EVALUATION OF CERTAIN STRUCTURAL CHARACTERISTICS OF RECYCLED PAVEMENT MATERIALS

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Evaluation of Certain Structural Characteristics of Recycled Pavement Materials

INTRODUCTION

In recent years the reuse or recycling of pavement materials has proved to be economically feasible and functionally successful. Characterization of recycling approaches is usually based on (a) the procedure used to recycle the material, (b) the type of paving material to be recycled and the end products they are to produce or (c) the structural pavement benefit to be gained from the recycling approach. Although each category has its own merit in describing the purpose and applicability of a given recycling approach, in the mind of the pavement engineer it is the structural benefit to the pavement system that is most critical.

A pressing need exists to characterize the structural properties of recycled materials such that the characterization parameters may be used by pavement engineers in conventional design approaches. This study attempts to compare certain structural properties of recycled pavement materials to conventional pavement materials. This was done by laboratory testing of field cores and by in situ testing using the Dynaflect.

SCOPE

Twenty-six field recycling projects were evaluated. These projects were located in eleven states. In fourteen of the twenty-six projects the recycled asphalt concrete was used as a surface. In the other twelve projects the old asphalt concrete was broken down and recycled either in-place or at a central plant and used as a base course.

Table 1 summarizes the jobs studied in terms of the recycling process used (i.e., central plants or in-place), the material recycled, additional or virgin material and stabilizers or asphalt rejuvenators added.

PURPOSE

Laboratory investigation of the Hveem and Marshall stabilities, indirect tensile strengths, water susceptibility and resilient moduli of the recycled asphalt concrete materials showed the recycled materials to be comparable to conventional mixes for the projects studied (1). A thorough evaluation of these mixture properties was performed on U.S. 277, Abilene, Texas; the Rye Grass, Washington, project and the Hillsboro to Silverton Highway, Oregon. Here cores of the pavement were compared before and after recycling, and laboratory recycled mixtures from the projects were compared with the properties of laboratory molded conventional materials from the respective locations (1).

The purpose of this investigation was to estimate the performance of the recycled materials in a standard structural system and to evaluate the structural capability of the recycled materials relative to conventional materials based on in situ testing.

The methods selected for this evaluation were (1) the development of AASHTO structural coefficients for the recycled pavement materials and (2) a non destructive, in situ deflection analysis comparing the response of a recycled pavement section to a control, conventionally constructed section.

Table 1. Summary of Projects Where Recycled Pavement Materials Were Evaluated.

Project	Date of Construction	Resulting Material Constructed	Percent and Type of Recycled Pavement Material	Percent and Type of New Material	Percent and Type of Additional Asphalt Modifier or Additive	Recycling Process Used
Interstate 8, Gila Bend Arizona	1978	Surface and Base	100 Asphalt Concrete		1.2 Cyclogen L	Central, Drum Dryer
U.S. 666, Graham County, Arizona	1977	Surface	80 Asphalt Concrete	20 Coarse Aggregate	1.4 AR - 2000 1.4 Extender Oil	Central, Drum Mixer
11th Avenue, Hanford, California	1976	Surface	100 Asphalt Road Mix		3.0 (Approx.) SC-800	In-place
Russell Avenue, Fresno County, California	1977	Base	100 Asphalt Concrete		1.1 Cyclogen H E	In-place
18th Avenue, LeMoore, California	1977	Base	100 Asphalt Concrete		3.5 Cyclogen H E	In-place
Highway 45, Yolo, California	1976	Base	100 Asphalt Concrete and Existing Base		4.0 (Approx.) Lime	In-place
Elkhart, Indiana	1976	Base	100 Asphalt Concrete		SA-1	In-place
Kossuth County, Iowa	1976	Surface	70 Asphalt Concrete	30 New Crushed Limestone	3.5 AC-10	Central Drum Mixer

(Continued)

Table 1 - Continued

Project	Date of Construction	Resulting Material Constructed	Percent and Type of Recycled Pavement Material	Percent and Type of New Material	Percent and Type of Additional Asphalt Modifier or Additive	Recycling Process Used
U.S. 56, Pawnee County, Kansas	1977	Base	100 Asphalt Concrete		Section 1: 2.0 cement Section 2: 1.5 cement Section 3: 1.0 MC-800 Section 4: 1.5 cement and 1.5 AC-7	In-place
Interstate 69, Flint, Michigan	1976	Base for Shoulder	100 Existing Base		AC-200 or MC-800	In-place
Trunk Highway 94 Minnesota	1977	Surface and Base	50 Asphalt Concrete 50 Existing Base		2.5 AC (200-300 Pen)	Central, Thermo Drum
Interstate 15, Henderson, Nevada	1974	Surface	100 Asphalt Concrete		1.5 AR-8000 0.75 Paxole	Central, Drum Mixer
U.S. 50, Dayton, Nevada	1975	Base	100 Asphalt Concrete and Existing Base		2.5 Cement	In-place
U.S. 93, Wells, Nevada	1975	Base	100 Asphalt Concrete and Existing Base		1.5 Cement	In-place
Ponderosa Avenue, Inclined Village, Nevada	1975	Base	100 Asphalt Concrete and Existing Base		4.0 Cement	In-place

(continued)

Table 1 - Continued

Project	Date of Construction	Resulting Material Constructed	Percent and Type of Recycled Pavement Material	Percent and Type of New Material	Percent and Type of Additional Asphalt Modifier or Additive	Recycling Process Used
Hillsboro to Silverton Highway, Woodburn, Oregon	1977	Surface	70 Asphalt Concrete	30 Crushed Limestone	1.8 AR-2000	Central, Drum Mixer
Interstate 20, Roscoe, Texas	1976	Base	69 Asphalt Concrete 16 Existing Aggregate Base	15 Crushed Limestone	2.3 AC-5	Central, Drum Mixer
Highway 36, Burleson County, Texas	1972	Surface	80 Portland Cement Concrete	20 Sand	4.8 AC-10	Central, Drum Mixer
U.S. 54, Dalhart, Texas	1972	Surface	100 Portland Cement Concrete		6.5 AC-10	Central, Drum Mixer
U.S. 84, Snyder, Texas	1976	Base	Section 1: 80 Asphalt Concrete	20 Base	5.0 EA-11 M	Central, Hot Pug Mill
			Section 2: 50 Asphalt Concrete	50 Base	6.0 EA-11 M	
			Section 3: 40 Asphalt Concrete	60 Base	6.0 AC-10	
			Section 4: 30 Asphalt Concrete	70 Base	7.0 AC-10	
			Section 5: 100 Asphalt Concrete		4.0 AC-10	

(continued)

Table 1 - Continued

Project	Date of Construction	Resulting Material Constructed	Percent and Type of Recycled Pavement Material	Percent and Type of New Material	Percent and Type of Additional Asphalt Modifier or Additive	Recycling Process Used
U.S. 277, Abilene, Texas	1975	Surface	100 Asphalt Concrete		5.0 EA-11 M 1.0 Reclamite	In-place
Loop 374, Mission, Texas	1975	Surface	100 Asphalt Concrete		Section 1: 1.6 Reclamite Section 2: 3.0 AC-5 Section 3: 2.0 Flux Oil	Central, Drum Mixer
U.S. 50, Holden, Utah	1975	Surface	Sections 7-17: 100 Asphalt Concrete		1.5(Approx.) AC-10	Central, Drum Mixer
			Section 18: 77 Asphalt Concrete	23 Aggregate	1.5(Approx.) AC-10	
			Section 19: 85 Asphalt Concrete	15 Aggregate	0.5 Softening Agent	
Blewitt Pass, Washington	1977	Surface	93 Asphalt Concrete Millings	7 Aggregate	0.5(Approx.) AC-5 0.5(Approx.) Softening Agent	Central, Drum Mixer
Rye Grass, Washington	1977	Surface	72 Asphalt Concrete Millings	28 Aggregate	0.75 Cyclopave	Central, Drum Mixer

DEVELOPMENT OF AASHTO STRUCTURAL COEFFICIENTS

General

Clearly the development of realistic AASHTO structural layer coefficients is a formidable task. In the first place these empirical coefficients which are the results of the AASHTO factorial experiments vary across a wide range. Secondly, the AASHTO Interim Guide (2), which is the design manual for the AASHTO pavement design method, provides no guidance for selecting structural coefficients for materials different from those used in the Road Test.

In order to estimate AASHTO structural coefficients a method must be developed which (1) is linked to the original AASHTO factorial experiments in terms of a performance related concept of pavement evaluation and (2) is based on some rational means of pavement evaluation.

The fundamental serviceability-performance equation developed at the AASHTO Road Test is the basis for developing structural coefficients.

$$\log N = \log \rho + \frac{G}{\beta}, \tag{1}$$

Where N = number of load repetitions

ρ and β = functions of load type, load magnitude and pavement sturcture

G = a damage function marked by loss in serviceability.

The structural coefficients developed for the AASHTO factorial experiments are used to develop the sturctural number, SN, which is in turn used to compute ρ and β. For a given type of loading at the Road Study site the performance of the pavement section was influenced solely by the structural number,

$$SN = a_1 D_1 + a_2 D_2 + a_3 D_3 \qquad (2)$$

Where D_1, D_2, D_3 = layer thickness

a_1, a_2, a_3 = layer structural coefficients of the asphalt concrete surface, granular base and subbase respectively.

