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Bituminous Pavement Permeability and Field Compaction Studies on Asphaltic Concrete

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Contents

Influence of Voids, Bitumen and Filler Contents on Permeability of Sand-Asphalt Mixtures

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•THE PURPOSE of this study was to ascertain experimentally the validity of Darcy's law for water flow through sand-asphalt mixtures as porous media and establish the relationship between the coefficient of permeability and the physical properties of the mixtures, namely, the bitumen, filler, and air voids (total mix) contents.

Mixtures were prepared with various amounts of bitumen and filler and the relation ship of these contents to that of air voids was established using the Marshall method of compaction (3 layers, 50 blows each).

The rate of water flow (distilled and de-aired) through the mixtures was measured with a specially constructed variable high-head permeameter and the coefficient of permeability of the water calculated.

The results of these tests have shown that sand-asphalt mixtures behave as porous media obeying Darcy's law; i.e., that the rate of water flow per unit time is proportional to the hydraulic gradient. It was also found from a plot of log total percent voids against log coefficient of permeability that the trend is linear and an increase in void content results in an increased coefficient of permeability, according to

$$\log_{10} K = m \log_{10} (p) + n \text{ or } K = 10^{n} . p^{m}$$

the general form being

$$K = a. (p)^{m}$$

in which K = Darcy's coefficient of permeability (cm/sec);

p = percent voids, total mix;

m, n = constants; and

a = the coefficient of permeability when p = 1 percent depending on the gradings of the mixtures tested.

The relationship of the bitumen and filler contents to the coefficient of permeability of these sand-asphalt mixtures was found to be directly related to its influence on the air voids (total mix) content.

Comparison of these test results with similar ones obtained by McLaughlin and Goetz (1) show obvious influence of the mix grading on the coefficient of permeability.

Properties of Mixture Ingredients

The sand-asphalt mixtures were composed of uniformly graded natural dune sand, crushed limestone dust as filler, straight-run bitumen (80/100 penetration) as binder. The natural dune sand served as skeleton of the entire asphalt mix; its apparent specific gravity is 2.66 and its dry density was found to be 1,650 kg per cu m (103 pcf) when compacted dry by the Marshall method described earlier.

The crushed limestone dust filler (by-product of local limestone quarries) has an apparent specific gravity of 2.81 and its grading is shown in Figure 1.

The straight-run bitumen with 80/100 penetration (product of the Haifa Refineries) has a specific gravity of 1.02 at 25 C, with physical properties according to ASTM specifications.

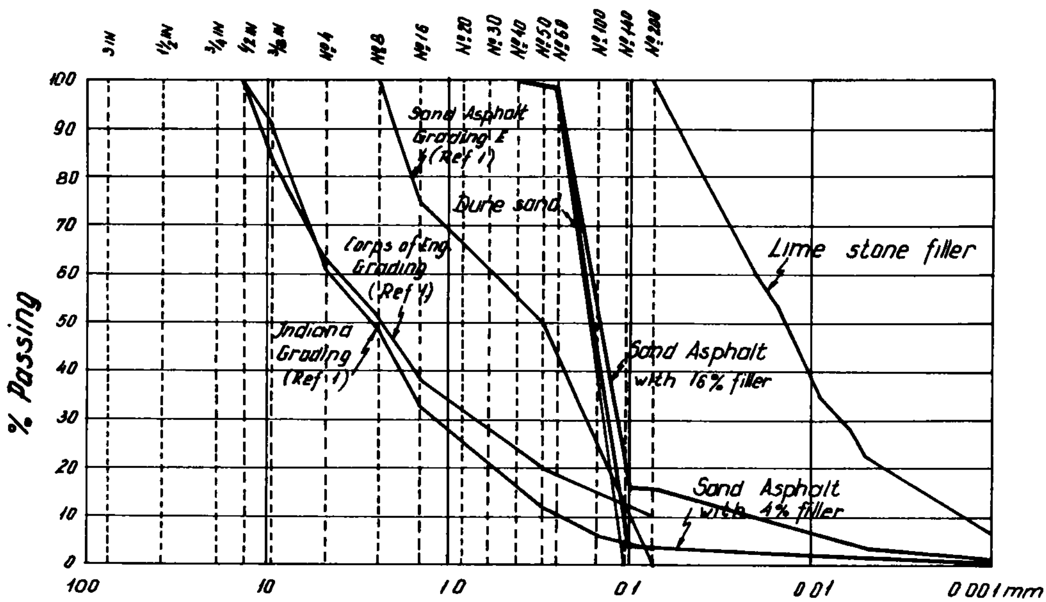

Figure 1. Grading chart.

Samples were prepared with different bitumen and filler contents (see Table 1), thus providing mixtures with different air voids contents in the total mix. The grading limits of the sand-asphalt mixtures are shown in Figure 1—the coarser limit for a minimum of 4 percent limestone dust filler and the finer limit for a maximum of 16 percent filler.

The following relationship between the voids content in the total mix with bitumen and filler content can be established on the assumption that the natural dune sand is the skeleton of the mix:

$$\frac{p}{100} = 1 - \frac{\gamma sa}{1 - \frac{wf}{100}} \; \frac{1}{\gamma_w} \left[\frac{Wb}{100} \; \frac{1}{G_b} + \frac{Wf}{100} \; \frac{1}{G_f} + \frac{1 - \frac{wf}{100}}{G_{sa}} \right]$$

in which

p = percent air voids, total mix;
γsa = dry density of the compacted sand (kg/m^3)
γ_w = density of water (kg/m^3);
W_f = filler content - percent (by weight) of dry mix (sand + filler);
W_b = bitumen content - percent (by weight) of dry mix;
G_b = specific gravity of bitumen;
G_f = apparent specific gravity of filler; and
G_{sa} = apparent specific gravity of sand.

This relationship for the range of Table 1 is shown graphically in Figure 2. Actual laboratory results have generally shown good agreement with these theoretical assumptions. All pertinent results are summarized in Table 1.

Measurements of Coefficient of Permeability

The Permeameter and Specimen Cell. — Figure 3 shows the variable-head device used for this study. It consisted of the specimen cell (J), two compressed air steel containers (a and b), CO_2 gas steel container (c), pressure regulator (e), two pressure gauges (g and p), Mercury manometer (m), two clocks (i and n), thermometer (f), two

TABLE 1

SUMMARY OF RESULTS OF PERMEABILITY TESTS OF SAND-ASPHALT MIXTURES

Grading	Bitumen (% wt of aggr.)	Filler (% wt. of aggr.)	Length of Spec. (cm)	Area of Specimens (sq cm)	Unit Wt. of Mix. (kg/m^3)	% Air Voids Total Mix	Coefficient of Permeability (cm/sec)
A_1	6.0	4.0	4.23	83.5	1,885	24.2	3.2×10^{-4}
A_2	6.0	4.0	4.22	83.4	1,775	27.5	2.0×10^{-4}
A_3	6.0	4.0	4.21	83.4	1,875	23.1	6.5×10^{-4}
C_1	10.0	4.0	4.18	83.5	1,910	17.5	5.5×10^{-6}
C_2	10.0	4.0	4.18	83.5	1,925	16.8	2.7×10^{-6}
C_3	10.0	4.0	4.08	83.5	1,933	16.5	1.3×10^{-4}
F_1	6.0	8.0	4.27	83.4	1,895	22.5	3.5×10^{-4}
F_2	6.0	8.0	4.33	83.4	1,892	22.7	1.4×10^{-4}
F_3	6.0	8.0	4.27	83.4	1,815	22.9	3.9×10^{-4}
G_1	8.0	8.0	4.27	84.0	1,940	15.7	1.6×10^{-4}
H_{11}	12.0	8.0	4.12	83.5	2,044	11.2	1.6×10^{-5}
H_{12}	12.0	8.0	4.12	83.5	2,044	11.2	1.1×10^{-5}
H_{13}	12.0	8.0	4.12	83.5	2,044	11.2	5.5×10^{-6}
H_{21}	12.0	8.0	4.21	83.5	2,046	11.0	3.6×10^{-5}
H_{22}	12.0	8.0	4.21	83.5	2,046	11.0	2.7×10^{-5}
H_{31}	12.0	8.0	4.16	83.5	2,052	10.5	1.8×10^{-5}
H_{32}	12.0	8.0	4.16	83.5	2,052	10.5	4.25×10^{-6}
K_1	6.0	12.0	4.42	83.4	1,937	21.0	6.1×10^{-4}
M_2	10.0	12.0	4.47	82.0	2,040	12.0	1.6×10^{-5}
R_2	10.0	16.0	4.22	83.5	2,077	10.8	8.2×10^{-7}
B_1	8.0	4.0	4.08	81.8	1,882	21.1	5.9×10^{-3}
D_1	12.0	4.0	4.13	83.7	1,935	15.0	2.65×10^{-5}
E_1	12.0	4.0	4.18	84.0	2,021	9.3	8.1×10^{-6}
G_2	8.0	8.0	4.13	82.3	1,950	18.7	2.5×10^{-4}
H	10.0	8.0	4.26	81.2	1,965	15.3	1.5×10^{-4}
I_2	12.0	8.0	4.35	82.0	2,040	10.4	1.4×10^{-5}
J_3	14.0	8.0	4.27	84.0	2,050	9.9	1.33×10^{-5}
L_2	8.0	12.0	4.41	84.0	1,965	17.5	1.0×10^{-4}
N_1	12.0	12.0	4.20	84.0	2,050	9.7	6.6×10^{-5}
O_1	14.0	12.0	4.37	84.0	2,060	7.4	3.7×10^{-8}
P_3	6.0	16.0	4.50	82.0	1,860	19.6	1.05×10^{-3}
Q_2	8.0	16.0	4.65	82.0	2,080	15.1	1.5×10^{-4}
R_1	10.0	16.0	4.27	83.5	2,064	11.3	8.0×10^{-6}
S_1	12.0	16.0	4.63	82.0	2,090	8.0	3.6×10^{-6}
T_2	14.0	16.0	4.63	82.0	2,120	5.3	1.25×10^{-9}
Ý$_1$	8.0	12.0	3.95	82.5	2,045	13.2	1.90×10^{-5}
Y_1	8.0	12.0			2,045	13.2	8.60×10^{-6}
Y_2	8.0	12.0	4.50	82.5	2,030	13.9	1.20×10^{-4}

