Effects of Soil Binder on Blackbase Mixture

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The findings of a study to evaluate the effects of soil-binder content on the behavior of blackbase mixtures used in Texas are summarized. The evaluation was based on a comparison and analysis of engineering properties, obtained by using the static and repeated-load indirect tensile tests, of mixtures with various soil-binder contents. For this study two blackbase mixtures, a rounded gravel with field sand and a crushed limestone, were investigated. Various engineering properties were evaluated at various soil-binder contents and asphalt contents. The engineering properties evaluated were tensile strength, static modulus of elasticity, fatigue life, and resilient modulus of elasticity. Most of the tests were conducted on air-dried specimens; however, a limited number of tests were conducted on pressure-wetted specimens to evaluate the influence of moisture content. Generally, the results indicate that the various engineering properties were maximized at relatively low soil-binder contents and at correspondingly lower asphalt contents. The optimum asphalt contents tended to decrease as the soil-binder content decreased. The optimum soil-binder contents for the various engineering properties ranged from 5 to 10 percent.

The primary objective of the study summarized in this paper was to evaluate the effect of soil-binder content on the behavior and design of blackbase paving mixtures used in Texas. Soil binder is defined as material that will pass the U.S. standard no. 40 sieve.

The acceptability of an aggregate gradation for use in asphalt mixtures often is judged by its conformity to specified size limits. These limits have generally been established either on the basis of satisfactory experience with materials that meet selected gradation specifications or in terms of selected gradation patterns of natural or crushed materials that are readily available. Thus, it is possible to have gradation limits that vary significantly but that will still produce satisfactory asphalt mixtures.

In Texas, a range of binder contents is specified as a part of the gradation requirements (specification no. 346, 1962). To determine the effect of soil-binder content on asphalt paving mixtures and to determine whether improved or less costly mixtures can be produced by specifying a binder content or by eliminating all specification requirements concerning binding content, a limited study (1) was conducted at the request of the Texas State Department of Highways and Public Transportation.

EXPERIMENTAL PROGRAM

The basic experimental approach was to determine the relationships between asphalt content and the engineering properties and to determine the optimum asphalt content for each property. These relationships and optima were then evaluated with respect to soil-binder content to determine whether properties could be improved by controlling the binder content. Finally, the effect of moisture on these relationships was evaluated.

The experimental study was a comparison of the engineering properties of asphalt mixtures composed of two representative types of aggregate, each with various soil-binder contents. By changing the quantity of soil binder, each selected aggregate gradation was mixed to produce laboratory specimens that had asphalt contents in the range generally used for design.

Materials

Two aggregates were used, a rounded river gravel and a crushed limestone, both of which are commonly used in pavement construction in Texas and have performed satisfactorily. The two aggregates were obtained from Eagle Lake and Lubbock, Texas, respectively.

The Eagle Lake aggregate was a mixture of four different aggregates, the combination of which can be generally described as a smooth-surfaced, angular, nonporous, crushed siliceous river gravel. The other material was a rough, subangular, porous crushed-limestone caliche obtained approximately 10 miles southeast of Lubbock.

The soil-binder contents selected for the Eagle Lake material were 0, 5, 10, 20, and 30 percent, and the binder contents for the Lubbock limestone were 0, 5, 10, and 25 percent. The various gradations and percentages of soil binders were obtained by the addition or removal of material finer than the no. 40 sieve while a constant amount of the coarser material was maintained. Gradations of the mixtures are shown in Figures 1 and 2.

The asphalt cement used with the Eagle Lake materials was an AC-20 produced at the Exxon refinery in Baytown, Texas; the asphalt cement used with the Lubbock limestone was an AC-10 produced by the Cosden Oil refinery in Big Springs, Texas.

Specimen Preparation

All specimens prepared for this investigation were mixed and compacted according to test method Tex-126-E by using the Texas motorized gyratory compactor (2). Subsequent to compaction, all specimens were allowed to cool in the compaction mold for 1 h before extrusion to prevent slumping of the specimen. After extrusion from the mold, the specimens were cured overnight at room temperature. Smaller test specimens for the indirect tensile test were then cut from the top and bottom portions of the compacted specimen. The densities of these test specimens were calculated from their weights and physical dimensions.

The top and bottom specimens were cured overnight at a room temperature of approximately 24°C (75°F). Thus, the total curing time was two days. The sawed specimens were 152 mm (6 in) in diameter and about 84 mm (3.3 in) in height and were tested by using the indirect tensile test.

To evaluate the effects of moisture, the exposed sawed faces of the test specimens were coated with a thin film of the asphalt cement used in the mixture. Then the specimens were subjected to pressure wetting (test method Tex-109-E, part IV). This procedure subjects a specimen to an 8274-kPa (1200-psi) hydrostatic water pressure at a water temperature of 65°C (150°F) for 15 min prior to actual testing by means of the indirect tensile test.

Indirect Tensile Test

The indirect tensile test is conducted by loading a cylindrical specimen with a single or repeated compressive load that acts parallel to and along the vertical diametral plane. The loading configuration develops a relatively uniform tensile stress perpendicular to the direction of the applied load and along the vertical diametral plane, which ultimately causes the specimen to fail by splitting along the vertical diameter. The basic testing equipment was

Figure 1. Gradations of Eagle Lake gravel mixture.

