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Field and Laboratory Studies

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Asphaltic Concrete Construction

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CORRELATION OF DENSITY AND STABILITY TESTS OF BITUMINOUS MIXTURES WITH SERVICE BEHAVIOR

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Full-Scale Asphaltic Construction in the Research Laboratory

R. J. SCHMIDT, California Research Corporation, Richmond, Calif.

The broad gap between research on asphaltic concrete specimens prepared in laboratory bench-type equipment and test roads or public highways has been narrowed by providing full-scale paving equipment in a research facility. This facility permits asphaltic concrete pavement construction under conditions of control which cannot be reached in field operations. The use of this equipment is described. Controls for all major construction variables are described and special techniques are detailed where appropriate. Methods include: large-scale aggregate handling, screening and gradation blending to laboratory tolerances; realistic drying and mixing, with close temperature controls; accurate measurement and recording of hot-mix temperatures throughout the mixing, placing, and rolling cycle; controlled steel- and rubber-wheeled compaction; sampling techniques for evaluation tests.

● WHENEVER something new appears, it is probably the result of a determined, well-planned research and development effort. If these developments are to be made rapidly and efficiently, the progress through the normal research sequence, from fundamental and exploratory to development, and finally to the field, must proceed without bogging down. The transitions between the early phases are straightforward, although they may require great effort and ingenuity.

The gap between the laboratory and the field is another matter. It is often the major obstacle limiting progress, although a large effort may be made to span it. Progress in asphalt paving research has been limited in this way; and the subject of this discussion is the progress in overcoming the problem.

Before a development is field tested, it should be explored by laboratory tests correlated with field performance. However, this is not always possible because correlated tests are not available. Even with good correlation, the test method may be questionable under any conditions except precisely those used in the correlation.

Sometimes it is impossible to develop a correlation because field conditions contain unknown factors not included in the test method. In this case, the development state in the laboratory is minimized in favor of development work in the field. However, this method also has serious disadvantages. For example:

1. The cost may be prohibitive because of the size or number of tests that must be made to give significant answers.
2. It may be impossible to cope with the uncontrollable or unknown variables.
3. Most field tests should be successful: failures are not good for public relations.
4. Field testing is slow and the rate of progress may not be fast enough.

A third alternate is to turn to functional tests which simulate field conditions as closely as possible under controlled conditions. An example of this is the successful use of test engines by the lubricating oil researchers.

This way has been chosen to bridge the gap between asphalt concrete (AC) laboratory work and field testing of finished pavements. The method is a stepping stone similar to the pilot plants used to develop large-scale chemical plant designs. It consists of making test sections with a small-size, full-scale hot mix plant and paving facilities in order to learn more about the methods of constructing AC pavements. Most of the problems connected with field testing are removed by controlling carefully all of the variables, including climatic conditions. The scale is large enough to prevent similtude problems. Figure 1 shows procedure used for this work.

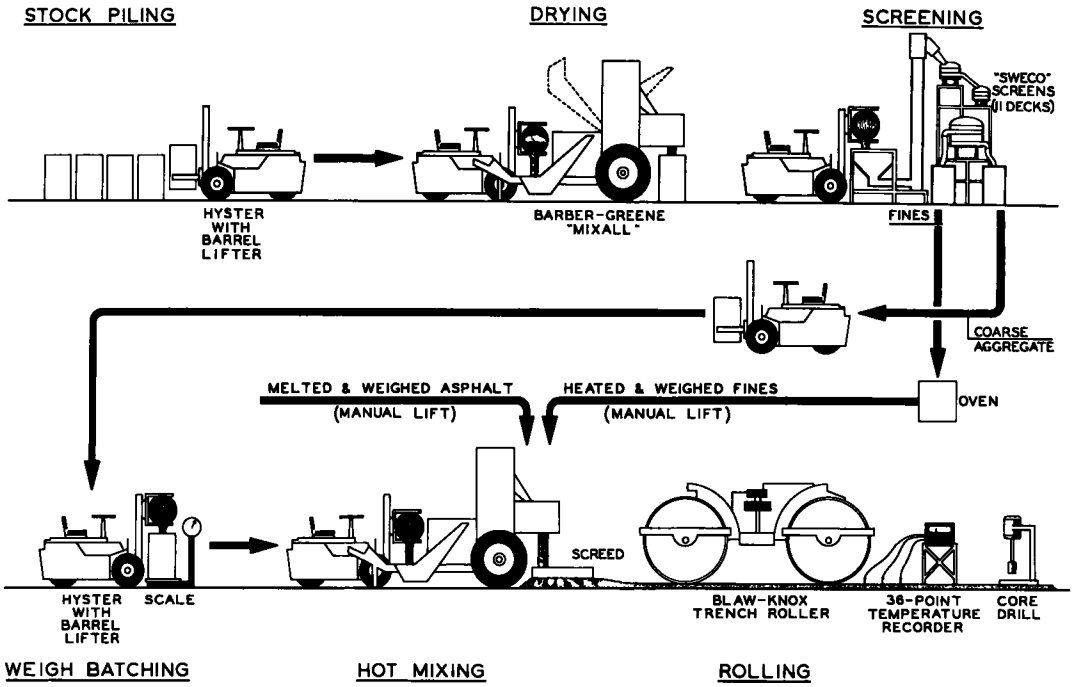

Figure 1. Construction of experimental asphalt pavements.

PROCEDURE USED

As indicated in Figure 1, the aggregates and hot mix are handled in steel drums by a lift truck equipped with hydraulically-operated barrel clamps. The aggregate is dumped directly from trucks into openhead drums and, if required, dried in a mixer. After cooling, it is screened into a number of separate sizes and, as needed, accurately recombined into individual, 300-lb batches. After an individual batch is preheated in the dryer, it is mixed with the required amount of hot asphalt and filler in the pug mill. The resulting batch of hot mix is screeded onto the test area and, along with several other batches, rolled with a trench roller. The temperature of the mix and base is automatically recorded throughout the paving operation. Tests are made both on the finished pavement and on the cores taken with a diamond drill.

CHOICE OF MAJOR EQUIPMENT

Commercially, aggregates are handled by conveyors and hoppers. However, this requires separate equipment at each step and the sequence is not easy to change. For the current purpose it was found best to handle all aggregate and hot mix in openhead drums carried about with a lift truck. In addition to being the lowest cost alternative, this method gave the greatest versatility in operations. The principal requirements on the lift truck were concerned with its ability to pick up an openhead drum of aggregate weighing 1,000 lb, transport it to another location, and pour it into a hopper. The lift truck shown in Figure 2, equipped with hydraulically-operated barrel clamps and a tilt mechanism, met these requirements.

One of the main problems in making experimental pavements is the accuracy with which aggregate gradings in mixes are reproduced. To do this, it is first necessary to screen the aggregates into a number of cuts and recombine them into separate batches. The screens used for this purpose should run continuously without having to change screen sizes at intervals and recycle the fine cut. Also, to keep the cost at a minimum the screens should be operated in series with only one feed elevator. Three

screening units, each containing three or four screens (Fig. 3) fill these requirements. The top two units are of 12 in. diameter; the bottom one, handling the fine gradings, of 24 in. diameter. Aggregates can be separated into as many as eleven cuts in one operation with this unit.

It is most important to reproduce the action occurring in commercial hot mix plants as closely as practical to avoid problems of similitude. The dryer should be of the same type as in a commercial plant, large enough in diameter to give the same aggregate wear, and fired with an adjustable open flame. Dust recovery and recycle are needed features on the plant; and the pug mill should be the heat-jacketed, twin-shaft type used in practice. Accurate control of aggregate and mix temperatures is of prime importance. The mixer shown in Figure 4 met these requirements after modifications had been made to collect the dust and to control the temperature of the mix.

The first requirement of the roller was that it be full size and weight so there would be no question of the functional value of the tests. Second, it should have two roller wheels arranged in such a manner that one roller at a time can be used; yet the second wheel must be easily interchangeable with the first on the test area within a minute's time so that more than one wheel can be used in a test. In addition, it should be possible to change the diameters and weights of the two wheels in a few hours without special facilities. The trench roller shown in Figure 5 meets these requirements. The front and rear pair of wheels steer separately so the roller can be operated with one set of wheels offset from the other. As shown in Figure 6, this permits one wheel to run alongside the test area on a ramp while the other wheel is on the test section. Adjustment of the steering permits an interchange of the wheels.

SPECIAL FEATURES

The mixer was modified to prevent excess dust loss and to allow good control of the final mix temperature. As shown in Figure 4, the flue and pug mill are completely hooded and discharged through a mechanically-driven centrifugal dust collector that returns the dust to the dryer or to a container as desired.

The thermocouples in the pug mill and in the dryer permit close control of the final mix temperatures provided the firing rate is carefully controlled by means of the flowmeter on the fuel supply. A thermocouple is imbedded in the wall of the pug mill and another extends into the mix. The thermocouple in the dryer is contained in a small chute which collects aggregate as it drops from the top of the rotating drum. This arrangement (Fig. 7) was required to prevent the thermocouple from sensing the temperature of the hot gases in the dryer instead of the temperature of the aggregate.

The wheels on the roller can be preheated by bubbling steam through the water or barium sulfate slurry used for ballast. This reproduces wheel temperatures experienced when a roller has been heated by high air temperatures and by the hot mix being rolled. Pneumatic compaction can be obtained on the experimental test sections by turning the roller end for end and using the pneumatic tires on the outrigger wheels. Tires of either 13.00-24 or 7.50-15 size are used.

The temperature profile in the mix before and during rolling is found from thermocouples buried at different levels in the mix. A special probe containing three thermocouples (Fig. 8) allows a temperature to be recorded at three levels simultaneously. One or more of these probes is inserted in each section being tested. In addition, thermocouples are permanently imbedded in the base at several levels. An example of a set of measurements taken during an experiment is shown in Figure 9.

The rate of heat loss from the unrolled hot mix on the test section can be controlled in several ways, thus simulating different weather conditions. The base may be preheated with a portable oven (Fig. 10), or cooled with dry ice to reproduce base temperatures found in the field. Alternatively, an insulating layer of paper or plywood may be put on the base to restrict the rate of heat loss, thus simulating the effect of a preheated base. The rate of cooling from the top surface is controlled by canvas, glass wool, or by pieces of plywood. The rate of heat loss is adjusted to reproduce the same temperature profiles previously measured under different weather conditions on mixes about to be rolled in the field.

Figure 2. Lift truck pouring hot mix from drum into screed box.

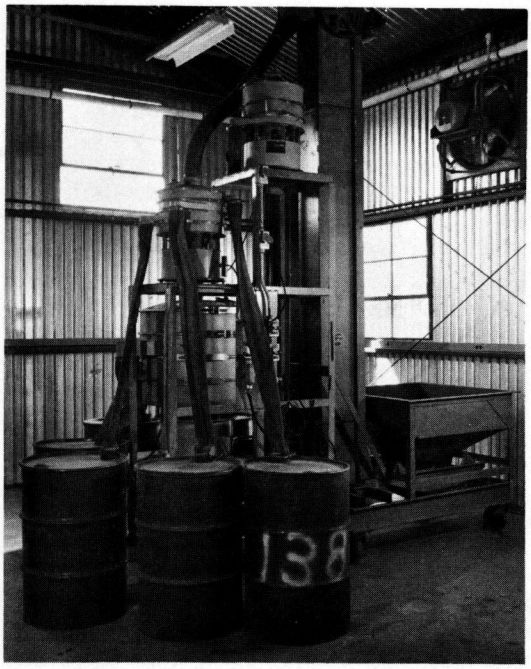

Figure 3. Screening facilities.

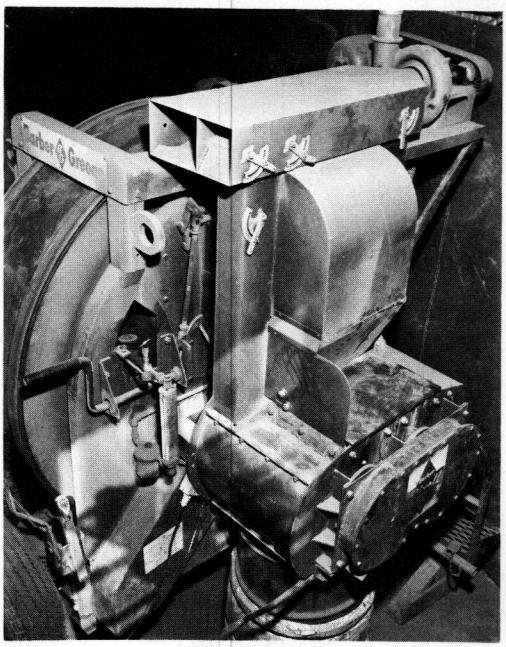

Figure 4. Hot mix plant and dust recovery system.

