- - - - - *LTRC* -

TECHNICAL ASSISTANCE REPORT

I-10 PAVEMENT DISTRESS
RAMAH - WESTOVER
EAST & WEST SECTIONS
STATE PROJECT NOS.
450-08-0022
450-07-0031
F.A.P. NOS.
IR-10-3-(152)141 & IR-10-3-(151)135

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ABSTRACT

A fourteen mile, 6.5-8.0 inch asphalt concrete overlay of continuously reinforced concrete pavement demonstrated delamination, leaching, and rutting distress prior to acceptance in June 1989. Twenty-six areas ranging from 20-1000 feet were either partially or full depth removed and replaced. An extensive field and laboratory investigation was conducted to determine the cause of the distress.

Full depth asphalt concrete overlay roadway cores were sampled at sites designated as either moisture damaged or rutted along with samples at non-distressed areas. Observations during this evaluation detected moisture damage within each of the lifts placed. Each core was returned to the laboratory where individual lifts were separated and the asphalt extracted. Asphalt content and gradations were determined. Materials were sampled from the contractor's plant and were tested for moisture susceptibility in the Louisiana boil test, modified Lottman test and Texas pedestal test.

Rut depths were determined for the entire project in 1989 and 1994 at 0.25 mile intervals in the outside wheelpath of the outside lane . Three trenches were cut across the outside lane in 1994 to determine the origin of the rutting problem.

The moisture susceptibility testing demonstrated that each of the individual materials was moisture susceptible by either the boil test or pedestal test. Modified Lottman tests indicated that the overall mixtures were also susceptible. The rutting was found to initiate in the wearing course mix. An analysis of the mix design found that the gradation approved for use met specifications but provided the maximum packing capacity leaving minimum void space. The asphalt content approved for use was found to be from 0.2 - 0.5 percent higher than optimum as determined by standard practice. The combination of minimum void space in the aggregate structure and excess asphalt provided the opportunity for permanent deformation. Recommendations are provided for revisions to the department's mix design procedures and guidelines for gradation and voids analysis.

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INTRODUCTION

State Project No. 450-07-0031 and State Project No. 450-08-0022 were constructed by Cook Construction Company and Dolphin Construction Company, respectively. Dolphin subcontracted its asphalt work to Cook such that Cook completed all asphalt concrete work on both projects. This work consisted of a 6.5 - 8.0 inch asphalt concrete overlay of 14.4 miles of continuously reinforced concrete pavement. The existing distress consisted of cracked and faulted concrete pavement caused from loss of subgrade support due to drainage conditions. Upon reviewing the projects for final acceptance in June 1989, a number of distressed areas were identified which included ravelling and what appeared to be delamination of the wearing course leading to pot holes.

Testing of these areas was initiated under the direction of Mr. Donald Carey, Bituminous Construction Engineer. Roadway cores were taken at twelve locations by District 61 laboratory personnel . A report {1} was written by Mr. Said Ismael, district lab engineer in August 1989. As this work was progressing the roadway continued to deteriorate and additional delaminated areas were identified. In addition, some rutting was noted at selected areas, generally after a construction joint and what appeared to be leaching was discovered. Additional sampling of the roadway continued with the involvement of both the Materials Laboratory and LTRC. In each case, numerous samples were cored to discover the reason for the distress and to define the bounds of the distressed area. Generally, cores were taken in both "good" and "bad" areas. Approximately 120 roadway samples were evaluated in all. Moisture damage was noted during the coring operation. At various locations either single lift or multiple lifts indicated damage. Each core was separated into individual lifts and the specific gravity was determined. All lifts were extracted and tested to determine compliance with specifications for asphalt content and gradation. The data from this evaluation are included in Appendix A.

Also, some testing for moisture susceptibility was accomplished using materials sampled from the contractor's plant. Boil tests were conducted on the plus No. 4 fractions of the coarse aggregate, modified Lottman tests were conducted on the binder and wearing course mixes and Texas pedestal tests were conducted on individual source aggregate materials.

The results of this evaluation revealed that all materials used on this project were moisture susceptible and their composite mixtures, which included liquid anti-strip agents, were also susceptible. With the exception of a few cores, all samples met roadway density requirements. In general, the extracted samples demonstrated that the mean asphalt content and gradations were within specification limits. However, individual samples were identified with high and low asphalt contents and gradations on specific sieve sizes that were either coarse or fine. Truck end segregation was visibly apparent on this project. In fact, after approximately 8.0 miles of wearing course construction in the west bound direction, the design optimum asphalt content of 5.1 percent asphalt cement content by weight was increased to 5.3 percent for the remainder of the project in an attempt to reduce the truck end segregation. Rutting of 0.5 inches or greater was identified at several locations.