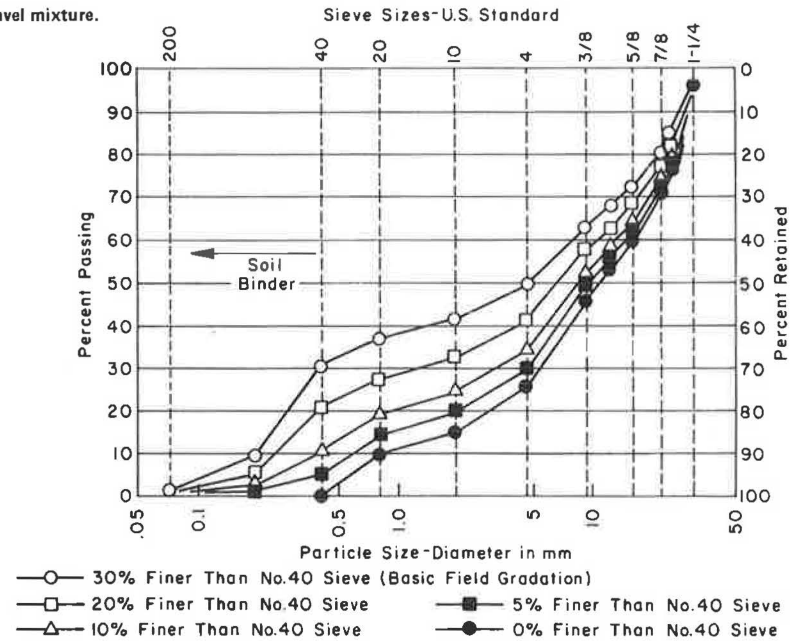
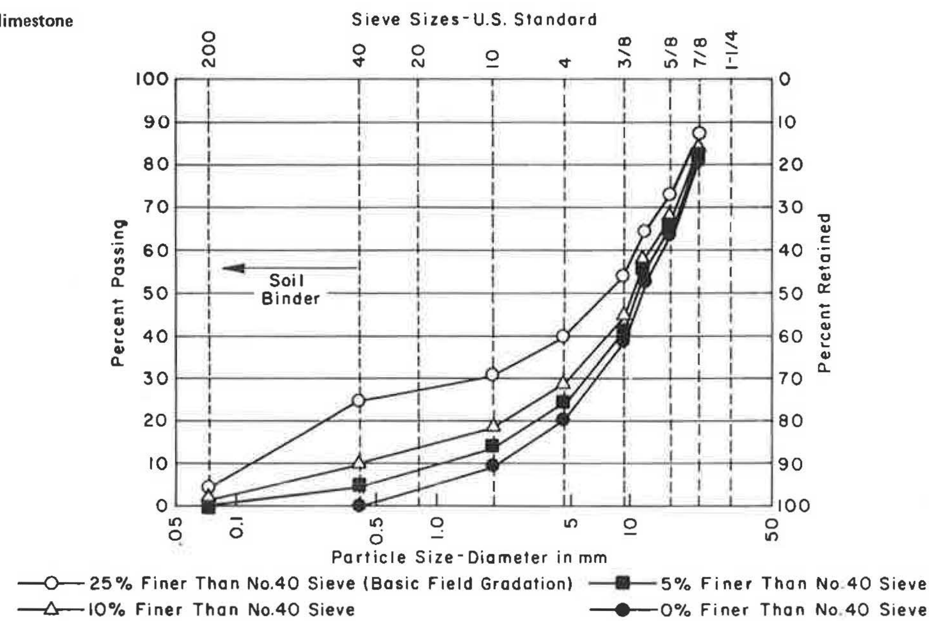


Figure 2. Gradations of Lubbock limestone mixture.



a Material Testing System (MTS) closed-loop electrohydraulic loading system.

The basic test procedure is described by Kennedy and Anagnos (3). A preload, which produced a tensile stress of approximately 5 kPa (0.6 psi), was applied to the specimens in the static tests to prevent impact loading and to minimize the effect of seating of the loading strip. The specimen was then loaded at a constant deformation rate of 51 mm/min (2 in/min). Vertical deformations were measured by a dc linear variable differential transducer (LVDT). Horizontal deformations were measured by two cantilevered arms with attached strain gauges. Both the load-vertical deformation and load-horizontal deformation relationships were recorded by a pair of X-Y plotters.

For the repeated-load tests, a light preload was also applied. The desired load was applied at a frequency of 1 Hz with a 0.4-s load duration and a 0.6-s rest period. Both the horizontal and vertical deformations were measured by dc LVDTs and were recorded on the X-Y plotters. All tests were conducted at 24°C (75°F)

Properties

Several of the properties analyzed are related to the relevant pavement distress modes of (a) thermal or shrinkage cracking, (b) fatigue cracking, and (c) permanent deformations, or rutting. The properties analyzed were

1. General: (a) density and (b) total air voids,
2. Static characteristics: (a) tensile strength and (b) static modulus of elasticity, and
3. Repeated-load characteristics: (a) fatigue

life and (b) resilient modulus of elasticity.

Density and Air Voids

The asphalt-voids ratio (AVR) density was calculated by using the mold diameter and the measured height, which was obtained while the specimen was subjected to the final compaction load of 3447 kPa (500 psi). The specimen weight was determined after extrusion from the mold. This density is also referred to as the in-mold density and is used to calculate percentage of total air voids as defined by test method Tex-126-E.

Tensile Strength

The ultimate tensile strength was calculated by using the following relationship for specimens 152 mm (6 in) in diameter and the load-deformation information obtained from the static indirect tensile test:

$$S_t = 0.105P_{ult}/t \tag{1}$$

where

S_t = ultimate tensile strength (psi),
P_{ult} = maximum load carried by the specimen (lb), and
t = thickness of the specimen (in).

Tensile stresses produced by loads less than the maximum load P_{ult} can also be calculated by using the above equation.

Static Modulus of Elasticity

The static modulus of elasticity was determined from the load-deformation relationships for static tensile tests. The equation used to calculate the static modulus was

$$E_s = (S_h/t)(0.27 - \nu) \tag{2}$$

where

E_s = static modulus of elasticity (psi);
S_h = slope of the relationship between axial load and horizontal deformation, i.e., the ratio of axial load to horizontal deformation within the linear range (lb/in); and
ν = static Poisson's ratio = (4.09/DR) - 0.27,

where DR = deformation ratio, i.e., slope of the relationship between vertical deformation and horizontal deformation (inches of vertical deformation per inch of horizonal deformation).

Fatigue Life

Fatigue life is defined as the number of load applications at which the specimen will no longer resist load or at which deformation is excessive and increases with essentially no additional loads.

Resilient Modulus of Elasticity

The resilient modulus of elasticity was calculated by using the resilient horizontal and vertical deformations, which are characteristic of the elastic deformations produced by repeated loads of short duration.