[a]After 1 month.

graduated glass standpipes of 70-cc capacity each (o and y), 20-1 volume bottle for distilled de-aired water (g), measuring cylinder (x) for collecting the outflowing water, vacuum pump (z), air compressor; gas stopcocks for preventing leakage and water connections made of plastic tubing.

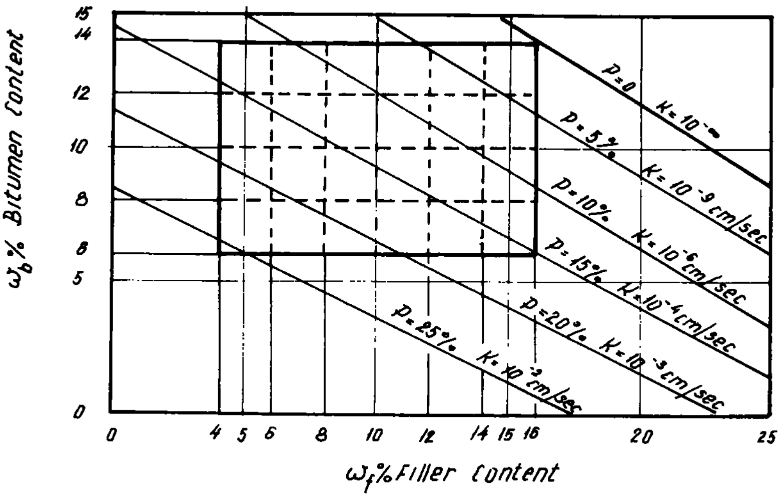

Figure 2. Percent voids total mix. Bitumen and filler contents.

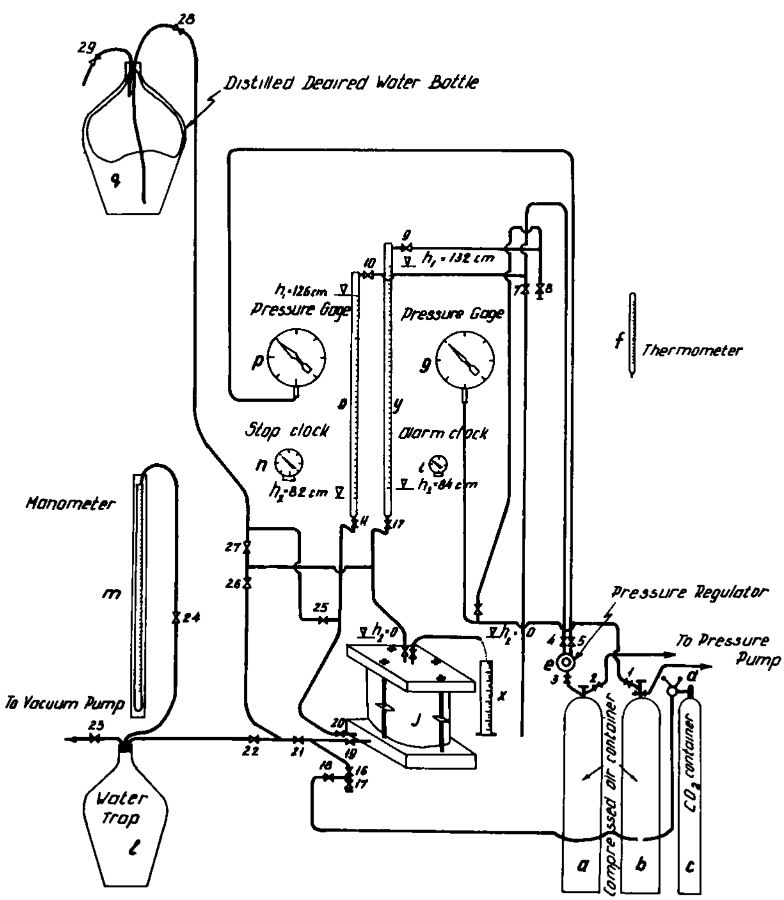

Figure 3. Setup for variable head permeability test.