The problem of establishing realistic structural coefficients becomes even more perplexing when one considers the sensitivity of the performance equation to the surface and base structural coefficients. This is clearly illustrated in terms of the a_2 by Darter and Devos (3). Darter and Devos used the special base study of the Road Test to estimate structural coefficients of the bituminous bases. This was done by comparing the performance of the various bases to the standard crushed stone base whose a_2 was established at 0.14. Based on this analysis the range of practical significance in a_2 for the various bases was 0.11 to 0.35. Darter and Devos evaluated the effects on the performance life of low volume roads of this range of a_2 values. The variation in performance life for one selected cross section was from less than one to well in excess of 20 years. A similar analysis (4) was performed over a realistic variation in a_1 values due to seasonal and temperature changes as determined by Van Til et al. (5). This analysis revealed a comparable sensitivity of the performance equation to a_1 values ranging from 0.30 to 0.50. Thus it is evident that the method selected to determine a_i's must be sensitive.

Criteria for Establishing Structural Coefficients

It is necessary to select a response from within the pavement structure to use as a basis of comparison when establishing structural coefficients. Three responses are generally considered in structural pavement analysis: (1) surface deflection, (2) maximum tensile strain in the bottom of the asphalt concrete and (3) vertical compressive strain.

Each of these responses was carefully evaluated as to its ability to estimate serviceability. Although the use of any one or all of the responses in establishing comparative criteria may be justified, the response of vertical deformation at the top of the subgrade was selected for the comparative analysis for several reasons. First, vertical subgrade deformation is directly correlated with performance, particularly in terms of riding quality and possibly rut depth. This point was verified by Jung and Phang (6) who studied the performance of pavement design in Ontario. Jung and Phang used layered elastic theory to arrive at stresses, strains and deformation within the pavement structure in hopes of establishing a distress mechanism that would provide a practical design criterion. Through this process of testing different cases, it was finally discovered that only the vertical deformation on the top of the subgrade emerged as the parameter which could be made to remain constant for a certain level of performance within each traffic or load class. This discovery pointed in the same direction as the results of previous research on the Brampton Road Test (7).

Second, vertical subgrade deformation has been shown by layered elastic theory to correlate well with performance loss in the AASHTO Road Test. In fact it has been shown to correlate better than maximum tensile strain in the bottom of the surface asphalt concrete layer (8,9).

Third, consider the AASHTO serviceability equation.

$$PSI = 5.03 - 1.91 \log (1+SV) - 1.38 \overline{RD}^2 - 0.01(C+P)^{\frac{1}{2}}, \qquad (3)$$

Where PSI = presents serviceability index

SV = slope variance

$\overline{RD}$ = rut depth

C+P = area of class 2 and 3 cracking plus patching per 1000 square feet.

This equation was developed statistically as a means to correlate the present serviceability rating, PSR, to physical pavement distress parameters. Obviously the serviceability is primarily sensitive to slope variance and is relatively quite insensitive to cracking, patching and rut depth. Mechanically, the criterion most closely related to slope variance is vertical subgrade deformation.

Although it may be argued that maximum tensile strain in the asphalt concrete should be the controlling criterion, the authors feel that the response of subgrade deformation is more sensitive to the contribution of any selected layer (surface or base) to the performance (according to the AASHTO definition) of the total system. Thickness of the asphalt bound layer as a portion of the total equivalent thickness probably is better determined by tensile strain.

The vertical subgrade strain is equivalent to subgrade deformation as a criterion of comparison when the subgrade modulus remains constant. Because the AASHTO subgrade has been shown to be highly stress dependent and thus to vary depending on the effective thickness of the pavement, the applied load and layer elastic moduli, it was decided to use subgrade deformation in lieu of subgrade strain. Thus, subgrade deformation is a normalized criterion.

Modeling the Pavement Structure

The structural layer coefficient of a recycled material used as a surface or base course must be evaluated within the same structural system as used for the AASHTO Road Test due to the system's interdependency. In order to do this it is necessary to model the AASHTO structural pavement system using rational pavement systems models. There are basically three models available for such analysis: visco-elastic layer models, stress dependent finite element models and stress dependent layered elastic models. Of course, linear elastic and finite element models are also available but are not considered adequate for this analysis.

The visco-elastic layer model requires special laboratory testing to develop visco-elastic input parameters. In addition, the laboratory material properties have not been correlated to field performance and experience with this type model is very limited.

Both the finite element and elastic layered stress sensitive models are adequate to model the AASHTO structural systems. The stress sensitive layered elastic model was selected because it is less expensive to run and due to the large number of runs required. The Chevron five layered elastic system program with iterative and superposition capability was used (10).

Material Characterization

Another important reason for selecting a stress dependent layered elastic model is that the materials used in the structural composition of the Road Test sections have been rather thoroughly characterized in terms of stress dependent elastic models.

One of the most thorough characterizations of the AASHTO pavement material was performed by Finn et al. (11). The results of this characterization is summarized in Table 2. Here the dynamic moduli of the various materials comprising the structural pavement system are characterized according to season. The resilient modulus models of the unbound base, subbase and subgrade materials are in reasonable agreement with the work of other researchers. (12,13).

The seasonal range in the resilient moduli of the subgrade was established by McCullough's procedure (14) based on a trial and error method of adjusting the subgrade modulus in order to obtain a reasonable match of measured deflection on the surface of the pavement.

Approach to Establishing AASHTO Structural Coefficients

The mechanistic response of subgrade deformation was selected as the response most closely associated with a change in pavement performance as measured by the serviceability concept. Thus an effort was made to correlate AASHTO Road Test performance data with vertical subgrade deformation as computed by the stress sensitive layer elastic Chevron program using the dynamic moduli in Table 2.

Loop 4 of the Road Test was selected for this analysis. The average annual pavement temperature throughout the duration of the Road Test (15) was used to select a weighted average dynamic modulus of the asphalt concrete surface as well as resilient moduli of the base, subbase and subgrade respectively. These average selected moduli are:

Table 2. Seasonal Dynamic Moduli of AASHTO Road Test Materials [after Finn, et al. (10)].

Material	Seasonal Moduli (Psi)			
	Oct-Nov (Fall)	Mar-Apr (Spring)	May-Aug (Summer)	Dec-Feb (Winter)
Asphalt Concrete, (E^*)	0.45×10^6	0.71×10^6	0.23×10^6	1.7×10^6
Base, M_R	$4000\Theta^{0.6}$	$3200\Theta^{0.6}$	$3600\Theta^{0.6}$	50,000[a]
Subbase, M_R	$5400\Theta^{0.6}$	$4600\Theta^{0.6}$	$5000\Theta^{0.6}$	50,000[a]
Subgrade, M_R	$27,000\ \sigma_d^{-1.06}$	$8000\ \sigma_d^{-1.06}$	$18,000\ \sigma_d^{-1.06}$	50,000[a]
Temperature (°F)	70	59	85	30

[a]Assigned values assuming frozen conditions.

(1 psi = 6,894 Pa)

$$E_{AC} = 700{,}000 \text{ psi} \qquad (5515 \text{ MPa})$$

$$E_{Base} = 4000\ \theta^{0.6}$$

$$E_{Subbase} = 5400\ \theta^{0.6}$$

$$E_{Subgrade} = 18{,}000\ \sigma_d^{\ -1.06}$$

Where θ = bulk stress and σ_d = deviatric stress.

Loop 4 was selected because the load applied to this loop was the standard 18,000 pound (80.0 KN) single axle load on lane 1 and a 32,000 pound (142.3KN) tandem on lane 2. In addition the computed structural coefficients for the structural layers within this loop were exactly the same as for the entire factorial experiment (i.e., $a_1 = 0.44$, $a_2 = 0.14$ and $a_3 = 0.11$) (15). Thus loop 4 is a very representative portion of the total experiment.

Based on the average selected dynamic moduli the subgrade deformations were computed for each section of loop 4. The deflection selected was the maximum vertical subgrade deflection beneath a dual wheel load of 4500 pounds (20.0 KN) per wheel. These deformations together with the weighted number of 18,000 pound (80.0 KN) single axle load applications to a terminal serviceability of 2.5 are shown in Table 3. These data are in turn plotted in Figure 1. As can be seen a rather sound correlation exists as is evidenced by the fact that 81 percent of the total variation in the number of 18,000 pound (80.0 KN) single axle equivalents to a terminal serviceability of 2.5, $N_{18(2.5)}$, can be explained by the best fit regression equation:

$$N_{18(2.5)} = 0.098\ e^{-3.39 \ln W_s},$$

Where W_s = vertical subgrade deformation.

Table 3. Summary of Computed Subgrade Deformation from Non-Linear Layered Elastic Theory and Actual Load Applications to a Terminal Serviceability of 2.5 (loop 4 of AASHTO Road Test)

Design Section			Subgrade Deformation, in.	N_{18}^{*} (2.5) lane 1	N_{18}^{*} (2.5) lane 2
3	0	4	0.03068	1,349	10,789
3	0	8	0.02371	34,198	40,179
3	0	12	0.01674	80,353	129,420
3	3	4	0.02588	40,179	43,152
3	3	8	0.01869	80,353	101,859
3	3	12	0.01282	476,431	529,663
3	6	4	0.02068	74,989	74,989
3	6	8	0.01411	88,512	186,209
3	6	12	0.01028	679,204	489,779
4	0	4	0.02006	63,242	82,985
4	0	8	0.01617	139,637	132,739
4	0	12	0.01232	194,984	476,431
4	3	4	0.01732	101,859	107,399
4	3	8	0.01334	129,420	147,911
4	3	12	0.01029	755,092	690,240
4	6	4	0.01435	101,859	138,038
4	6	8	0.01108	872,971	809,095
4	6	12	0.00933	778,036	—
5	0	4	0.01402	104,713	132,739
5	0	8	0.01191	151,356	165,196
5	0	12	0.00976	557,186	712,853
5	3	4	0.01249	161,436	181,134
5	3	8	0.01034	331,131	724,435
5	3	12	0.00852	503,500	957,194
5	6	4	0.01090	622,300	770,903
5	6	8	0.00899	529,663	532,108
5	6	12	0.00749	—	—

(1 in. = 25.4 mm)

* Number of 18 kip (80.0 kn) single axle load applications to a terminal serviceability of 2.5.