On the basis of this evaluation, 26 areas ranging from 20 feet to 1000 feet were recommended for repair. Depending on the distress and the observations during coring, full depth or partial removal and replacement was accomplished.

Dual wheel rutting became increasingly prevalent after the first several summers. Mr. Gordon Nelson, District Maintenance Engineer, requested complete removal and replacement of the full depth overlay in April 1994. In July and August 1994, district maintenance forces and LTRC personnel further investigated the rutting situation by cutting trenches and removing the overlay at three locations. It was determined that the permanent deformation was only occurring in the wearing course. It was also noted that moisture damage was indicated in the binder course and levelling course mixtures by the lack of bond between lifts and the lack of cohesion in the asphalt concrete. Additionally, rutting measurements were taken at 0.25 mile intervals for the entire project for comparison to the 1989 rutting survey.

Recommendations were forwarded to cold mill the existing wearing course and replace it with a less rutting susceptible surface course such as a stone mastic asphalt, SMA. It is anticipated that moisture damaged areas may be uncovered during the milling operation which will require patching to the depth of damage. It is recognized that additional maintenance may be required in the future due to moisture damage but that these costs should be less than complete removal of the existing overlay.

This report documents the laboratory and field evaluations conducted from 1989 to the present. Additional recommendations with respect to specifications are provided.

DISCUSSION OF RESULTS

MOISTURE SUSCEPTIBILITY

Field Evaluation

The initial distress observed on these projects was manifested as delaminations of the wearing course which had begun to develop pot holes. Also, a white colored substance appeared to be leaching from the surface in several of the areas where the delaminations occurred. In response, the district lab cored twelve locations to evaluate the in-place materials at the direction of Mr. Donald Carey, Bituminous Construction Engineer. Cores at locations 2, 3, 4, 5, 6, 8, 9, 10 and 12 were identified in reference {1} as being porous looking, breaking up or with lack of cohesion. A review of the wearing course gradations provided in the reference, however, indicates that with the exception of one core being fine passing the No. 200 sieve and one core with a high asphalt content, the cores were within job mix formula limits.

In an effort to define the extent of moisture damage, additional areas were cored by either the Materials Lab or LTRC. As this information was being obtained, evidence of delaminations or leaching was discovered in other areas. In each case a series of cores was taken to determine the extent and severity of possible moisture damage. These cores were designated as series 20, 21, 23, 24, 25, 30, 80, 140, and 150. Series 31, 32 and 33 were taken in areas which did not appear to have any damage. Observations were made at the time of coring. The specific gravity was determined for each lift of each core. Individual lifts were then extracted to determine asphalt content and gradation for compliance to specifications.

As shown in Appendix A, moisture damage was found in each of the suspected moisture susceptible areas while no damage was observed in areas 31 - 33. A statistical analysis of gradations and asphalt content was performed. In general, there was no correlation between gradations for those cores with moisture damage versus those cores without moisture damage. Extracted gradations were found to be outside JMF limits both in "good" areas and "bad" areas. Sieve sizes most prevalently out of JMF limits were the Nos. 4, 10 and 40 sieves. There were 11 wearing course, 4 binder course and 5 levelling course samples with out of tolerance gradations. There were 2 levelling course with high asphalt content, 2 levelling course with low asphalt content, 2 binder course with high asphalt content and 4 binder course with low asphalt content. The dispersion of data in the gradation and asphalt content analysis indicates that truck end segregation was apparent in the roadway samples. There was no correlation between out of tolerance mix and "good" and "bad" performance. Only two cores for each individual course were found to be out of compliance with the minimum density requirements.

Laboratory Evaluation

The JMF indicated that the coarse aggregates used in the wearing course passed the specification required Louisiana boil test with the sandstone and siliceous limestone having 98 and 95 percent retained asphalt. Upon discovering the field moisture damage and leaching and after evaluating the cores, Mr. Said Ismael boiled and soaked a roadway core for several cycles. The core appeared slightly stripped and the fine sand appeared to be leaching. At that point, the district lab obtained samples of the fine and coarse sands from the contractor's yard. Boil tests on these sands provided no useable information.

LTRC was completing a research study at this time in which a number of moisture susceptibility tests were evaluated. It was decided to examine the Cook materials in a series of tests from this study. In addition to the Louisiana boil test, a modified Lottman test was used to evaluate the full mix design and the Texas pedestal test developed by Kennedy at the University of Texas was used to evaluate individual aggregate source materials.