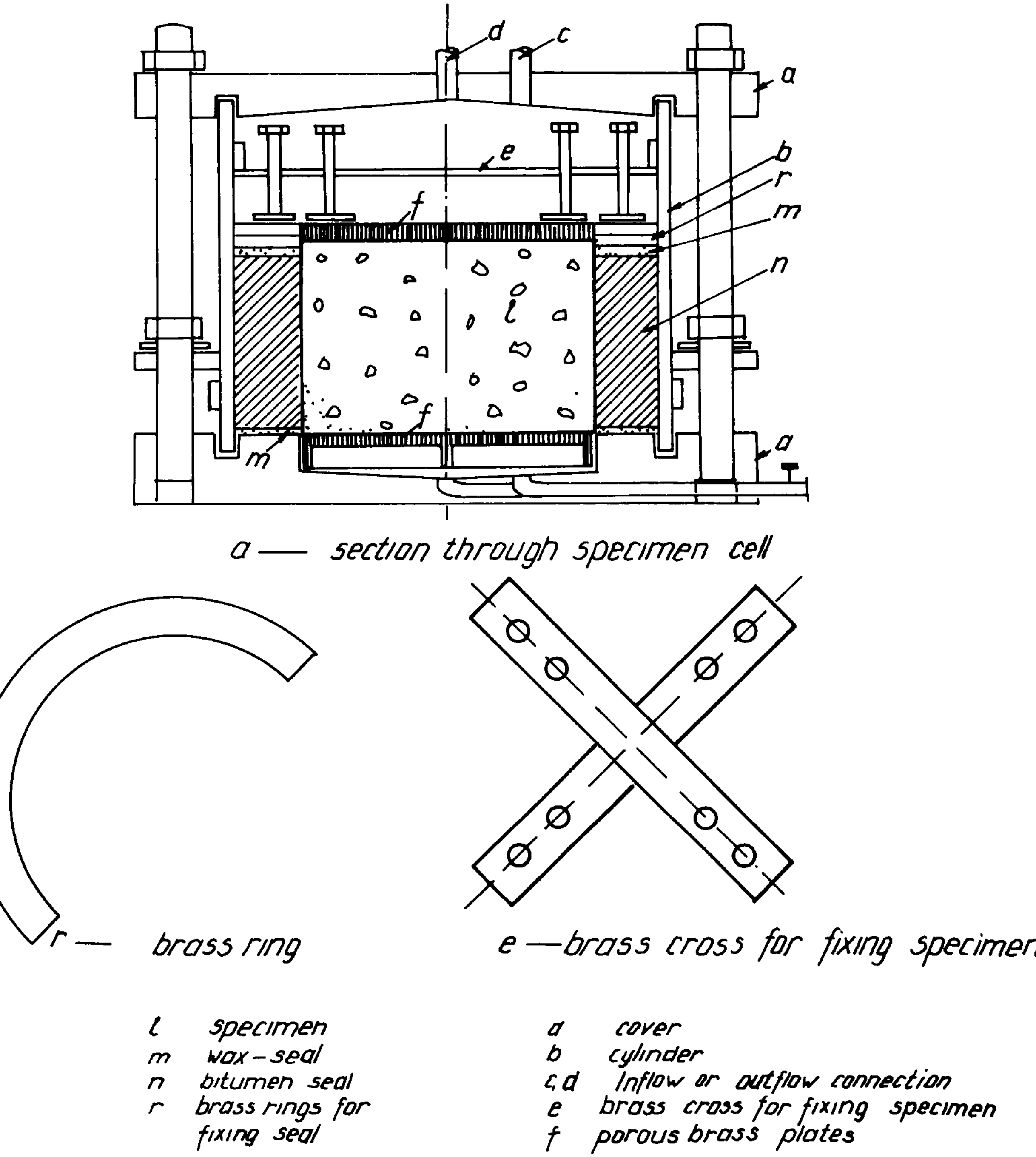

Figure 4. Details of specimen cell.

Figure 4 shows a detailed section of the specimen cell (J in Fig. 3); the cell consists of a 6-in. internal diameter steel cylinder (b) a cover with a conical undersurface (a) provided with water inlets (c and d), as well as vacuum and air exits.

The bottom is recessed to take a porous brass plate (f) on which the specimen 1 rests. The space between the 4-in. diameter specimen and the 6-in. cylinder is sealed with a mixture of wax and bitumen. A brass cross (e) over the specimen fixes it in place.

Experimental Procedure.—The following experimental procedure was used. First, vacuum was applied to the cell for about 15 min; then the specimen was saturated, allowing water to penetrate upwards through its bottom, thus driving out air bubbles; after water had started flowing into the graduated measuring cylinder (x) the rate of flow was measured and checked; the test was re-run thrice for each hydraulic gradient. The coefficient of permeability K, cm per sec at the temperature of the water was calculated according to (2).

$$K_t = 2.3 \frac{a.L}{A(t_1 - t_o)} . \log_{10} \frac{h_o}{h_1}$$

The permeability obtained for temperature T, K_T was reduced to that at 20 C, $K_{20\,C}$ by (2).

$$K_{20\,C} = K_T \frac{\mu_T}{\mu_{20\,C}}$$

When a high hydraulic gradient was required, use was made of appropriate compressed air pressure, taking care to keep air from penetrating the water passing through the specimen.

TEST RESULTS

The rate of flow over a range of various hydraulic gradients was measured in specimens having different voids contents in the total mix and a good linear relationship was found between the hydraulic gradient and rate of flow, thus confirming the validity of Darcy's law for the sand-asphalt mixtures tested. Figures 5, 6, 7, 8, and 9 show the results of the tests in specimens H_{31}, H_{32}, y_1, and y_2. Specimen y_1 (see Fig. 7 for results) was stored, after its test, for one month in a saturated condition and then retested at various hydraulic gradients; again, a linear relationship was found between hydraulic gradients and the rate of flow, but the coefficient of permeability was lower (see Fig. 8).

In Figure 9, test results of specimen y_2 are shown. Experiments with this specimen comprised series with the hydraulic gradient varied in small intervals; the rate of flow for each value of the gradient was found linear but its absolute values decreased through the series, reaching a constant value after 10 variations. This phenomenon was observed on most of the specimens tested.

Permeability vs Percent Voids, Total Mix

To establish the relationship of the coefficient of permeability calculated from the experiments a log log scale plot was chosen: a linear trend was found between log percent voids, total mix, and log coefficient of permeability (Fig. 10). This linear relationship can be expressed by

$$\log_{10} K = m \log_{10} p + n$$

or

$$K = 10^n . p^m$$

the general expression being

$$K = a . p^m$$

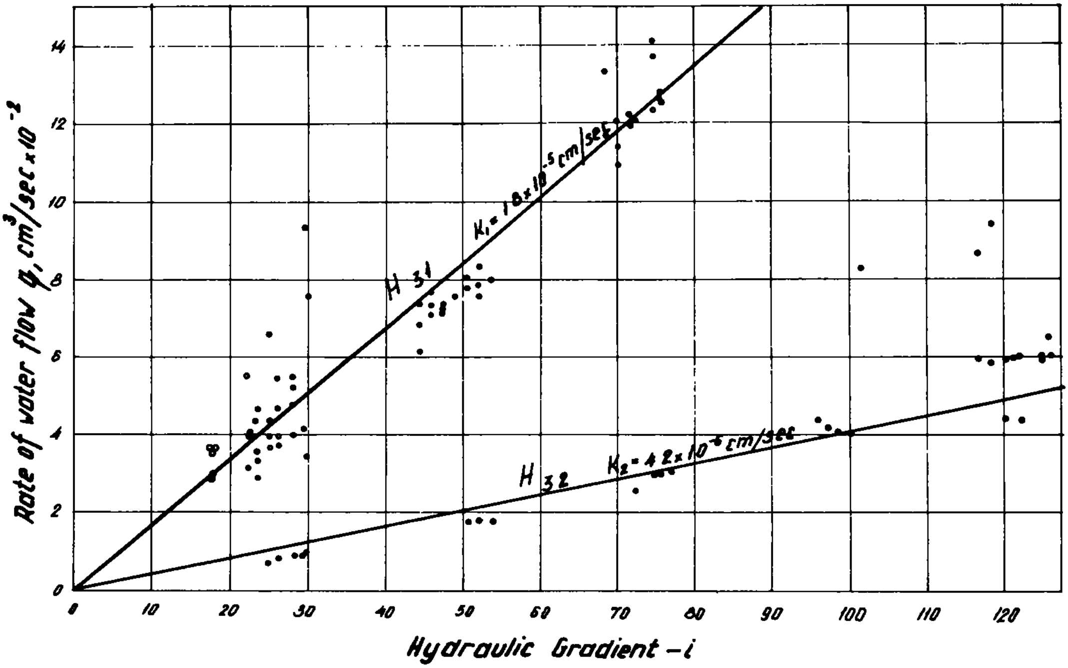

Figure 5. Rate of water flow vs hydraulic gradient (specimens—H_{31} and H_{32}).

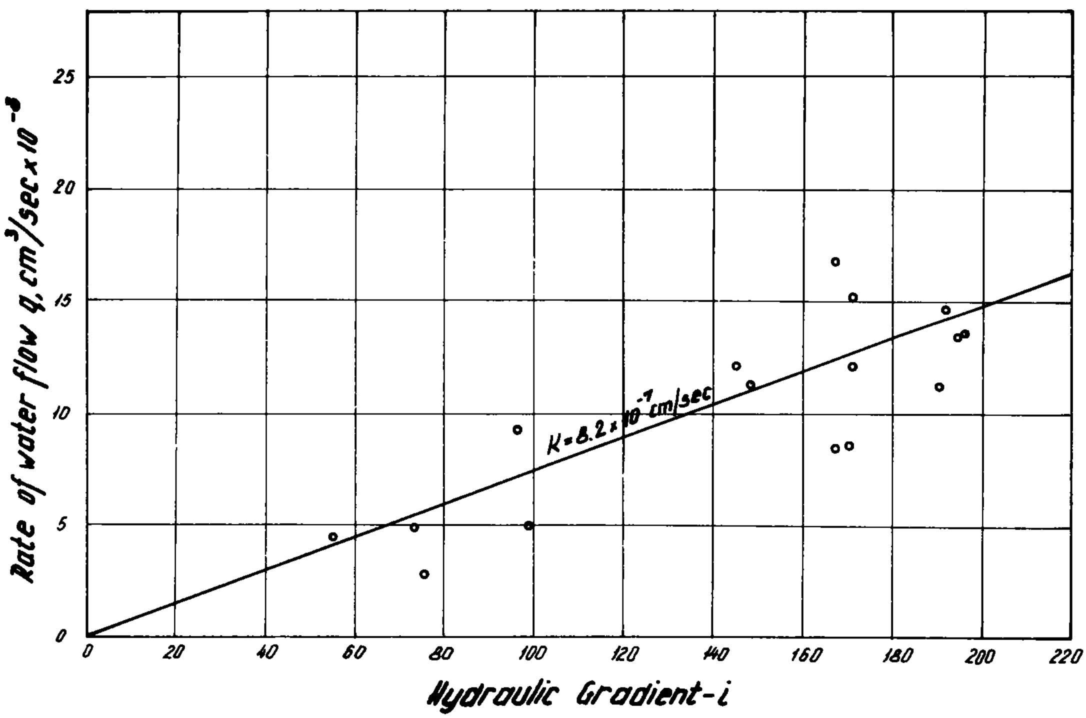

Figure 6. Rate of water flow vs hydraulic gradient (specimen R_2).

