Discussion

Waheed Uddin

In the state-of-the-art report on stripping of asphalt pavements, the authors have presented an excellent overview of the different types of tests to evaluate the stripping potential of an asphalt concrete mix. In their comments on qualitative coating and stripping evaluation tests, it is mentioned that the ASTM D1664 standard test is frequently criticized. Nevertheless, this test is treated as a standard in many countries around the world and is a quick way to examine the stripping potential while using the material testing facilities on a paving site.

In Saudi Arabia, on the projects of the Civil Aviation Department, a modified stripping test is included in the standard specifications prepared by their consultants, Netherlands Airport Consultants. This test falls in the same category as ASTM D1664. The test is performed on laboratory-prepared as well as plant-mix samples. The test procedure is outlined below.

> In the laboratory, samples of asphaltic concrete mixture are prepared using the same procedure as for Marshall specimens. The optimum asphalt content is used in the preparation of the test samples. The mixture is then spread in loose thin layers to be cured in air for 24 hours. Half of a 600 ml clean glass jar is filled with the sample and covered with distilled water at the room temperature. After 24 hours immersion in water, the jar is vigorously shaken for about 15 minutes and the sample is then examined visually for stripping. This is done by looking through the water in the standard way. Further examination is done by spreading the mixture again on a clean flat surface.

This test method has been successfully used during the construction of pavements on civil aviation projects. The writer used this procedure along with ASTM D1664 on some of these jobs. The modified test has two distinct advantages over the ASTM standard method: (a) The test sample is prepared according to the design mix at the optimum asphalt content, and (b) the test is also performed on the plant mix samples. In general the modified procedure gives a better estimate of stripping potential of the design asphalt concrete mix under conditions closely related to actual field condition.

Qualitative test methods are still necessary especially in the developing countries or anywhere when the more sophisticated testing facilities are not readily available. A qualitative test method can also be used to develop a rating scheme for various aggregate sources in a project area.

Author's Closure

Mark A. Taylor

Uddin's defense of qualitative coating evaluation tests is well received and certainly justifiable. Our intent was not to discredit this classification of stripping tests, but rather to point out the dissatisfaction of many investigators with the reproducibility of the tests and their correlation with field performance. Limitations on the paper's length prevented a thorough discussion of many aspects of the stripping problem. Uddin's discussion is greatly appreciated since it provides a different perspective on one of the many categories of stripping tests.

Publication of this paper sponsored by Committee on Flexible Pavement Construction.

Investigation of Moisture Damage to Asphalt Concrete and the Effect on Field Performance—A Case Study

THOMAS W. KENNEDY, ROBERT B. McGENNIS, AND FREDDY L. ROBERTS

An investigation of premature distress and failure of an asphalt concrete overlay placed in 1979 and 1980 is summarized. The primary objective of the study was to determine the probable cause of the distress. The investigation involved an analysis of construction records and laboratory test results performed during and after construction. In addition, specimens and material were obtained from the roadway for use in a laboratory evaluation. The sampling program included the collection of cores, slabs, and stockpile or pit materials. A description and summary of the pavement and distress, construction procedures, and mixture characteristics, along with the findings related to the probable causes of the distress is presented. The basic causes were that (a) all aggregates and the resulting aggregate-asphalt combinations were highly susceptible to moisture damage and (b) the antistripping additive used in the mixture was not effective.

In the fall of 1979, an overlay project was undertaken to rehabilitate a section of roadway that had been in service for 13 yr. The project was 9 miles long and consisted of overlaying a continuously reinforced concrete pavement (CRCP) with hot-mix asphalt concrete.

In June 1980, before completion of the contract, distress began to develop on certain sections of the highway (Figures 1-4). Distress was in the form of rutting, shoving, and bleeding. Initially, distress occurred in small areas of the outside westbound lanes; however, distress subsequently developed in

Figure 1. Typical rutting and corrugation distress.

Figure 2. Typical rutting distress.

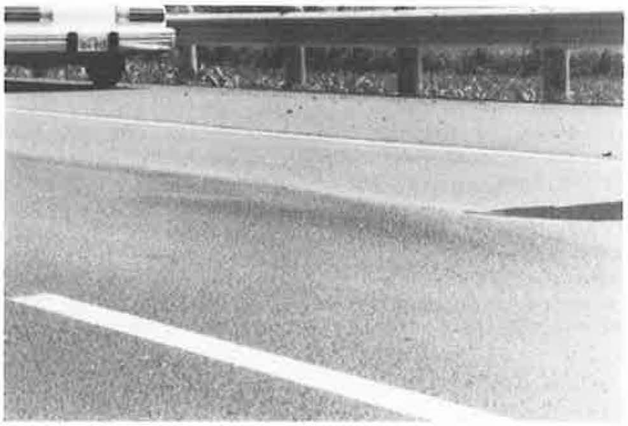


Figure 3. Magnitude of the rutting distress.

Figure 4. Planning that outlines the longitudinal variation in rutting and shoving distress.

other areas. Minor maintenance operations were performed on the most highly distressed areas, and signs were installed to advise traffic to change lanes to avoid these distressed areas.

This paper is a summary of an investigation to determine the causes of this premature distress. The investigation included analyses of construction records and laboratory test results performed during and after construction. It should be noted that project records indicated that virtually all materials and construction were within applicable specifications.

