded aggregate (Figure 5). Open-graded, untreated aggregates are dependent on aggregate interlock for strength. Confinement or cohesion, or both, is needed to maintain interparticle contact.

The cohesion required is apparently small, as 28 kPa (4 psi) was sufficient for Aggregate C (Table 4). This small amount of cohesion significantly increased the shear strength, as indicated by the penetration rate. The DCP penetration rate decreased from 76 mm (3.0 in.) to 13 mm (0.5 in.) (Figure 5). The cohesion permits open-graded EAMs to function as a stabilized pavement layer.

The relation indicated in Figure 6 can be used to convert DCP penetration rate to California bearing ratio (CBR). Figure 6 was developed from work by Kleyn (*23*) for a wide range of stabilized materials; similar correlations exist for unconfined compressive strength (*24*, *25*). The EAM penetration rates in Table 5 correlate to a CBR of approximately 100 for the dense-graded EAMs and 24 for the open-graded EAMs.

MIXING EFFECTS

Table 4 indicates the trends for the plant-mixed EAMs, Aggregates D and E. Aggregates D and E had been stockpiled for approximately 2 months before sampling and testing. The plant-mixed specimens indicate the same trends and responses as the laboratory-

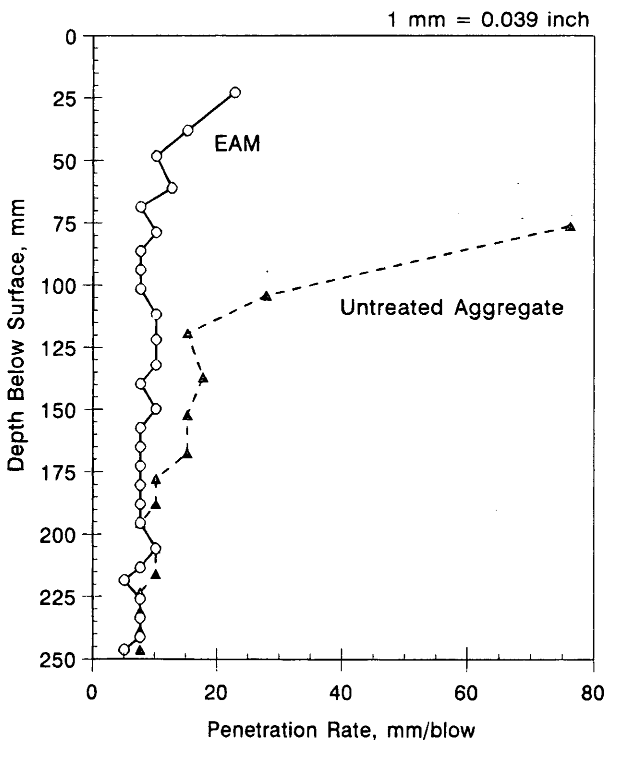

FIGURE 5 Aggregate C DCP.

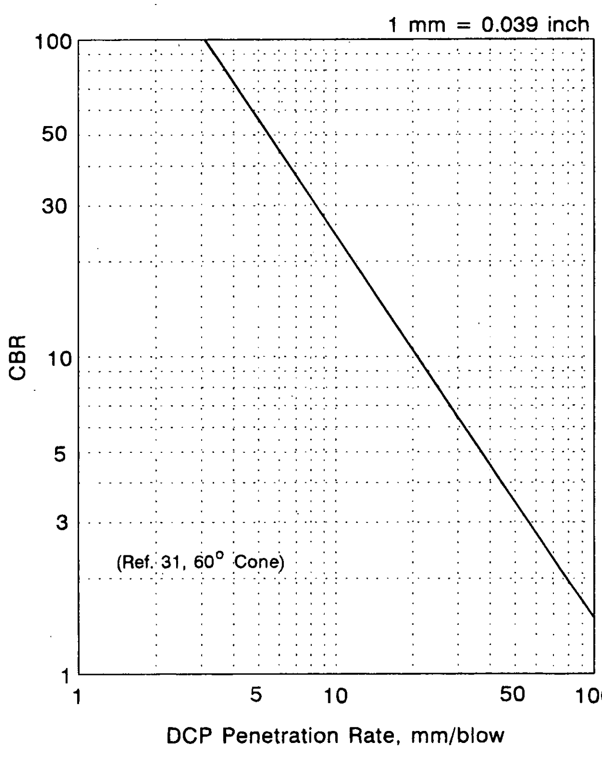

FIGURE 6 DCP-CBR conversion.

mixed specimens; emulsion improves the engineering properties of the aggregate.