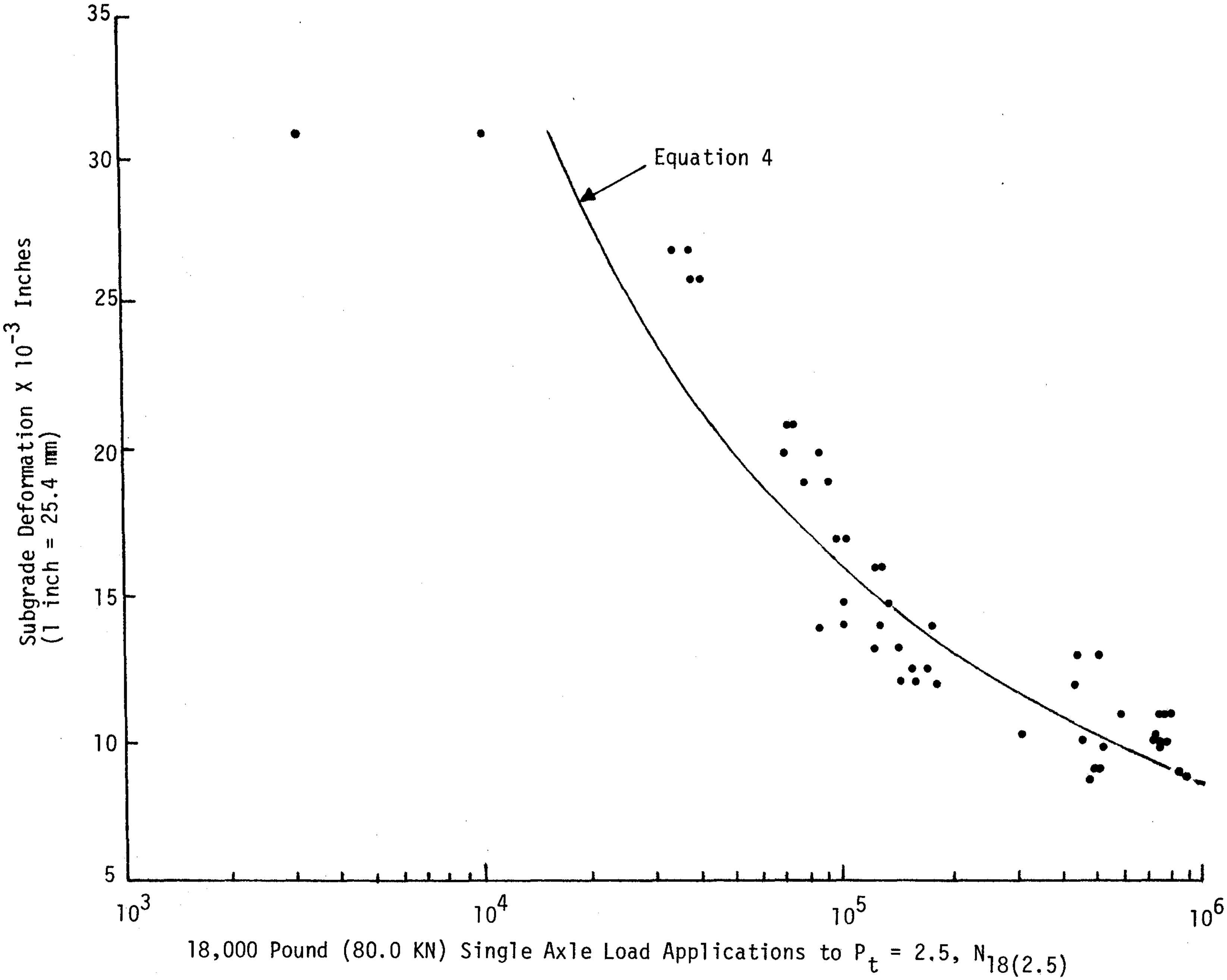

Figure 1. Relationship Between Subgrade Deformation and Load Applications to Terminal Serviceability in Loop 4.

Thus it appears that a reasonable correlation exists between the subgrade vertical deformation based on rationally derived pavement material dynamic moduli and the rate of pavement serviceability loss.

The next step was to characterize the elastic properties of the recycled asphalt materials and to substitute these properties in place of those of the corresponding conventional material used at the Road Test and to evaluate by the stress sensitive layer elastic model the response W_s. Knowing W_s, equation 4 may be used to evaluate $N_{18(2.5)}$. Finally, the general AASHTO flexible pavement performance equation written in terms of an 18,000 pound single axle load and a terminal serviceability of 2.5 was used to select a value of a_i (recycled layer) which would yield the same value of $N_{18(2.5)}$ as was obtained from equation 4. This was possible as the only unknown structural coefficient was that of the recycled layer. All other layers were assigned the weighted average structural coefficient established for that layer at the Road Test.

Characterization of Recycled Asphalt Materials

The resilient modulus of the recycled asphalt concrete materials was determined by the diametral resilient modulus test (16). The diametral test is basically a repetitive load test using the stress distribution principles of the indirect tension test. Schmidt (16) has shown that the results of the diametral resilient modulus are in reasonable agreement with the triaxial compression repeated load tests.

The major disadvantages of diametral testing is the effects of confining pressure on the resilient modulus cannot be determined. It is important to note, however, that even though the lower portion of a full depth asphalt pavement may be in a triaxial stress state, the compressive vertical stress and tensile radial and tangential stresses are similar to the stress state of the diametral specimen (3).

Of course, the major advantages of the diametral tests are that it is more rapid and convenient than the triaxial or flexural beam test. Thus it was possible to evaluate a large number of field core samples from each project over the following temperature range: -10°F (-12°C), 32°F (0°C), 73°F (23°C) and 100°F (33°C).

It is recognized that to base any structural analysis solely upon a measure of stiffness is a shortcoming of the analysis. The major structural properties of asphalt stabilized layers include the resilient modulus, fatigue and permanent deformation. A complete structural coefficient, a_i, of an asphalt stabilized layer is probably related to all of these factors in a composite way since they all affect performance. However, all of these structural properties are interrelated for asphalt stabilized materials such as the dependence of fatigue and permanent deformation on the resilient or dynamic modulus. Even though a_i may be dependent on a variety of factors and interactions of factors, the resilient or dynamic modulus <u>may</u> be the <u>single</u> most significant property to be correlated to a_i.

The weighted average resilient modulus of the recycled material in question was substituted for the corresponding AASHTO material as previously discussed. This weighted average was computed based on a pavement temperature histogram developed at the Road Test. Thus the weighted average resilient modulus of the recycled material represents a reasonable characterization of this material had it been used in loop 4 of the AASHTO Road Test.

Results of a_i Determination

To verify that the procedure illustrated in Figure 2 yields realistic values of the structural coefficients of surface and base layers, coefficients were calculated for each layer and design section of the model of loop 4. Table 4 summarized these results. Equation 1 in terms of 18 kip (80.0 KN) single axle load applications was used to caluclate a_i of the layer in question so that the value of $N_{18(2.5)}$ computed from equation 4 as a function of W_s agreed with the value of $N_{18(2.5)}$ computed from equation 1 for that design section. This was accomplished by using the weighted average structural coefficient established at the Road Test for the other two layers and computing the a_i for the layer in question.

The range of structural coefficients computed for all design sections illustrates the variation in the cofficients between design sections. It is also important to note that the average computed structural coefficients are in agreement with the established coefficients.

Having verified that the procedure selected yields realistic values when the reference materials were input, the next step was to input the weighted average resilient modulus determined for each recycled material. This value was substituted for the AASHTO surface layer or the AASHTO base and subbase layer depending upon its actual use in the pavement system.

Results of this procedure are summarized in Table 5 for recycled surface courses and in Table 6 for recycled bases.