PROJECT DESCRIPTION

Overlay operations consisted of a level-up followed by a 2.5-in. hot-mix asphalt concrete (HMAC) binder course producing an average overall depth of about 3 in. A 1-in. wearing course provided the final surface (Figure 5).

Construction of the level-up course began on October 15, 1979. Before placement of the binder course a light tack coat was placed on the CRCP; however, no individual cracks were sealed. The level-up layer varied in thickness from about 1 to 3 in. and placement was completed on April 8, 1980.

Construction of the 2.5-in. binder course began on October 22, 1979, at approximately the midpoint of the job and proceeded westward. Figure 6 illustrates the construction sequence. A laydown machine was used to place the mixture and a tandem double drum vibratory roller and a medium pneumatic roller were used to compact the mixture. The eastbound and westbound lanes were laid in three mats of 15 ft, 12 ft, and 11 ft for the inside lane and shoulder, outside lane, and outside shoulder, respectively. On January 18, 1980, base course paving operations were suspended due to poor weather conditions. On March 10, 1980, paving operations resumed and continued until completion on May 5, 1980. Therefore, the portion of the binder course placed prior to January 18 remained exposed to the elements until the end of May when it was covered by the wearing course.

Construction of the 1-in. wearing course began on May 16, 1980, and continued until July 1, 1980, when paving operations were halted because of the observed distress.

TYPE AND SEVERITY OF DISTRESS

The types of distress observed on the surface were rutting, shoving, and bleeding (Figures 1-4). Four condition surveys were conducted by project personnel from the Center for Transportation Research (CTR) between October 9, 1980, and September 9, 1981. Results are shown graphically in Figure 7. Four pavement distress conditions were designated as described in Table 1. These distress conditions were (a) minor or none, (b) moderate, (c) moderate to severe, and (d) severe. In some areas of the westbound lane the severe distress was so extensive that traffic was diverted to adjacent lanes.

The progressive nature of the distress can be seen by observing that areas with initial distress designated as minor or moderate gradually became more severe. Overlay construction began on October 15, 1979, and the first evidence of distress was observed in June 1980 in the westbound outside lane. By April 1980 distress was observed in practically all sections of the project that were constructed between October 1979 and January 1980, i.e., everything west of Station 1043 (Figure 6).

Figure 5. Existing and proposed pavement structures.

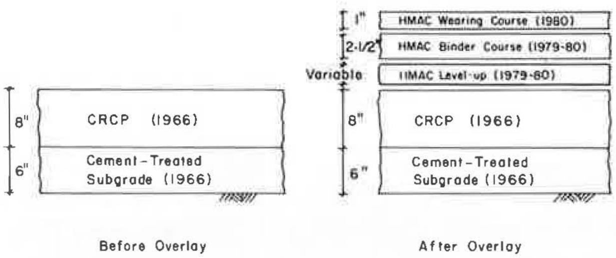
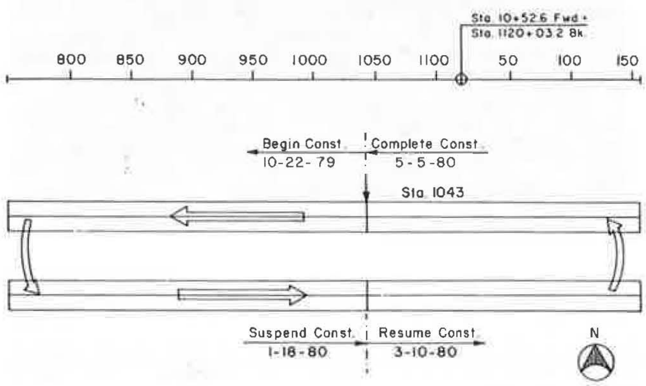


Figure 6. Construction limits and sequence for binder course mixture.



STUDY APPROACH

Particular attention was paid to the binder course because initial observations made by CTR project staff indicated that the cause of distress was contained in this layer. Loose samples of the binder course collected by local officials in July 1980 and examined by CTR personnel indicated extensive stripping. Aggregate appeared to be free of any asphalt cement; and, in addition, the asphalt cement remaining lacked adhesive qualities.

All pertinent construction data were collected for analysis. These data were separated by mixture design and location on the project and analyzed to determine if any patterns emerged that might relate construction data to distress occurrence.

Cores were obtained at 26 locations (Figure 8) that included all four of the distress categories. Approximately 18 cores were taken at each sampling location. In many highly distressed areas, cores could not be secured because the mixture disintegrated into individual components during the coring process. In such cases, cores were secured as close to the area as possible. Cores were sawed into their constituent layers before testing and specimens were selected for each proposed test.

Bag samples of all materials used on the project were obtained from stockpiles and specimens were compacted in the laboratory. The prepared specimens were then tested using a variety of laboratory test procedures to provide additional data for use in correlating test measurements to probable causes of the distress.

MIXTURE DESIGN

Three different mixtures were used in the binder course of this overlay project, designated as designs 1, 2, and 5 (Table 2 and Figure 9). Mixture

Table 1. Distress conditions.

Distress Condition	Rut Depth (in.)	Shoving
Minor to none	0-0.5	None
Moderate	0.5-1.0	None
Moderate to severe	1.0-2.5	Slight
Severe	1.0-2.5	Pronounced

Figure 7. Condition survey.