Boil tests were conducted on a proportional blend of the coarse aggregates used in the binder/base course mix. Tests were conducted on the aggregate with no liquid antistrip material, 0.8 percent Permatac (Permatac was the antistrip used by Cook), and 1.0 percent hydrated lime applied to the aggregate in slurry form. Table 1 presents the results of retained coating. It is noted that these results indicate that the minimum required retained coating of 90 percent was only achieved with the use of hydrated lime. As such, the coarse aggregates used for these mixes were moisture susceptible.

TABLE 1. BOIL TEST, RETAINED COATING	
No Antistrip	30%

0.8% Permatac Antistrip	72%
1.0% Hydrated Lime	90%

Based on the work in the district lab and the leaching observed on the roadway, it was believed that the fine sand was the primary source of the stripping problem. A modified Lottman test and the Texas pedestal test were used to evaluate the total mix and individual sands, respectively. In the Lottman testing, samples were fabricated at both the 7 percent air voids recommended in the test procedure which represents typical mix after construction and at 10 percent air voids which would represent the maximum allowable air voids under the specification. In addition to the materials used for these projects, fine sands from two other contractors were evaluated to determine differences in performance.   Typically, a tensile strength ratio of wet strength to dry strength of 75 percent is considered acceptable. Table 2 presenting the Lottman results indicates that regardless of air void content or fine sand source all mixes are moisture susceptible with or without liquid antistrip agents. The use of hydrated lime produced acceptable results. Since the fine sand did not influence the results, the coarse aggregate and coarse sand are implicated.

The Texas pedestal test was then used to evaluate the coarse and fine sands individually. In this test a small pill sample is submerged in water in a closed container and then subjected to a minimum of 20 freeze/thaw cycles or until the sample fails. Typically, failure in less than 10 cycles indicates moisture susceptibility, 20 plus cycles indicates non-moisture susceptibility and 11-19 cycles indicates possible moisture damage. As shown in the results provided in Table 3, the coarse sand is moisture susceptible and the fine sand could be susceptible. Neither the addition of liquid antistrip nor hydrated lime improved these results.

TABLE 2. MODIFIED LOTTMAN RESULTS			
Mix Tested	Wet Strength PSI	Dry Strength PSI	Tensile Strength Ratio
Cook F.S., 0.5%A/S, 10% Air Voids	32	90	36
Delta F.S., 0.5%A/S, 10% Air Voids	29	80	36
Cook F.S., 0.8%A/S, 10% Air Voids	28	92	30
Cook F.S., 0.5%A/S, 7% Air Voids	43	139	31
Barber F.S., 0.5%A/S, 7% Air Voids	34	123	28
Cook F.S., 1% Lime, 10% Air Voids	80	90	89
Delta F.S., 1% Lime, 10% Air Voids	77	80	96

TABLE 3. TEXAS PEDESTAL, CYCLES TO FAILURE		
Treatment	Coarse Sand	Fine Sand
No Antistrip	5	18
Antistrip	6	19
Hydrated Lime	6	19

During the August 1994 field rutting evaluation, trenches were cut across the outside lane to determine the origin of the rutting problem. It was noted that moisture damage was indicated in the binder and levelling course mixes in that there was a lack of bond between lifts and the mixtures seemed to be soft and lack cohesion. The slabs were returned to LTRC where cores were drilled. The cores were then cut into the various lifts. Modified Lottman tests were conducted on the binder and wearing course lifts; the levelling course was not suitable for testing. The results presented in Table 4 confirm the moisture susceptibility found previously.

TABLE 4. MODIFIED LOTTMAN TESTS, ROADWAY			
Mix Tested	Wet Strength PSI	Dry Strength PSI	Tensile Strength Ratio
Binder Course	109	182	60
Wearing Course	74	107	69

Summary

Observed field delamination and leaching problems were investigated through an extensive sampling program which demonstrated moisture susceptibility. The roadway samples were evaluated in the laboratory for compliance to specifications with respect to gradation, asphalt cement content and density. While a small percentage of the field samples were found to be out of JMF tolerance limits, no correlation existed between "good" and "bad" areas on the roadway. The dispersion of data from the roadway samples indicates that truck end segregation was evidenced within the sampled areas.

A laboratory analysis of both individual materials and mix designs used on this project demonstrated without qualification that each coarse aggregate and the coarse sand were moisture susceptible. The fine aggregate demonstrated borderline moisture susceptibility. Mixtures prepared in the laboratory meeting JMF designs also demonstrated moisture susceptibility problems. Laboratory testing of roadway samples also indicated that the in-place mix was moisture susceptible.

As a result of the work completed in research study 85-1B, "Compatibility of Aggregate, Asphalt Cement and Antistrip Materials" {2} and the findings on this project, the standard use of the modified Lottman test was recommended for immediate implementation. The modified Lottman test was incorporated into specifications by early 1990.