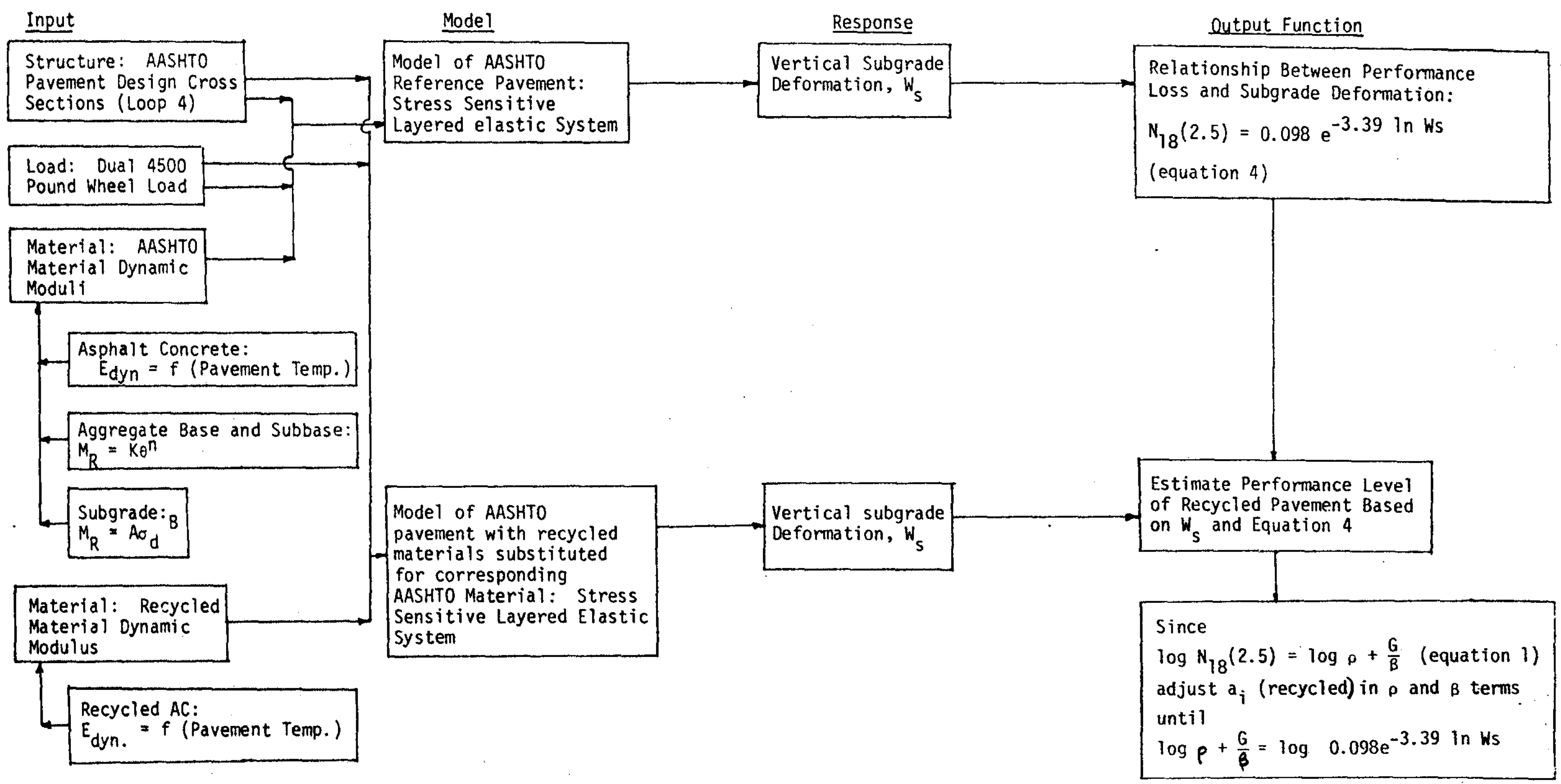

Figure 2. Methods Used to Estimate Structural Layer Coefficients for Recycled Materials.

Table 4. Verification of Procedure's Ability to Predict Realistic a_i's.

Design Section			Subgrade Deformation, W_s, in.	* $N_{18}(2.5)$	Structural Coefficients a_1	a_2	a_3
3	0	4	.03068	13,414	0.50	–	0.15
3	0	8	.02371	32,163	0.45	–	0.11
3	0	12	.01674	104,802	0.47	–	0.12
3	3	4	.02588	23,894	0.46	0.12	0.09
3	3	8	.01869	72,107	0.42	0.12	0.10
3	3	12	.01252	259,160	0.47	0.12	0.12
3	6	4	.02068	51,152	0.38	0.17	0.07
3	6	8	.01411	187,183	0.43	0.13	0.11
3	6	12	.01028	562,991	0.48	0.16	0.12
4	0	4	.02006	56,719	0.50	–	0.17
4	0	8	.01617	117,879	0.47	–	0.13
4	0	12	.01232	296,619	0.48	–	0.12
4	3	4	.01732	93,362	0.45	0.15	0.12
4	3	8	.01334	226,449	0.45	0.15	0.11
4	3	12	.01029	546,455	0.46	0.16	0.11
4	6	4	.01435	176,770	0.42	0.13	0.10
4	6	8	.01108	425,148	0.43	0.13	0.10
4	6	12	.009331	761,614	0.41	0.13	0.10
5	0	4	.01402	191,292	0.51	–	0.20
5	0	8	.01191	332,720	0.48	–	0.14
5	0	12	.009755	655,003	0.47	–	0.12
5	3	4	.01249	283,140	0.47	0.19	0.15
5	3	8	.01034	537,539	0.45	0.16	0.12
5	3	12	.008518	1,037,732	0.44	0.15	0.11
5	6	4	.01090	449,448	0.44	0.13	0.10
5	6	8	.008996	862,222	0.42	0.13	0.10
5	6	12	.007479	1,613,608	0.42	0.12	0.10
				Average	0.45	0.14	0.11

* $N_{18}(2.5)$ = Number of 18 kip(80.0KN) Single axle applications to a terminal serviceability of 2.5 as computed by Equation 4.

Table 5. Structural Coefficients of Recycled Asphalt Concrete Surfaces.

Recycled Pavement	Description	a_1
Interstate 8, Gila Bend, Arizona	100% Recycled AC + 1.2% Cyclogen L	0.46
U.S. Highway 666, Graham County, Arizona	80% Recycled AC + 20% Virgin Aggregate + 1.4% AR = 2000 + 1.4% Extender Oil	0.47
Kossuth County, Iowa	70% Recycled AC + 30% New Crushed Limestone + 3.5% AC-10	0.45
Trunk Highway 94, Minnesota	50% AC + 50% Existing Base + 2.5% AC (200-300 pen)	0.45
Interstate 15, Henderson, Nevada	100% AC + 1.5% AR - 8000 + 0.75% Paxole	0.55
Hillsboro to Silverton Highway, Woodburn, Oregon	70% AC + 30% Crushed Limestone + 1.8% AR - 2000	0.49
Highway 36, Burleson County, Texas	80% Crushed PCC + 20% Sand + 4.8% AC - 10	0.54
U.S. Highway 54, Dalhart, Texas	100% Crushed PCC + 6.5% AC - 10	0.44
U.S. Highway 277, Abilene, Texas	100% AC + 5.0% EA - 11M + 1.0% Reclamite	0.46

Table 5 - Continued

Recycled Pavement	Description	a_1
Loop 374, Mission, Texas	100% AC + 1.6% Reclamite (section 1) + 3.0% AC - 5 (section 2) + 2.0% Flux Oil (section 3)	 0.44 0.37 0.44
U.S. Highway 50, Holden, Utah	100% AC + 1.5% (Approx.) AC -10	0.59
Blewitt Pass, Washington	93% AC Cold Millings + 7% Virgin Aggregate + 0.5% (Approx.) AC - 5 + 0.5% (Approx.) Softener	0.46
Rye Grass, Washington	72% AC Cold Millings + 28% Virgin Aggregate + 0.75% Cyclopave	0.49

Table 6. Structural Coefficients of Recycled Bases Where the Recycled Bases were Stabilized with a Bituminous Binder (Characterized by Diametral Resilient Modulus versus Temperature).

Recycled Base	Description of Recycled Base	Reference Base Thickness, Index	Compute a_2
18th Avenue,	Crushed AC +	4	0.46
Le Moore,	3.5% Cyclogen	8	0.42
California	(Rejuvenator)	12	0.38
		avg.	0.38
Russel Avenue,	Crushed AC +	4	0.42
Fresno County	1.1% Cyclogen	8	0.38
California	(Rejuvenator)	12	0.35
		avg.	0.35
U.S. Highway 56,	Crushed AC + 1.5%	4	0.49
Pawnee Co., Kansas	Cement and 3.8%	8	0.46
(Section 2)	Water	12	0.42
		avg.	0.43
U.S. Highway 56,	Crushed AC + 1%	4	0.45
Pawnee Co., Kansas	MC - 800	8	0.41
(Section 3)		12	0.37
		avg.	0.38
U.S. Highway 56,	Crushed AC + 1.5%	4	0.49
Pawnee Co., Kansas	Cement + 1.5% AC - 7	8	0.45
(Section 4)	and 4% Water	12	0.39
		avg.	0.40
Trunk Highway 94,	Crushed AC +	4	0.50
Minnesota	Existing Base + 2.5%	8	0.45
	AC	12	0.41
		avg.	0.42
I20, Roscoe,	Crushed AC +	4	0.45
Texas	Existing Base +	8	0.40
	2.8% AC - 3	12	0.36
		avg.	0.37
U.S. Highway	Crushed AC +	4	0.56
84, Snyder, Texas	Base + 5%	8	0.51
	Asphalt Emulsion	12	0.48
	(Section 1)	avg.	0.49

In Situ Evaluation of Dynamic Modulus

In certain cases for the recycled pavements evaluated it was impossible to obtain suitable samples of recycled base materials by coring. This is because the materials would often crack during the coring process rendering the specimen irreparably damaged. The Dynaflect was used to approximate an in situ dynamic modulus of these recycled bases.

The Dynaflect is a trailer mounted steady state dynmaic force generator. It exerts two vertical loads separated by 20 inches (508 mm) and varying sinusoidally in phase at 8 Hz. The total load, extended by a rotating weight, varies from 500 pounds (2.22 KN) upward to 500 pounds (2.22 KN) downward. Five geophones located at 0, 1, 2, 3 and 4 feet (0, 0.35, 0.610, 0.914 and 1.219 M) measure the accelerations of the pavement surface at these locations. The geophones are connected to an analog-type integrator; the double integration of ground acceleration reflects the deflection of the monitoring point. Since the distance from the vibrator is known, a deflection profile can be plotted.

This deflection profile was used to evaluate the elastic modulus of the recycled layer. This was done by a trial and error analysis of selecting moduli of the layers within the system until the measured deflection basin was matched. This procedure would be nearly impossible if one had to search for the modulus of each layer in the pavement structure due to the interdependency of the effects of these moduli on the deflection basin shape. Fortunately a procedure was used so that only the modulus of the recycled layer was unknown.

The diametral resilient modulus over a range of temperatures was evaluated for the asphalt concrete sufaces from field cores of the pavement evaluated in this manner. Since the pavement temperature at the time of testing was known, a good approximation of the surface layer's resilient response to the Dynaflect load was obtained.