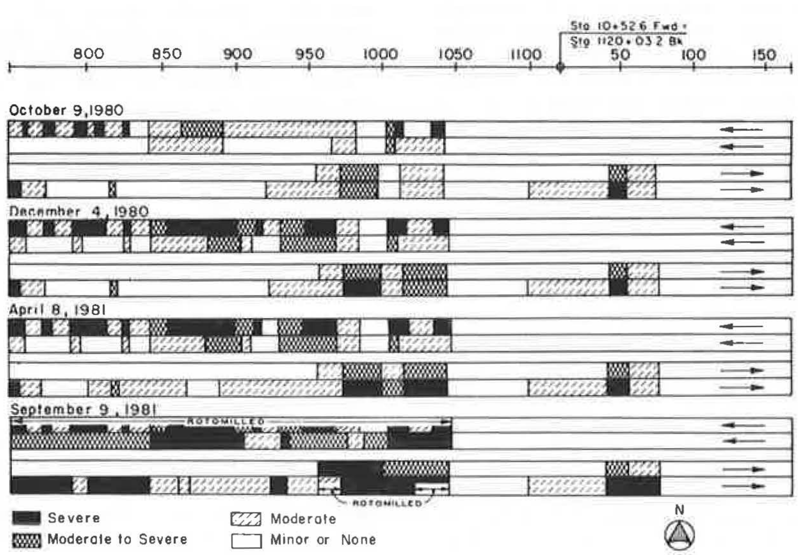


Figure 8. Sampling locations and corresponding condition survey.

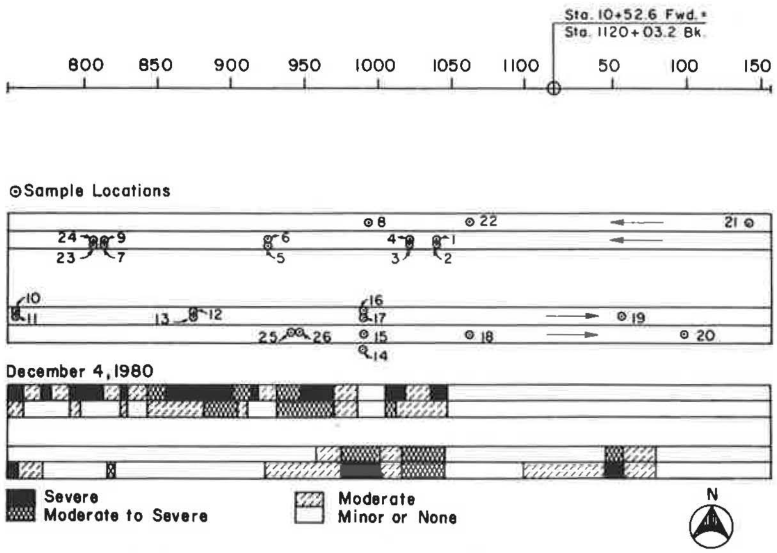


Table 2. Binder course mixture design.

	Percent by Weight of Total Mixture				
Design	Coarse Aggregate	Gem Sand	Coarse Sand	Field Sand	Asphalt Content[c]
1	38.1	25.7	12.4	19.1[a]	4.7
2	35.2	24.7	10.5	24.7[a]	4.9
5	35.4	25.0	10.6	25.0[b]	4.0

[a]Pit No. 1 field sand.
[b]Pit No. 2 field sand.
[c]1 percent liquid antistripping agent by weight of asphalt cement.

designs conformed to Texas Department of Highways and Public Transportation Specification Number 340, "Hot Mix Asphaltic Concrete Pavement." The design procedure used was a modified Hveem method. No routine test for moisture susceptibility of aggregates was performed. Designs 1 and 2 used the same materials but had asphalt contents of 4.7 and 4.9 percent, respectively. Design 5 was markedly different from the other two; it contained a different field sand and a much lower asphalt content of 4.0 percent. The areas on the project in which each design was placed are shown in Figure 10.

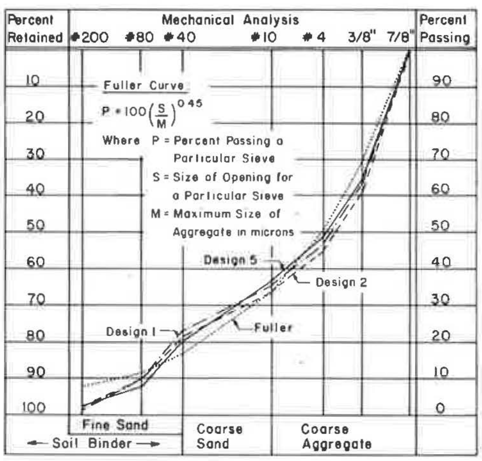
ANALYSIS AND EVALUATION OF FIELD DATA

As shown in the condition surveys (Figure 7), the distress was concentrated in the base course in designs 1 and 2, and the major distress began in design 1. No major distress occurred in design 5, although slight rutting was visible at the time of the last condition survey. As previously noted, designs 1 and 2 were similar (Table 2 and Figure 8), with design 2 having only 0.2 percent more asphalt. Design 5 was markedly different, containing a different field sand and a much lower asphalt content.