RUTTING

Field Evaluation

The initial investigation of rutting on these projects consisted of the evaluation of a non-rutted area and a rutted area identified in Appendix A as areas 26 and 27, respectively. Roadway cores were taken transversely across the outside lane at the inside edge, inside wheelpath, center lane, outside wheelpath and the outside edge. Each core was saw cut into separate lifts and specific gravities were determined. The inside and outside edges in both areas were found to be less densified than the wheelpaths, which is reasonable. For the wearing course mix, the air voids were found to be higher in the non-rutted area than the rutted area demonstrating the additional compaction in the rutted area. Also, the asphalt content in the rutted area was 0.2 percent higher than the non-rutted area.

As the laboratory work on moisture problems was progressing, additional rutting was detected on the roadway. It was noted that often the rutting appeared immediately after a construction joint, continued for several hundred to a thousand feet and then stopped, indicating that perhaps morning start up problems were contributing to the problem. Fourteen additional sites were evaluated with paired rutting and non-rutting areas identified. Generally, the rutted areas measured 0.15 - 0.20 inches deeper than the non-rutted areas. At each site three cores were drilled in the outside wheelpath, inside wheelpath and center lane, designated A, B, and C, respectively. These data are identified as 1A, 1B, 1C, 2A, etc. in Appendix A with the rutted areas having odd numbers and the non-rutted areas having even numbers. Again, each core was divided into individual lifts, the asphalt was extracted and gradations determined. Only 6 of 126 samples were found to be out of gradation tolerance limits, mostly in levelling course mix. In three of the seven paired areas, though, the asphalt content was found to exceed the JMF limits in the rutted areas. A statistical analysis using t-test procedures indicated no significant differences between gradations or asphalt content in the rutted versus non-rutted areas in either the binder or wearing course mix with the exception of the Nos. 40, 80 and 200 sieves in the wearing course mix. For these exceptions the rutted areas were less than 1.0 percent finer.

Figure 1 presents the specific gravities of the wearing course samples. There is no consistent pattern in density between rutted and non-rutted locations. Only 6 of 126 samples inclusive of all lifts did not achieve the desired density. It is noted that the air void content for wheelpath samples is generally less than 3.0 percent. In-place air voids less than 3.0 percent can lead to plastic flow and the rutting distress observed in subsequent evaluations.

Rutting measurements were then taken at 0.25 mile intervals in the outside wheelpath of the outside lane for the entire length of the project in both directions to document initial rutting and serve as base line data for future evaluation. This data is included in Appendix B. Table 5 presents the average and range of rut depths measured. It is noted that the average measured rut depth in the eastbound direction is 0.26 inches and 0.16 inches in the westbound direction. Closer observation of Appendix B indicates that the rut depths for approximately the first eight miles, westbound, ranged from 0.05 to 0.15 inches with rut depths of 0.20 to 0.40 inches thereafter. This is known to be directly related to the design asphalt cement content being supplied to the mix. Because of truck end segregation in the first eight miles, the contractor was directed by department personnel to increase asphalt content 0.2 percent above optimum, or from 5.1 to 5.3 percent asphalt cement by weight. It was believed that the additional asphalt content would increase film thickness and reduce the segregation problem.

TABLE 5. SUMMARY OF RUT DEPTHS, 1989

Roadway	Average	Range
Westbound	0.16	0.05-0.40
Eastbound	0.26	0.20-0.40

In July 1994 rutting measurements were again examined at 0.25 mile intervals in the outside wheelpath of the outside lane in both directions. This data is also included in Appendix B and is summarized in Table 6. An increase in rut depth was found throughout the entire project. There are significant areas with rut depths exceeding 0.5 inches which could pose a hazard to the public. It is again observed that the first eight miles westbound have significantly less rut depth than the remainder of the project. With several exceptions, the measured rut depth is 0.25 inches or less within this area which is considered normal and due to typical consolidation by traffic.

TABLE 6. SUMMARY OF RUT DEPTHS, 1994

Roadway	Average	Range
Westbound	0.39	0.05-1.2
Eastbound	0.50	0.20-1.0

Trenches were cut in the roadway in August 1994 at three locations to determine the origin of the rutting. The slabs were removed to the LTRC laboratory where lift thicknesses were measured. In each case, while moisture damage was indicated in the binder and levelling course layers, the rutting was measured to occur in the wearing course mix only.