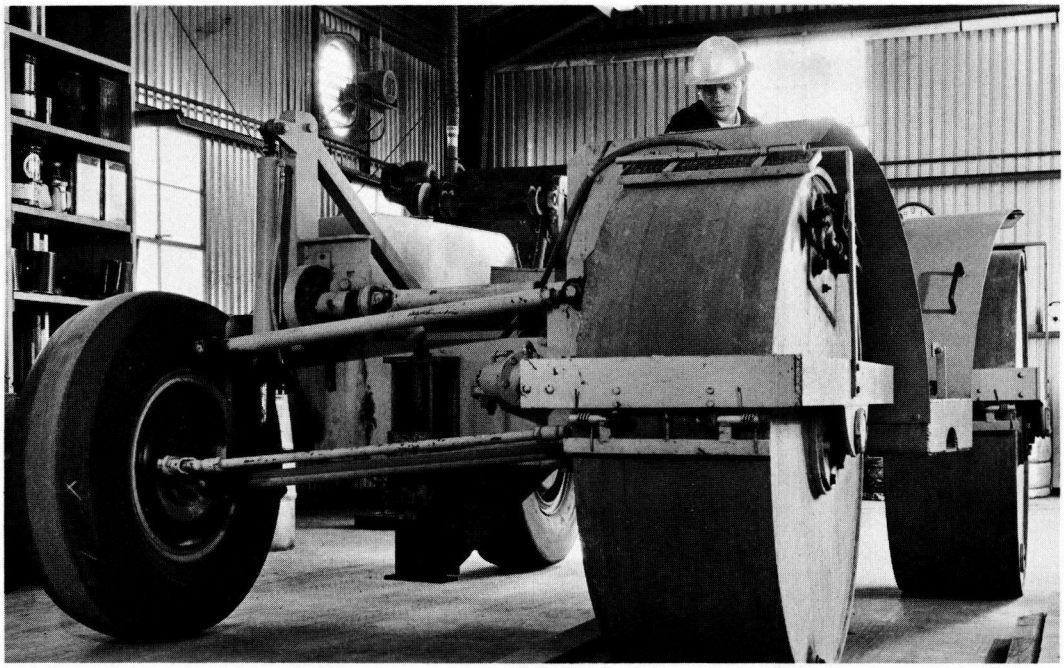

Figure 5. Experimental roller.

Figure 6. Rolling an experimental test section.

Although the equipment is normally housed and used in the building (Fig. 11), it is portable and can be moved into the field where experimental test sections can be made for traffic and durability studies.

The thickness of the AC being rolled is accurately controlled by the height of the screed with respect to the guiderails. This height is easily changed and can be continously varied by using tapered guiderails.

The base on which the experimental pavements are made can be changed to meet the requirements of the experiment. It can be either AC, portland cement concrete, crushed rock, or, if desired, a resilient or unstable base.

PRELIMINARY OPERATION

It has been possible to study as many as six different mixes or rolling temperatures simultaneously in one experiment; at the same time, two different roller weights or two degrees of rolling were also studied.

The temperature of hot mix delivered from the pug mill is within $\pm$ 5 F of the desired temperature; and by proper scheduling, rolling temperatures within $\pm$ 10 F of the desired temperature are obtained.

Before experimental pavements are

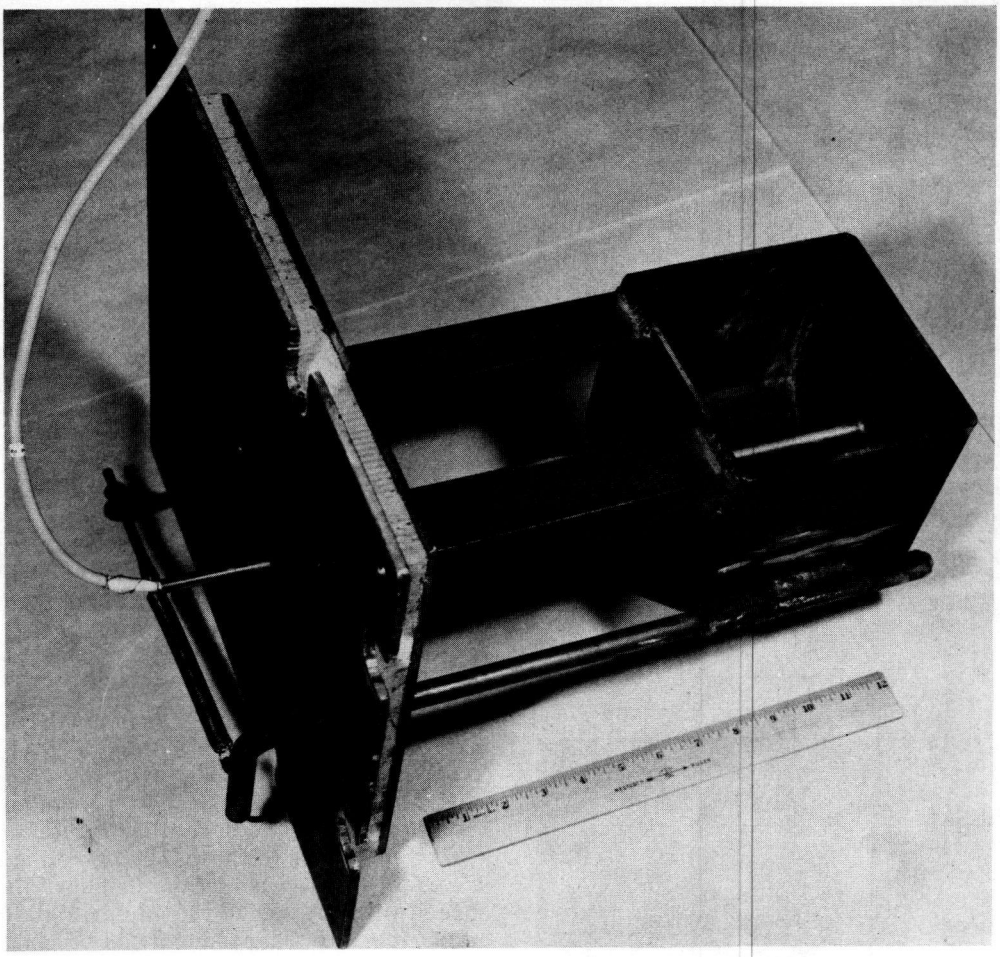

Figure 7. Continuous sampler (contains thermocouple) for sensing aggregate temperature in drier.

cored, they may be photographed, tested for softness, and their permeability measured. The cores may be tested for density, permeability, or stability by the Marshall or Hveem methods. Sometimes Abson extractions are made on the cores and on samples of the hot mix taken for this purpose.

The reproducibility is good, as indicated by the core densities shown in Table 1, which compares cores taken from two separate but closely controlled experiments. A similar spread in individual core densities is found on most field corings. However, when sections are cored to the extent shown in Figure 12, statistically meaningful comparisons may be made.

Examples of the kind of work possible with these facilities are described in another paper ("Behavior of Hot Asphaltic Concrete Under Steel-Wheel Rollers") in this bulletin, and in a report (1) given elsewhere on observed peculiarities in determining core densities.

No completely satisfactory comparison with field construction has been made because of the paucity of precise data on typical projects. The field conditions and results are difficult to establish and control, except on very special occasions. Nevertheless, the

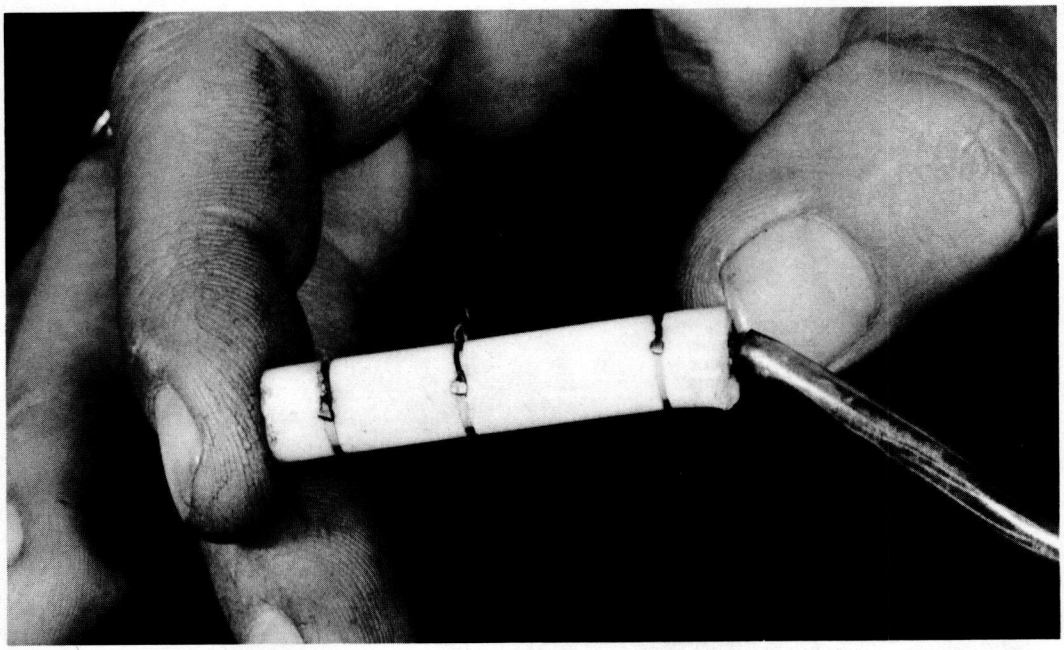

Figure 8. Three-level thermocouple probe.

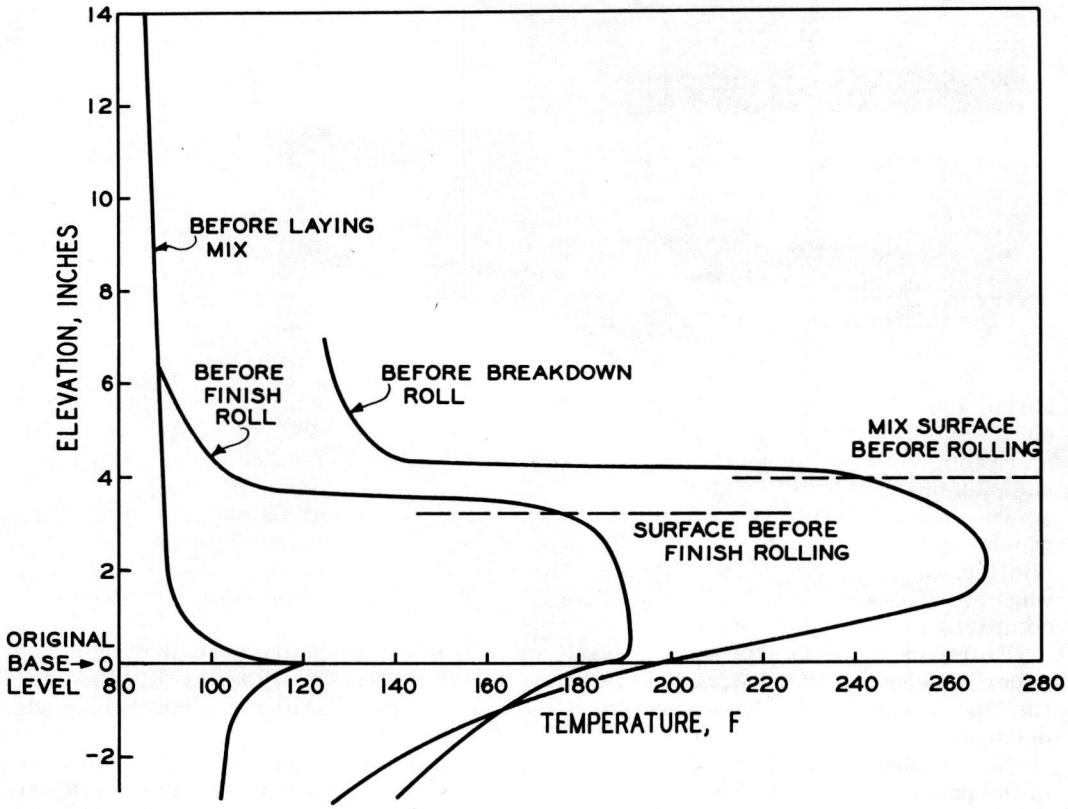

Figure 9. Variation in temperature during rolling of hot asphalt concrete.