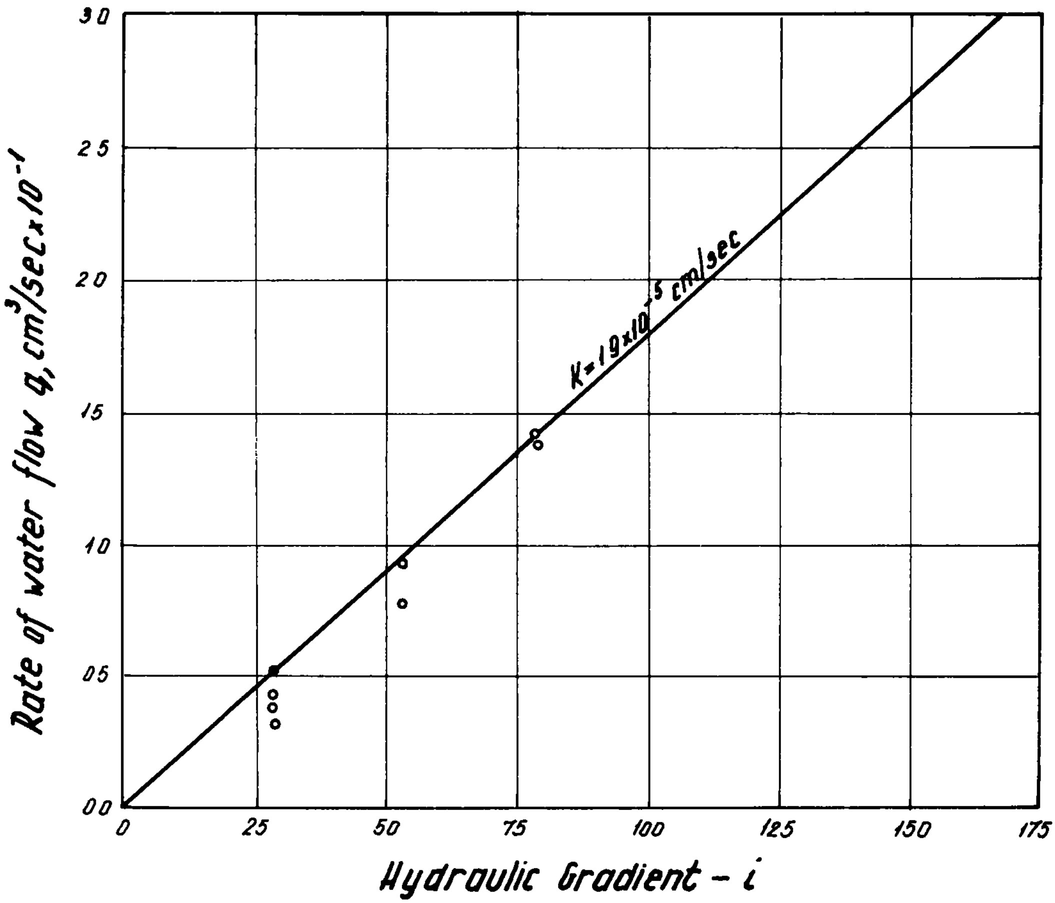

Figure 7. Rate of water flow vs hydraulic gradient (specimen Y_1).

For the range of mixtures tested (Fig. 9) this expression becomes

$$\log_{10}K = 10 \log_{10}p - 16$$

or

$$K = 10^{-16} \cdot p^{10}$$

Values of K for a range of voids contents are given in Figure 2.

Permeability vs Bitumen and Filler Contents

The influence of the bitumen and filler contents on the permeability of the sand-asphalt mixtures is expressed indirectly by its influence on the voids content in the total mix (Fig. 2). A combination, for instance, of 6 percent bitumen and 16 percent filler gives a mixture with 15 percent air voids and the estimated coefficient of permeability based on these experiments, would be $K = 5.5 \times 10^{-5}$ cm per sec. The same value can be obtained by using 8 percent bitumen and 12 percent filler, or 10 percent bitumen and 9 percent filler; i.e., for the same voids content 2 percent of filler is equivalent to about only 1 percent of bitumen.

Permeability vs Grading

To evaluate the influence of the grading of the mixture (in addition to that of the

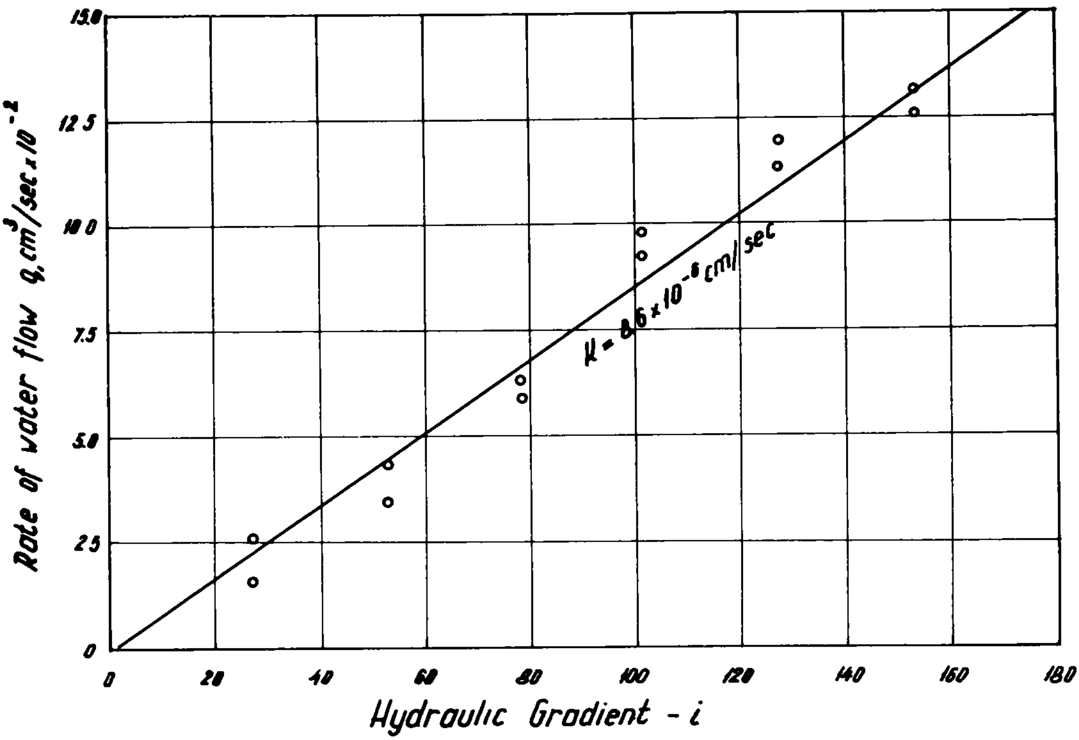

Figure 8. Rate of water flow vs hydraulic gradient (specimen Y_1 after one month).