Summary

The rutting distress was evaluated in 1989 and 1994 through the measurement of rut depths and the examination of roadway samples. The 1989 evaluation demonstrated field consolidation beyond that normally observed in Louisiana mixes (typically 0.25 inches). With a few exceptions, no statistical differences could be found in densities, gradations or asphalt content between the rutted and non-rutted locations. It was noted that the first eight miles westbound of the project had significantly less rutting than the remainder of the project. At that point the mix design was changed raising the asphalt content to 0.2 percent above optimum. The contractor added the additional asphalt at the request of the department to alleviate a segregation problem at truck exchanges. In 1994, the rutting had progressed such that potentially hazardous areas were identified.

ANALYSIS OF THE RUTTING PROBLEM

Background

In March and April 1986 the DOTD Secretary and the Construction and Maintenance Engineer requested that the asphalt specifications committee make changes to the asphalt specifications to require the following:

- 100 percent of aggregate material to pass the 3/8 inch sieve in wearing course mixes similar to Mississippi DOT mix designs;

- minimum roadway 96 percent compaction of plant density;

- two percent hydrated lime; and,

- minimum percent asphalt cement content.

During this same time period, the FHWA issued Technical Advisory T 5040.24 which was a compilation of standard practices found to contribute to good mix design, construction and placement of asphalt concrete. Essentially, DOTD already complied with 90 percent of these recommendations. DOTD did not comply, however, with the use of the 0.45 power curve for gradation design and the use of voids in the mineral aggregate, VMA.

After consideration of these requests, a supplemental specification was approved by the department in December 1986. This specification incorporated most of the requests as follows:

- While the FHWA 0.45 power curve was not directly specified, wearing course gradations were modified to allow 70-100 percent aggregate passing the 3/8 inch sieve. Some fractions were retained on the 1/2 inch along with minimal retention on the 3/4 inch sieve to account for the 3/4 inch material contained in reclaimed asphalt pavement materials. The specification limits were generally equidistant from the 0.45 power curve using the 1/2 inch sieve as the nominal maximum size aggregate.
- Voids requirements were changed to require 2-4 percent air voids in the mix, 75-85 percent voids filled with asphalt and roadway compaction to be 96 percent of plant

density. It was believed that these requirements would produce a final in-place mix meeting the requirements of the FHWA TA of 6-8 percent air voids and meet the minimum percent asphalt content requested by the department.

- Additionally, hydrated lime was required at the rate of 2 percent (this requirement was later dropped)

- A new high type wearing course was designated, type 8, which would include aggregate classified by its ability to provide surface friction properties. Most of the aggregates which met the classification requirements for type 8 mix were absorptive.

- Percent crushed aggregate minimums were also increased.

These specifications were not approved by the FHWA as they did not include the specific use of the 0.45 power curve, 3-5 percent air voids, reduced material permitted to pass the No. 200 sieve and a dust/asphalt ratio as specified in the TA. However, the FHWA did conditionally approve the use of the December 1986 specifications on a project by project basis to gather information. The December 1986 specification was used on these projects. Upon the department's decision to approve the 0.45 power curve as specified in the TA, the FHWA approved the supplemental specifications in December 1987. In August 1988 a new Application of Quality Control Specifications for Asphalt Concrete Mixtures {3} was issued which included the use of the 0.45 power curve.

Upon observing the failures and completing the evaluation of the roadway samples on these projects in the fall 1989 and early 1990, it was realized that the specification requirements had gone too far in an attempt to increase asphalt content and the voids requirements were returned to 3-5 percent air voids and 70-80 percent voids filled with asphalt. In addition, in an effort to open up the mix to accommodate additional asphalt the use of VMA as recommended in the FHWA TA was included. Also, with the recognition of the moisture problems on these projects and the completion of the LTRC study on compatibility of materials, the modified Lottman test was recommended for immediate implementation. New supplemental specifications were approved for use in December 1990 and September 1991 which incorporated these changes.

Review of Project Data

Appendix C contains the JMF and the mix design optimum asphalt content curves used for the wearing course mix on these projects. It is noted that this was one of the first state projects to use the new specifications which required the use of an aggregate according to its friction rating. Unfortunately, Louisiana does not have a native aggregate which meets the friction criteria for a type 8 wearing course. With the advent of this mix type, imported aggregates began to be used with which the department had no experience. The friction aggregate used on this project was absorptive. As most of the Louisiana experience is with non-absorptive aggregates, apparent gravities are used to compute the theoretical gravity of a mixture rather than effective gravities. However, noting that an absorptive aggregate was being used, the department amended its mix design procedure to use an effective gravity for all absorptive aggregates in the calculation of the theoretical gravity. This was the case for these projects.