The equation used to calculate the resilient modulus was

$$E_R = (P_R/tH_R)(0.27 + \nu_R) \tag{3}$$

where

E_R = resilient modulus of elasticity (psi),
P_R = applied repeated load (lb), and
ν_R = resilient Poisson's ratio = 4.09 $(H_R/V_R) - 0.27$

where H_R and V_R are the resilient horizontal and vertical deformations for the loading cycle that corresponds to 0.5 N_f. The equation is the same as that used for the static Poisson's ratio; however, since the relationship between load and deformation is essentially linear, the equation was modified.

Testing Program

The variables included in this study were aggregate type, soil-binder content, asphalt content, and moisture content. For the repeated-load tests, two stress levels that would produce reasonable fatigue lives were selected. Only a limited number of mixtures were tested to evaluate the effects of moisture. These mixtures contained the optimum asphalt contents for maximum tensile strength. All tests were performed at room temperature, 24°C (75°F).

ANALYSIS AND DISCUSSION OF TEST RESULTS

Density and Design Optimum Asphalt Content

The in-mold AVR densities were generally larger than the densities of the top and bottom sections of the specimens, and the densities of the bottom specimens were generally greater than those of the top specimens. Mixtures that had high soil-binder contents had the lowest in-mold AVR densities, whereas the mixtures that had relatively low binder contents had the highest in-mold AVR densities. The relationships between asphalt content and total air voids were determined for each aggregate gradation (Figure 3). The laboratory AVR-design optimum asphalt contents (2) are slightly greater than the asphalt contents that correspond to the inflection point on the straight-line section of the AVR curves.

Eagle Lake Gravel

These relationships indicate that (a) as the amount of soil binder decreased from 30 percent to 5 percent the total air voids decreased and (b) the total air voids increased appreciably as the amount of soil binder decreased from 5 percent to 0 percent. The total air voids were approximately the same for binder contents of 5 and 10 percent. The relationships between soil-binder content and total air voids at asphalt contents of 3.0, 3.5, and 4.0 percent are shown in Figure 4. These relationships indicate that the minimum total air voids occurred at a binder content of about 7 percent for mixtures containing 3.0 and 3.5 percent asphalt and at a binder content of about 10 percent for mixtures containing 4.0 percent asphalt. The laboratory AVR design optimum asphalt content decreased from 4.5 percent for a 30 percent binder content to a minimum value of 3.5 percent for a 5 percent binder content and then increased slightly to 3.6 percent for 0 percent binder content (Figure 5a). The corresponding total air voids at laboratory AVR design optimum asphalt content remain constant at 1.6 percent for values of soil-binder content ranging between 5 and 20 percent but increase appreciably for 0 percent and 30 percent binder contents (Figure 5b).

Figure 3. Relationships between asphalt content and total air voids.

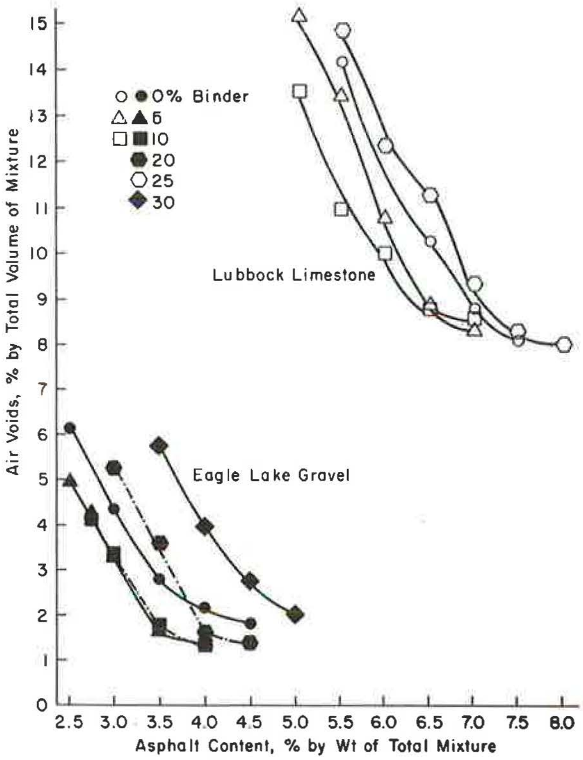
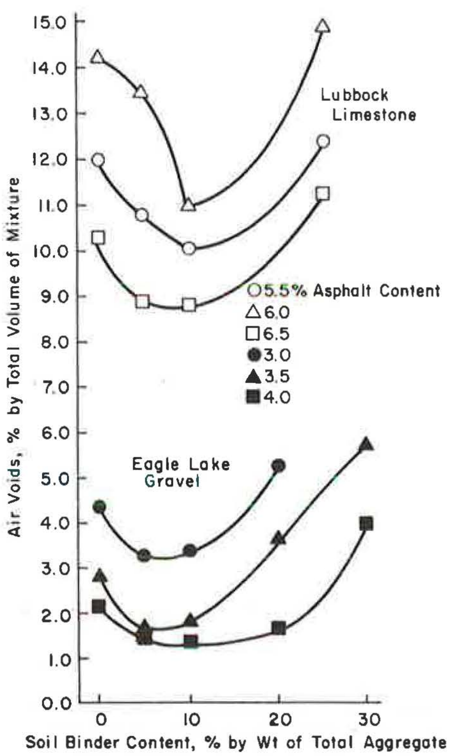


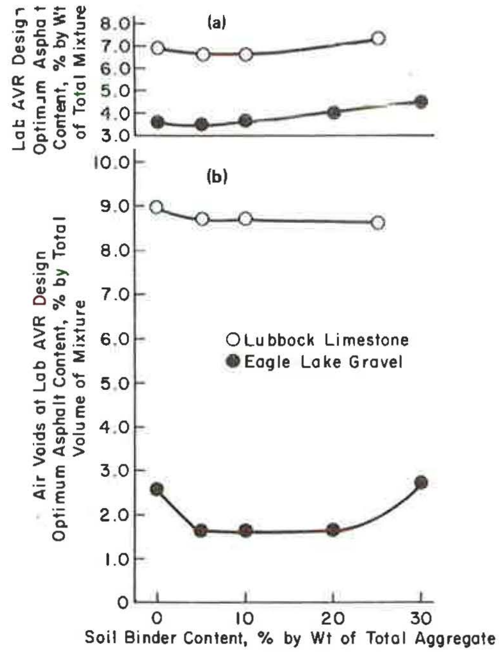
Figure 4. Relationships between soil-binder content and total air voids.