The resilient modulus of the subgrade under the light Dynaflect load was approximated with the help of layered elastic theory. To begin with, it can be easily shown using Boussinesg one-layer theory that the Dynaflect sensor at 4 feet (1.22 M) from the center of loading is a unique indicator of the elastic response of the subgrade to the Dynaflect loading. That is, under certain conditions, this deflection is insensitive to the material above the subgrade. This condition is expressed by the equation.

$$E_o\ (r) = \frac{\sigma_o\ ^2\ (1 - \mu^2)}{d(r)\ x\ r} \tag{5}$$

Where $E_o\ (r)$ = the modulus of the equivalent supporting layer yielding the surface deflection measured

σ_o = the surface stress level

α = radius of loading area

μ = Poisson's ratio of subgrade

$d\ (r)$ = surface deflection at radius r

r = distance of sensor 5 from center of load.

For this approximation to be correct the equivalent thickness of the pavement structure in terms of the subgrade, $h_{e,s}$, must be less than the distance r, $h_{e,s} < r$. This can be checked by Odemark transformation (17). For all pavement structural sections in this analysis r was greater than $h_{e,s}$. Thus, the fifth sensor reading was determined to be uniquely related to the subgrade modulus. A similar use of this concept was employed by Ullidtz (18) and Majidzadeh (19).

Figure 3 was developed using the Chevron 5 layer linear elastic computer program and was used to approximate the subgrade modulus under

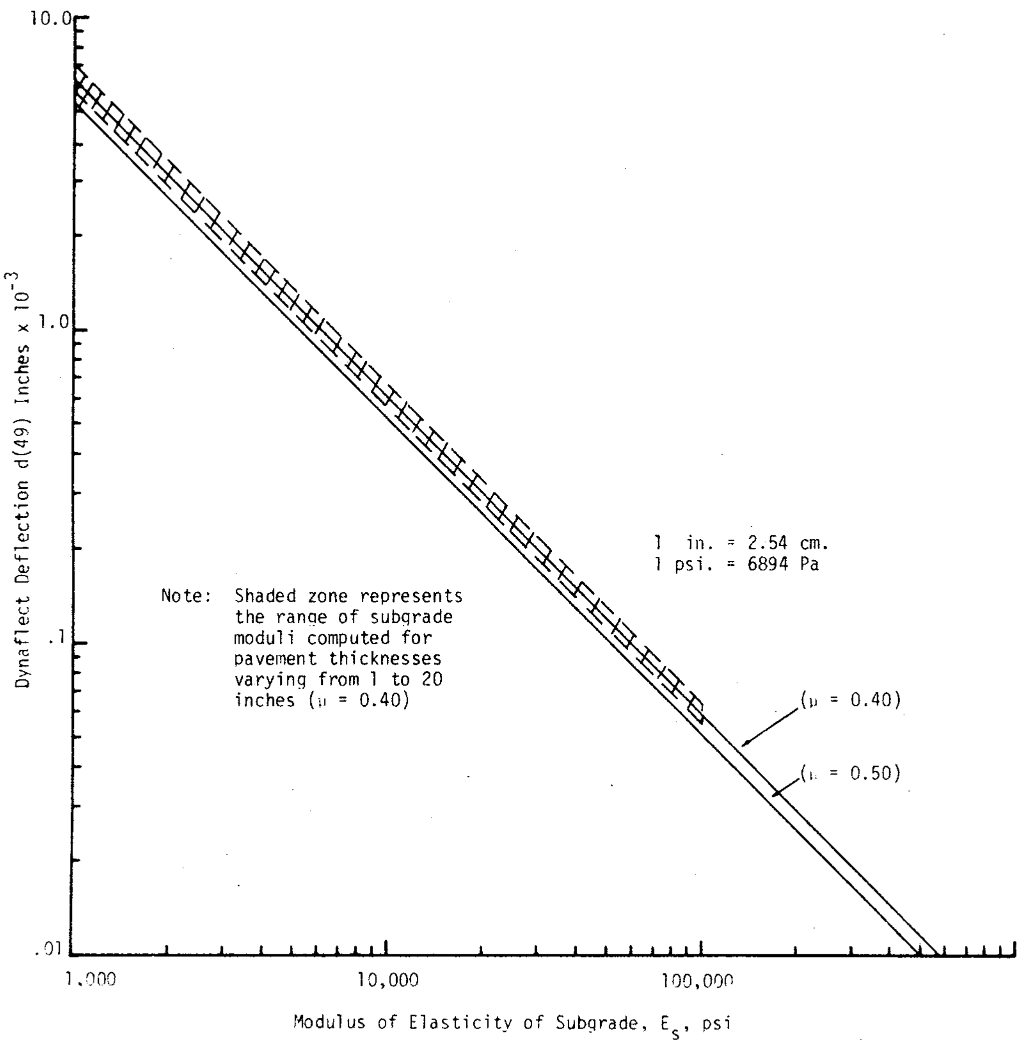

Figure 3. Relationship Between the Dynaflect Deflection of Sensor 5, d(49), and the Subgrade Modulus of Elasticity.

Dynaflect loading for the pavement in question as a function of the deflection recorded by geophone 5.

Since all pavement analyzed in this manner could be successfully approximated as 3-layer systems (surface, recycled and subgrade) the only unknown response was that of the recycled layer. The authors realize that this technique is crude in that it roughly approximates these moduli under a relatively light load. However, the recycled layers evaluated in this manner were either cement stabilized, lime stabilized or stabilized with a combination of chemical stabilizers. Thus, these fully cured bases should, if uncracked, exhibit a relatively small degree of stress sensitivity as well as temperature susceptibility. At least this was assumed.

The resulting in-situ dynamic modulus of these recycled materials together with the computed a_2 are shown in Table 7. The in-situ dynamic modulus was assumed to remain constant and was, therefore, input directly into the analysis for determining structural coefficients.

Discussion of the Results

Based on the structural coefficient evaluation , recycled materials used as surface courses are comparable to conventional asphalt concrete surfaces. In fact, the structural coefficients generally exceed that of 0.44 established for the Road Test surface layer. This essentially indicates that the recycled material are stiffer over the frequency of temperature evaluated. Generally, however, the recycled material is not greatly stiffer than the AASHTO standard. This may be important as very stiff recycled layers, especially thin layers, may insinuate potential fatigue cracking problems. Since some of the pavements evaluated had

Table 7. Structural Coefficients of Recycled Bases Where the Recycled Layer was Characterized by In-Situ Dynamic Testing.

Recycled Base	Description of Recycled Base	In-Situ Dynamic Modulus, psi	Reference Base Thickness, Inches	Computed a_2
Ponderosa Avenue, Inclined Village, Nevada	Crushed AC + Existing Base + Cement (Approx. 4%)	190,000	4 8 12 Avg.	0.34 0.31 0.27 0.26
U.S. Highway 50, Dayton, Nevada	Crushed AC + Existing Base + Cement (Approx. 2.5%)	120,000	4 8 12 Avg.	0.29 0.26 0.23 0.23
U.S. Highway 93, Wells, Nevada	Crushed AC + Existing Base + Cement (Approx. 2.5%)	700,000	4 8 12 Avg.	0.49 0.46 0.42 0.42
Highway 45, Yolo, California	Crushed AC + Existing Base and Some Native Subgrade + Approx. 4% Lime	600,000	4 8 12 Avg.	0.47 0.43 0.39 0.40
Elkhart, Indiana County Road 3	Crushed AC + Existing Base Stabilized with SA-1 Stabilizer	140,000	4 8 12 Avg.	0.31 0.28 0.25 0.26
Flint, Michigan Interstate 69	Crushed AC + Existing Base Stabilized with Asphalt (used as Shoulders	116,000	4 8 12 Avg.	0.27 0.24 0.22 0.22

been in service for as many as seven years and several had been in service for more than two years at the time of testing without significantly visible signs of distress, the stiffness response and the resulting structural coefficient is all the more meaningful.

The structural coefficient of the recycled bases is perhaps a more meaningful structural indicator than it is for surface layers. This is because it is the primary role of the base to protect the subgrade by adequately distributing stress and thus reducing strain. The high values of the recycled bases' structural coefficients indicate the great stress relieving ability of these base materials. This is obviously due to the much stiffer effects of the recycled, stabilized layers when compared to the AASHTO granular base.

The analysis used to evaluate base layer structural coefficient is highly sensitive to the thickness of the reference base as shown in Tables 6 and 7. The structural coefficient of the thinner bases are significantly greater than for the thicker bases. This is due to the much greater stiffness of the recycled bases than the reference unbound aggregate base and subbase system at the Road Test. The effect is somewhat mitigated at greater thicknesses.

Based on the average values of the structural coefficients for the recycled bases, these bases should be expected to perform comparably to the bituminous stabilized, and cement and lime stabilized bases at the AASHTO Road Test. For example, the structural coefficients of the bituminous stabilized bases at the Road Test ranged from 0.32 to 0.38 with a mean of 0.35 (1). The structural coefficients for cement treated bases ranged from 0.15 to 0.23 and for lime treated bases from 0.15 to 0.30 (1).

STRUCTURAL IN-SITU EVALUATION

Dynaflect Study of Recycled Pavements

The Dynaflect was selected as the method by which to evaluate the in situ recycled pavements. The Dynaflect was selected because of its versatility, accuracy and availability. The Dynaflect is used by many state highway departments where recycled pavements were evaluated and thus may be used for follow-on evaluation of the recycled materials.

For each pavement evaluated 30 Dynaflect readings within 1000 linear feet (307 M) were recorded at random locations on the recycled section as well as on an adjoining section of conventional pavement. That is, all pavement layers in the adjoining section were constructed of conventional materials and in a conventional manner. In most cases the adjoining section was cross-sectionally identical to the recycled section. In all cases, the adjoining section was free of visual signs of distress and appeared to be functionally satisfactory. The adjoining sections were used as reference or control sections with which to comparatively evaluate the recycled layer.