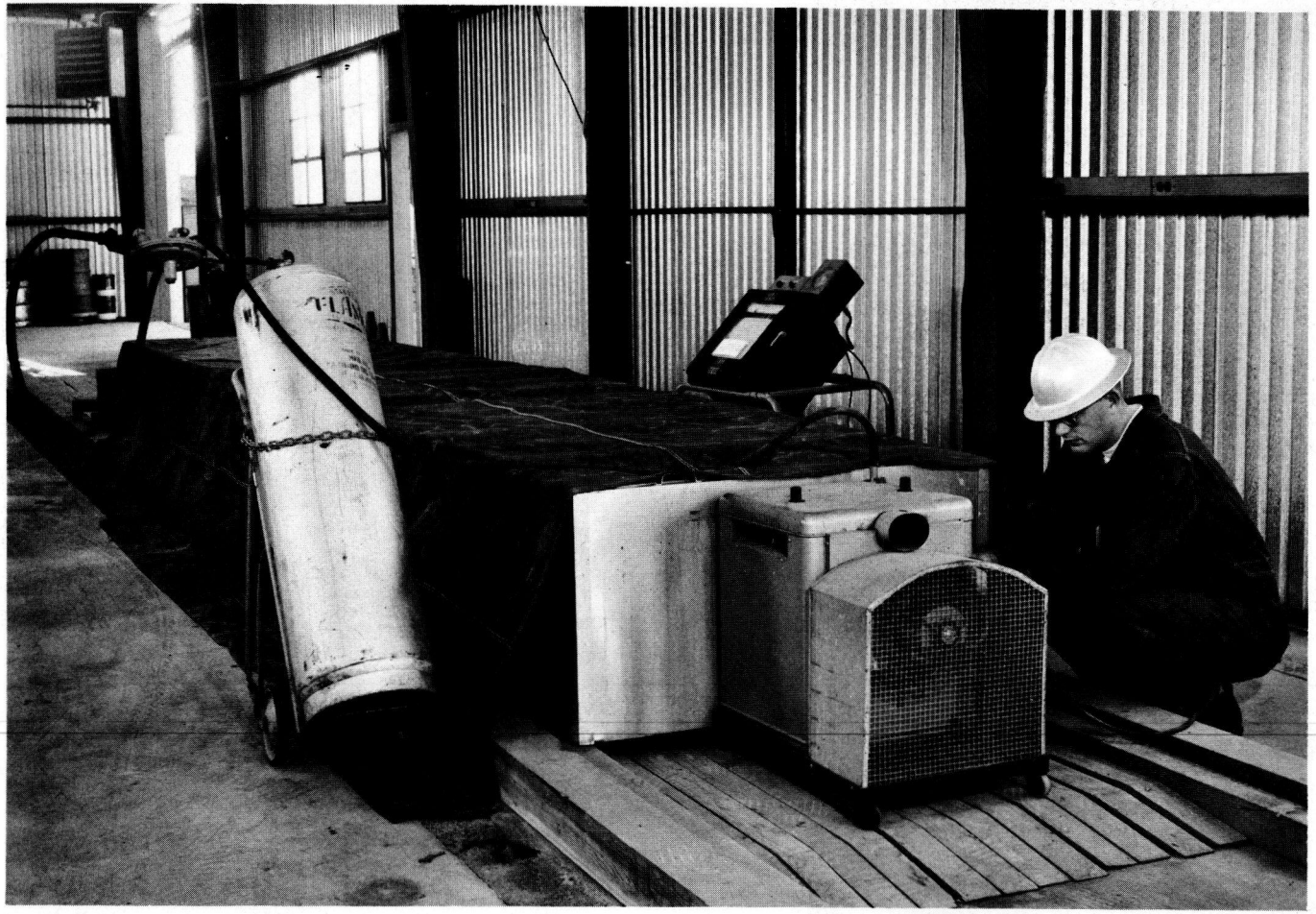

Figure 10. Preheating the base.

Figure 11. Interior of full-scale paving research laboratory.

TABLE 1

PAVEMENT EVALUATION DATA SHOW COMPARABLE RESULTS FOR EQUAL CONSTRUCTION PROCEDURES

Item	Test 01-10A	Test 01-10C
Aggregate	Cache Creek Gravel	
Grading	WB - dense	WB - dense
Asphalt	B-14277	B-14277
Asphalt added (%)	5.60	5.63
Asphalt extracted (%)	5.57	5.66
Extracted pen. at 77 F	45-44-44	44-45-45
Mixing temp. (F)	352	340
Breakdown temp. (F)	263	262
Roller passes:		
400 lb/lin in.	6	6
Pneumatic	4	4
300 lb/ lin in.	2	2
Core Densities (pcf):		
1	140.07	139.11
2	140.94	140.67
3	140.81	140.51
4	140.37	140.75
5	142.15	142.02
6	140.66	141.69
Average	140.83	140.79

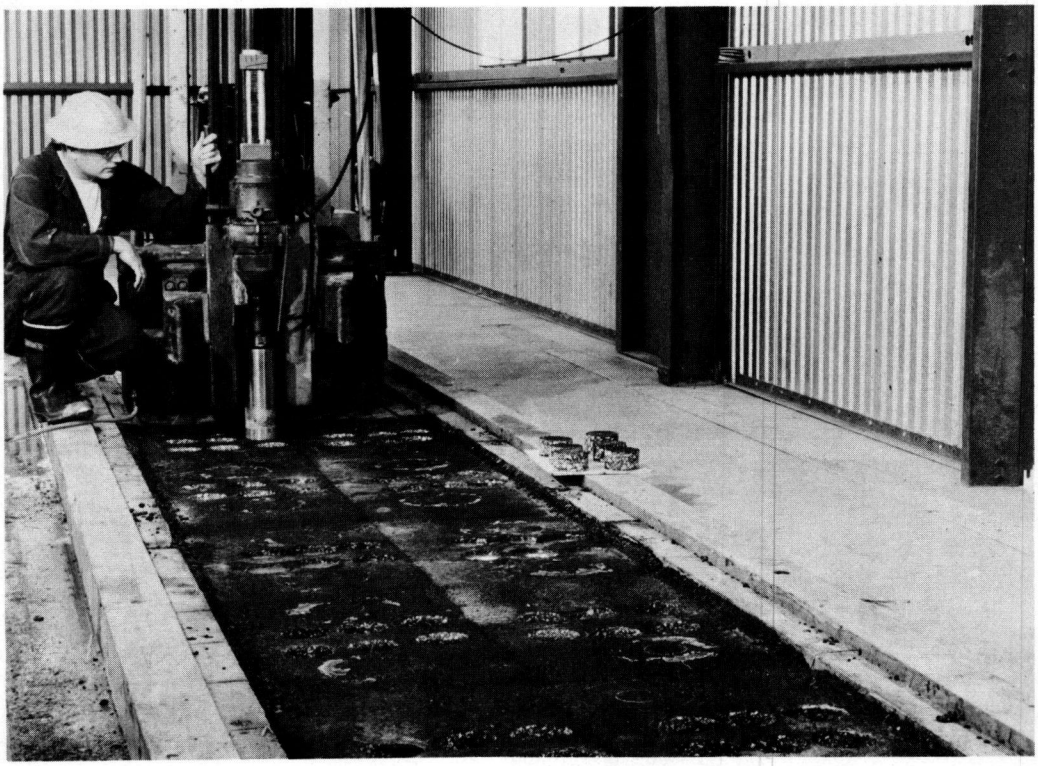

Figure 12. A cored test section.

experiments on construction behavior of hot mixes made with these facilities are giving information that is being applied directly to field problems without intermediate field testing. The procedures used are realistic and carefully controlled so there is more confidence in them than in the full-scale field tests.

REFERENCE

1. Hein, T.C., and Schmidt, R.J., "Density Changes in Asphalt Pavement Core Samples." Preprint, ASTM Annual Meeting, Atlantic City, N. J. (June 21-26, 1959).

Laboratory and Field Densities of Hot-Mix Asphaltic Concrete in Texas

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Laboratory and field data have been collected from twelve regular contract hot-mix asphaltic concrete paving projects in various areas of Texas for the purpose of comparing laboratory and field densities. Data from road samples taken from two to nine months after the roads were built show that five of the pavements exceeded laboratory density by 1 to 3 percent as determined by calculation methods currently used by the Texas Highway Department. None of the road samples measured by this method had a density lower than 97.2 percent (2.8 percent voids). The average density of the twelve different samples was 100.0 percent; the maximum density measured 103+ percent. The specific gravities of the mixes were then redetermined so as to include most of the absorption of asphalt cement by the aggregate. These data show an average of 94.6 percent, with a maximum of 97.3 percent. The range of differences brought out by the two methods was from 1.9 to 12.7 percent.

Data on mix design, traffic and field construction are included for background information.

●THE DATA PRESENTED in this paper are concerned with a portion of the research conducted for the Texas Highway Department by the Texas Transportation Institute on hot-mix asphaltic concrete. Studies are being made on the aggregates, the asphalt cements, the asphaltic concrete mixtures resulting from a combination of these materials, and the service behavior of the complete pavements in twelve different areas of Texas. Hot-mix asphaltic concrete designed to meet the current standard specifications falls into the general classification of dense-graded mixtures. As shown by data to follow, the specific mixes laid in different areas of the state show the considerable range in physical characteristics possible for such a material falling within a given set of specifications.

The particular point brought out in this part of the study deals with densification of the pavements by construction equipment and subsequent densification by vehicular traffic. Factors affecting pavement densification are considered and comparisons are drawn. The effectiveness of adequate and efficient compaction is shown and the need for more restrictive specifications is emphasized.

REVIEW OF THE LITERATURE

In 1937 Vokac (1), in a study of the service characteristics of 106 different cold mixtures with one year of service, made the following statement about voids in the finished mix:

> According to the data presented in this paper it is indicated that the one-year performance of a surface mixture is not critical to a reasonable precentage of voids. It is believed, however, that excessive voids would prove detrimental in a longer period of observation as excessive voids will undoubtedly permit the intrusion of moisture into the mix which may cause serious volume changes upon freezing. Moisture will also have a tendency to strip the bitumen from hydrophilic aggregates. The gradation of the aggregate is obviously an important factor in controlling the percentage of voids in the finished mix. Obviously, any expedient which will effect a reduction of the voids in a given aggregate will result in a greater strength with a given percentage of asphalt.

He also pointed out the importance of the compressive strength characteristic of a paving mixture and further stated that pavement resilience was not nearly as important as resistance to load. This was not qualified sufficiently and may be contradictory under certain circumstances.

Hveem (2), in 1940, observed that satisfactory aggregate grading curves tend to pass close to a point represented by the coordinates, 31 percent of the material passing a size equal to 0.031 of the particular maximum size of the gradation. Such curves are plotted on semi-log paper with the percentage retained on the arithmetic scale. Practical use of this observation is still made. For given materials and construction procedures, grading is an inseparable factor from pavement densification. All aggregate gradings studied in this project were checked against Hveem's "general grading chart." Reasonable compliance was found in all cases, although a number of the designers of these mixes were not aware of the existence of such a chart.

Kampf and Raisch (3), in their study of voids in paving mixtures, emphasize the importance of construction compaction and design of the mixture in the control of the voids of a finished asphaltic concrete pavement. In the compaction operation, the three items mentioned were weight of roller, temperature of mix during compaction, and stability of the mixture. The factors affecting voids in mix from the design point of view were listed as (a) the amount of asphalt cement, (b) the amount of filler (material passing the No. 200 sieve), and (c) available voids in matrix material.

In the research under consideration, the material passing the No. 200 sieve by wet sieve analysis was taken as filler in all instances.

Pauls and Rex (4) pointed out some of the problems associated with use of local aggregates. The questionable advantages of additives were discussed and it was mentioned that the aggregate, as well as the asphalt, is a factor in the selection of an effective additive. For one series of subangular aggregates tested, the use of 40-50 penetration asphalt was found advantageous from the stability viewpoint. For highly textured materials, penetration of the asphalt is a very minor factor in its effect on stability.

Hveem and Vallerga (5) stated that the terms density, permeability and compaction are related but not necessarily synonymous as they apply to bituminous mixtures. It was further stated that stability was neither related nor synonymous to the foregoing properties. That aggregate gradation has little predictable influence on stability was also mentioned. Experience with various aggregate gradings in Texas indicates that it is possible to produce stable mixtures although an aggregate grading may vary considerably between fixed maximum and minimum sizes.

Nevitt (6) mentioned the potential of the Texas gyratory shear compactor as an apparatus for laboratory compaction of hot-mix asphaltic concrete. This apparatus is simple and lends itself to field use quite readily. Correlation with field samples is shown later in data of this paper.

The data of Ortolani and Sandberg (7) compare favorably with the data presented, but indications are that present compaction equipment and procedures coupled with higher density and heavier traffic give higher field core densities than are obtained on laboratory specimens from the same mix design.

DESCRIPTIONS OF TEST SECTIONS

Table 1 gives the average daily traffic (ADT) for 1957 and 1958, the type of aggregate used in the various designs, and the type of base under each pavement. A study of the data to follow did not reveal any definite correlation between type of base and rate of densification, or type of base and density to date. The thickness of the layers placed, no doubt, is the prime factor for this lack of correlation.

It will be noted that the greatest thickness placed in any of the twelve roads was 2 in. The breakdown pass on all test roads was made with a 10-ton 3-wheel steel roller. In four instances a pneumatic roller was used for final construction densification. The effectiveness of this method is revealed by data to be presented. Actually, it is considered advisable by a number of asphalt paving technologists to use the pneumatic roller between the breakdown pass of the steel roller and the final rolling with a steel tandem roller. This procedure was observed in the construction of test sections 9 and

10. The rolling temperature given was the temperature of the mix immediately in front of the breakdown roller. No temperature measurements were made after rolling was completed.