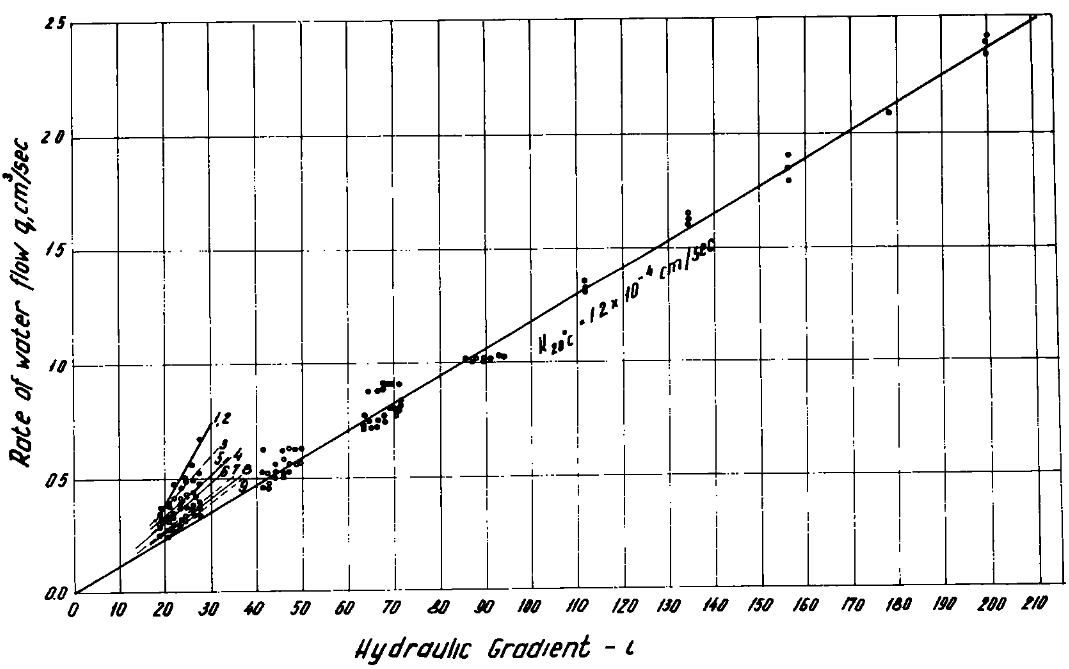

Figure 9. Rate of water flow vs hydraulic gradient (specimen Y_2).

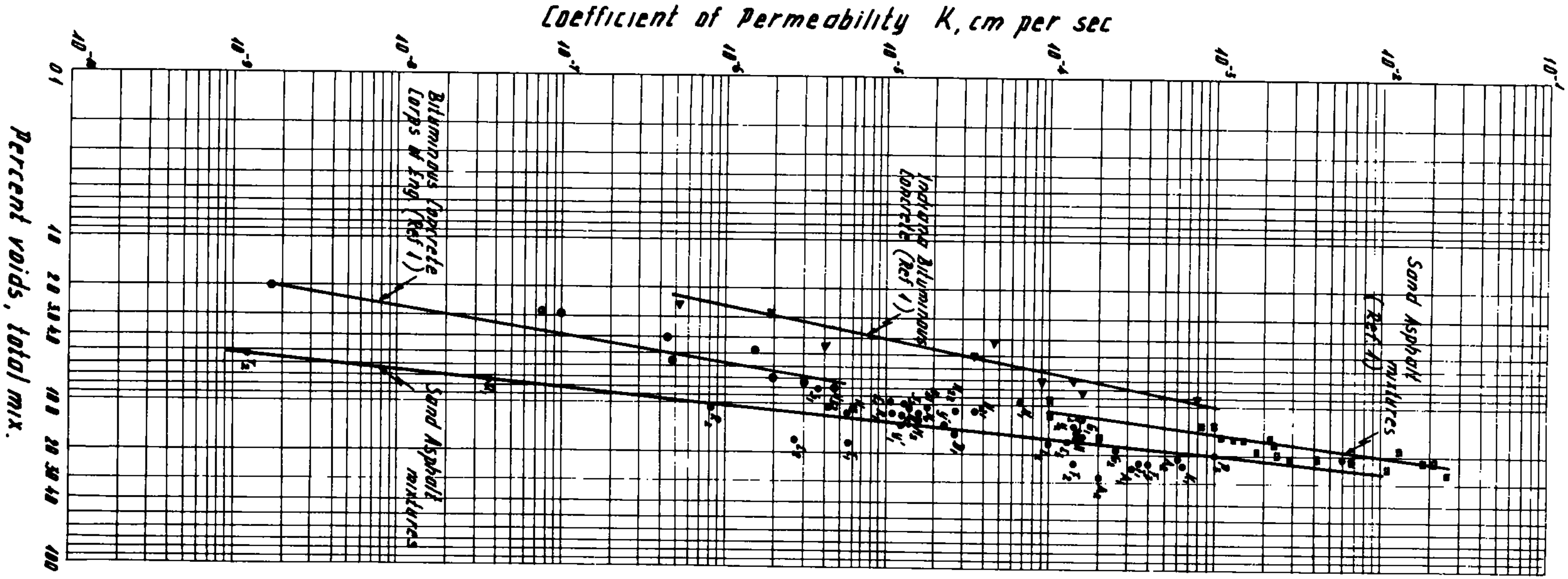

Figure 10. Coefficient of permeability K (cm/sec) vs percent voids total mix. (log-log scale).

voids content) on the value of coefficient of permeability, comparison is made with results of similar test by McLaughlin and Goetz (1).

Figure 1 gives the gradings of the bituminous concrete and sand-asphalt mixtures of (1) and Figure 10 their respective coefficients of permeability vs percent voids total mix (log log scale). The comparison clearly shows that for a given grading, the coefficient of permeability is dependent on the voids content and that varying gradings correspond to different coefficients of permeability—for the same voids content in the total mix, the finer the grading the smaller its coefficient of permeability. Figure 10 shows that for 5 percent voids the coefficient of permeability of the Indiana bituminous concrete grading is $K = 3 \times 10^{-5}$ cm per sec that of the Corps of Engineers bituminous concrete grading 5×10^{-7} cm per sec; that of the sand-asphalt mixtures, about 2×10^{-7} cm per sec; and that of the sand-asphalt mixtures of the present study (which had the finest grading), 1×10^{-9} cm per sec.

CONCLUSIONS

From the limited results of the present study and those elsewhere (1), the following conclusions can be drawn:

1. Permeability measurements showed a linear relationship between the hydraulic gradient and rate of water flow—the medium is porous and Darcy's law is valid.
2. At the beginning of the permeability tests, the rate of flow of water was high; on repeating the test, a gradual decrease was observed tending to a constant limit.
3. For a given grading, a linear trend was found on a log log scale, between the coefficient of permeability K (cm per sec) and p - voids content of the total mix, the equation of the line being K (cm per sec) = $a.p^m$ with the constants a and m depending on the parameters of the mixture including the bitumen and filler contents.
4. The bitumen and filler contents of the sand-asphalt mixtures tested affect the coefficient of permeability indirectly; namely, through their influence on the voids content. For a given mixture, an increase in bitumen content by 1 percent decreases the voids content by almost the same amount as an increase of 2 percent in dust filler, thus rendering the coefficient of permeability more sensitive to changes in the bitumen content to those in filler content.

In the light of the limited but encouraging results, further research is called for, mainly with a view to establishing a more definite correlation between the coefficient of permeability, voids content and grading parameters over a wide range of fine, medium, and coarse mix gradings each covering a wide range of percent voids.

ACKNOWLEDGMENT

This paper is based on the M.Sc. thesis of the second author, under the supervision of the first author.

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2. Lambe, T. W., "Soil Testing for Engineers." Wiley.

Compaction Studies of Asphalt Concrete Pavement as Related to the Water Permeability Test

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In constructing a stable and durable asphalt concrete pavement, it is imperative that the mixture be correctly designed and, during the laydown operations, be properly compacted. If, in the finished pavement, the void content is high, particularly if the voids are interconnected, the entrance of air and water may adversely affect the service life of the pavement mixture. To guard against excessive voids or porosity of the pavement many organizations specify some minimum percent relative compaction of the finished pavement, the percentage of relative compaction being measured against some standardized laboratory procedure. On review of these time-consuming methods, it became apparent that a simpler approach to evaluate this property of the pavement was desirable. This paper presents data relating to the void content of the pavement as influenced by different rolling procedures and also stresses the importance of having newly laid asphaltic pavement subjected to traffic before the wet weather season. A simple test method is presented which measures the porosity of the pavement and can be used as an aid during the compaction procedure.

• IN CONSTRUCTING a stable and durable asphalt concrete pavement, two important steps are necessary: (a) the mixture must be correctly designed, and (b) it must be properly compacted. In the design of the mixture, such factors as gradation of the aggregate, particle shape characteristics and surface texture, absorption of asphalt by the aggregate, and optimum asphalt content are important considerations. In the laydown operations, temperature of the mix, type of compaction equipment, and air temperature are of paramount importance. If, in the finished pavement the void content is high, particularly when the voids are interconnected, the passage of air and the admittance of water will adversely affect the durability and ultimate life of the pavement mixture. The entrance of air into a permeable pavement contributes to the rapid hardening of the asphalt binder primarily through oxidation and evaporation. This fact has been cited extensively in the literature and needs no further elaboration here (1), (2), (3). This paper deals primarily with the compaction and its influence on water permeability of asphalt concrete pavements.