Construction Sequence

All of design 1 and a large portion of design 2 for the binder course were placed prior to January 18, 1980, at which time construction was suspended because the weather was too cold and wet for satisfactory placement. The remainder of design 2 and all of design 5 were placed after construction resumed

Figure 9. Comparison between binder course design gradations and Fuller gradations.



on March 10, 1980. The binder layer served as a temporary surface and was exposed to the action of both the environment and traffic until it was covered by the wearing surface beginning on May 6, 1980.

A review of the condition surveys (Figure 7) indicates that the majority of the distress occurred in those sections which were exposed during the winter months of 1979-1980. In addition, distress occurred first in those sections which were laid first, i.e., those exposed for the longest period of time.

Asphalt Content

Since the distress was primarily distortional in nature, i.e., rutting and shoving, it was believed that some distress could be the result of excessive asphalt. Asphalt contents as reported in daily construction records were superimposed on distress and no definite relationship between asphalt content and distress was evident. However, the extracted asphalt contents for design 1 were generally above the design value; for design 2 they were about equal to the design value; and for design 5 they were essentially below the design value.

Figure 10. Locations of mixture designs.

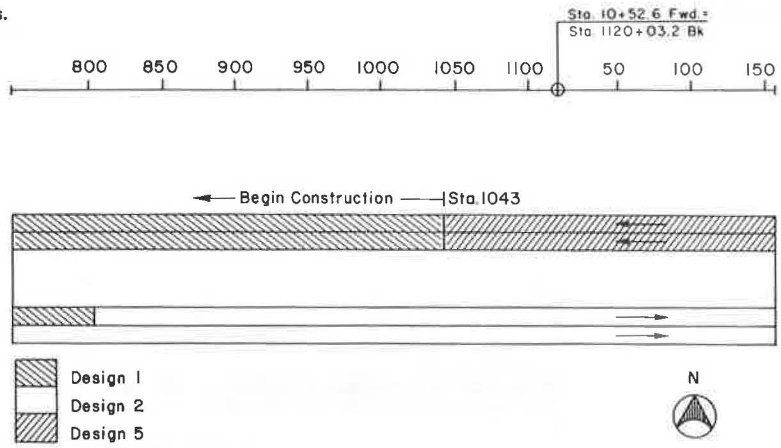
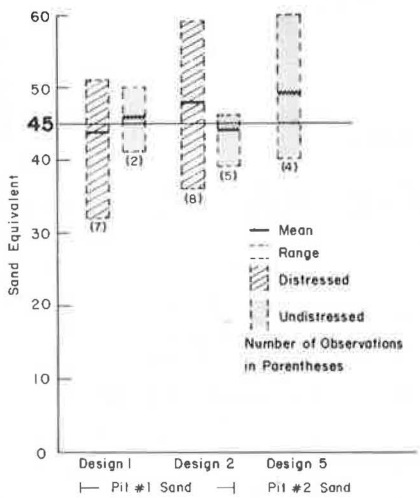


Figure 11. Sand equivalent values.



Mixture Voids

Using the daily extraction values and known specific gravities of all mixture constituents, the voids in the compacted mineral aggregate (VMA) and percent voids filled with asphalt (VF) were computed. No differences in VMA were detected for the various mixture designs or distress conditions. In addition, throughout the project the VMA ranged from 14.7 to 19.1 percent and averaged 17.4 percent, which is almost 4 percent higher than the minimum of 13.5 percent recommended by the Asphalt Institute (1). Similarly, no differences in VF were detected for the various mixture designs or distress conditions. Monismith (2) has suggested that a VF range of 65 to 75 percent for base courses and 75 to 85 percent for surface courses is required in order to produce mixtures that perform satisfactorily in the field. The VF ranged from 49.6 to 64.8 percent and averaged 57.5 percent (9.6, 5.2, and 7.4 percent air voids, respectively); this is below the recommended values creating mixtures that may be (a) permeable to water and potentially moisture susceptible, and (b) susceptible to raveling.

Gradation

A comparison between the design gradation and the Fuller gradation, which produces dense, well-graded mixtures, is shown in Figure 9. All three mixture designs have more material retained on the No. 4 sieve than the Fuller gradation. All three mixture designs are deficient in material passing the No. 200 sieve. It is felt that this lack of minus No. 200 material could have contributed to the overall damage to these moisture susceptible mixtures. This fine material normally helps to fill the space between larger aggregates, thus impeding the permeation of moisture throughout the mixture by minimizing the extent of interconnected voids.

Sand Equivalents

Surface coatings inhibit adhesion of asphalt cement on aggregate and thus increase the potential for stripping if moisture is present. Therefore, sand equivalent test values were examined to determine if there was a detectable relationship between sand equivalent and distress.

The average and range of the sand equivalent values, as computed by the residency laboratory during construction, are shown in Figure 11 for each design and category of distress. Designs 1 and 2 contained pit No. 1 sand and design 5 contained pit No. 2 sand. The average sand equivalents for the pit No. 1 and pit No. 2 sands were 45 and 49, respectively. Although these mean values were above the normally acceptable value of 45, 50 percent of the tests on the mixtures containing pit No. 1 sand (designs 1 and 2) were below 45 with values as low as 32 and as high as 59, indicating a high degree of variability in this sand (Figure 11). For the pit No. 2 sand only 25 percent were below 45 and values ranged from 40 to 60.