Lubbock Limestone

The total air voids relationships for Lubbock limestone shown in Figure 4 indicate that, as the amount of soil binder decreased from 25 percent to

Figure 5. Relationships between soil-binder content and (a) AVR design optimum asphalt content and (b) total air voids.



10 percent, the total air voids decreased to a minimum and then increased as the amount of soil binder decreased further from 10 to 0 percent. The mixture containing 10 percent binder had the lowest total air voids, regardless of the percentage of asphalt content.

The relationships between soil-binder content and (a) laboratory AVR design optimum asphalt content and (b) total air voids at laboratory AVR design optimum asphalt content are also shown in Figure 5. Laboratory AVR design optimum asphalt contents (Figure 5a) were 6.6 percent for both 5 percent and 10 percent soil-binder contents; however, the optimum asphalt contents are higher for 25 percent soil-binder content (7.3 percent) and 0 percent soil-binder content (6.9 percent). For each soil-binder content, the total air voids at laboratory AVR design optimum asphalt content are very close; they ranged from 9.0 percent for 0 percent soil-binder content to 8.6 percent for 25 percent soil-binder content.

Static Indirect Tensile Test Results

The engineering properties, tensile strength, and static modulus of elasticity were estimated by using the static indirect tensile test.

Tensile Strength

Optimum asphalt contents for maximum tensile strength were found for each soil-binder content and each aggregate type. Values of optimum asphalt content for the Eagle Lake mixture ranged from 3.0 percent for soil-binder contents of 5 and 10 percent to 4.0 percent for a binder content of 30 percent and from 5.5 percent for a binder content of 10 percent to 7.0 percent for a binder content of 25 percent for the Lubbock limestone mixtures (Figure 6a). The maximum tensile strength was 1365 kPa (198 psi) for the Eagle Lake gravel mixture and 1367 kPa (200 psi) for the Lubbock limestone mixture, both of which occurred at a binder content of 5 percent (Figure 6b).

Figure 6. Relationships between binder content and (a) optimum asphalt content and (b) corresponding maximum tensile strength.

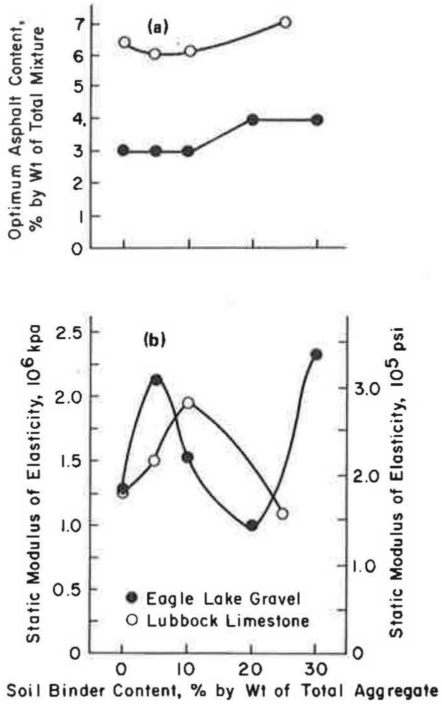
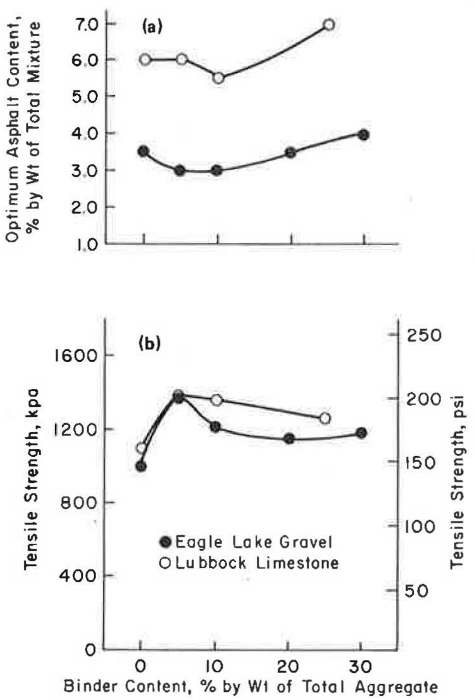


Figure 7. Relationships between soil-binder content and (a) optimum asphalt content and (b) corresponding static modulus of elasticity.



The maximum tensile strengths of the Eagle Lake mixtures at binder contents of 30, 20, and 10 percent were essentially equal at about 1190 kPa (173 psi); however, the optimum asphalt contents were 4.0, 3.5, and 3.0 percent, respectively. As the soil-binder content decreased from 10 to 5 percent, the strength increased by 180 kPa (26 psi), and the optimum asphalt content remained constant at 3.0 percent. Without any soil binder, the ultimate tensile strength of the Eagle Lake gravel mixture decreased significantly, and the mixing optimum asphalt content increased from 3.0 to 3.5 percent.

Similar trends were found for the Lubbock limestone mixtures, except that the optimum asphalt contents for 5 and 0 percent binder contents were the same (6.0 percent).

Static Modulus of Elasticity

For all mixtures there were optimum asphalt contents for maximum static moduli of elasticity (Figure 7). These optimum asphalt contents for Eagle Lake gravel mixtures ranged from 3.0 percent for 0, 5, and 10 percent binder contents to 4.0 percent for 20 and 30 percent binder contents (Figure 7a). For Lubbock limestone mixtures, the optimum ranged from 6.0 percent for 5 percent soil-binder content to 7.0 percent for 25 percent soil-binder content.