The Materials and Research Department of the California Division of Highways has collected over the years a great deal of evidence which leads to a rather conclusive opinion that many asphalt pavement failures are attributable directly to the presence of excessive amounts of water that entered the pavement structure after construction.

On breaking a chunk of pavement from many failed areas, colloidal fines were often found in the intimate part of the mix and particularly in the lower course of the asphalt pavement. This is caused by pumping action resulting from deflection under heavy loads. The infiltration of fines from muddy water into small cracks of the pavement mixture will considerably reduce the cohesion of the mixture and also prevent any possibility of the cracks "healing" under traffic action during summer temperatures.

It is the general assumption that the permeability of the compacted pavement and its durability are more or less proportional to the percentage of air voids. This statement should only be accepted in a general sense. Certain size dimensions of the individual voids, and the lack of interconnection of the voids could easily produce a pavement of relatively high void content and a low permeability. In other words, low density and permeability are not necessarily the same thing. In this case, it is to be expected that the hardening of the bituminous binder will progress at a relatively slow rate. One other important phase of the permeability-void-durability relationship which should be stressed is that the preceding statements are generally true when the same asphalt and aggregate mixture is used. On the other hand, it should not be overlooked that the source or method of manufacturing the bituminous binder may far overshadow the effects of permeability and air voids as far as durability of the pavement is concerned (4, 5).

To guard against excessive voids of the pavement, many organizations specify some minimum relative compaction of the finished pavement, the percentage of relative compaction being measured against some standardized laboratory compaction or in some cases against theoretical density. To determine this relative compaction, it has been necessary to obtain samples of the compacted mixture by either chipping out blocks or obtaining a core. Although both methods have been used with some degree of success, there are definite drawbacks. In breaking out a block, the compacted mixture is very often disturbed, which may lead to erroneous results in the specific gravity determination. When obtaining cores by drilling, water is introduced into the specimen and considerable time is required to dry the core at low temperature. This causes some delay in determining relative compaction results.

After reviewing existing methods, and through the field studies it became evident that if a physical check on compaction during construction was to be possible, a rapid method of measuring relative compaction of pavement mixtures was needed. The purpose of this report is to present the studies as they relate compaction and water permeability of asphaltic concrete pavements, and to discuss the factors that influence the permeability during construction and service life of the pavement. A simple test method is presented which is equally applicable to new or existing pavements.

TABLE 1

PERMEABILITY OF PAVEMENT IMMEDIATELY AFTER CONSTRUCTION[a] COMPARED WITH MOISTURE CONTENT IN PAVEMENT AFTER WINTER RAINS

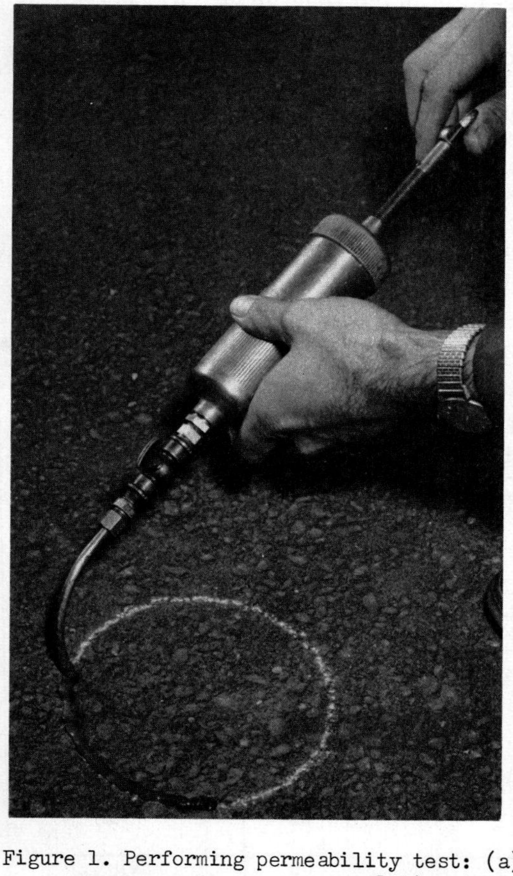
Permeability Measurement Date	Permeability (ml/min)	Sample Date for Moisture	Percentage of Pavement Moisture
Dec 1956	10	March 1957	1 37
	10		1.50
	15		2 50
	35		2.96
	55		3 10
	105		2.69
	112		2.18
	170		3 14
	610		3.37

[a]Average percentage of moisture in paving mixture during construction = 0.2 percent.

TEST METHOD

In the preliminary studies, it was noted that water poured on a new asphaltic concrete pavement did not readily wet the surface and very little entered the mix. Later studies showed that traffic action, together with the presence of dust on the surface tends to change the interfacial tension relationship, and water will readily enter a permeable surface during the first rains. The problem of devising a test for permeability of the surface was solved by the addition of a small amount of detergent to reduce the surface tension of the water used in the test. On a number of jobs involving a relatively impermeable base, such as cement-treated base, the values obtained by this method show good relationship between the permeability and the moisture content found in the mix following rains (see Table 1).

The test method is detailed in Appendix A. The equipment has been assembled in a compact kit and is readily portable. The general technique was originally developed in connection with seal coat studies (6) and has been in use by this department for the past six years. Briefly, the test is performed by forming a small reservoir by means of a "grease ring" or dam around a previously marked 6 in. circle on the pavement (Fig. 1a). The ring may be easily placed with a grease gun using ordinary cup grease, or it may be completely formed in one operation by a special gun. The ring grease is sealed to the surface by running the finger around the outside edge of the grease. A special graduated cylinder containing the test solution and equipped with a drain tube is placed beside the ring, and the operator feeds the solution into the area within the ring, starting a stop-watch at the start of flow of the liquid from the graduate (Fig. 1b).



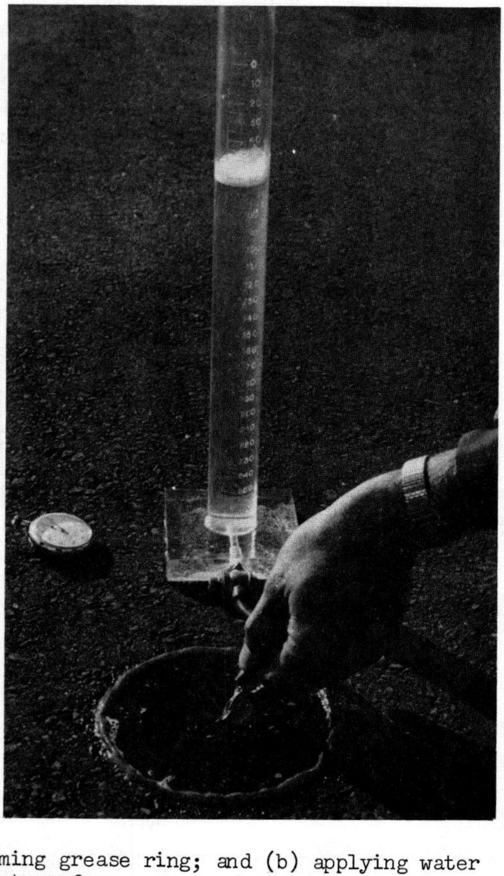


Figure 1. Performing permeability test: (a) forming grease ring; and (b) applying water solution to pavement surface.

The area within the ring is kept moist during a test period of 2 min. The film of liquid in the ring should only be thick enough to present a glistening appearance. In other words, the water is fed in only as fast as it is absorbed by the pavement and the test is conducted under zero pressure. At the end of the 2-min period the total solution used is divided by two and the permeability reported in milliliters per minute for a 6-in. diameter circle. The complete test may be performed in 3 to 5 min.