During construction the contractor was required to change to pit No. 2 because of difficulty with localized areas of dirty sand. Although the above suggests that the observed distress could quite possibly be related to the sand, no definite relationship was found between the average sand equivalent values and observed distress for designs 1 and 2. In addition, inspection of the individual sand grains under a microscope indicated that both sands had essentially the same shape. Thus, it was felt that the sand was not a direct cause of the distress unless the variation in fines in designs 1 and 2 could have caused problems.

Density

Daily construction records were examined to obtain density of laboratory-prepared Hveem stability specimens. It was noted that the densities of design 1 mixtures were always less than the preferred 97 percent of theoretical maximum density, design 2 densities were slightly greater than 97 percent, and all of the design 5 densities were much greater than 97 percent. In addition, it should be noted that design 2 densities for mixtures placed after construction resumed on March 10, 1980, were higher than those measured prior to January 18, 1980. In fact, the densities for designs 2 and 5 began to increase as soon as construction resumed. Although no satisfactory explanation is available, this behavior suggests a change in compaction conditions or materials.

Densities were determined for each core. Average core densities for the three designs are shown in Figure 12. Average core density for design 5 (150.9 pcf) was significantly higher than the overall average core density for the undistressed areas of either designs 1 or 2 (146.5 and 146.2 pcf, respectively). Thus, in addition to not being exposed to the winter and spring weather conditions, design 5 areas were so dense that any water present could not penetrate the mixture.

Stability

The Hveem stabilities of laboratory specimens prepared daily during construction were examined. Most of these values were between 35 and 45 and thus met the minimum stability requirements. Plots were made to investigate potential relationships between stability and distress location within designs 1 and 2 but none were found.

Hveem stabilities were also determined for two or three cores at most of the 26 sample locations shown in Figure 8. It should be noted that these Hveem stability values cannot be interpreted in terms of specifications because the tests were on cores; however, they can be evaluated on a comparative basis. The results indicate that the average stability for design 5 was higher than that for either design 1 or design 2, but only two cores were tested for design 5. Stabilities were generally lower in the distressed areas than in the undistressed areas and the variation tended to be slightly smaller. This could possibly be attributed to the fact that cores from distressed areas were damaged by stripping. Many of the values were quite low, ranging down to 10.

Tensile Strength

Tensile strength was determined for cores taken from the roadway. Static and resilient moduli of elasticity were also measured and showed the same trends as tensile strength. The dry tensile strengths of cores from distressed and undistressed areas were not significantly different for designs 1 and 2. However, cores from design 5 had a higher overall average dry tensile strength than cores from either design 1 or design 2. Wet tensile strengths, i.e., tensile strengths of cores subjected to moisture conditioning, were also evaluated and are discussed under moisture susceptibility.

Moisture Content

In situ moisture content was determined for slab samples of the binder course mixtures obtained at each coring location. The results summarized in Figure 13 indicate that design 1 and 2 mixtures contained more moisture than the design 5 mixture. This is not unexpected since the design 5 binder course mixture was covered by the wearing course almost immediately after placement and because its density was significantly greater than the densities from the other two designs. In addition, the areas where the design 2 mixture was covered contained less moisture than those areas which were not covered. Thus, it appeared that moisture content was related to distress and needed to be evaluated.

EVALUATION OF MOISTURE SUSCEPTIBILITY TESTS

To evaluate the moisture susceptibility of the various mixtures, the Texas freeze-thaw pedestal test and indirect tensile test were performed.

Texas Freeze-Thaw Pedestal Test

Pedestal tests (3) were performed on both the individual and the combined aggregates of each binder course mixture, with and without the antistripping additive. Previous work at the University of Texas at Austin has demonstrated that the test can distinguish between those aggregates that are susceptible to stripping and those that are not (3). Aggregate-asphalt mixtures that fail in less than 10 to 15 cycles generally can be considered to be susceptible to moisture damage or stripping. The results of the study are summarized in Figure 14. The results indicate that all three designs were susceptible to moisture damage and that all four individual sands were moisture susceptible, especially the gem sand and the coarse sand. It can also be seen that there was no difference in the moisture susceptibility of the field sands from the two different pits. The inclusion of the antistripping additive did not change the test results and it apparently was not effective in increasing the resistance of the mixtures or the individual aggregates to water damage.

Figure 12. Core densities for the various mixture designs.

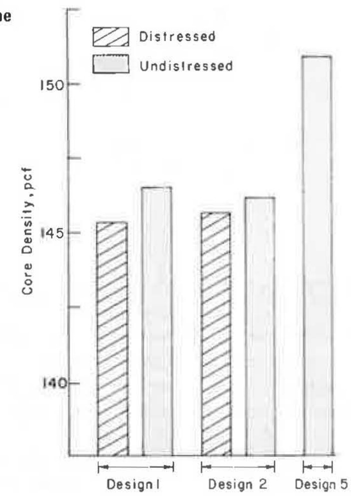


Figure 13. Moisture content of roadway slabs of the binder base course.