By recording 30 delfection readings within a 1000 ft. (307 M) lineal expanse of pavement, the deflection parameters could be statistically evaluated. A deflection parameter, for instance maximum deflection, could be evaluated from these 30 readings with a high probability that it represented the true deflection of the population recorded over the same 1000 ft. (307 M) section. In fact with 30 measurements the probability that the mean deflection parameter measured would be within twenty percent of one standard deviation from the population mean was over 95 percent. This was highly acceptable. The purpose of selecting a 1000 ft. (307 M) lineal distance was to minimize construction variables and concentrate on material properties.

The 30 deflection readings within each project were analyzed in terms of their distribution frequency. The Chi square goodness of fit, skewness and Kurtosis tests were performed, and the result was that the deflection data were approximately normally distributed. In fact normality could not be rejected in almost 80 percent of the cases. Thus, the statistical procedures developed for normally distributed data were valid.

The student t-test was used to determine whether a statistical difference existed between the recycled and control section of pavement. An example of this process is shown in Tables 8 and 9 where in each two cross-sectionally identical sections were compared. In the case of U.S. Highway 84, Snyder, Texas, the recycled section was superior in that the maximum Dynaflect deflection, d(10) (symbolizing the surface deflection at geophone 1 midway between the dual load wheel or 10 inches (254 mm) from each load); the surface curvature index, SCI (difference between the surface deflections recorded from geophones 1 and 2); and spreadability, S (ratio of the average surface deflection at each geophone location to d(10)) were all superior. That is d(10) and SCI were lower and S was greater for the recycled section. In addition, the estimated elastic response of the subgrade, E_s, under the Dynaflect load showed no statistical difference between sections indicating an identical foundation. This is crucial due to the great sensitivity of deflection to E_s.

On the other hand, the deflection parameters of U.S. 50 near Holden, Utah, indicate that the recycled section 7-17 are superior to control section 1-6 in terms of each parameter: d(10), SCI, S and E_s. Thus, it

Table 8. Statistical Comparison of Dynaflect Parameters Between Control and Recycled Section 4, U.S. Highway 84, Snyder, Texas.

Parameter Compared	Degrees of Freedom	Student's T-Statistic	Is the Difference Significant? $\alpha = 0.05$	$\alpha = 0.01$	Superior Section[1]
d(10)	53	5.295	Yes	Yes	Recycled
SCI	50	12.206	Yes	Yes	Recycled
S	56	10.579	Yes	Yes	Recycled
E_s[2]	56	0.891	No	No	-

[1]Section which is structurally superior on the basis of the parameter indicated.

[2]Used to evaluate the change in structural characteristics due to the subgrade modulus.

Table 9. Statistical Comparison of Dynaflect Parameters Between Control Sections 1-4 and Recycled Sections 7-17 of U.S. Highway 50, Holden, Utah.

Parameter Compared	Degrees of Freedom	Student's T-Statistic	Is the Difference Significant? $\alpha = 0.05$	$\alpha = 0.01$	Superior Section[1]
d(10)	87	17.016	Yes	Yes	Sections 7-17
SCI	80	12.628	Yes	Yes	Sections 7-17
S	130	4.504	Yes	Yes	Sections 7-17
E_s[2]	122	23.166	Yes	Yes	Sections 7-17

[1]Section which is structurally superior on the basis of the parameter indicated.

[2]Used to evaluate the change in structural characteristics due to the subgrade modulus.

becomes more difficult to differentiate between the contribution of the pavement structure from that of the subgrade. In turn, it is more difficult to compare two sections where a statically significant difference exists between the subgrade support.

Basis for Comparative Dynaflect Evaluation

All of the parameters which describe portions of the Dynaflect deflection basin are heavily influenced by the subgrade and by the interaction of the pavement system. A detailed analysis of the Dynaflects' deflection basin revealed that the method best suited for evaluating the structural response of the pavement was one which used the entire basin. The deflection basin shape is a valuable analytical tool particularly when normalized by considering, in addition the maximum deflection.

Perhaps the most successful analytical tool which can be used as a comparative evaluation of Dynaflect data is the two parameter or dual parametric analysis similar to that developed by Vaswani (20). This analysis is based on linear layered elastic theory. The two parameters used are the maximum Dynaflect deflection and the spreadability.

If a uniform pavement layer of a given elastic modulus were increased in thickness above a semi-infinite subgrade the maximum deflection would decrease and the spreadability would increase as thickness increased. Figure 4 illustrates an analysis chart based on these principles and was developed using the Chevron layered elastic computer program.

Evaluation of the Dynaflect deflection basin (21) has shown that the basin below a dual wheel load of 4,500 pounds (20 KN) per wheel can be approximated by multiplying the maximum Dynaflect deflection by 22.5. This was verified on several projects in this study. Thus, the analysis chart in Figure 4 was developed for a dual 4,500 pound (20 KN) wheel load, typically used in numerous design schemes. The analysis chart, also assumes that the Dynaflect deflection basin's spreadability is approximately equal to that measured under the dual 4500 pound (20 KN) wheel load.

The reason for trying to relate the Dynaflect parameters to a dual 4,500 pound (20 KN) wheel load was to approximate the deflection response of the pavement under analysis to a typical design load. However, since the analysis herein is a comparative analysis this is of little consequence.

In order to compare the in situ structural response of a recycled layer to a control layer at the same location, a concept which evaluates equivalent thickness was used. This essentially is an equivalent thickness concept based on in situ stiffness. The basis for this analysis is the dual parametric chart, Figure 4.

In Figure 4 the locus of any combination of d(10) x 22.5 and S determines an effective thickness of pavements above the semi-infinite subgrade. This effective thickness, D, in terms of the composite or average modulus of the pavement above the subgrade was used in the development of the chart. The effective thickness may be thought of as:

$$D = \sum_{i=1}^{N} b_i t_i, \tag{6}$$

where b_i is the effective thickness factor of a given layer, t_i is the actual layer thickness and N is the number of pavement layers.

In the cases evaluated the effective thickness factor based on stiffness of the control or reference layer, b_i, and the recycled layer, b_i', were the only two unknowns in the two systems making the equation solvable for b_i' in terms of b_i. The ratio of b_i to b_i is defined as the thickness equivalency ratio. Thus, if $b_i'/b_i = 2.00$, then the recycled

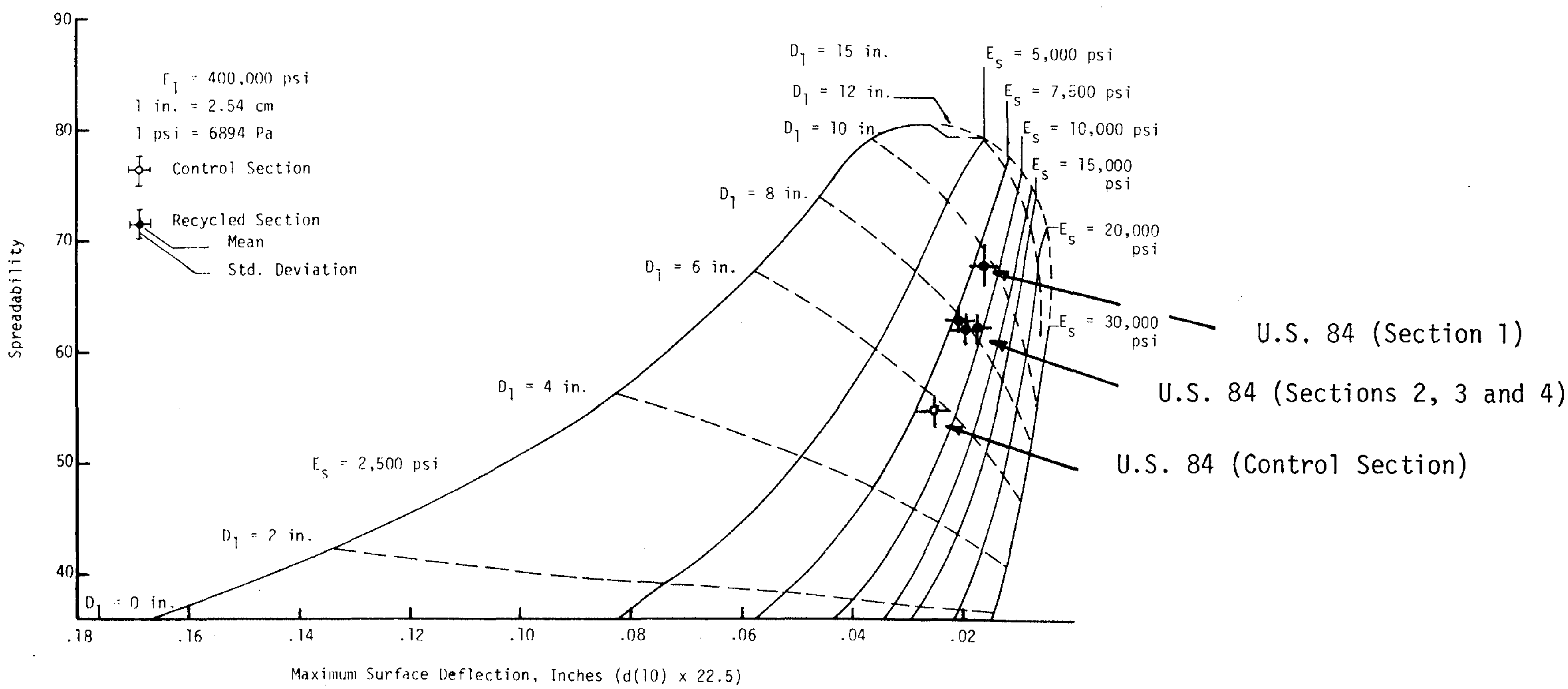

Figure 4. Dual Parametric Analysis Chart Used to Compare Dynaflect Responses of Recycled and Control Pavement Sections. (The Five Pavement Cross-Sections Compared Above are Geometrically Identical - Note the Increased Effective Thickness Caused by Recycling.)

material has a structural response which indicates twice the effective thickness of the control. All the control pavement systems evaluated contained only one stabilized layer. It was assumed that the stabilized layer was the dominant contributor to the pavement's effective thickness, D. This is a conservative estimate in terms of the stiffer layer (recycled or conventional) and is thought to be valid as, for example, unbound granular bases under the light load of the Dynaflect are likely to contribute little to the effective structural thickness when compared to the much stiffer stabilized layer.