Table 2 indicates considerable variation in the surface area of the aggregates used in the several test roads. The listed surface areas were calculated using the surface area factors suggested by the California Materials Manual (8). It is interesting to note that in the case of test section 4, with an asphalt content of 4.7 percent, there is an average film thickness of 4.81 microns, whereas test section 11 has a 4.4 percent

TABLE 1
TEST SECTION CHARACTERISTICS AND CONSTRUCTION DATA

Test Section	Traffic ADT 1957	Traffic ADT 1958	Aggregate Type	Base Type	Thickness of Surface, in.	Rolling Temp (F)	Roller Type
1c	4230	4600	Gravel and crushed limestone	PCC	1	310	10-ton, 3-wheel
1d	4230	4600	Gravel	Over 1c	3/4		10-ton, 2-wheel Light pneumatic
2	520	450	Gravel	PCC	1 3/8		
3	4220	4210	Crushed limestone	Flex.	2	310	10-ton, 3-wheel 15-ton, 2-wheel 20-ton, pneumatic
4	5410	5360	Gravel	PCC	1 3/8	300	10-ton, 3-wheel 8-ton, 2-wheel
5	1990	2280	Crushed limestone	Flex.	7/8 7/8	275	10-ton, 3-wheel 10-ton, 2-wheel
6	960	1060	Crushed limestone	Flex.	1 3/4	300	10-ton, 3-wheel 8-ton, 2-wheel
7	6040	6170	Shell	Flex.	1 1/2		10-ton, 3-wheel 10-ton, 2-wheel
8	6690	7090	Crushed limestone	Brick over PCC	1 1/4	250	10-ton, 3-wheel 10-ton, 2 wheel
9	20,000	21,660	Crushed basalt	2 in A C. over Flex	1/2	260	10-ton, 3-wheel pneumatic 260 lb/in 8-ton, 2-wheel
10	15,000	5970	Gravel	Flex	1 1/2	310	10-ton, 3-wheel 8-ton, 2-wheel 25-ton pneumatic
11	17,500	18,200	Crushed limestone	PCC	1	300	10-ton, 3-wheel 8-ton, 2-wheel
12	5710	6430	Gravel	PCC	1 5/8	275	10-ton, 3-wheel 8-ton, 2-wheel

TABLE 2
AGGREGATE AND ASPHALT DATA

Test Section	Design Asphalt Content[1] (%)	Agg. Surf. Area (sq ft/lb)	Avg. Film Thickness (μ)
1c	6.2	36.36	8.75
1d	5.5	34.70	8.06
2	5.5	41.78	6.68
3	5.5	29.88	9.36
4	4.7	49.42	4.81
5	4.3	32.18	6.72
6	5.0	43.36	5.83
7	7.5	35.38	11.07
8	4.8	35.64	6.80
9	5.0	26.48	9.55
10	5.5	30.48	9.16
11	4.4	30.00	7.36
12	5.0	38.38	6.62

[1]By total weight.

asphalt content with a resulting average film thickness of 7.36 microns. The tendency in Texas is to go to thicker films in order to improve the flexibility characteristics and extend the useful life of the pavement. This, of course, can be done at no increase in amount of asphalt cement used by simply reducing the surface area of a mix design by aggregate gradation changes. Some consideration also is being given to use of lower penetration asphalt cement. The penetration grade now in general use in Texas is 85-100. The apparent advantages of a harder asphalt are attractive. The effective film thickness most suitable cannot be a singular one; this is evident from the host of variables already listed for the few test roads under study. The proper film thickness may vary 100 percent or more from one aggregate and service condition to another aggregate and service condition. Such factors as shape, surface texture and absorption of aggregate, weight and density of traffic, and climate, must be taken into account. As noted in Table 2, the variation in film thickness for the twelve roads under study is from about 5 to 11 microns. Careful evaluation of these pavements indicates that all but possibly one (test section 11) would give considerably better service over a longer period of time if the film thickness had been increased. Test section 2 is the most outstanding example of this need. Low traffic density points to the need for greater film thickness. Well rounded aggregates used in this design indicate the need of a harder asphalt, because stability as determined by the Hveem method is improved for such aggregates with lower penetration asphalts.

Table 3 emphasizes the variation in specific gravities obtained by different methods. The values in Col. a were obtained from laboratory compacted specimens by weighing in air and water, those in Col. b by formula consideration of the individual specific gravities of the various components making up a given mix design. If absorption is neglected—and it is for the values in Col. b—this would give the specific gravity of a voidless mix. In practice, however, absorption is a factor that must be considered if an accurate measure of final voids is to be obtained. In spite of the need for consideration of absorption, it is the practice in Texas with some exceptions, to disregard aggregate absorption.

Col. c gives the specific gravities of the mixes under study as obtained by measuring the weight and volume characteristics of the composite mixtures in the loose state. A

TABLE 3
SPECIFIC GRAVITY DATA

(a) Lab. Sp.Gr (gm/cc)	(b) Theoretical Sp Gr. THD (gm/cc)	(c) Imp. Sp.Gr. (gm/cc)	(d) Lab Theor. x 100	(e) Lab. Imp x 100	(f) Road Sample Sp. Gr. Mo. Value	Mo. Value	(g) Max.Rd. Lab. x 100
2.25	2.30	2.42	97.8	93	9mo 2.25	20mo 2.24	100.0[1]
2.31	2.38	2.42	97.2	95	8mo 2.34	20mo 2 34	101.2
2 22	2.35	2.42	94 5	91.8	5mo 2.04	19mo 2.09	94.2
2.28	2.35	2.41	97.0	94.6	8mo 2.26	20mo 2.26	99.2[1]
2.34	2.44	2.48	96.2	94.4	7mo 2.42	16mo 2 40	103 3
2.37	2.49	2.53	95 4	93.8	7mo 2.33	17mo 2.39	100.7
2.38	2.44	2.45	97.6	97.2	7mo 2.31	9mo 2.40	101 0
2.17	2.25	2.34	96 6	92.8	8mo 2.12	8mo 2 16	99.5
2.39	2.43	2.46	98 5	97.2	4mo 2.34	16mo 2.37	99.2
2.39	2.50	2.57	95.7	93.1	const. 2.25	19mo 2 42	101.2[1]
2.32	2.39	2.42	97.2	96.0	2mo 2.35	14mo 2.35	101.2[1]
2.43 THD	2.47	2 49	98.4	97.6	2mo 2.42	16mo 2.45	100.8
2.29	2.43	2.44	94 3	94.0	7mo 2.35	10mo 2.31	102.7

[1] Pneumatic
[2] Composed of two courses: laboratory data are on lower course, road data on combined courses.

wetting agent and vacuum are used in getting the displaced volume of the samples after the vacuum-saturation procedure of Rice (9) and Benson and Subbaraju (10). Although this method, which is used extensively in the materials research laboratory of the Texas Transportation Institute, is not assumed to give the final answer on degree of absorption, it does give an answer that is more realistic than that used in Texas as well as in several other states. The discussions to follow point out the reason for this statement.

Texas specifications (11) require density as measured on laboratory compacted specimens to be in the range of 94 to 98 percent, with 96 percent as the "optimum". Values of these measurements are shown in Col.d, Table 3. These values do not take into account any absorption of asphalt by the aggregate. Only samples 8 and 11 fall outside this range. On the other hand, these values shown in Col.e do consider the major portion of the absorption that takes place. Voids content as high as 8 percent is revealed, with a low value of 2.4 percent.

Up to this point the discussion has been concerned with laboratory measurements on mixes that duplicate the job formula. Because the practical aspects of the problem are concerned with what happens in the field, data from field specimens are necessary.

Road samples were taken from the test section at ages as shown in Col. f. Densities as obtained by a ratio of maximum road specific gravity over laboratory specific gravity (expressed as a percent) are shown in Col. g. The average of the 13 values shown is 100+ percent. This average has no practical meaning, but the value is mentioned for comparison purposes with the data of Philippi (12), who stated that field measurements indicated that field densities in Texas had approached laboratory densities on the average to within 0.8 percent. It was further stated that not within the life history of any bituminous concrete pavements had the field density exceeded the laboratory density. The data presented here indicate that more recent construction equipment and compaction methods make it possible not only to reach but also to exceed (with the aid of traffic) the laboratory density as indicated in Col. g.

Examination of the data indicates that the values in Col. e actually represent a more nearly correct picture of the percentage of solids in the laboratory specimens, inasmuch as these values have as their basis a voidless mixture (practically speaking). The values of Col. g should therefore be corrected to the same basis as that used in Col. e. This correction would result in a series of values ranging from 86.5 to 98.5 percent. This is another way of saying that the voids in the completed pavements may be said to vary from 1.5 to 13.5 percent rather than 1.5 to 5.7 percent as shown by the data of Col. d.

Which method is correct? In reality, neither is. Repeated tests with a variety of aggregates and mix designs by numerous laboratories in Texas have shown that it appears possible to have mixtures with well over 100 percent solids where mixes of increasing asphalt content are analyzed using the equation values of Col. d. This is possible only because absorption of the asphalt by the aggregates is neglected. If, however, these same mixes are analyzed by use of the equation of Col. e, none of the mixtures will show as much as 100 percent solids. It has not been found possible in this laboratory to prepare a voidless specimen by any known means of compaction, even though the asphalt content is allowed to vary from a low value (very dry mix) to a very high value (more than enough asphalt to overfill the total voids in the mineral aggregate).

The real picture of field density and laboratory density is revealed when these values are compared when both are calculated using the specific gravity as obtained by the vacuum-saturation method as the common basis. Such calculations show that eight of the twelve test sections have higher densities than the corresponding laboratory samples.

CONCLUSIONS

Realizing that these data are limited and incomplete in the sense that traffic will cause some further densification in some of these pavements, the following conclusions are presented:

1. Absorption of asphalt by the aggregate should be taken into account in all labora-

tory and field evaluations of bituminous concrete. The methods currently used by the Texas Highway Department for density evaluations do not account for asphalt absorption by the aggregate. This leads to fictitious values of percent solids—values as high as 103 or 104 percent.

2. The vacuum-saturation procedure of determining the specific gravity of the composite loose asphaltic concrete mix is reliable and gives practical specific gravity values that consider most of the absorption of the asphalt by the aggregate.

3. Texas Highway Department standard procedure for forming laboratory specimens of job formula mixes does not, as a general rule, produce ultimate density of the mix, because field samples of companion mixes from pavements less than two years old consistently show somewhat higher densities. This apparent contradiction of previous findings is explained by changes in construction equipment and increased weight and density of traffic.

4. The use of self-propelled pneumatic rollers between the breakdown pass and final rolling with a tandem roller is strongly recommended. In effect this hastens surface sealing and little change in density occurs in pavements so compacted.

ACKNOWLEDGMENT

The author is grateful to the many members of the Texas Highway Department District Offices who gave freely and willingly of their time and effort in securing data and samples on the test sections. Particular thanks are due Jeff Seay, Supervising Field Engineer, Texas Highway Department, who supervised the placing of the test sections and assisted in sampling and evaluating the pavements.

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Behavior of Hot Asphaltic Concrete Under Steel-Wheel Rollers

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Maximum compaction of hot asphalt concrete is obtained under steel-wheel rollers when the roller weight and number of passes are optimum for the roller diameter and load bearing capacity of the mix. When more than the optimum roller effort is used, low densities or checked surfaces result.

Some mixes behave in an abnormal manner by shoving excessively under rollers they should tolerate. According to ordinary design tests, these abnormal mixes may be quite stable. Sometimes these mixes are referred to as "slow setting."

The paper proposes a mechanism by which mixes can be unstable at high temperatures during rolling and still be quite stable by normal design criteria. The difference is attributed to a transient high "pore pressure" caused by unabsorbed asphalt.

The condition can be corrected by changes in the mix design, filler content, or aggregate dryness. The character of the asphalt does not appear to be of great importance Several examples of field problems corrected by application of some of these principles are given.

●MODERN HIGHWAY ENGINEERS use every device available to assist them in designing high quality, asphalt concrete pavements. They are assured that their designs and specifications are met by means of inspections made during and at the end of construction. In nearly all cases, these practices result in high quality surfaces because minor difficulties or unusual behavior of the mix under the steel-wheel rollers are resolved by field engineers and construction crews as soon as detected. Changes in mix design, temperature, layer thickness, or rolling procedure are routinely made to obtain the desired density, toughness, or appearance of the finished pavement.

Occasionally, however, an abnormal situation arises when these normal measures do not correct the difficulty as expected, although no obvious errors in mix composition or procedure were made. As a result, the finished pavement may have a low density, be tender, and easily marred by high pressures or power steering turns, or have a checked (fissured) surface. When these situations arise, both the mix and asphalt used are often called "slow setting." This term can be misleading because it implies the presence of questionable hardening qualities in the asphalt binder that have not been clearly demonstrated. Actually, all of the reports of abnormal behavior that have been received and the several instances that have been observed in the field resolve into failures of the mix to compact properly under steel-wheel rollers.

To understand this problem, a study was undertaken in the research laboratory. Under controlled conditions, many of the variables of asphalt concrete design and compaction were studied in laboratory and full-scale tests (1, 2). It was found that the porosity of the aggregate, the water content of the aggregate, the type and amount of fines, the aggregate gradation and the roller characteristics are all important in determining how well a mix compacts. Other factors might also contribute, but they have not been identified in the recent work.

The understanding gained in the laboratory has been applied to field problems with success. This paper describes the concept of the compaction mechanism, the data that were obtained and the conclusions that were reached. Several field studies have verified the principles postulated.

MECHANISM OF HOT ASPHALT CONCRETE COMPACTION

Steel-wheel rolling of hot asphalt concrete is a means of applying pressure and kneading action to a mix so that compaction will occur. When high roller pressures are used, shear deformations tend to occur in front and behind each wheel. Shear deformations in the mix under these conditions cause an expansion of the mix which results in decompaction. The intimate particle contact lost by the shearing action is not restored until pressure is again applied to the decompacted zone.

The conditions existing in a dymanic situation under a moving roller are shown in Figure 1. Most of the decompaction occurs in front of the roller, although a minor amount occurs in the rear. This action gives the appearance of a low wave traveling along, not unlike the bow wave in front of a moving boat. This wave is a symptom of a zone of decompaction preceding each pass of the roller.