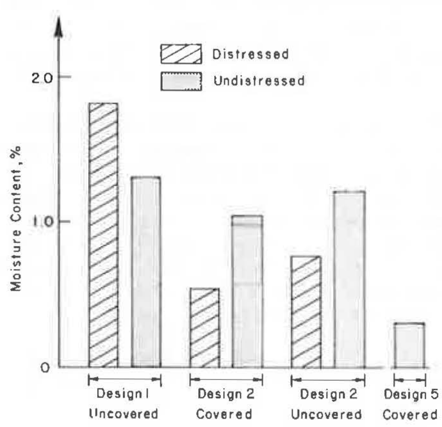


Figure 14. Texas freeze-thaw pedestal test results for individual aggregates and for the binder course mixtures.

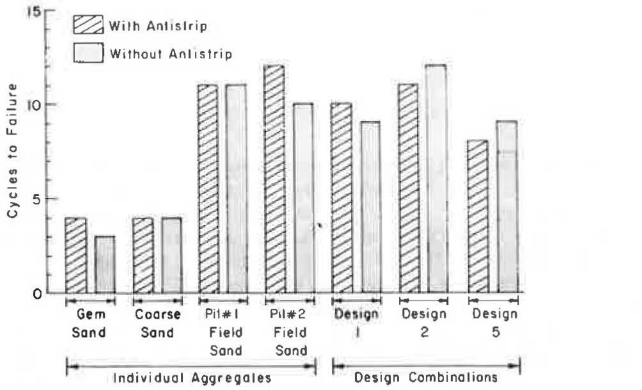


Figure 15. Average dry and wet core tensile strengths for each mixture design.

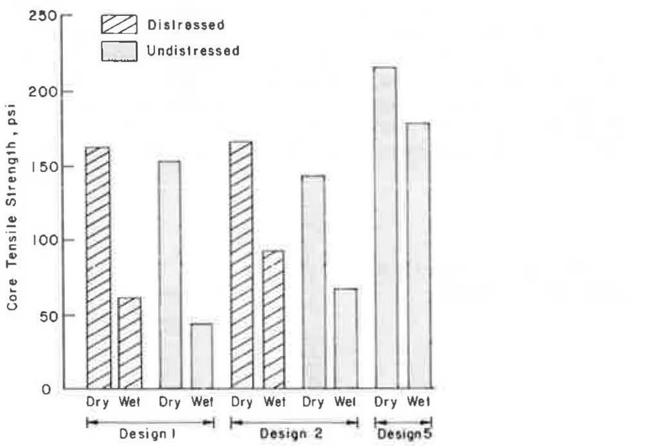
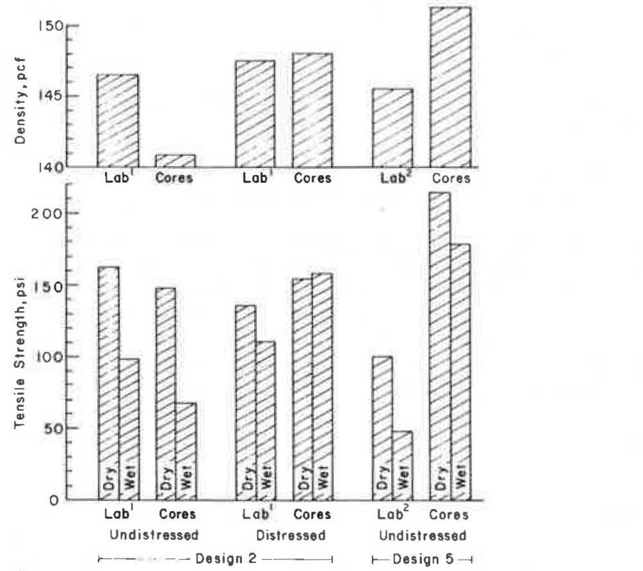


Figure 16. Effect of density on tensile strength after moisture conditioning.



[1]Standard compaction effort

[2]Core density could not be achieved with laboratory compaction equipment

Wet Tensile Properties for Cores

In addition to tests on dry specimens, tensile tests were performed on cores from each location after the specimens were subjected to moisture conditioning. Three methods of moisture conditioning were included:

1. Vacuum saturated freeze-thaw (VSFT). Specimens were soaked for 30 min under a 15-in. mercury (Hg) vacuum and soaked for an additional 30 min at atmospheric pressure at 75°F, frozen 15 hr at 0°F, soaked 24 hr in a water bath at 140°F, and finally soaked 3 hr in a water bath at 75°F before testing.
2. Seven day high vacuum saturation. Specimens were soaked for 7 days at room temperature, 75°F, under a 15-in. mercury vacuum. After the vacuum saturation period, the specimens were immediately tested.
3. Thermal cycling (TC). Specimens were soaked for 30 min under a 15-in. mercury vacuum and soaked for an additional 30 min at atmospheric pressure at 75°F, followed by 18 cycles of freezing and thawing. One freeze-thaw cycle consists of 4 hr at 10°F followed by 4 hr at 120°F. Every other cycle was followed by 8 hr at 75°F. The specimen was placed in a 75°F water bath during the last 8 hr rest period. Specimens were then immediately tested.

After conditioning, the tensile strength of each core was determined using the indirect tensile test. The results are shown in Figure 15.

Cores for design 5, after being moisture conditioned, retained a higher percentage of original dry tensile strength than those for either of the other mixtures. Lottman (4) has suggested that retention of less than about 70 percent of the dry tensile strength values when tested wet indicates a mixture that is moisture susceptible. When subjected to the most severe moisture conditioning methods, cores of mixtures from designs 1 and 2 generally retained much less than 70 percent of the dry tensile strength, thus indicating their moisture susceptibility.