BINDER EFFECTS

Table 4 indicates comparisons between HFE-300 and CSS-1. The consistent trend is that CSS-1 emulsion improves EAM shear strength and modulus response more than HFE-300. This trend is more apparent for dense-graded than open-graded EAMs. However, both types of emulsions provide acceptable EAM performance.

Table 4 also compares the EAMs with BAM-type mixes using AC-5. There is substantial improvement in all responses using asphalt cement to treat the aggregate. The BAM shear strengths are an order of magnitude greater than those of the EAMs. The modulus responses are also increased significantly, and the permanent deformation potential is reduced greatly.

To evaluate an EAM "fully cured" strength, additional specimens were compacted at the 50 percent moisture-loss condition and then cured to constant weight for several months. The curing conditions were 38°C (100°F) and 25 percent relative humidity.

Table 6 demonstrates the results of the fully cured EAM specimens. Comparison with the AC-5 results in Table 4 indicates that the EAM properties are significantly less than the BAM mixes. The important conclusion is that EAMs do not have the potential to ultimately attain properties similar to HMAC.

CONCLUSIONS

EAM properties and responses are not constant, but vary with moisture content. Tables 4–6 summarize typical EAM engineering properties at various cure conditions. Specific properties and trends that were identified are as follows:

1. EAM engineering properties improve with increased moisture loss.
2. The magnitude of EAM shear strength is not improved greatly over the untreated granular material, but the EAM secant modulus is significantly higher than the untreated aggregate.
3. Dense-graded EAM shear strength generally exceeds open-graded EAM shear strength.
4. EAM modulus is improved over the untreated aggregate modulus.
5. EAM modulus is stress hardening, that is, E_R increases with an increase in bulk stress, θ.

The obvious trend is that EAM engineering properties (shear strength, resilient modulus, secant modulus, rutting potential, DCP) improve with increased moisture loss. EAM engineering properties continue to improve with curing after compaction. However, even after curing to constant weight, EAM properties are not equivalent to those of HMAC. Thus, EAMs are displaying characteristics of an improved granular material.

TABLE 6 Fully Cured EAM Triaxial Properties

		Shear Strength				Repeated Load		
Agg.	Emulsion	ϕ^{oa}	C^a kPa	E_{secb} MPa	Shear Str.^{c} kPa	K^d MPa	n^d	$\epsilon_p\%^e$
A Dense	HFE-300	---	---	675	2370^f	114.2	0.313	0.1
	CSS-1	---	---	413	5126^f	35.9	0.507	0.02
B Dense	HFE-300	36.7	427	138	1907	27.8	0.461	0.3
	CSS-1	---	---	482	3879^f	45.4	0.501	0.1
C Open	HFE-300	37.9	324	262	1544	126.7	0.239	1.4
	CSS-1	43.5	124	200	882	65.8	0.341	1.7
D Dense	HFE-300	44.0	296	158	1709	56.2	0.354	0.2
E Open	HFE-300	32.9	34	131	290	85.9	0.247	3.6

1 Mohr-Coulomb friction angle, ϕ degrees, and cohesion, C.
2 Secant modulus, E_{sec} at 50% of peak stress.
3 Deviator stress at σ_3=69 kPa (10 psi).
4 Resilient modulus constants, $E_R = K\theta^n$.
5 Predicted permanent strain, $\epsilon_p\% = AN^b$, at N =1000.
6 Unconfined compressive strength using 445kN ram.

--- Specimen tested unconfined

1 kPa = 0.145 psi

ACKNOWLEDGMENT

This paper is based on research results previously developed in Project IHR-527, Mechanistic Design for Local Roads, sponsored by the Illinois Department of Transportation.

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Publication of this paper sponsored by Committee on Characteristics of Bituminous Paving Mixtures to Meet Structural Requirements.