For the Eagle Lake gravel mixtures, the maximum static modulus of elasticity probably occurred at 5 percent (Figure 7b). The optimum binder content was 10 percent for Lubbock limestone mixtures.

Repeated-Load Indirect Tensile Test Results

Repeated-load indirect tensile tests were conducted to evaluate the fatigue life and resilient modulus of elasticity. A discussion of the resistance to permanent deformation for each of the mixtures may be found in Ping and Kennedy (1).

Fatigue Life

To eliminate the effect of stress and to determine the effect of asphalt content and binder content, the fatigue lives of the mixtures were evaluated for a tensile stress of 100 kPa (14.5 psi) for each binder content.

The optimum asphalt content for maximum fatigue life was found for each of the mixtures of Eagle Lake gravel and Lubbock limestone. The relationships between soil-binder content and the optimum asphalt contents are shown in Figure 8. Optimum asphalt contents for the Eagle Lake mixtures ranged from 2.9 percent for 5 percent binder content to 4.6 percent for 30 percent binder content and from 4.5 percent for 10 percent binder to 7.5 percent for 25 percent binder for the Lubbock limestone mixtures. For both mixtures the optimum asphalt content for 0 percent soil binder was higher than the optimum for mixtures with 5 and 10 percent soil binder.

The optimum soil-binder content for maximum estimated fatigue life was 5 percent for both types of aggregate. The maximum estimated fatigue life was about 8.7 kHz for the Eagle Lake gravel at 2.9 percent asphalt content and about 980 kHz for the Lubbock limestone at 6.0 percent asphalt content.

Resilient Modulus of Elasticity

The relationships between asphalt content and the resilient modulus of elasticity at 0.5 N_f indicated that the optimum asphalt content for maximum resilient modulus was not well defined; most of the relationships are flat. This behavior is consistent with the behavior previously reported (1,4).

Nevertheless, in order to analyze the effects of binder content, an attempt was made to pick an asphalt content that produced the maximum modulus. The resulting relationships between soil-binder content and optimum asphalt content for maximum resilient modulus of elasticity for the loading cycle corresponding to 0.5 N_f are shown in Figure

Figure 8. Relationships between binder content and both optimum asphalt content and the corresponding fatigue life for (a) Eagle Lake and (b) Lubbock limestone mixtures.

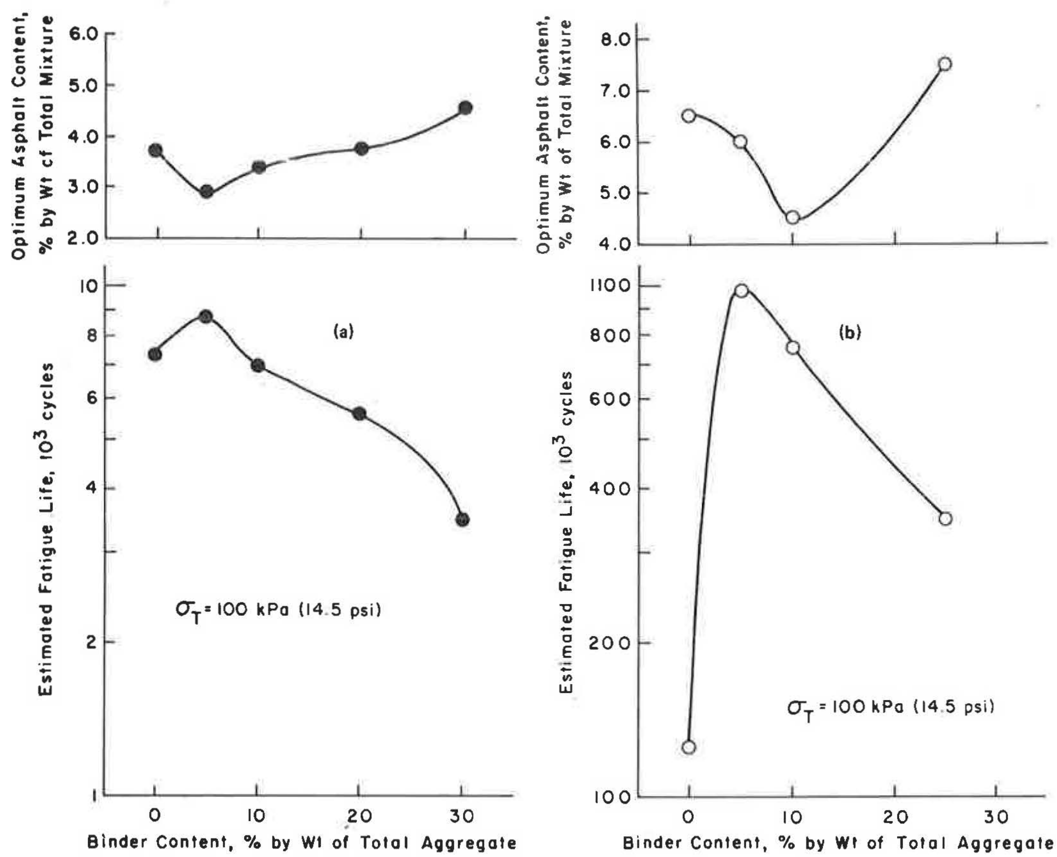
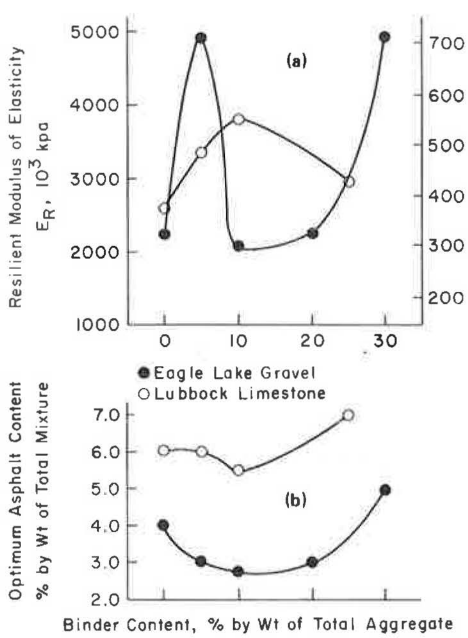


Figure 9. Relationships between soil-binder content and (a) maximum resilient modulus of elasticity at 0.5 N_f and (b) corresponding optimum asphalt content.