Wet Tensile Properties for Laboratory-Prepared Specimens

Laboratory specimens were prepared to evaluate the effect of density on the moisture susceptibility of two of these binder course mixtures. Specimens were compacted at a standard compactive effort and at an effort that produced densities equal to those observed in the field for designs 2 and 5. The results are summarized in Figure 16.

The tensile strengths of the laboratory specimens of the design 2 mixture were similar to that of the cores; however, the tensile strengths for design 5 mixtures prepared at standard compaction effort show a significant loss (greater than 50 percent) after specimens were moisture conditioned (Figure 16). For design 5 mixtures the densities prepared with standard compaction effort were much lower than the field core densities. In fact, it was not possible to compact laboratory specimens to the field densities measured for design 5. These test results substantiate the findings from the pedestal test and suggest that the strength retention of the design 5 cores is due to the high field density which prevented reduced moisture penetration. Thus, if moisture penetrates the mixture, severe damage would be expected in all three mixtures.

CONCLUSIONS

Based on an analysis of the construction data and the laboratory findings, it is concluded that the primary cause of distress was moisture-induced

Table 1. Estimates of sulfur use in the state of Washington for various percentages of SEA paving mixtures.

Percentage of Total Hot-Mix Using Sulfur-Extended Asphalt	Total Sulfur Required (tons)	Total Asphalt Savings (tons)[a]
5	4,900	2,500
10	9,750	5,000
25	24,400	12,500
50	48,800	25,000
75	73,100	37,500
100	97,500	50,000

Notes: 1 ton = 0.91 metric tons. Estimates are based on annual hot-mix production of 5 million tons (4.5 million metric tons).

[a]Based on the assumption that optimum content requires the same volume of binder. Binder content for conventional hot-mix is assumed to be 5.5 percent (by weight of total mix) and 6.5 percent (by weight of total mix) for sulfur-extended asphalt (30/70 ratio).

A third study has been initiated in the state of Washington to investigate the use of SEA paving mixtures. The study is being conducted by FHWA, Western Direct Federal Division, and involves the construction and evaluation of an SEA paving mixture placed on the Baker River Highway near Concrete, Washington (constructed in the summer of 1981).

The SEA studies conducted in this state and others are being used to examine the strength and durability of these paving mixtures. The added question is whether enough sulfur will be available at an acceptable price to use in the replacement or extension of asphalt cement.

If the sulfur supply predictions prove adequate, how much asphalt cement might be saved by using SEA mixtures? Table 1 provides an estimate of the saving for various percentages of SEA mixtures as a function of the total annual hot-mix production for the state (estimated at 4.5 million metric tons or 5.0 million tons). If all of the hot-mix production were SEA mixtures with a 30:70 ratio, the state of Washington requirement for sulfur would be slightly less than 90,000 metric tons (97,500 tons), and the corresponding asphalt saving would be 45,400 metric tons (50,000 tons). Using a current asphalt price of \$190 per metric ton (\$170 per ton), the break-even price for sulfur is about \$95 per metric ton (\$85 per ton). However, price is only one issue; if asphalt is placed on an allocation system, price may be delegated to a somewhat secondary issue. If all hot-mixes produced in the state were SEA mixtures, an amount equal to about 1.5 percent of the total sulfur production of Alberta would be needed for this purpose.

SUPPLY AND DEMAND

Sulfur is one of the most important of the industrial raw materials. To quote the Kirk-Othmer Encyclopedia of Chemical Technology:

> Sulfur, one of the most versatile and essential elements on this planet, is the chemical industry's most widely used raw material. In fact, its applications are so widespread that sulfur consumption is often used as a measure of a nation's economic activity.

Historically, an adequate supply of sulfur has been available to meet the demand at an annual growth rate of 4 to 4.5 percent. In 1974-1975, however, the supply of sulfur decreased and the price of sulfur increased as a result of supply and demand. The price continued to be high through 1976, and a debate occurred as to whether the world would experience a glut or shortage of sulfur in the next 10 to 20 years. This lack of consensus was summarized in an article published in 1976 (2); two paragraphs from that article are as follows:

> On the glut side: existing world stockpiles of a record 26 million metric tons (28.6 million tons) of surplus sulfur; the likelihood of large tonnages of recovered sulfur being produced in the Middle East as a result of sweetening operations at natural-gas-exploitation projects underway there; the probability of more sulfur being recovered as a result of the U.S. and Western European environmental legislation targeted at powerplant stack-gas emissions; the virtual certainty that more high-sulfur crude will be processed in the U.S.; and the expected growth of recovered sulfur supplies following commercialization of projects for synthetic natural gas, coal liquefaction, shale oil and other alternative energy sources.
>
> On the shortage side: predicted declines in sulfur associated with dwindling gas-fields in currently strong producing areas such as France and Canada; doubts about the rate of recovered sulfur in the next decade; lack of strong economic incentives for continued growth of Frasch (mined) sulfur production in the U.S.; reservations about the marketability of small tonnages of recovered sulfur from scattered individual sources; and rapidly inflating capital costs for Frasch facilities, recovery units and shipping and transportation.