The specific gravity used for the AA35, sandstone aggregate appears high compared to LTRC experience with this aggregate source. Generally, LTRC has found this aggregate's specific gravity to range from 2.58-2.60. Specifically, material tested from this project sampled from the contractor's yard tested as 2.60 in the LTRC laboratory. Since the Rice method to determine effective gravity was new at this time frame to most of the district and the materials labs, this variation is understandable. The difference in gravity, though, would have the affect of reducing the mix theoretical gravity to 2.452. If this gravity was used, the voids characteristics would be changed such that design air voids would be 2.1 percent and voids filled with asphalt, VFA, would be 85. In all probability, this job mix at the extreme level of the specifications would not have been approved.

The optimum asphalt content as determined from the specification criteria for the four point average was 5.06 which was rounded to 5.1. This asphalt content was used for approximately the first eight miles of construction in the westbound direction. At that point as indicated by project records, the asphalt content was increased 0.2 percent at the direction of the construction section to mitigate a segregation problem occurring at the end of every haul truck and the beginning of the next haul truck. The asphalt content then used for the remainder of these projects was 5.3. It is noted that the section of roadway placed using 5.1 percent asphalt is demonstrating consolidation of 0.25 inches which is consistent with experience.

Further observation of the optimum asphalt curves indicates that if the asphalt content for air voids is determined at four percent (air voids 3-5) and for VFA at 75 (VFA 70-80) as required by the revised December 1990 specification, the asphalt content would be 4.5 and 4.8, respectively. Further, using some subjectivity, choosing the asphalt content for density at 5.0 percent, the first of the highest two measurements of density, and averaging these values with 5.0 percent for stability, the average asphalt content would be 4.8 percent. Therefore, the production asphalt content for 75 percent of the project was over designed by 0.5 percent according to the revised December 1990 specifications.

Use of the 0.45 Power Curve and Voids Analysis

Prior to the development of mix design methods, gradation was used as the primary means to determine asphalt content in bituminous mixtures. Soon on air voids were recognized as a mix parameter which directly contributed to field performance. Incorporating these thoughts, much research was conducted from the 1920s through the 1960s in the development of mix designs with respect to the ideal packing capability of various gradations to optimize asphalt content for durability and air voids for performance { 4, 5, 6, 7, 8, 9 }. The 0.45 power chart as developed by Goode and Lufsey {9} has remained the primary tool for evaluating gradations. When gradations are plotted on a log-log chart of percent passing versus sieve size the densest aggregate packing will be obtained on a line with a 0.45 slope. Unfortunately the original researchers never determined how the 0.45 power curve should be drawn. As a result there are numerous methods being used routinely.

In the development of the December 1986 specifications there was also disagreement on how to draw the maximum density line and the 0.45 power curve was not instituted. It was essentially used, though, because the newly developed specification bands (allowing 100 percent of the material to pass the 3/8 inch sieve per the request from the Secretary) was drawn equidistant from an ideal 0.45 line drawn from 0 to a point with 100 percent passing the 1/2 inch sieve, i.e., using the 1/2 inch sieve as the nominal maximum size aggregate. With the tolerance limits imposed in a job mix formula and the criterion that tolerance limits could not be expanded beyond the specification limits, a contractor was essentially forced to produce a gradation on, or parallel to, the 0.45 power curve. In fact the initial October 1988 Application Manual enforces this concept as it directs "Theoretically, if the mix were formulated with the gradation represented by the 0.45 power curve, there may be inadequate voids for asphalt cement or additives. Therefore, all points for a job mix formula should plot parallel to, slightly above or below, and reasonably equidistant from the 0.45 power curve... Job mix formula gradation proposals which when plotted on the Asphalt Concrete Gradation-0.45 Power Curve, and compared to the 0.45 power curve show humps or which cross the power curve (with the exception of the No. 200 sieve and the top sieve), can be indicative of resulting mixture problems. Such proposed gradations should be adjusted." The effect of these instructions, because of the tolerance and specification limitations, which are still being used to date by many contractors, is that very dense mixtures are being produced which have insufficient void space for asphalt cement; a parallel line drawn on the 0.45 power chart still produces a slope of 0.45 which is the densest possible packing.

Although misapplying the 0.45 power chart concept, it was also recognized that there was a need to generate enough space in the aggregate structure to accommodate enough asphalt in order to produce a durable mix. It was decided that the best method to achieve this goal was the use of voids in the mineral aggregate, VMA as desired by the FHWA in their letter disapproving the December 1986 specifications. Again there was disagreement within the specification committee. The concept of requiring a minimum amount of void space in the aggregate structure has been used frequently in order to justify additional asphalt cement in the mix. McLeod successfully argued the case in Canada of using minimum VMA requirements to provide thicker asphalt films to provide a more durable mix to help alleviate low temperature cracking problems. However, recognizing the mathematical inter-relationship of void parameters (air voids, VFA, VMA), McLeod {10, 11} provided that when two of these parameters are specified, the third parameter is determined. When all three parameters are specified, the contractor is limited to an extremely narrow operating band which is generally smaller than normal test variation. Noting this idea, but wanting to satisfy the FHWA request, the committee decided to incorporate VMA as a mix design criteria only. VMA was instituted in the December 1990 specifications and is discussed in a Supplemental Application Manual {12}.