9. The maximum resilient moduli of elasticity for Eagle Lake mixtures with 5 and 30 percent soil-binder contents were about 2.5 times those for 0, 10, and 20 percent soil-binder contents. Thus, either 5 or 30 percent may be chosen as the optimum soil-binder content; however, the curve was not well defined. The optimum binder content for maximum resilient modulus of elasticity for the Lubbock limestone mixtures was 10 percent with 5.5 percent asphalt content.

Since the actual asphalt content is not critical to the resilient modulus of elasticity and since the actual values of modulus were relatively constant, the minimum values of asphalt content should be used.

Moisture Damage

This study generally indicated that the optimum soil-binder contents for maximum engineering properties were relatively low, in the range of 5 to 10 percent. In addition, low binder contents required less asphalt and, therefore, improved the economy of the mixtures. However, the specimens tested were dry and had not been subjected to moisture. A limited series of tests was therefore conducted to evaluate the effects of moisture on the engineering properties of the two materials. A series of specimens for each aggregate type at the optimum asphalt content for the maximum ultimate tensile strength was subjected to pressure wetting and then was tested to obtain static indirect tensile results and resilient moduli of elasticity.

Test results are shown in Figures 10 through 13. The asphalt contents of tested specimens were lower than the optimum asphalt contents for the maximum densities, and the air-void contents were higher. Contents after pressure wetting were proportional to the total air voids, i.e., the higher the total air voids, the higher the water contents.

A definite effect of moisture was apparent on the static modulus of elasticity and the ultimate tensile strength (Figures 10 and 11). A strength loss of about 250 kPa (36 psi) occurred for Eagle Lake gravel mixtures with 5 percent soil binder and a loss of about 500 kPa (72 psi) for mixtures with 30 percent soil binder. However, pressure wetting did not produce a loss of tensile strength for

Figure 10. Relationships between binder content and moisture content on (a) the static modulus of elasticity and (b) the ultimate tensile strength for Eagle Lake gravel mixtures.

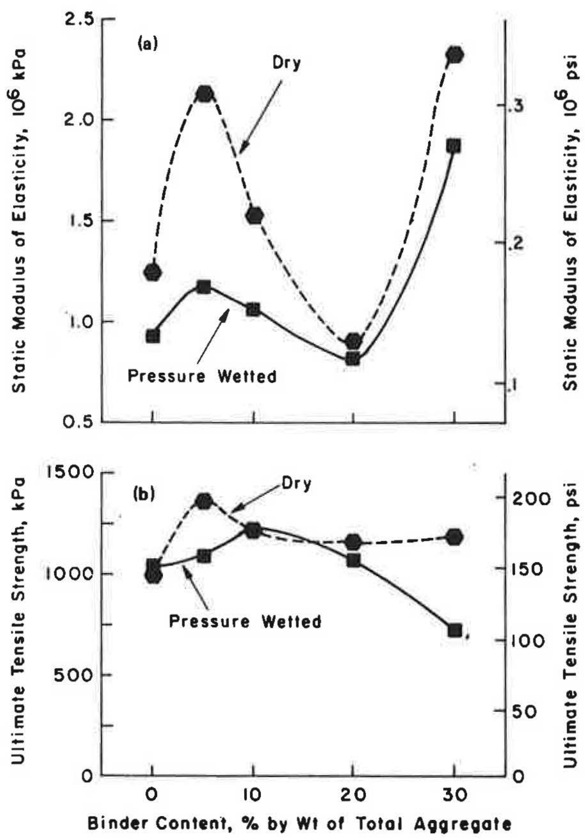


Figure 11. Relationships between binder content and moisture content on (a) the static modulus of elasticity and (b) the ultimate tensile strength for Lubbock limestone mixtures.

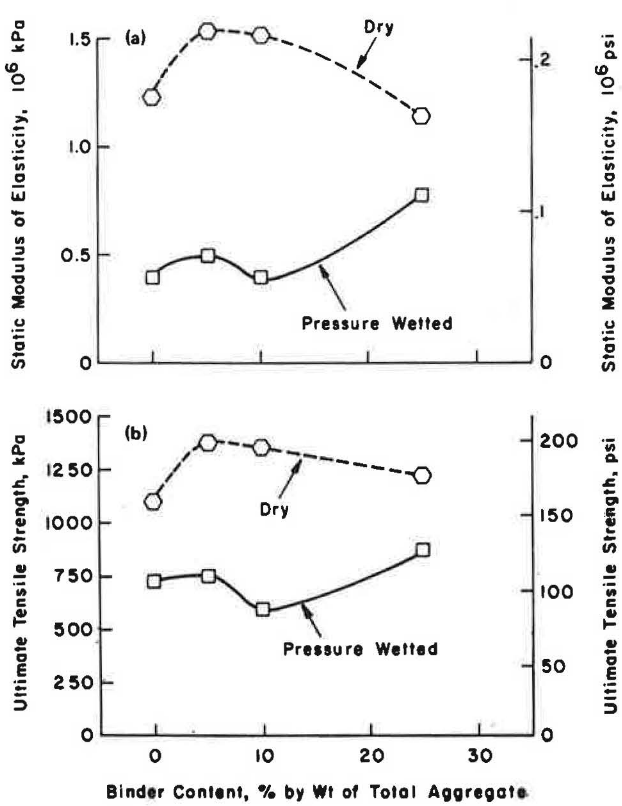


Figure 12. Relationships between binder contents and air voids, water content, and resilient modulus of elasticity for Eagle Lake gravel mixtures.