The supply of sulfur remains low and the lack of agreement regarding future sulfur availability continues. Numerous individuals in several organizations continue to analyze historical data, projected world trends, and technological improvements in sulfur recovery in attempts to resolve the uncertainty of the future availability of sulfur. The motivation for examining this future availability is extensive ongoing research into the potential use of sulfur as a construction material.

This paper provides background regarding the uses of sulfur, types of sulfur deposits, identified sulfur reserves, current sulfur recovery technology, and the importance of price in maintaining an adequate supply of sulfur. The paper concludes with projected sulfur supply and demand trends for the world, the United States, and the state of Washington.

USES

Sulfur is unusual when compared with most major mineral commodities because it is used largely as a chemical reagent rather than as a finished component. This means that before it can be used in industry it must be converted to an intermediate chemical. Sulfuric acid is the most important of these intermediate chemicals. In 1978 approximately 85 percent of the sulfur consumed in the United States was either produced directly in or converted to this form.

The distribution of U.S. sulfur consumption in 1978 by end-use categories may be broadly summarized as follows (3):

1. Agriculture accounted for more than 60 percent.
2. Petroleum refining includes petroleum refining and associated chemical processes where process streams may serve both the refinery and the chemical complex. Sulfuric acid requirements for these processes accounted for 8 percent.
3. Nonferrous metal production, which includes the leaching of copper and uranium ores with sulfuric acid, accounted for 6 percent.

stripping of the asphalt cement from the aggregate. A summary of findings that support this conclusion are as follows:

1. Design 5 and the portion of design 2 placed after the winter had a lower field moisture content than the rest of the project. This was due to (a) both sections had higher densities although the density of design 5 was substantially higher, (b) neither design section was exposed during the winter, and (c) both design sections were covered soon after being placed in the spring.
2. The densities of design 5 and the portion of design 2 placed after the winter were significantly higher than for the rest of the project.
3. The tensile strengths and the portion of the tensile strength retained after being subjected to moisture were much greater for design 5. This is attributed to the increased density which produced lower voids, higher strengths, and reduced moisture penetration. The Hveem stabilities on the cores for design 5 were higher than for designs 1 and 2. There were essentially no differences among Hveem stabilities of laboratory-prepared, job control specimens.
4. The Texas freeze-thaw pedestal test values and the retained tensile strength after moisture conditioning indicated that all aggregates are highly susceptible to moisture damage.
5. All laboratory tests and field observations indicated that the antistripping agent was not effective in preventing moisture damage.

EPILOGUE

During July 1982, an extensive amount of this roadway was removed by a cold milling operation. After the top 1 to 2 in. was removed, the underlying materials were almost devoid of asphalt indicating almost total stripping of the binder course.

ACKNOWLEDGMENT

This investigation was conducted at the Center for Transportation Research, Bureau of Engineering Research, The University of Texas at Austin. We wish to thank the sponsor, the Texas State Department of Highways and Public Transportation, for permission to publish the results of this study.

The contents of this paper reflect our views and we are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Texas State Department of Highways and Public Transportation. This paper does not constitute a standard, specification, or regulation.

REFERENCES

1. The Asphalt Institute. Mix Design Methods for Asphalt Concrete and Other Hot-Mix Types. Manual Series No. 2, Fourth Edition, March 1974, p. 32.
2. C.L. Monismith. Asphalt Paving Mixtures, Properties, Design, and Performance. Prepared for the Short Course in Asphalt Paving Technology, The Institute of Transportation and Traffic Engineering, University of California at Berkeley, 1961-1962, 143 pp.
3. T.W. Kennedy, F.L. Roberts, and K.W. Lee. Texas Freeze-Thaw Pedestal Test for Evaluating Moisture Susceptibility for Asphalt Mixtures. Research Rept. No. 253-3, Center for Transportation Research, University of Texas at Austin, Feb. 1982.
4. R.P. Lottman. Predicting Moisture-Induced Damage to Asphaltic Concrete. NCHRP Rept. 192, 1978.

Publication of this paper sponsored by Committee on Flexible Pavement Construction.

Availability of Sulfur for Sulfur-Extended Asphalt Paving in Washington State

JOE P. MAHONEY

The availability and pricing of sulfur with respect to sulfur-extended asphalt paving mixtures are assessed. The assessment includes a review of past and current trends as well as estimates of the availability of sulfur up to the year 2000 for the United States and specifically the state of Washington.

CURRENT RESEARCH

The current research related to sulfur in the state of Washington examines the potential of using sulfur for partially replacing or extending the asphalt cement in asphalt concrete. The first field experimental work was sponsored by the Washington State Department of Transportation (WSDOT). The results were reported in a study entitled "Sulfur Extended Asphalt Binder Evaluation." In this experiment, sulfur-extended asphalt (SEA) paving mixtures were placed in August 1979 at two test sites near Pullman, Washington (1). One site was an existing state highway (SR 270) and the other was the Washington State University (WSU) Test Track.

Based on initial findings from the first SEA project, a second study was initiated by WSDOT with the University of Washington (UW) entitled "Sulfur Extended Asphalt Laboratory Investigation." The stated goals of this study are to

1. Further evaluate the applicability and desirability of using SEA paving mixtures in the state of Washington.
2. Develop design criteria that will improve the utilization of SEA mixtures.
3. Assess the availability and pricing of sulfur in the state of Washington.

The third goal is addressed in this paper.