The implementation of the VMA criteria has provided problems as contractors have struggled to generate VMA. Recognizing the error of the "plot parallel" instructions in the Application Manual, revised instructions in the 1992 LA DOTD Mix Design Procedures {13} advise that the gradation "should form a line which curves away from the midpoint of the power curve." Additional instructions still require that a gradation which crosses the maximum density line will not be acceptable. While not written, contractors were additionally advised to produce mixes on the coarse side of the maximum density line. In many cases, such attempts resulted in lower VMA.

During this time period, the FHWA TA concepts were being strongly recommended nationwide. Huber and Shuler, principal researchers at The Asphalt Institute, investigated the problems associated with the 0.45 power curve, definition of the maximum density line and VMA {14}. Their work clearly demonstrates that the definition of the maximum density line promoted in TAI publications and subsequently promoted by the FHWA did not generate a line of maximum packing capacity. As a result, instructions to "move away from the maximum density line" often reduces VMA rather than increasing VMA. Figure 2 and Table 7 taken from reference {14} presents this finding convincingly. In this figure the maximum density line as defined by TAI, FHWA and DOTD is drawn from 0 to 100 percent of the sieve which first retains material. The gradations on this figure were designed to progress away from this line. It is found that rather than increasing VMA, VMA is highest, closest to the line and farthest away from the line (Table 7). In fact, the maximum packing appears to be somewhere in the mid point of the curves. Essentially, the researchers found that the best fit of minimum VMA and maximum density lines was that proposed by McLeod {15}. In this definition, the nominal maximum size is the next larger standard sieve on which at least 10 percent of the total aggregate is retained. The maximum sieve size would be one size larger than the nominal maximum size, which would be the smallest sieve size through which the entire amount of aggregate must pass. The maximum density line on the 0.45 power curve would be drawn from 0 to the maximum size. This

definition when tested with three data bases including that of Goode and Lufsey and the FHWA Demonstration Project No. 74, Field Management of Asphalt Mixes, statistically demonstrated the best fit with minimum VMA. It is this definition which is used in the Strategic Highway Research Program, SHRP, "Superpave" system.

TABLE 7. VMA RELATED TO DISTANCE FROM MAXIMUM DENSITY LINE {14}

Increasing Distance From Maximum Density Line	Minimum VMA, %	
	Crushed Aggregate	Uncrushed Aggregate
E	13.9	12.8
D	12.6	11.0
C	11.6	10.4
A	11.5	10.8
B	12.1	10.4
F	14.4	12.4

Figure 3 presents the type 8 wearing course gradation used on the Ramah-Westover projects. For this particular case because of the fineness of the coarse aggregate, the maximum density line defined by reference {14} and the Superpave system and the line defined by TAI, FHWA and DOTD are the same. It is noted, as discussed earlier and consistent with the instructions provided in the department's Applications Manual, that the contractor's gradation runs parallel to and slightly away from the maximum density line. Similarly, the contractor has, thus, attained the maximum possible packing of the aggregate system leaving very little void space for asphalt. When this information is combined with either the 0.2 percent additional asphalt to alleviate the segregation condition or the 0.5 percent additional asphalt provided by the specification voids requirements analyzed in the previous section, the rutted condition of the roadway is easily understood. Interestingly, this curve does not pass through the "forbidden" zone of the SHRP specification. According to SHRP procedures, however, the mix would have to demonstrate that it does not over-compact when subjected to gyratory compaction.

In an effort to adjust specification gradations and increase nominal size aggregate, Mr. Sam Cooper, Bituminous Construction Engineer, and Moore and Associates laboratories developed several mix designs including 3/4 inch top size aggregate {16}. These gradations are presented in Figure 4 with the SHRP maximum density line and exclusion zone depicted. It is noted that these gradations cross the maximum density line three times violating Application Manual guidelines. In addition, both gradations pass through the exclusion zone. As reported, these mixtures were tested with Koch Materials Laboratory's Hamburg Rut Wheel Tester, which has been documented to identify mixtures subject to field rutting. After 20,000 passes these mixes attained less than four millimeters of deformation indicating no susceptibility to rutting. These results suggest that the Application Manual guidelines need immediate revision. Further, they suggest that the use of the 0.45 power curve and the SHRP exclusion zone are tools solely to be used to evaluate possible gradations. As such, they are not absolute. The results suggest that once mixture and voids criteria are met, additional testing of the mix needs to be accomplished to test for resistance to shear whether that testing be in the gyratory compactor or some other surrogate rut testing device such as the Hamburg device.