Obviously the subgrade modulus varies somewhat throughout any construction project or test site. In addition, the subgrade due to compaction variation and material variation is not a homogeneous semi-infinite half-space as assumed by theory. Although changes in the elastic modulus of a true semi-infinite half-space do not affect the effective thickness of a pavement as evaluated through Figure 4, vertical anisotropy of the subgrade does. For example, if the subgrade is homogeneous under one test section yet has a stiff, dessicated crust under an otherwise cross-sectionally identical comparison section, the effective thickness of the stiff, dessicated crust would be reflected in the comparative analysis. By evaluating adjoining sections with a statistically designed experiment it is believed that this problem was minimized. In addition, although several subgrades were evaluated as statistically different between the recycled and control section, the difference was small.

Thus, the graphical procedure, Figure 4, should be generally reliable to compare the structural responses of two cross-sectionally comparable pavements. The results of this analysis are tabulated in Table 10.

Results of the Comparative Analysis

All of the recycled surface layer asphalt concretes considered in this study, except one, were recycled through a central plant operation. The lone in-place surface job was that of 11th Avenue in Kings County, California, which was an in-place operation using SC-800 cutback and the additional binder to restabilize an old, distressed road mix.

Generally, the eleven recycled central plant produced surfaces are equivalent to the conventional surface quality asphalt concretes they were compared against, based on the effective thickness ratio, b_i'/b_i. Seven of the eleven recycled layers had effective thickness ratios of between 0.9 and 1.10. Only two of eleven or eighteen percent have ratios of 0.90 or lower while the other eighteen percent had ratios of 1.10 or greater.

In the case of the recycled road mix asphalt concrete surface; 11th Avenue, Kings County, California; the effective thickness ratio was 1.00 between a conventional and recycled road mix.

Twelve of the fourteen recycled bases in Table 10 were recycled in-place. Of these twelve the material recycled was at least partly composed of an old asphalt concrete surface or asphalt penetration surface or some combination thereof. As is characteristic of in-place recycling, the processes used to recycle the old material into a new base were quite varied.

Ten of the in-place recycled bases based their ratios of stiffness on a surface quality asphalt concrete. Sixty percent of these had ratios equal to or greater than 1.00. The remaining fourty percent had ratios of approximately one-half.

The central plant recycled bases of Interstate 20, Roscoe, Texas (3 sections) and U.S. Highway 84, Snyder, Texas (4 sections), were compared

Table 10. Summary of Thickness Equivalency Ratios.

Project Designation	Recycled Layer Analyzed	Control Section Reference Layer	Thickness Equivalency Ratio, B
Interstate 8, Gila Bend, Arizona Section 2 vs. Section 1	Recycled Asphalt Concrete Surface	Conventional Asphalt Concrete Surface	1.02
Interstate 8, Gila Bend, Arizona Section 4 vs. Section 3	Recycled Asphalt Concrete Surface	Conventional Asphalt Concrete Surface	0.90
Interstate 8, Gila Bend, Arizona Section 5 vs. Section 6	Recycled Asphalt Concrete Surface	Conventional Asphalt Concrete Surface	1.03
Interstate 8, Gila Bend, Arizona Section 5 vs. Section 3	Recycled Asphalt Concrete (Full Depth)	Conventional Asphalt Concrete (Full Depth)	0.91
11th Avenue, Hanford, California	Recycled Asphalt Road Mix	Conventional Asphalt Road Mix	1.00
Russell Avenue, Fresno, County, California	Recycled Asphalt Stabilized Base	Conventional Road Mix Asphalt Stabilized Base	3.44
18th Avenue, LeMoore, California	Recycled Asphalt Stabilized Base	Conventional Aggregate Base	2.40

Table 10 (continued)

Project Designation	Recycled Layer Analyzed	Control Section Reference Layer	Thickness Equivalency Ratio, B
Highway 45, Yolo, California	Recycled Base Lime Stabilized	Conventional Asphalt Concrete Surface	1.24
U. S. 56, Pawnee County, Kansas	Recycled Asphalt Concrete Base (Section 1)	Conventional Asphalt Concrete (Full Depth)	1.25
U. S. 56, Pawnee County, Kansas	Recycled Asphalt Concrete Base (Section 2)	Conventional Asphalt Concrete (Full Depth)	0.98
U. S. 56, Pawnee County, Kansas	Recycled Asphalt Concrete Base (Section 3)	Conventional Asphalt Concrete (Full Depth)	1.12
U. S. 56, Pawnee County, Kansas	Recycled Asphalt Concrete Base (Section 4)	Conventional Asphalt Concrete (Full Depth)	1.12
Interstate 15, Henderson, Nevada	Recycled Asphalt Concrete Surface	Conventional Asphalt Concrete Surface	0.87
U. S. 50, Dayton, Nevada	Recycled Base Cement Stabilized	Conventional Asphalt Concrete Surface	0.42

Table 10 (continued)

Project Designation	Recycled Layer Analyzed	Control Section Reference Layer	Thickness Equivalency Ratio, B
U. S. 93, Wells, Nevada	Recycled Base Cement Stabilized (Section 1)	Conventional Asphalt Concrete Surface	1.15
U. S. 93, Wells, Nevada	Recycled Base Cement Stabilized (Section 2)	Conventional Asphalt Concrete Surface	1.54
Ponderosa Avenue, Inclined Village, Nevada	Recycled Base Cement Stabilized	Conventional Asphalt Concrete Surface	0.56
Hillsboro to Silverton Highway, Woodburn, Oregon	Recycled Asphalt Concrete Surface	Conventional Asphalt Concrete Surface	0.90
Interstate 20, Roscoe, Texas	Recycled Asphalt Concrete Base (Section 1)	Conventional Asphalt Concrete Surface	1.08
Interstate 20, Roscoe, Texas	Recycled Asphalt Concrete Base (Section 2)	Conventional Asphalt Concrete Surface	1.24
Interstate 20, Roscoe, Texas	Recycled Asphalt Concrete Base (Section 3)	Conventional Asphalt Concrete Surface	1.05

Table 10 (continued)

Project Designation	Recycled Layer Analyzed	Control Section Reference Layer	Thickness Equivalency Ratio, B
U. S. 54, Dalhart, Texas	Recycled Asphalt Concrete Surface	Conventional Asphalt Concrete Surface	0.98
U. S. 84, Snyder, Texas	Recycled Asphalt Concrete Base (Section 1)	Conventional Asphalt Concrete Surface	1.98
U. S. 84, Snyder, Texas	Recycled Asphalt Concrete Base (Section 2)	Conventional Asphalt Concrete Surface	1.57
U. S. 84, Snyder, Texas	Recycled Asphalt Concrete Base (Section 3)	Conventional Asphalt Concrete Surface	1.59
U. S. 84, Snyder, Texas	Recycled Asphalt Concrete Base (Section 4)	Conventional Asphalt Concrete Surface	1.59
U. S. 277 Abilene, Texas	Recycled Asphalt Concrete Base	Conventional Asphalt Concrete Surface	0.54
U. S. 50, Holden, Utah	Recycled Asphalt Concrete Surface (Sections 7-17)	Conventional Asphalt Concrete Surface (Sections 1-4)	1.40

Table 10 (continued)

Project Designation	Recycled Layer Analyzed	Control Section Reference Layer	Thickness Equivalency Ratio, B
U. S. 50, Holden, Utah	Recycled Asphalt Concrete Surface (Section 18)	Conventional Asphalt Concrete Surface (Sections 1-4)	1.59
U. S. 50, Holden, Utah	Recycled Asphalt Concrete Surface (Section 19)	Conventional Asphalt Concrete Surface (Sections 1-4)	1.54
Blewitt Pass, Washington	Recycled Asphalt Concrete Surface	Conventional Asphalt Concrete Surface	0.34
Rye Grass, Washington	Recycled Asphalt Concrete Surface	Conventional Asphalt Concrete Surface	1.80

to conventional surface grade asphalt concrete in their respective areas. All of the resulting effective thickness ratios were greater than one, fifty-seven percent over greater than 1.50.

Overall, 22 of the 32 effective thickness ratios computed or sixty-nine percent were greater than one. Although the standard of comparison was different in each case, the fact that the recycled layer was equal to or stiffer than a comparable conventional layer sixty-nine percent of the time is significant.

LABORATORY TESTS

Fatigue Potential

As the authors have tried to carefully acknowledge a severe limitation of any analysis based on material stiffnesses is that it does not effectively consider the fatigue or fracture potential of the material. The recycled materials evaluated herein responded to laboratory and field induced loading with a stiffness comparable to that of conventional asphalt concrete and conventional asphalt stabilized bases.

Probably all of the recycled pavement sections analyzed in this study are thick enough to be governed by a controlled stress mode of flexural fatigue. This is true as the bituminous or stabilized materials involved provide the primary structural support to the roadway. The recycled materials should perform well in such pavement sections as their stiffness under loading is probably great enough to prevent fracture under normal design traffic loading.