Eventually, after enough passes have been made, an equilibrium is established in which the recompaction and decompaction are balanced. As discussed later, the state of compaction at equilibrium is dependent on the load-bearing capacity of the mix, as well as the roller weight and diameter. A small roller sinks deeper into a mix causing more extensive decompaction than does a larger roller of the same weight. This results in the larger bow waves observed with small rollers.

Because of the roller curvature, horizontal stresses are visualized in a mix under tension—like splitting wood with a wedge. If the resultant tensile strains are large enough, the mix surface pulls apart, causing fissures of checking to appear after rolling. These horizontal forces are larger with the sharper curvature of small rollers. This agrees with the field observation that checking is more severe with small-diameter rollers.

FACTORS INFLUENCING COMPACTION

There appears to be an optimum load-bearing capacity (or stability) which permits a maximum compaction to occur under a particular roller weight and size. To illustrate this point by extremes, a mix can be so stable (as occurs at ambient temperatures under traffic) that negligible compaction takes place. On the other extreme, the mix can have such a low stability that the roller sinks deeply into the mix.

A relative measure of this stability or bearing capacity of a mix is the familiar Coulomb equation (3), which expresses the stress, necessary to cause shear failure in a mix

$$S = C + \sigma \operatorname{Tan} \phi$$

in which C is the cohesion, σ is the confining pressure, and ϕ is the angle of internal friction of the mix.

Hot-mix plant operators have little control of ϕ because it is a consequence of the specified mix design. They do, however, have considerable control over the cohesion. They can vary this by changing the mixing temperature, the asphalt type or grade, or by changing the amount and kind of filler used. The effects of these factors are discussed in more detail later.

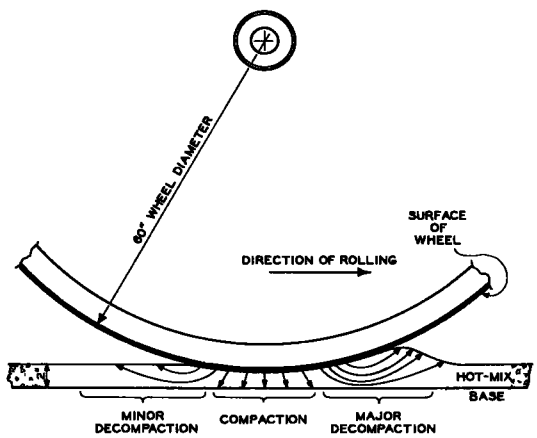

Figure 1. Behavior of hot-mix during rolling.

The contribution of cohesion to the high-

temperature stability of a mix can be estimated from McLeod's (3) work, which shows that doubling the cohesion doubles the bearing capacity of thin layers of asphalt concrete on a solid base under normal loading conditions. It seems reasonable, therefore, that this effect will hold at higher temperatures. If this is the case, higher cohesion will reduce decompaction in front of the roller, as well as checking in the surface directly under the roller. However, if the cohesion increases, as it does when put into service under traffic, it will eventually become large enough to prevent compaction. There appears to be an optimum mix cohesion to obtain a maximum compaction under a given set of rolling conditions.

Even more favorable conditions will exist if the cohesion is shear dependent (breaks down with shear), in which case compaction can occur easily directly under the roller because the local high shear rates reduce cohesion. In front of the roller the shear rates are lower (less shear breakdown) and the resultant higher cohesion is more effective in preventing decompaction. This ideal situation would have a low stability directly under the roller and high stability (confining forces) adacent to the roller. How this can occur is discussed in the next section.

FACTORS INFLUENCING COHESION

During rolling the cohesive properties of an asphalt concrete are dependent on the flow properties of the asphalt and filler mixture (binder). Three characteristics considered important in modifying the rheology of the binder are: (a) filler content and particle size distribution, (b) rolling temperature, and (c) nature of asphalt. Nijboer (4) studies the influence of the properties of the binder on the cohesion of a mix by means of triaxial and other laboratory experiments. He refers to cohesion as the initial resistance. Extrapolation of Nijboer's data to higher temperatures gives an indication of the relative effect of the different factors under rolling conditions. When the asphalt content is 6 percent and the mix filler content varies from 2 to 5 percent, Figure 2 shows the tenfold increment in cohesion. Reducing the average filler particle size from 100 to 10 microns doubles the cohesion, according to Figure 3. The effect of reducing the rolling temperature from 300 F to 200 F is shown in Figure 4 and results in a fourfold increase in cohesion. Figure 4 also shows that the nature of the asphalt has an effect on cohesion that might reach a factor of three.

The increase in cohesion at mix temperatures obtainable by changing from a 200-300 to a 40-50 penetration grade asphalt can be estimated from temperature viscosity curves. This is equivalent to about a 30 F difference in mix temperature (drop from 280 F to 250 F). This indicates that less than a twofold increase in cohesion can be expected by changing the grade of asphalt a maximum amount.

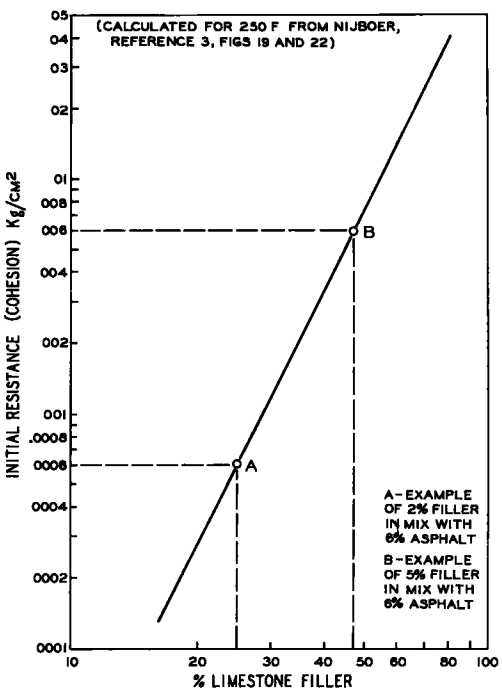

Figure 2. Effect of filler in binder on cohesion of mix.

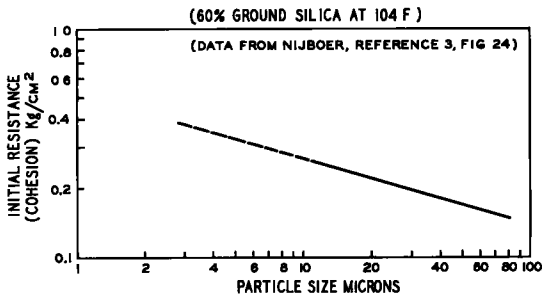

Figure 3. Effect of filler particle size on cohesion of mix.

If these extrapolations of Nijboer's data to higher temperatures are valid, clearly when the filler content is low raising it is the most effective way of increasing the hot mix cohesion.

The strain rate of a filled asphalt is not directly proportional to the stress at all levels (5). The rate of strain stays very low as the stress is increased until a point where shear breakdown occurs. Beyond this point, strain is proportional to stress in the customary manner. This is shown (Fig. 5) in illustrating experiments using the microviscometer (6). The breakdown point is proportional to the amount of filler included in the binder. All ordinary unfilled paving asphalts have very low, almost insignificant, shear breakpoints. Further work may show that quite different breakdown points are obtained using the same type and amounts of filler but with different asphalts.

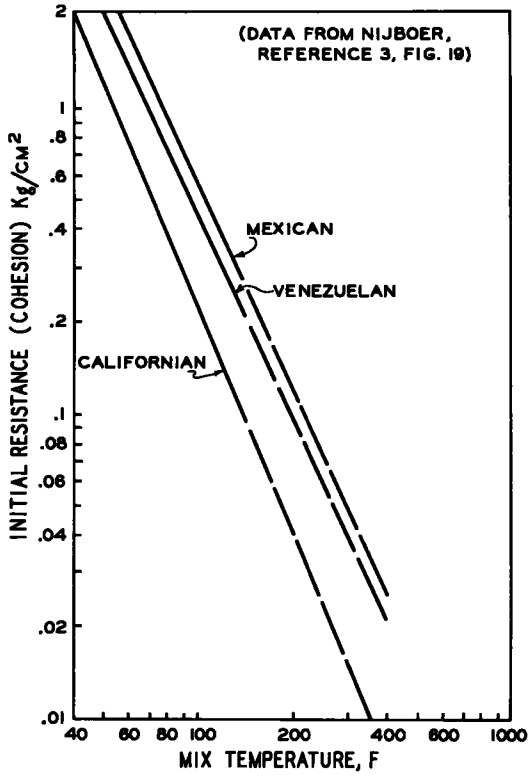

Figure 4. Effect of temperature and asphalt source on cohesion of mix.

NORMAL BEHAVIOR UNDER STEEL-WHEELED ROLLERS

A mix is considered to act normally when its behavior under a roller can be predicted from ordinary laboratory stability tests. This means that very stable mixes should tolerate heavy rollers or many passes before excessive decompaction occurs. Those with a low stability will tolerate only light rollers or a few passes.

For convenience, the behavior of these normal mixes is considered in two classes because they respond differently to severa variables. They are considered understressed when the mix is lightly stressed in the range where a greater compactive effort will increase the degree of compaction obtained. They are considered overstressed when an additional compactive effort causes a drop in the degree of compaction in a mix. The optimum amount is the transition point where maximum compaction is obtained between the under- and overstressed conditions.

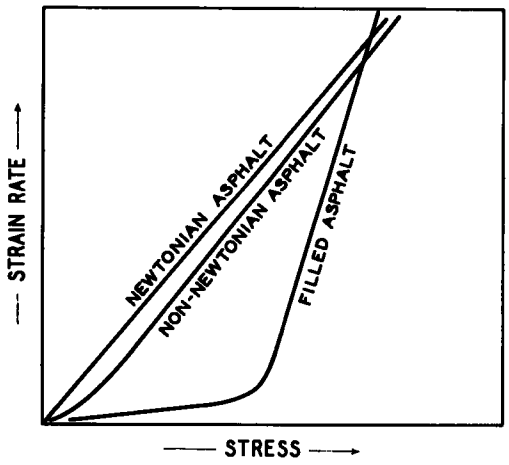

Figure 5. Filled asphalt strain rate drops rapidly with stress.

UNDERSTRESSED

In the range where the stability of a mix is more than adequate to support the compactive effort imposed during rolling (or in a laboratory compactor where shear deformation is limited by the containing mold), increasing compactive efforts give higher densities (4). Figure 6 and Table 1 show this effect. The data, shown in solid

TABLE 1

EFFECT OF NUMBER OF PASSES[1] ON CORE DENSITY

Core	Core Density (pcf) 1 Roller Pass	2 Roller Passes	4 Roller Passes	5 Roller Passes
A	129 77	130.88	132.68	134.42
B	130.88	131.15	133.91	133.08
Avg.	130.32	131.01	133.29	133.75

[1]Asphalt A, Table 3, used in these experiments

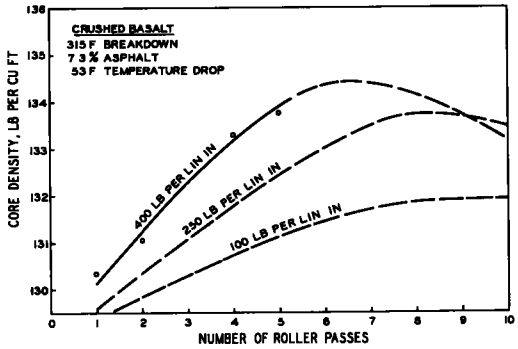

Figure 6. Core density vs number of roller passes for different wheel pressures.

lines, were developed using full-scale equipment described in Ref. (2). The dotted lines and extensions are estimates based on limited experiments and on field experience. Figure 6 shows that by increasing either the number of roller passes or the roller weight, up to a certain point, higher densities are obtained. This finding is consistent with Nijboer's studies (4), which appear to be limited to the understressed class.

Under these conditions the temperatues of rolling have a pronounced effect upon increasing the density obtained during rolling. However, as shown in Figure 7 and Table 2, the effect varies, depending on the temperature range and compactive effort. As the temperature increases, cohesion drops; and the stability of the mix decreases slightly, thus permitting more compaction to occur under the roller. The remaining stability is still high enough to support the roller without excessive decompaction.

Nijboer (4) shows that in these understressed conditions an increased compaction for a given roller weight can be obtained by reducing the roller diameter. This will produce the increased deformation required under these conditions. However, as previously discussed, smaller rollers increase the checking and the amount of decompaction in front of the roller at the same time.

In this situation, where the cohesion of the mix is not a major factor, the source of the asphalt appears to have little effect. This point is confirmed by the results of full-scale experiments summarized in Figure 8 and Table 2 which relate density obtained to rolling temperature when asphalts of different types are used. Table 3 lists the properties of asphalts used.

Normal Behavior When Overstressed and Stressed at Optimum During Rolling

As shown in Figure 6, increasing the number of passes of a low-pressure roller will increase the density up to a limited value. No amount of further rolling will increase it more. The plateau at which this maximum density occurs depends on the roller pressure and diameter, the mix properties and temperature, and the thickness of the layer of asphalt concrete being rolled.