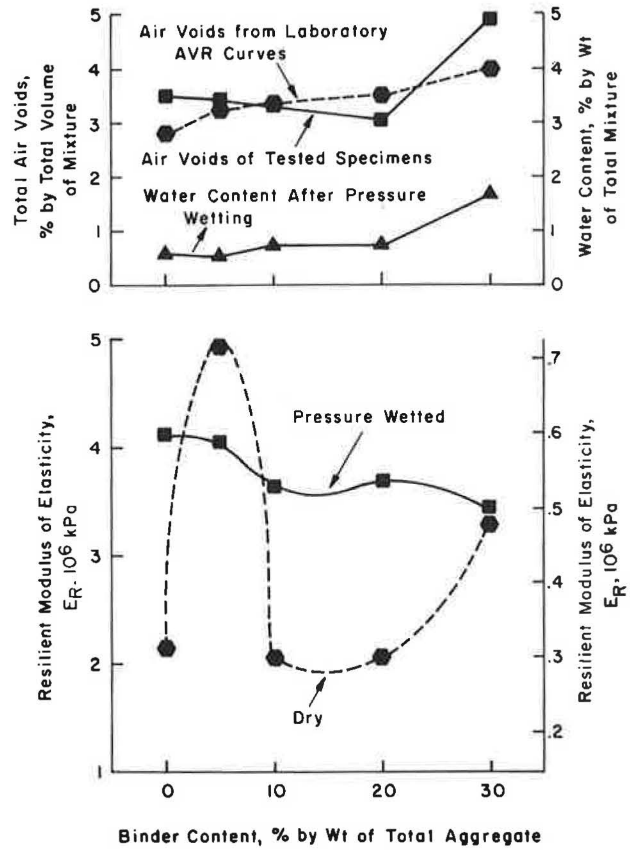
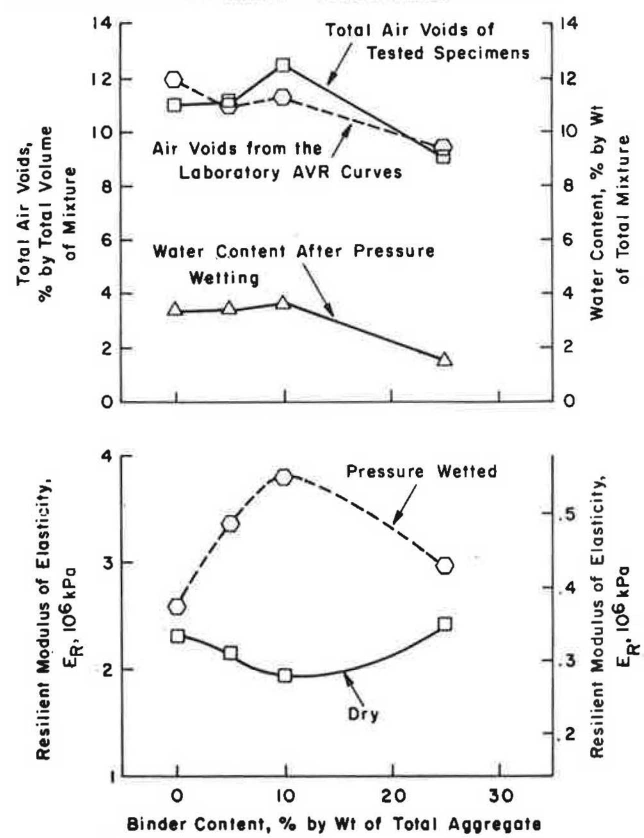


Figure 13. Relationships between binder content and air voids, water content, and resilient modulus of elasticity for Lubbock limestone mixtures.



mixtures with 0, 10, and 20 percent soil binder (Figure 10). For the Lubbock limestone mixtures the losses were more consistent, varying from 750 kPa (110 psi) to 400 kPa (58 psi). The effect of pressure wetting on static modulus of elasticity was more significant (Figure 11). Losses in modulus for the Eagle Lake mixtures ranged from 100 000 kPa (14 500 psi) to slightly less than 1 000 000 kPa (145 000 psi). Similarly, for the Lubbock limestone the losses ranged from about 400 000 kPa (58 000 psi) to 1 000 000 kPa (145 000 psi). No consistent or explainable relationships were observed for the resilient modulus of elasticity (Figures 12 and 13). In most cases the pressure-wetted specimens exhibited higher moduli than did the dry specimens. This was especially true for the Lubbock limestone mixtures.

The shape of the density relationships for tested specimens and the shape of the relationships for tensile strength and the static modulus of elasticity after pressure wetting (Figures 10 and 11) were similar.

Moisture damage apparently was dependent on the density of the mixtures or air void content. The highest density for Eagle Lake gravel mixtures was achieved at 5 percent soil-binder content and at 10 percent soil-binder content for Lubbock limestone mixtures. This would suggest that, as long as the mixture has adequate density, substantial moisture damage will not occur; however, additional study is required.

CONCLUSIONS

The conclusions based on the findings of this investigation are summarized below.

General

1. The laboratory AVR design optimum asphalt contents ranged from 3.5 percent to 4.5 percent for Eagle Lake gravel mixtures and from 6.6 percent to 7.3 percent for Lubbock limestone mixtures. Total air voids were affected by both the soil-binder content and the asphalt content. With proper asphalt content, the minimum total air voids occurred at soil-binder contents between 7 and 10 percent for Eagle Lake gravel mixtures and at 10 percent soil-binder content for Lubbock limestone mixtures. The corresponding total air voids for Lubbock limestone mixtures with the laboratory AVR design optimum asphalt content were from 8.6 percent to 9.0 percent, which was much higher than those for Eagle Lake gravel mixtures (1.6 percent to 2.7 percent).
2. An optimum binder content for the maximum AVR density existed for the two materials. The Eagle Lake gravel mixture with 5 percent soil-binder content had the lowest optimum asphalt content (3.5 percent) and the highest density, and the Lubbock limestone mixture with 10 percent soil-binder content had the lowest optimum asphalt content (6.5 percent) and the highest density.
3. There was a tendency for the optimum asphalt content to decrease as the soil-binder content decreased; however, when the mixture contained little or no soil binder, the optimum asphalt content increased.