On the other hand, relatively thin bituminous surfaces may give little or not support, and the pavement deflects an amount controlled by the subgrade, base material and subbase. In this mode a recycled material's ability to perform well is more dependent on its resistance to load induced fatigue than its ability to distribute vertical stresses due to stiffness. To determine whether or not recycled materials will perform adequately in these situations, extensive fatigue testing of recycled mixtures is required. This testing must focus on the role of the rejuvenators or additives as well as the recycling process used (i.e., central plant, in place, surface).

Several researchers (20, 21) have reported on the potential of the indirect or splitting tension test as an indicator a fatigue potential. Maupin and Freeman (20) concluded that the indirect tension test was the best simple test available by which to characterize the fatigue behavior of bituminous concrete. Their main conclusion was that the indirect tensile test could be used successfully to present the fatigue characteristics of bituminous concrete.

Indirect tension tests were performed on the recycled pavements studied. These indirect tension data were recorded at 77°F(25°C) and at a loading rate of 2 in/minute (0.84 $\frac{mm}{sec}$). Tests were performed on field cores before and after Lottman freeze-thaw conditioning (22). These results one compared to sixteen Texas pavements, Table 11, which have performed well in terms of load associated fatigue cracking. These Texas pavements form the basis for Figure 5 and range in age from 8 to 17 years with a mean age of 10 years. The rate of crack growth was examined on each pavement by field visual evaluations. All pavements presented have experienced minimal crack growth and have thus performed very satisfactorily.

Figure 5 shows that the indirect tension data for the recycled

Table 11. Texas Pavements Used in Comparative Analysis

Roadway Designation	Location	Pavement Age, Years
IH 10	Pecos	8
US 60	Whitedeer	8
US 70	Matador	10
US 83	Abilene	8
US 84	Waco	9
US 87	Lubbock	8
US 96	Euadale	8
US 281	Faysville	17
US 287	Childress	14
SH 36	Milano	14
SH 37	Corpus Christi	8
SH 79	Hearne	9
SH 259	Orr City	17
SH 358	Corpus Christi	8
FM 493	Donna	8
FM 730	Azle	8

Note U.S. 277

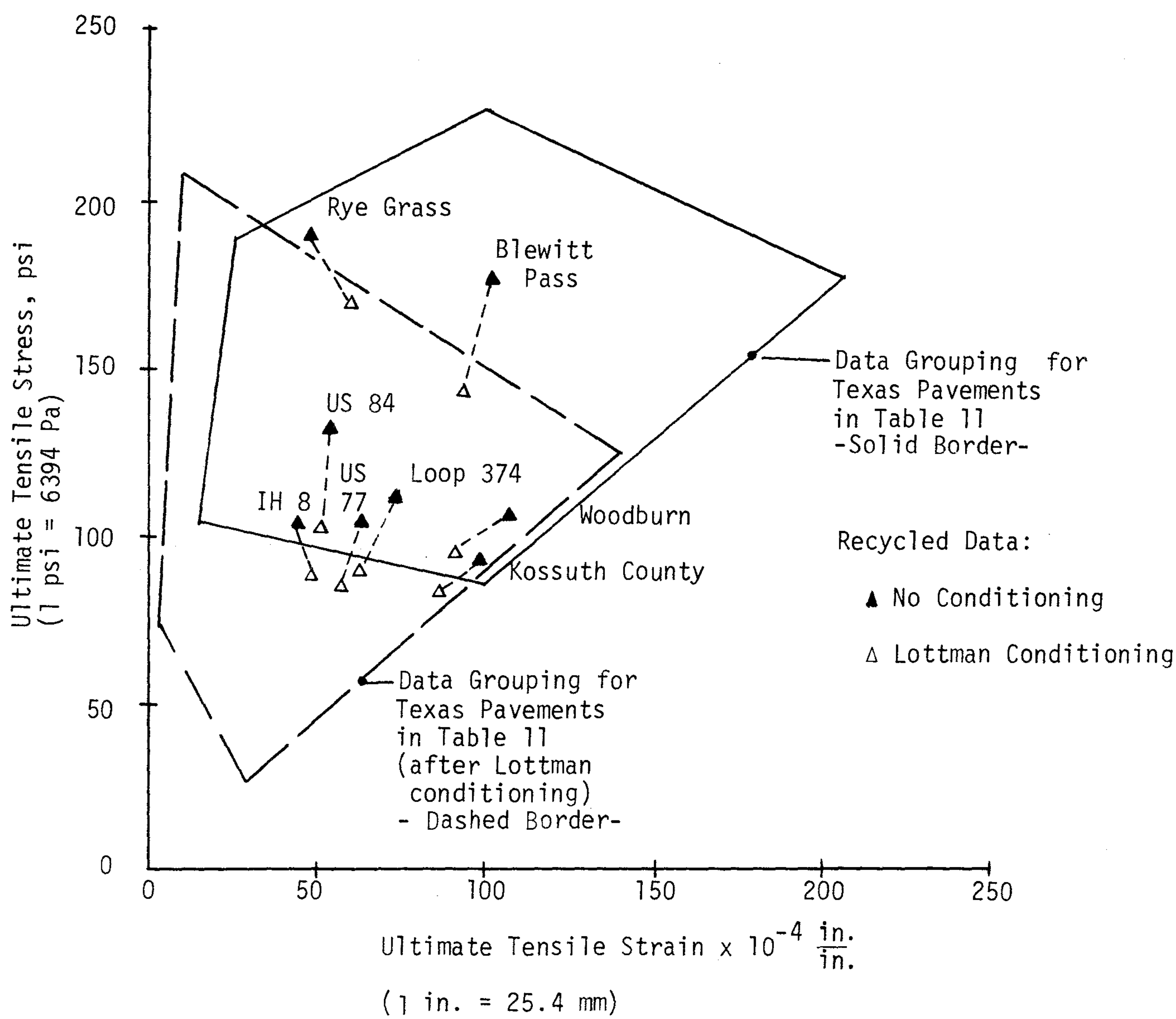

Figure 5. Comparison of Indirect (Splitting) Tension Data of Recycled Surfaces and Successful Texas Asphalt Concrete Surfaces.

pavements is in line with that of the successful pavements listed in Table 11. Figure 5 also shows that after Lottman conditioning, the recycled materials have about the same magnitude decrease in ultimate stress and are less stiff than the old, reference pavements. This could be due to the effects of the rejuvenators in the recycled materials which leaves the mixture less brittle after conditioning.

Although these data cannot substitute for fatigue testing, they indicate favorable tensile failure properties when compared to successfully functioning conventional pavements.

Stability

As the stiffness of a pavement material used for a surface course increases so do the shearing stresses in the layer. Thus the surface must have adequate stability to function successfully.

Table 12 compares the Hveem stabilities of cores of the recycled pavements used as surfaces or bases with the sixteen successful conventional pavements in Table 11. The recycled stabilities are slightly lower but are within a reasonable range. In addition the effects of Lottman freeze-thaw conditioning is seen to be, percentage-wise, slightly less detremental for the recycled pavements.

CONCLUSIONS

Laboratory derived material properties such as Hveem and Marshall stabilities, indirect tensile strengths, moisture susceptibility and resilient moduli, indicated that recycled asphalt concrete could be expected to replace conventional asphalt concrete in the pavement structure with satisfactory results. This study substantiates this expectation.

Table 12. Comparison of Hveem stabilities between Recycled and Successful Conventional Asphalt Concrete Pavements

Pavement	Hveem Stability	Hveem Stability after Lottman Conditioning
16 conventional Asphalt Concrete Surfaces (n = 48)	$\overline{X}$ - 32 S = 9 CV,% = 28	$\overline{X}$ = 23 S = 11 CV,% = 48
14 Recycled Asphalt Concrete Surfaces and Bases (n = 56)	$\overline{X}$ = 27 S = 6 CV,% = 22	$\overline{X}$ = 20 S = 8 CV,% = 40

Recycled asphalt concrete used as both surface and base courses appear to be able to function as well as conventional materials based on a comparison with the standard paving materials used at the AASHTO Road Test. This comparison was made using the structural coefficients concept. Table 13 summarizes the structural layer coefficients calculated for the recycled material and compares them with the structural coefficients developed at the Road Test. Although there are obvious limitations in comparing materials only in this way, the structural coefficient is based on the most thorough study of pavement performance available and is believed to give a realistic first approximation of the performance of recycled materials.

The greater stiffness of the recycled materials studied is evident in Table 10. Here the effective thicknesses (based on stiffness) of most recycled materials were greater than the conventional layer used for comparison. Stiffness is once again of primary influence in this analysis which is a limitation. However, it is reasonable to infer that the recycled pavements, which have functioned successfully for as many as seven years and which maintain a stiffness comparable to or greater than the conventional layer evaluated against, are structurally as sound as conventional materials. Of course, this must be verified by more thorough laboratory fatigue, creep and permanent deformation testing. Such characterization should be evaluated in both layered elastic and viso-elastic structural pavement analyses.

Table 13. Summary Table of AASHTO Structural Coefficients Determined for the Various Types of Recycled Materials Studied.

Type of Recycled Material	Layer Used As	Range of a_i Computed	Average Computed a_i	a_i for Corresponding Layer and Material at AASHTO Road Test
Central Plant Recycled Asphalt Concrete	Surface	0.37-0.59	0.48	0.44
In-Place Recycled Asphalt Concrete Stabilized with Asphalt and/or an Asphalt Modifier	Base	0.22-0.49	0.39	0.35
In-Place Recycled Asphalt Concrete and Existing Base Material Stabilized with Cement	Base	0.23-0.43	0.33	0.15-0.23
In-Place Recycled Asphalt Concrete and Existing Base Material Stabilized with Lime	Base	0.40	0.40	0.15-0.30

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