Dense-graded bituminous surfaces that are covered with an open-graded mix may also be tested by chipping away the open-graded mix, just outside of the 6-in. ring, down to the dense-graded surface. (A ½-in. thick open-graded mix, employing ¼-in. maximum aggregate, is being placed extensively in California over newly placed dense-graded asphalt concrete.) The ring of open-graded mix removed is about ½ to ¾ in. wide. This annulus is filled with the grease to form the seal, and the permeability is determined through the open-graded mix within the area of the ring. It was found necessary to perform the test in this manner, because removal of all the open-graded surface with a chisel within the ring area tends to seal the underlying surface with a glaze of asphalt and results in erroneous readings.

The present practice is to perform tests successively at intervals of 25 ft in the wheel tracks and at a point midway between. A total of six readings constitute a "set" for any one area, and the average is then obtained. A series of these sets should be obtained over the length of the job.

On a multi-lane road, one of the important areas for checking should be between the wheel tracks in the passing lane. An initially high permeability of a pavement may be reduced to a satisfactory low value in the wheel tracks by traffic action. However, the between-wheel-track areas may remain relatively unchanged and water may enter here, cross-flow through the pavement, on top of the base, and collect beneath the wheel track areas.

FACTORS INFLUENCING PERMEABILITY OF PAVEMENT DURING CONSTRUCTION AND SERVICE LIFE

The field studies have uncovered a number of variables that influence the permeability of the pavement during construction and its service life:

1. Segregation of mix during placing.
2. Temperature of mix during breakdown rolling.
3. Temperature of mix during pneumatic rolling.
4. Weight of breakdown roller.
5. Tire or contact pressure of pneumatic roller.
6. Ambient temperature during placing of mix.
7. Void content of the compacted mix.
8. Amount of traffic before winter rains.

Even though every effort is made to maintain uniform construction procedures, individual permeability test values may be still quite variable.

On one project the variations in a single load of mix were determined by taking readings every 5 ft in a longitudinal direction and every 2 ft transversely. This was performed for three separate loads of mix in different test sections. The average values for one load of mix in a transverse and longitudinal direction are shown in Figure 2. The frequency of values for an individual load in each of three different test sections is shown in Figure 3. The results indicate an increasing spread of values with increasing permeability, with the spread being greatest for values above the average. It is necessary to obtain a sufficient number of readings to insure a reliable average reading, if it is desired to evaluate compaction by the permeability test.

Also, permeability can not be estimated by visual inspection of the surface appearance, with the possible exception of obvious rock pockets. Figures 4a and 4b show the large variation in permeabilities for the same general surface texture on a particular project.

The method of placing the mixture may influence the permeability values transversely,

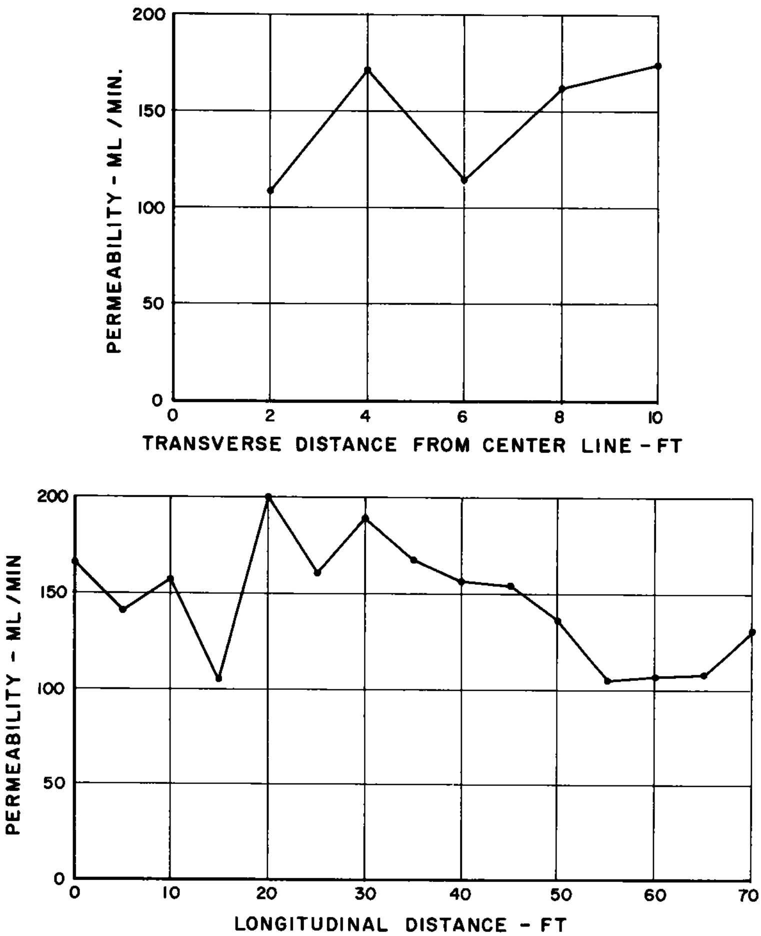


Figure 2. Variation in average permeability after spreading and rolling a single load of mix.

across the lane, as shown in Figure 5. In the normal paving procedure with end dump trucks, the initial permeability is generally higher in the future wheel track areas, probably due to some segregation of the mixture near the edges by the lateral distribution device in the paver. However, this is reversed when the bottom dump method is used. There is apparently a greater amount of segregation in the latter method, and this is manifested by a higher permeability value immediately in back of the pickup or conveyer equipment of the paver.

A major factor is the temperature of the mix at the time of breakdown and pneumatic rolling operations. The results of varying the breakdown temperatures are shown in Table 2 and Figures 6 and 7. The change in permeability values for base and surface courses having different gradings and asphalt contents, but rolled with the same equipment are shown in Figure 6. The reduction in permeability with increase in breakdown temperature is very definite for both types of mixtures. The importance of this factor is further indicated by results shown in Figure 7. The average permeability value after completion of high temperature breakdown rolling in Section 2 is almost as low as the complete rolling schedule in Section 1 where breakdown temperatures were much lower. These results indicate that the permeability test does provide an indication of the degree of densification during the breakdown rolling operation.

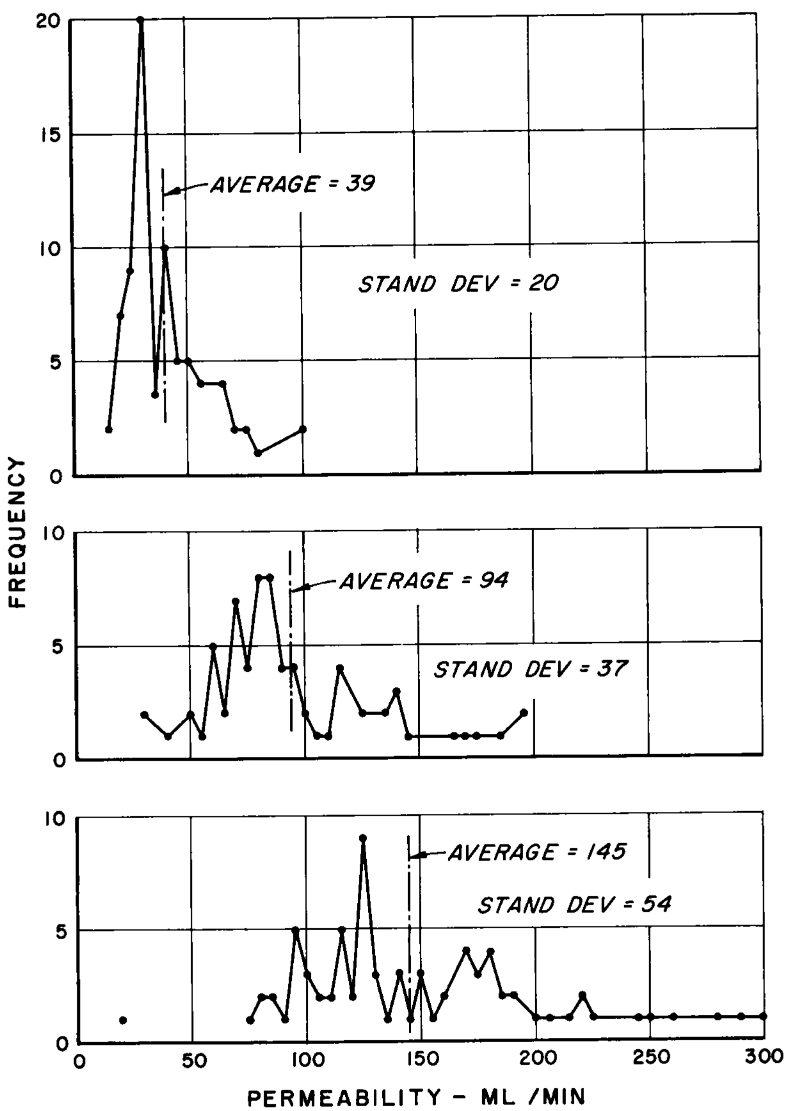


Figure 3. Variation in permeability after spreading and rolling a single load of mix from three test sections on same project.