Static Characteristics

1. Both Eagle Lake gravel and Lubbock limestone mixtures exhibited essentially equal maximum static tensile strength; however, the asphalt content required for the Lubbock limestone mixtures (6.0 percent) was much higher than that for Eagle Lake gravel mixtures (3.0 percent).
2. Within the limits of this study, no definite relationship could be found between static modulus of elasticity and soil-binder content for Eagle Lake gravel mixtures; however, an optimum soil-binder content (10 percent) for maximum static modulus of elasticity existed for Lubbock limestone mixtures.

Repeated-Load Characteristics

1. A definite optimum binder content for maximum fatigue life existed for both the Eagle Lake gravel and the Lubbock limestone mixtures. At the same stress level of 100 kPa (14.5 psi), the fatigue life of Lubbock limestone mixtures was much higher than that of Eagle Lake gravel mixtures; i.e., the maximum fatigue life of Lubbock limestone mixtures at 5 percent binder content was about 110 times that of Eagle Lake gravel mixtures.
2. For resilient modulus of elasticity and static modulus of elasticity, no well-defined optimum soil-binder content existed for the Eagle Lake gravel mixture; however, an optimum soil-binder content (10 percent) was found for the Lubbock limestone mixture.

Moisture Damage

1. The moisture damage for the Lubbock limestone mixture was more severe than that for the Eagle Lake gravel mixture.
2. The moisture damage appeared to be directly related to the total air voids of the mixture; i.e., the damage resulting from water was greater for mixtures with higher total air voids.

Optimum Asphalt Content

1. Optimum asphalt contents were found to occur for the following material properties: (a) AVR density, (b) tensile strength, (c) static modulus of elasticity, and (d) fatigue life. No well-defined optimum occurred for the resilient modulus of elasticity.
2. The optimum asphalt content for mixtures with higher soil-binder content was generally larger than the optimum for mixtures with lower soil-binder content.
3. In general, the lowest optimum asphalt content occurred at 5 percent soil binder for the Eagle Lake gravel mixture and at 10 percent soil binder for the Lubbock limestone mixture.

Optimum Soil-Binder Content

1. For the Eagle Lake gravel mixture, the optimum soil-binder content was 5 percent for AVR density, tensile strength, fatigue life, and permanent deformation.
2. For the Lubbock limestone mixture, the optimum soil-binder content ranged from 5 to 10 percent for the various engineering properties.

ACKNOWLEDGMENT

This investigation was conducted at the Center for Transportation Research, Bureau of Engineering Research, University of Texas at Austin. We wish to thank the sponsors, the Texas State Department of Highways and Public Transportation and the Federal Highway Administration of the U.S. Department of Transportation.

The contents of this paper reflect our views, and we are solely responsible for the facts and the ac-

curacy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This paper does not constitute a standard, specification, or regulation.

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

Permanent Deformation Characterization of Bituminous Mixtures for Predicting Pavement Rutting

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A general overview of rutting models available for predicting rutting in flexible pavements, the permanent deformation characterizations used by these rutting models or subsystems, and the general limitations of these characterizations is given. Discussed in detail are VESYS and DEVPAV. The most intensive development of a rutting model has been for the VESYS subsystem through support of the Federal Highway Administration. Since numerous researchers are involved in its further development and implementation, the published results of permanent deformation testing by a number of researchers have been transformed to the VESYS material characterization parameters ALPHA(1) and GNU(1). The relationships between these and other significant variables such as deviator stress, temperature, resilient modulus, and confining stress are reviewed and summarized. ALPHA(1) and GNU(1) were found to generally decrease with increasing deviator stress or temperature; the increase in ALPHA(1) dominated the effect on permanent strain predictions. ALPHA(1) increases with increasing confinement, and GNU(1) appears to decrease. Both parameters appear to increase as the material stiffness increases.

The advent of pavement models capable of making predictions of permanent deformations in pavement materials has created a need for characterizing materials in terms of permanent strains that result from repeated wheel loadings. A number of researchers have studied this problem both in the laboratory and through theoretical studies and comparisons with field measurements, and several procedures have been applied with varying degrees of success.

Virtually all models that offer predictions of rutting in pavements use elastic-layer theory to arrive at predictions of stresses or strains and then use these predictions and characterizations of the permanent deformation potential of pavement materials to arrive at predictions of rutting on a pavement surface. All of these procedures are limited (a) because the models are not as yet developed to the level of complexity imposed by the real physical system and (b) because of shortcomings in the characterizations of the permanent deformation potentials of the pavement materials. This does not imply that the characterizations are not useful but simply means that further development and improvements are needed to increase the confidence level in their predictions. The same is certainly true of fatigue-life predictions and, to a lesser extent, of most other outputs from analytical models used for pavement analysis and design.

The purposes for this paper are twofold. The first is general in nature and is intended to offer a brief overview of available pavement models developed for predicting rutting in flexible pavements and the characterizations used for permanent deformation potential of bituminous mixtures. The second is to update the state of knowledge of those characterization parameters used specifically by the VESYS rutting prediction subsystem. The former should be useful to researchers interested in flexible pavement models and the latter to the Federal Highway Administration of the U.S. Department of Transportation, state departments of transportation, and private and academic researchers working to improve and implement VESYS.

RUT-PREDICTION MODELS AND THEIR MATERIAL CHARACTERIZATIONS

A number of general approaches to the permanent deformation characterization of bituminous materials have been applied; they may be classified as follows:

1. Creep compliance characterization: These compression tests are generally run with one long-term static load but have also been run incrementally with specified load and unload durations (1) and with repetitive static loads. Characterization is in terms of a creep modulus as a function of load duration.
2. Repetitive-load test characterization: Many cycles of stress are applied for very short intervals (usually 100 000 or more cycles at 0.06- to 0.2-s load durations) in a factorial format to take into account the effects of stress state and sample temperature. Characterization is in terms of permanent strain as a function of cycles of loading.
3. Regression model characterization: Rutting is predicted on the basis of parameters such as stress and material stiffness. Permanent deformation potential is implicitly considered through its correlations with the significant parameters used.