As the roller pressure is increased beyond a critical point for a given mix, the density of the mix is no longer unaffected by an excess of passes. It begins to drop. As Figure 6 shows, only a few passes of the highest pressure rollers can be tolerated.

This drop in density with excess rolling appears to be caused by the formation of surface checks or fissures. It can occur on a cold mix with a heavy roller if enough passes are made. The concept on how this occurs is as follows: As the mix becomes

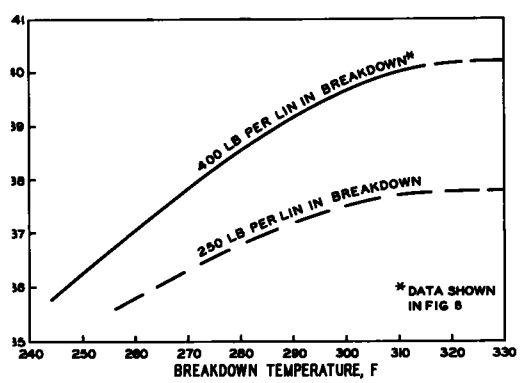

Figure 7. Normal behavior: Density increases with breakdown temperature and roller pressure.

more stable under compaction or by cooling the roller does not sink into the mix so much and its full weight is concentrated in a narrow band. Thus, the horizontal components of force causing tensile stresses are concentrated and the mix fails under high local tension. An example of this decompaction at the surface is shown in Figure 9. This shows the varying densities in a core sliced into thin horizontal sections with a diamond saw. The density is lowest at the surface.

Increasing roller pressure up to a point will raise the density obtained for a given mix. However, as schematically shown in Figure 10, a maximum value will be obtained where the decompaction resulting from the increased weight offsets the compaction expected from higher pressure. Eventually, with a further increase in pressure, the density actually drops. Each mix then has an optimum roller pressure which gives a maximum density to the mix.

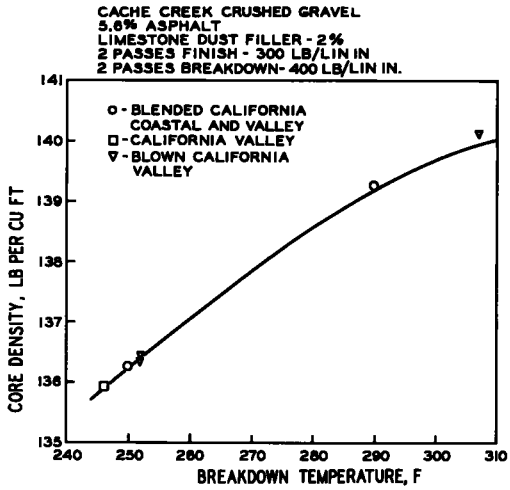

Figure 8. Normal behavior: Density increases with breakdown temperature regardless of asphalt source.

TABLE 2
CORE DENSITY AND BREAKDOWN TEMPERATURE[1]

	Core Density (pcf)						
	Asphalt A[2]		Asphalt B[3]		Asphalt C[4]		
Core	250F[5]	290F[5]	246F[5]	303F[5]	252F[5]	252F[5]	307F[5]
A	136.48	139.48	135.65	140.08	137.34	135.46	139.68
B	136.39	138.81	135.84	139.33	136.79	134.89	139.89
C	135.94	139.55	136.44	139.05	136.06	135.78	140.76
D	135.99	-	134.38	-	135.00	136.82	-
E	136.78	-	136.25	-	136.37	138.70	-
F	136.93	-	136.86	-	136.07	136.87	-
Avg.	136.41	139.28	135.90	139.48	136.27	136.42	140.11

[1]Data for Figs. 7 and 8.
[2]Blended California coastal and valley asphalt.
[3]California valley asphalt.
[4]Blown California valley asphalt.
[5]Breakdown temperature, in °F.

Figure 11 shows the relation between the stability of a mix and its optimum roller weight. It indicates that more stable mixes tolerate heavier rollers. This relation is clear if the reader will recall from his own experience, the very low roller weights or low mix temperatures required to roll unstable mixes made with uncrushed gravel. Recall also the high pressures required to compact well graded, crushed, aggregate systems having high stabilities. The stability of a normal mix, as measured by normal mix design tests, is related to the optimum roller weight that will give the mix a maximum density using a fixed number of passes. The ranges shown are estimated from field observations and intended to be illustrative only.

It should be kept in mind that the values shown will be quite different for rollers with different diameter wheels. With large wheel diameters, higher pressures can be used and higher densities obtained before excessive shear deformation occurs. Wheels with small diameters cause excessive shear failure at rather low loadings and will

TABLE 3

PROPERTIES OF ASPHALTS USED IN FULL-SCALE EXPERIMENTS[1]

Identification Tests	Asphalt A[2]	Asphalt B[3]	Asphalt C[4]
Penetration at 77 F, 100 g, 5 sec	96	91	91
Penetration at 39.2 F, 200 g, 60 sec	24	25.5	20
Penetration ratio	25	28	22
Flash point, Pensky-Martens (°F)	445	465	440
Viscosity at 275 F (SSF)	138	126	119
Heptaine-xylene equivalent	20/25	25/30	20/25
Softening point, ring and ball (°F)	110	116	Not tested
Thin film oven test, 325 F, 5 hr:			
Weight loss (%)	0.51	0.53	0.42
Penetration retained (%)	53	53	55
Ductility of residue	100+	150+	150+
Miscellaneous tests:			
Oliensis spot test	Neg.	Neg.	Neg.
Sulfur, bomb method (%)	3.0	1.6	1.3
Construction data:			
Penetration at 77 F (before hot mix)	92	89	82
Penetration at 77 F (extracted)	48	47	43
Penetration retained (%)	52	53	53

[1]Construction data shown in Table 2 and Figs. 6, 7 and 8.
[2]Blended California coastal and valley asphalt.
[3]California valley asphalt.
[4]Blown California valley asphalt.

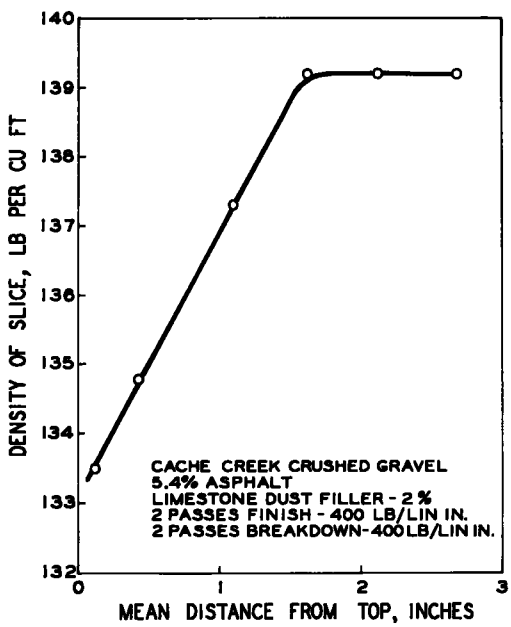

Figure 9. Density of sliced cores increases with distance from top of mix.

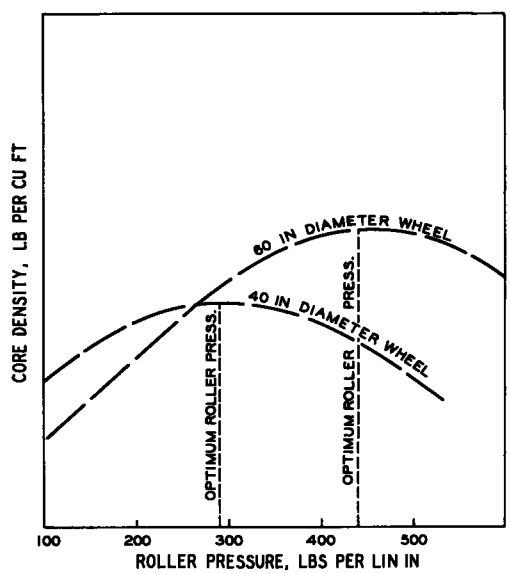

Figure 10. Density varies with roller pressure and wheel diameter.

give low maximum densities (Fig. 10).

Asphalt concrete is most often laid in courses about 2 in. thick; they are rarely greater than 3 in. or less than ¾ in. As shown in Figure 12, the thickness is quite important in determining the maximum roller weight that can be used effectively. Very thin layers are shown to tolerate much higher pressures than thick ones. This is because the close proximity of the plane of the stable base inhibits shear deformation and decompaction markedly. McLeod (3), in theoretical considerations, shows that thin layers of a given asphalt concrete have a much higher resistance to shearing displacement than thick ones.

At the optimum, where the maximum roller pressure is applied consistent with maximum compaction, the cohesion of the mix has become the factor limiting the resistance to shearing displacement (stability) of the mix. Accordingly, as the cohesion increases, so will the optimum roller pressure.

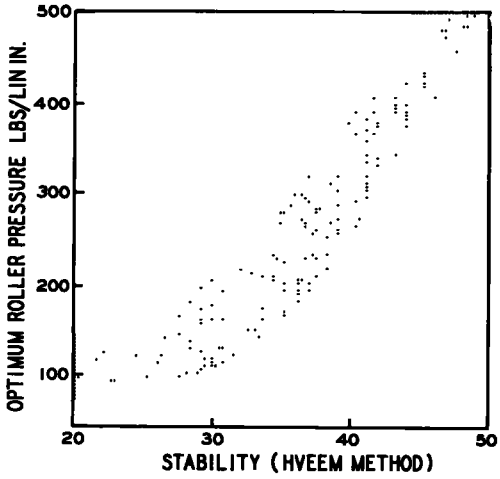

Figure 11. Optimum roller pressure depends on mix stability.

An example of the effect of reducing the cohesion of heavily stressed mixes by increasing temperature is shown in Figure 13 and Table 4. This figure, giving the results of several full-scale experiments, shows that no change in the density is obtained as a result of increasing the rolling temperatures. Also, both 300- and 400-lb per lin in. rollers are shown to give the same results. Possibly, these roller weights straddle the optimum for the mix being rolled.

As discussed previously, the cohesion of a given mix is controlled principally by amount and particle size of the filler and to a less extent by the mix temperature or penetration grade of asphalt. The source of asphalt is indicated to be a minor variable. However, unlike the previous case with understressed mixes, asphalt source would be expected to have some effect on determining the optimum roller weight. The effect should be obscured by minor changes in filler type or amount. Work is in progress to establish these points.

ABNORMAL MIXES

Occasionally, a mix does not behave in the expected normal fashion; by conventional tests it appears quite stable,

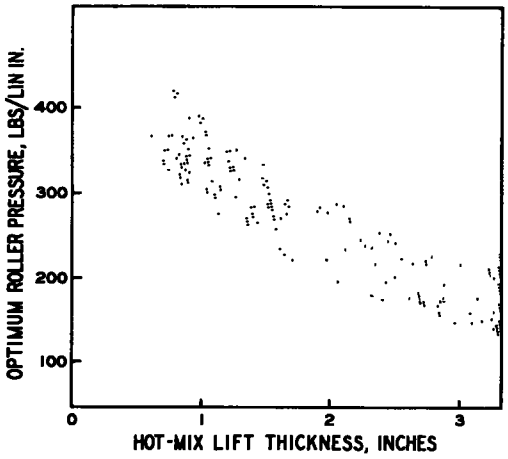

Figure 12. Optimum roller pressure depends on lift thickness.

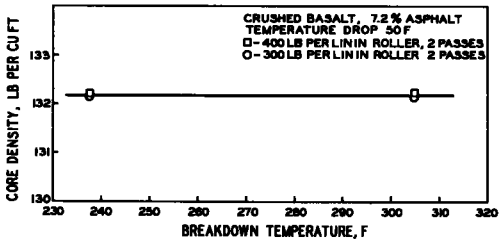

Figure 13. Overstressed condition density vs breakdown temperature.

TABLE 4

DENSITY VERSUS BREAKDOWN TEMPERATURE[1,2]

Core	Core Density (pcf)			
	300-lb/lin in. Wheel		400-lb/lin in. Wheel	
	238 F[3]	305 F[3]	238 F[3]	305 F[3]
A	133.03	132.84	133.62	131.32
B	132.28	133.76	132.38	133.00
C	130.81	130.95	132.45	131.86
D	131.40	130.89	130.66	131.37
E	132.98	132.94	132.24	131.71
F	132.56	131.40	131.71	133.95
Avg.	132.17	132.13	132.17	132.20

[1]Asphalt A, Table 2, used in these experiments.
[2]Data for Fig. 13.
[3]Breakdown temperature, in °F.

but under a heavy roller it shoves excessively. Such mixes are represented by points falling below the shaded area in Figure 11. According to normal stability tests, the mix shown should support a heavy roller and achieve high density. In practice, it will tolerate only a light roller and, accordingly, achieves a low density.

This discrepancy appears to be a result of significant differences between the conditions used in determining laboratory stability and the conditions occuring during field rolling. Some of these differences are:

1. Laboratory stability tests are run at 140 F; rolling temperatures are in the range of 225-300 F.
2. Laboratory tests are run on throughly dried aggregates; this is not always the case on aggregates used in plant mixes (7, 8).
3. Laboratory voids calculations are based on tests run at 77 F; in the field, mixes are normally compacted 200 F higher than this.