Summary

Analysis of the wearing course mix design and the optimum asphalt curves indicates that the mix contained from 0.2 to 0.5 percent too much asphalt for the gradation of aggregate used. The excess asphalt was because of a number of factors which included lower specification air voids, higher specification VFA, the use of a higher than actual theoretical specific gravity and the direction to use excess asphalt to mitigate a segregation problem. It is noted that the specification changes in voids criteria initiated in December 1990 should alleviate the excessive asphalt problem. The current specified use of the materials transfer device should stop the practice of adding asphalt to prevent segregation.

A review of the department's gradation specification and mix design procedures indicated that rather than creating sufficient void space for the asphalt cement, the current specification band and instructions on the application of the 0.45 power curve and VMA tend to produce a mix with the densest possible packing. There is an immediate need to institute revisions in the gradation specification and the use of the 0.45 power curve. Because of the amount of time

passed, the existing practice has become ingrained. A special training session on mix design practice and voids analysis is strongly recommended.

The rutting experienced on these I-10 projects is directly attributable to a densely packed aggregate with minimum void space combined with excessive asphalt cement content. It is believed that the mix is still plastic and will continue to deform. The wearing course should be removed and replaced at the earliest opportunity.

CONCLUSIONS

1. Field evaluations in 1989 and 1994 demonstrated that moisture damage was occurring in the levelling, binder and wearing course mixtures on these products. Roadway cores were soft or pliable and sometimes readily broke in pieces with uncoated aggregate. Trenches cut in the roadway displayed similar weakness.

2. Laboratory evaluations using the Louisiana boil test, modified Lottman test and the Texas pedestal test demonstrated that each of the aggregate materials were moisture susceptible. Subsequently, specifications were changed to require the use of the modified Lottman test to examine the moisture susceptibility of all asphalt concrete mixtures in addition to using the boil test to evaluate the coarse aggregate.

3. A rutting investigation conducted in 1989 demonstrated that initial consolidation of the wearing course was greater than that normally found in Louisiana with several areas experiencing 0.4 inch ruts. A second investigation in 1994 identified significant portions of the wearing course with rut depths greater than 0.5 inches indicating that the mix is in a plastic condition. Trenches cut in the pavement indicate that the rutting is limited to the wearing course mix.

4. Analysis of the project records, project specifications and application guidelines indicates that the gradation used produced the densest possible packing thereby providing minimum void space in the aggregate. Possible testing errors in the theoretical specific gravity determination and the specification design air void and VFA requirements produced a mix with excessive asphalt. The excessive asphalt content was further compounded by the field directive to increase asphalt content 0.2 percent above optimum to alleviate a segregation problem. The combination of minimal voids and excessive asphalt content produced a mix subject to permanent deformation. Specification void and VFA requirements have since been revised to address this situation. Problems still exist in the practice of the 0.45 power curve and VMA analysis.

CONCLUSIONS (Cont'd)

5. Testing of the roadway samples indicated that the contractor generally met all specification requirements. While a small percentage of samples were found to be out of job mix tolerance limits, no correlation could be found between performing and non-performing areas.

RECOMMENDATIONS

1. Revisions to the wearing course specification gradation band and the Applications Manual regarding the use of the 0.45 power curve and the use of VMA should be undertaken as soon as possible.

2. As demonstrated by the work of Huber and Shuler, the SHRP Superpave method of determining the maximum density line for the 0.45 power curve provides a better determination of the maximum density line. This method should be adopted immediately. However, the use of the 0.45 power curve and the SHRP exclusion zone should be used as tools and not as an absolute in the mix design process. Proposed mixtures should be subsequently tested to determine compaction properties.

3. Training should be initiated for both department and contractor personnel to teach the concepts of mix design practice incorporating the SHRP 0.45 power curve and voids analysis.

4. Additional development of gradation specifications incorporating a larger nominal size aggregate should continue. This work should include the subsequent testing of proposed specification bands with equipment such as the SHRP shear tester, the various rut test devices (Hamburg, French, Georgia) or the gyratory test machine to determine mix resistance to permanent deformation.

5. The extent of moisture susceptibility problems encountered on these projects emphasizes the importance that should be placed on the boil test and modified Lottman test by both district laboratory engineers and contractor personnel during the mix design process .

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APPENDIX A

ROADWAY CORE DATA

APPENDIX B
RUTTING MEASUREMENTS

APPENDIX C

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