As a result, the volume of asphalt in the voids may be excessively high because the asphalt is not absorbed into porous aggregate until a sizeable drop in temperature occurs and until all steam has evolved from the aggregate. The effect is similar to the low stability observed in wet soils having high pore pressures.

EFFECT OF AGGREGATE POROSITY

Whenever the air voids in a mix become too low, the stability drops (9). This principle is well recognized both in aggregate-asphalt and soil-water systems (10). Figure 14 shows this effect in a dense mixture of aggregate and asphalt.

Unstable conditions caused by low voids

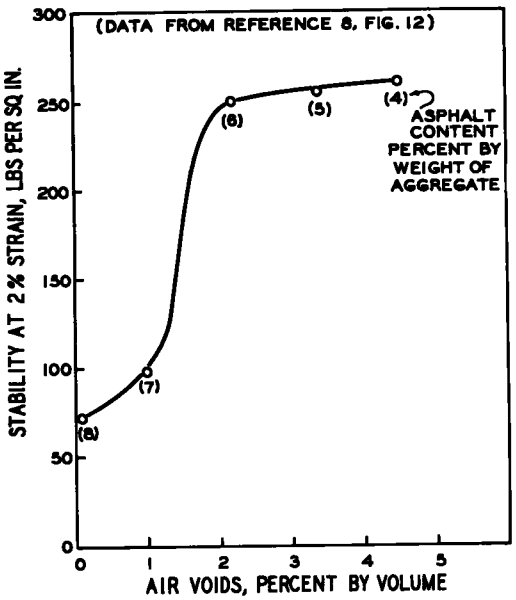

Figure 14. Mix stability is low when voids are filled with asphalt.

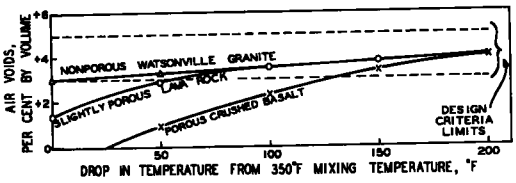

Figure 15. Air voids depend upon porosity and drop in temperature after mixing (dense mix).

can be present at the high rolling temperatures existing shortly after mixing and not be evident later at room or design temperatures. Examples of how this occurs are shown in Figure 15, where the effective voids contents at different temperatures are shown for aggregates having several porosities. The data used to construct these curves were obtained by immersing the aggregate mix at high temperatures in asphalt in calibrated flasks and observing changes in volume as the temperature was dropped in increments (11). Care was taken during preparation that the only air remaining in the mixture was in the pores in the aggregate.

With the nonporous Watsonville granite, there is little change in the effective voids content as temperature drops. This would be predicted from calculations based on the thermal expansion of the asphalt and granite. However, using a porous aggregate, striking changes in the effective voids content of the system are shown. This result also can be predicted if the thermal expansion of the entrapped gases is considered.

Dense mixes made with porous aggregates are shown to have zero voids contents at high temperatures. There is inadequate void space to accommodate the asphalt present. Under these conditions a mix would have a very low load bearing capacity.

Figure 16 was constructed from the same data, except that the gradation assumed was for an open mix. Under these conditions, only the most porous aggregates are shown to approach a critical voids content at high temperatures. This conclusion agrees with the observations that abnormal behavior does not occur in open-type mixes.

These experiments indicate that a greater absorption could be obtained if the asphalt and aggregate were mixed together at a higher temperature, thus allowing the mix to cool longer before rolling. The bigger drop in temperature, ΔT, causes absorption of more asphalt. Figure 17 and Table 5 give the results of full-scale experiments made to clarify this theory. The crushed basalt used (see Fig. 18 for grading) was the same as the aggregate used in the absorption experiments previously described. The curve shows that a maximum density is obtained for this particular mix and roller when the ΔT between mixing and rolling is about 65 F. Points on the curve include rolling temperatures varying from 222 F to 305 F. They fall close together if the ΔT is the same.

The series of experiments not only confirms the theory that a higher temperature drop would remove the excess asphalt from the voids, but also demonstrates that there is an optimum amount of asphalt to remove in a particular mix.

The abscissa in Figure 17 is plotted from left to right on the basis of a greater ΔT between the pugmill temperature and rolling temperature. Also plotted is the amount of asphalt estimated to be in the voids for the corresponding temperature drop. This was calculated from Figure 15, which was based on the same aggregate, gradation, and asphalt.

Also indicated is the increase in stability corresponding to the reduced amount of asphalt in the voids. As more than the optimum amount is removed, the mix becomes lean in asphalt and more difficult to compact.

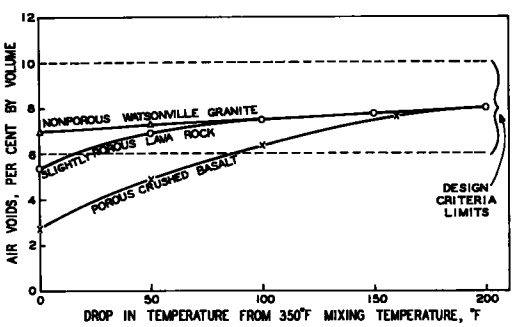

Figure 16. Air voids depend upon porosity and drop in temperature after mixing (open mix).

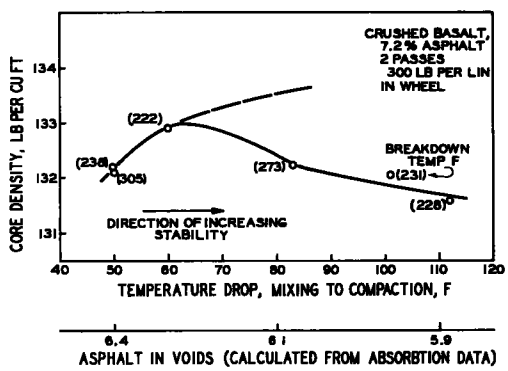

Figure 17. Asphalt absorbtion caused by drop in mix temperature influences core density.

TABLE 5

DENSITY VERSUS TEMPERATURE DROP[1,2]

Core	Core Density (pcf)					
	Temp. Drop 50 F Bkdn. Temp. 238 F	Temp Drop 50 F Bkdn. Temp. 305 F	Temp Drop 60 F Bkdn. Temp. 222 F	Temp. Drop 83 F Bkdn. Temp. 273 F	Temp. Drop 101 F Bkdn Temp. 231 F	Temp. Drop 112 F Bkdn. Temp. 228 F
A	133.03	132.84	133.54	130.95	131.97	131.71
B	132.28	133 76	132.21	131.98	131.98	131.58
C	130.81	130.95	(136.04)[3]	133.07	132.36	130.93
D	131.40	130.89	133.40	132.73	131.00	131.56
E	132.98	132.94	132.44	131.06	132.69	131.41
F	132.56	131.40	132.86	133.59	132.47	132.46
Avg.	132.17	132.13	132.89	132.23	132 07	131.60

[1]Asphalt A, Table 3, used in these experiments.
[2]Data for Fig. 17.
[3]This value rejected from calculation of average on basis of Q test at 95% confidence level (12, 13).

This finding is similar to what would happen if more or less asphalt were added in the beginning to an ordinary, nonporous aggregate. With the porous aggregate, the effect is created by controlled adsorption.

The rise in the curve in the range of small ΔT's (up to 65 F) shows that the particular roller pressure used was more than optimum for the stability of the overrich mix. As the asphalt was removed by a greater ΔT, the stability finally increased until the roller used was just optimum. Removal of more asphalt by a still greater ΔT increased the stability to the extent that the roller pressure was below optimum. In this case, the density achieved was lower than was obtained with a less stable mix with the same roller pressure. The dotted line illustrates the increase in density that might have occurred if the roller pressure had been increased to the optimum amount corresponding with the increasing stability.

EFFECT OF MOISTURE ON ABNORMAL BEHAVIOR

Sometimes aggregate mixes are incompletely dried in the plant before they are mixed with asphalt. Some of the ways this can occur are:

1. The cold feed to the plant is saturated with water.
2. The aggregate is porous and difficult to dry.
3. The dryer is overloaded because of production requirements.
4. The hot coarse aggregate is held for an insufficient period in the hot bins for the water to be completely desorbed.
5. The temperature of the aggregate coming from the dryers is held down to avoid hot mix hardening.

When the aggregate is incompletely dried at the time it is mixed with asphalt, water in the pores will continue to be vaporized or desorbed until equilibrium is reached. Evolution of steam may continue, although the temperature may have dropped substantially. During this time, absorption of asphalt will not occur in the pores evolving vapor.

It seems likely that the steam evolved from the rock pores may contribute to the volume of asphalt binder in still another manner besides preventing absorption. When steam evolves from the

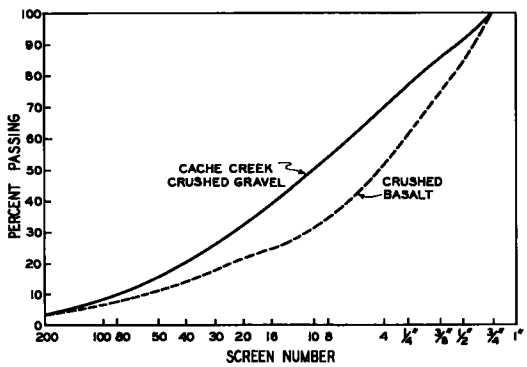

Figure 18. Gradings of aggregates in full scale experiments.

micropores, some of it is in the form of colloidal-size bubbles and remains for a while in the asphalt without breaking. This is suggested by the light-brown color asphalt acquires under these conditions. The volume of steam entrapped in the asphalt contributes to the total volume of liquid in the voids in proportion to the pressure on the system. Directly under the roller, where pressures are high, its volume would be reduced; in the low-pressure areas adjacent to the roller, its volume would be larger. Both of the suggested mechanisms give the effect of increasing the volume of asphalt in the voids at higher temperatures. A discussion of methods used to improve aggregate dryness before mixing is shown in Field Study C.

FIELD STUDIES

Typical case histories of field problems included in the appendix support several of the principles postulated in this discussion. The belief that a "tender" pavement is a poorly compacted pavement is supported by Field Study A, where penetration tests were made on areas with different amounts of traffic and subsequently by laboratory tests in which the same mixes were compacted different amounts. Another Field Study B, shows that the relative compaction is low on a mix reported to be tender. This example is typical of many others not included.

Field Studies C and D both substantiate the propositions that moisture remaining in the aggregate under the asphalt prevents absorption and causes low mix stability at high temperatures.

Field Study E confirms the importance of the filler content on the cohesion and bearing capacity of the hot mix. This overstressed mix was easily shifted towards a more optimum condition by any one of several fillers. Under these same conditions, changing the asphalt type had no noticeable effect.

SUMMARY

This discussion has covered many of the factors influencing the behavior of hot asphalt concrete under steel-wheel rollers. It includes situations where the mix is under- and overstressed and when it behaves in a predictable fashion or abnormally in a way that is neither predictable nor easily explained. General conclusions on how to obtain the best compaction cannot be given for all conditions. Instead, the following guidance is suggested for the several situations encountered in rolling hot mix:

A. For Normal Mixes Stressed Below Optimum

1. Increase roller weight.
2. Decrease roller diameter (usually not recommended).
3. Increase number of passes of roller.
4. Reduce cohesion of mix by using either a softer grade of asphalt or less filler, or by increasing the rolling temperature.
5. If roller weight cannot be increased, reduce stability of mix within specification limits by changing either aggregate grading or aggregate angularity, or by increasing asphalt content. This change is suggested only when there is no danger of subsequent instability under traffic.

B. For Normal Mixes Overstressed

1. Increase roller diameter.
2. Decrease number of roller passes.
3. Decrease roller weight.
4. Decrease thickness of each layer of asphalt concrete rolled.
5. Increase cohesion of mix by using a harder grade asphalt, adding or changing the type of filler, or lowering the rolling temperature.
6. Increase stability of mix by changing aggregate grading or aggregate angularity, or by decreasing asphalt content.

C. To Make Abnormal Mixes Behave More Nearly as Predicted from Design Tests

1. Adjust mix design to maximum permissible final voids content.
2. Dry aggregate thoroughly.
3. Allow an optimum temperature drop between mix and rolling temperatures.
4. Add a fine-grained filler if examination shows mix is low in fines.

D. To Reduce the Appearance of Surface Checking

1. Increase roller diameter.
2. Decrease roller weight.
3. Decrease number of roller passes.
4. Reduce sand fraction. (Use a mix gradation less prone to show checking; a coarser, more roughly textured mix.)

Pneumatic compactors are rapidly gaining popularity in compacting hot asphalt concrete. If correctly used, these rubber-tired rollers will give much greater compaction than is possible with steel-wheel rollers. They must be heavy enough and have high enough contact pressures. Furthermore, the mix should be rolled at the highest possible temperature; otherwise, many passes are required. Usually, poor compaction resulting from improper steel-wheel rolling can be readily corrected by proper pneumatic compaction if the pavement is still warm.

ACKNOWLEDGMENTS

The authors wish to acknowledge the valuable assistance of W.H. Ellis and R.S. Winniford, of the California Research Corporation, Richmond, Calif.; and of C.L. Monismith, Professor of Civil Engineering, University of California, Berkeley.

Appendix

FIELD STUDY A

Project Location

Access road, airport, San Joaquin Valley, California.

Complaint

Prolonged tenderness of the pavement. Pavement was soft and could easily be scuffed three days after construction.

Mix Composition

Aggregate Type—Crushed silicious gravel. Aggregate separated into three bins.
Aggregate Grading—The grading is shown in Figure 19.

Miscellaneous Mix Characteristics:

Asphalt content, %	4.9
Stability:	
Hveem stabilometer, S-value	52
Hveem cohesiometer, C-value	178
Penetration of extracted asphalt (77F)	73

Field and Laboratory Studies

Field studies were made to determine the relative resistance to displacement under load of the pavement, which had been rolled with a 51-in. diameter, 200-lb per lin in. roller. For this study, the Soiltest penetrometer was used. The time for penetration to a depth of ¼ in. under a load of 320 psi was measured. Tests were made across the pavement, from one edge to the other.

Laboratory specimens were compacted using various pressures on the Triaxial Institute kneading compactor. The penetration time of the Soiltest penetrometer at 320 psi was compared with stability and density values.

Laboratory Data

Foot pressure (psi) with 150 blows of Triaxial Institute kneading compactor	100	200	300	400	500
Density (lb/cu ft)	136.7	137.7	142.5	143.8	144.0
Hveem stabilometer value	22	28	33	35	35
Hveem cohesiometer value	63	87	106	152	165
Time per ¼-in. penetration (sec at 320 psi)	1.2	3.6	9.9	30+	30+

Field Data

Field Conditions	Penetration Time (sec) (load of 320 psi)
1. Tender areas; sections that could be scuffed	3 to 15
2. Areas that received traffic were tough, dense, and scuff resistant	60+

Discussion and Conclusions

Tenderness of asphalt concrete surface mixes is a result of inadequate compaction. Areas receiving rubber-tire compaction and well-compacted laboratory specimens were tough and scuff resistant. Inadequately compacted areas and specimens were tender and could be easily displaced.

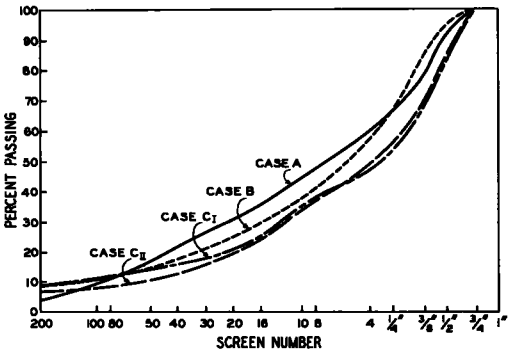

Figure 19. Gradings of aggregates in field problems A,B,C.

FIELD STUDY B

Project Location

Commercial airport, Hawaiian Islands.

Complaint

Prolonged softness of pavement. At high ambient temperatures the pavement was soft, lacked cohesion, and was easily scuffed.

Mix Composition

Aggregate—The aggregate was a very dense crushed lava rock. Extracted grading is shown in Figure 19.

Asphalt content, 5.3 percent.
Density measurements (pcf):

Core from pavement slab	147.9
Recompacted, 75-blow Marshall compaction	158.7
Recompacted in Triaxial Institute kneading compactor, California procedure	169.7

Test	Core	Laboratory Recompacted
Marshall stability	875	1,000+
Marshall flow	19	15
Hveem stability	12	43

Changes Made and Effect

Subsequent rubber-tired compaction eliminated tenderness.

Discussion and Conclusions

Prolonged tenderness and softness of the mix are a result of low density. Laboratory recompacted samples indicated that greatly increased field density could be attained. Rubber-tired rolling later improved density and eliminated tenderness.

FIELD STUDY C

Project Location

Primary state highway, Hawaiian Islands.

Complaint

The paving contractor had frequently built pavements which remained tender for prolonged periods after construction and pavements which were difficult to compact to the required densities.

Aggregate Composition

Crushed lava rock. Moisture absorption of coarse grading 2.6 percent; high specific gravity. The grading is shown in Figure 19.

Changes Made and Effect

Changes—The hot-mix plant operation was deliberately controlled to obtain conditions in which the mix would be dry in one case and have a significant water content in the other.

In the first case, the cold feed to the dryer was barely moist. It was dried at the normal rate and then allowed to "heat soak" in the hot bins for 15 min prior to mixing with the asphalt in the pugmill.

The same procedure was used in the second case, except that the cold feed was water saturated, and no "heat soaking" period was allowed. Both mixes were rolled with the same roller and number of passes within a period of one-half hour of each other.

Effect of the Changes—

	Case I	Case II
Visual appearance of hot mix	Black	Brown, tiny bubbles visible on close inspection
Temperature of hot mix from pugmill (°F)	300	280
Field compaction temperature (°F):		
Top ½ in. of mix	245	245
Center of mix	280	270
Bottom ½ in. of mix	250	250
Analysis of pavement specimens:		
Moisture content, ASTM D 95 (%)	Trace	0.0065
Asphalt, Abson recovery (%)	6.1	6.1
Aggregate grading, after Abson recovery	See Figure 19	
Recompacted by Hveem kneading compactor:		
Hveem stabilometer value	59	60
Hveem cohesiometer value	461	343
Condition of finished pavement	Very tough and dense, resists movement	Soft, tender, easily displaced

Conclusions and Discussion

The only significant difference between these two mixes appears to be moisture content. Yet, there was a striking difference in the "tenderness" of the finished pavement. Moisture apparently causes the softness because it prevents asphalt absorption. Also, it causes the mix to be unstable at high temperatures and poorly compacted under the roller.

Incomplete aggregate drying is due to either insufficient dryer capacity of inefficient operation. More complete water removal can be obtained by recycling the aggregate, by using two dryers in series, or by using a longer dryer. The operating efficiency can sometimes be increased by decreasing the dryer tilt, which increases the residence time of the aggregate in the dryer. Also, intermittent operation causes poor drying because in the operating conditions are not uniform.

Advantage is seldom taken of the water removed from the aggregate during storage in the hot bins. "Heat soaking" of the coarse aggregate in the hot bins for at least 15 min allows the larger rock to reach equilibrium with respect to temperature and moisture content. The coarse bins should be run as full as practical in order for the rock to "soak" for a maximum period.

FIELD STUDY D

Location

Ohio.

Complaint

The mix pushed and shoved during rolling and had a "light-colored, dull appearance."

Mix Composition

Aggregate Type—The mix was made by combining two bins. The coarser one consisted of a "not particularly silicious" crushed gravel; the fine one, of natural sand.

Grading—The grading (Fig.20) shows the mix to be dense, probably with low voids in mineral aggregate.

Stability—Not established.

Moisture content of mix—0.4 percent water by ASTM D 95.

Change Made and Effect

When the plant throughput was cut from 130 to 110 tons an hour to more thoroughly dry the aggregate, both the off-colored mix and rolling difficulties disappeared.

Discussion and Recommendations

The mix behaved in an abnormal manner because asphalt was prevented from being absorbed into the aggregate by the evolving steam. The mix was unstable at high temperatures. Thorough drying of the aggregate is the best solution to the problem. However, the mix would not be as sensitive to a small amount of water in the system if a more open grading were used, as in Field Study E.

FIELD STUDY E

Location

Ohio.

Complaint

1. The asphalt collected in small areas of the coarse aggregate particles. Black glossy coatings delivered at the mix plant turned light in color en route to the job.
2. The mix did not have good "handling" properties in areas where hand raking was necessary.
3. The mix pushed excessively in front of the roller.
4. The pavement remained tender and was damaged easily for several weeks.

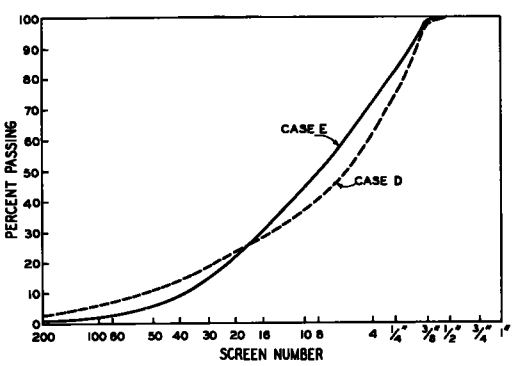

Figure 20. Gradings of aggregates for field problems D, E.

Mix Composition

Aggregate Type—The mix was made by combining two bins. The coarse bin consisted of slick uncrushed silicious gravel; the fine one, of a natural sand. In one instance there was 0. 4 percent water in the aggregate in the coarse bin.

Design Grading—The grading is shown (Fig. 20) to be low in the sand fraction (-16 mesh). It is especially low in the fines (-200 mesh) content.

Design Stability of Mix

1. Hveem stability, about 30.
2. Cohesion (Hveem), less than 200.

3. Marshall stability, about 700.
4. Marshall flow, about 18.

Design Asphalt Content

5.8 percent of 70-85 penetration grade.

Design Voids in Total Mix

Approximately 7 percent air voids were calculated for specimens compacted by both the Marshall and kneading compactors.

Effect of Making Changes

1. The asphalt source was changed from a Venezuelan to a Mid-Continent type. No differences were apparent either in laboratory studies or in the behavior in the field.
2. All symptoms causing complaint disappeared when 1.5 to 2 percent of limestone dust, hydrated lime, or natural fines were added to the mix. The mix compacted to a tough surface resisting damage; the raking became normal, and the mix appeared black on delivery.

Discussion and Conclusions

An economical solution to the problem was made by obtaining a sand fraction containing more natural fines. Although the coarse aggregate in the hot mix contained substantial water, the high voids content in the mix prevented this from causing a critically low stability. By the definition used, this was not an abnormal mix. The difficulties were caused primarily by a low mix stability and low cohesion at high temperature, which was corrected by increasing the filler content.

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Discussion

W.H. CAMPEN, Omaha Testing Laboratories, Omaha, Neb.—The authors are to be complimented for having done a fine job of showing the interrelationships of roller, stability and density of bituminous pavement mixtures. They have put in writing in a systematic manner what experienced designers and field inspectors have had to know in order to construct stable and durable asphalt pavements.

For instance, they give a number of corrective measures which might be taken if a mixture is understressed or, in other words, if the mixture can not be densified properly. This condition came into being in recent years when mixtures had to be made stronger (had to have more stability) to prevent shoving and rutting. The remedial measures suggested are sound. However, if the stability and other properties are to be preserved, the only remedial measure known at this time is the use of pneumatic-tired equipment having tires inflated to 90-psi pressure, and loaded to 6,000 to 8,000 lb per tire, in addition to heavy-steel rollers.

In regard to overstressed mixtures, rather than to adjust the mixture or change rollers, the practical thing to do is to allow the mixture to cool until it can be densified without tearing it up. This procedure is widely used.

In connection with any discussion on understressed and overstressed mixtures it must be understood that mixtures are usually designed not only for stability but also for voids. Changing gradation, filler, or the asphalt content will also change the voids. For this reason, the changing of the mixture should not be recommended. It is assumed also that the original designer was thoroughly qualified.

As far as correcting abnormally acting mixtures is concerned, it would seem that the authors prove by their own field studies that the abnormality is due to improper rolling, insufficient drying of the aggregate, or improper mixture design. Field studies A and B show that the tenderness or lack of stability is due to lack of density. Studies C and D show that moisture in the mixture as laid is the cause of lack of cohesion, density, and stability. Study C also shows that improper grading and excessive asphalt are the cause of shoving and other signs of weakness. It appears, therefore, that the abnormality lies in the method of preparing and laying of the mixtures rather than in the lack of correlation between the predicted and actual behavior

The authors also speak of mixtures which are inclined to tear even if proper rollers are used. There are such mixtures, and experience has shown that these mixtures are either deficient in asphalt or contain an excess amount of materials passing the No. 40, No.80 and/or No.200 sieves. For instance, a ¾-in. maximum size asphaltic concrete containing 60 percent on the No. 10 sieve, 70 percent on the No. 40 sieve, 80 percent on the No. 80 sieve and 90 percent on the No. 200 sieve, would tear. By increasing the amount retained on the No. 40 sieve to 80 percent and making corresponding increases on the other fine sieves, but keeping the amount on the No. 10 sieve at 60 percent the tearing condition would be corrected.

CLOSURE, R.J. Schmidt, W.J. Kari, H.C. Bower, and T.C. Hein—Mr. Campen's comments are appreciated, since they represent many years of experience in constructing and testing asphalt pavements. His statements lend added weight and emphasis to several of the corrective suggestions made in the paper.

Specifically, concurrence with his comments on pneumatic rolling is indicated by the final paragraph of the paper. Also, it is agreed that the short-range solution to rolling an overstressed mixture is to allow it to cool before rolling. However, this solution is not always practical on large projects because of the excessive roller operator overtime involved. On one project, this required as much as 4 hr overtime per day and was easily corrected by a slight change in filler content of the mix.

Mr. Campen's extensive experience with paving may lead him to conclude that there was no discrepancy between predicted and actual behavior of the mixes mentioned in the field studies. However, the authors know of no laboratory tests that would predict

the field problems described in the report. It was only after the problem occurred and further investigations were made that it was possible to arrive at the proper corrective action in each case. Inasmuch as the mixes were the results of normal good design practice, it is felt appropriate to describe their behavior as abnormal until proper corrective action is taken. We are also indebted to Mr. Campen for his detailed explanation on how to redesign mixes which are inclined to tear. Our corrective Step 4 was intended to summarize this procedure.

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