Further reduction in permeability following breakdown compaction may be achieved by pneumatic rolling. Experience has clearly indicated that traffic action is very effective in achieving a "tightening" or "sealing" of the surface and one reason for pneumatic rolling is to obtain this during construction. The requirement for pneumatic-tired rolling in the California Division of Highways 1960 Standard Specifications (Appendix B) was based on evidence that this form of rolling is an effective way to reduce permeability.

Some typical results of permeability-rolling combination studies obtained under the 1954 specifications are shown in Figure 8. Although an average reduction in permeability is noted with increased rolling, it is not as great as would be expected. Unfortunately, the pneumatic roller tire pressures did not exceed 35 psi. However, on one project it was possible to boost the tire pressure up to 50 psi and a definite reduction of about 150 ml per min was attained when compared to the 35-psi tire pressure.

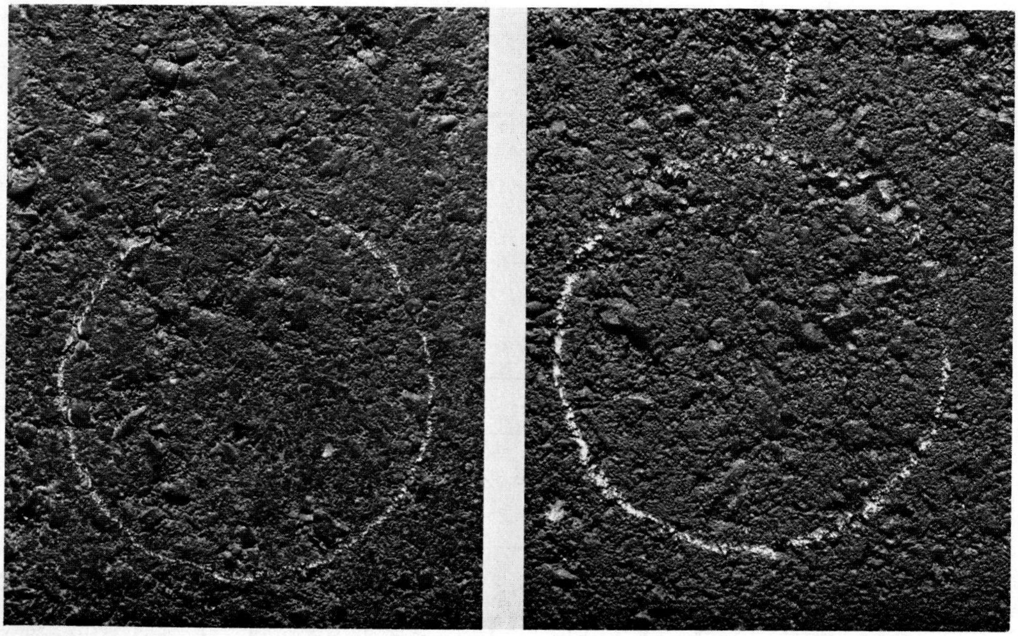


Figure 4. Permeability-surface texture relation: (a) 10 ml per min; and (b) 580 ml per min (both figures from same project).

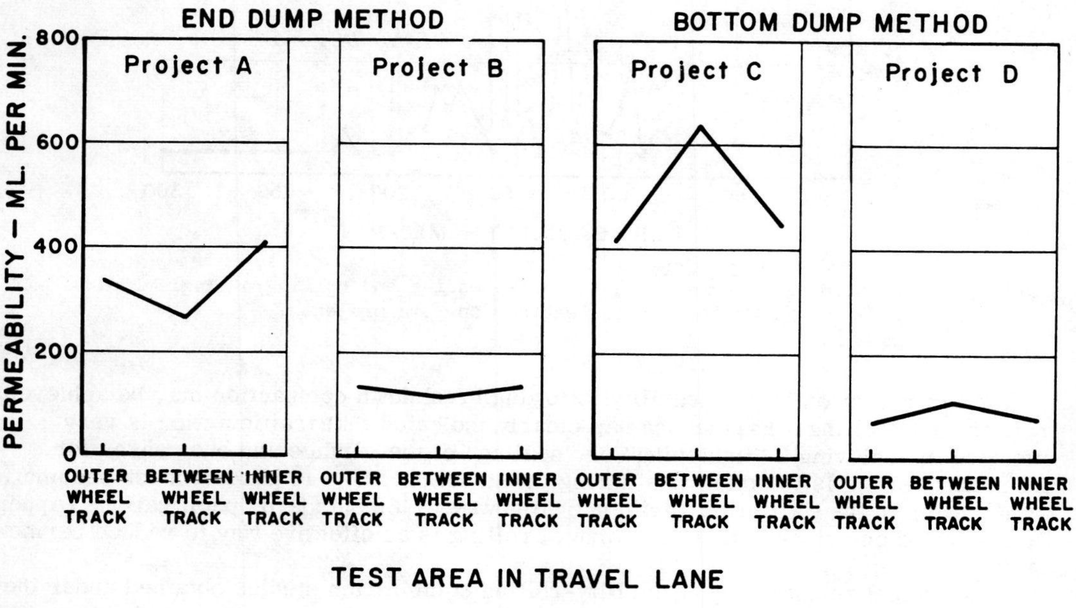


Figure 5. Average permeability values for two different paving methods.

TABLE 2

ROLLING STUDIES, PROJECT E

Paving Date	Max. Mix Type (in.)	Test Sect.	Lane	Course	Type of Rolling	Average Rolling Temp. (°F)	Avg. 24-Hr Perm. (ml/min)
June 1960	2 1/2	A	Passing	Base	Breakdown, 1 coverage,	278	
					finish with tandem	160	227
		B	Travel	Base	Breakdown, 1 coverage,	298	
					finish with tandem	168	162
		B-1	Travel	Base	Breakdown, 1 coverage,	281	
					pneumatic, 1 coverage,	176	
					finish with tandem	155	155
		B-2	Travel	Base	Breakdown, 1 coverage,	289	
					pneumatic, 3 coverages,	168	
					finish with tandem	155	135
	3/4	1	Passing	Surface	Breakdown, 1 coverage,	218	
					finish with tandem	163	352
		1-1	Passing	Surface	Breakdown, 1 coverage,	215	
					pneumatic, 1 coverage,	174	
					finish with tandem	138	183
		1-2	Passing	Surface	Breakdown, 1 coverage,	214	
					pneumatic, 3 coverages,	164	
					finish with tandem	142	147
		2	Travel	Surface	Breakdown, 1 coverage,	243	
					finish with tandem	160	179
		2-1	Travel	Surface	Breakdown, 1 coverage,	254	
					pneumatic, 1 coverage,	166	
					finish with tandem	152	136
		2-2	Travel	Surface	Breakdown, 1 coverage,	250	
					pneumatic, 3 coverages,	164	
					finish with tandem	139	77

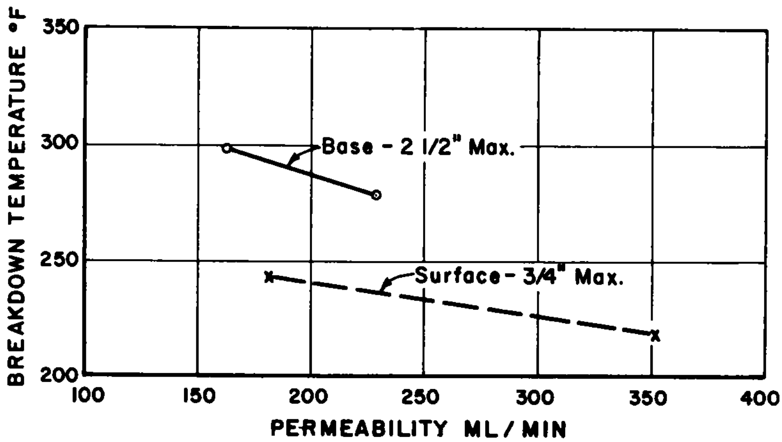


Figure 6. Effect of breakdown temperature on permeability, Project E.

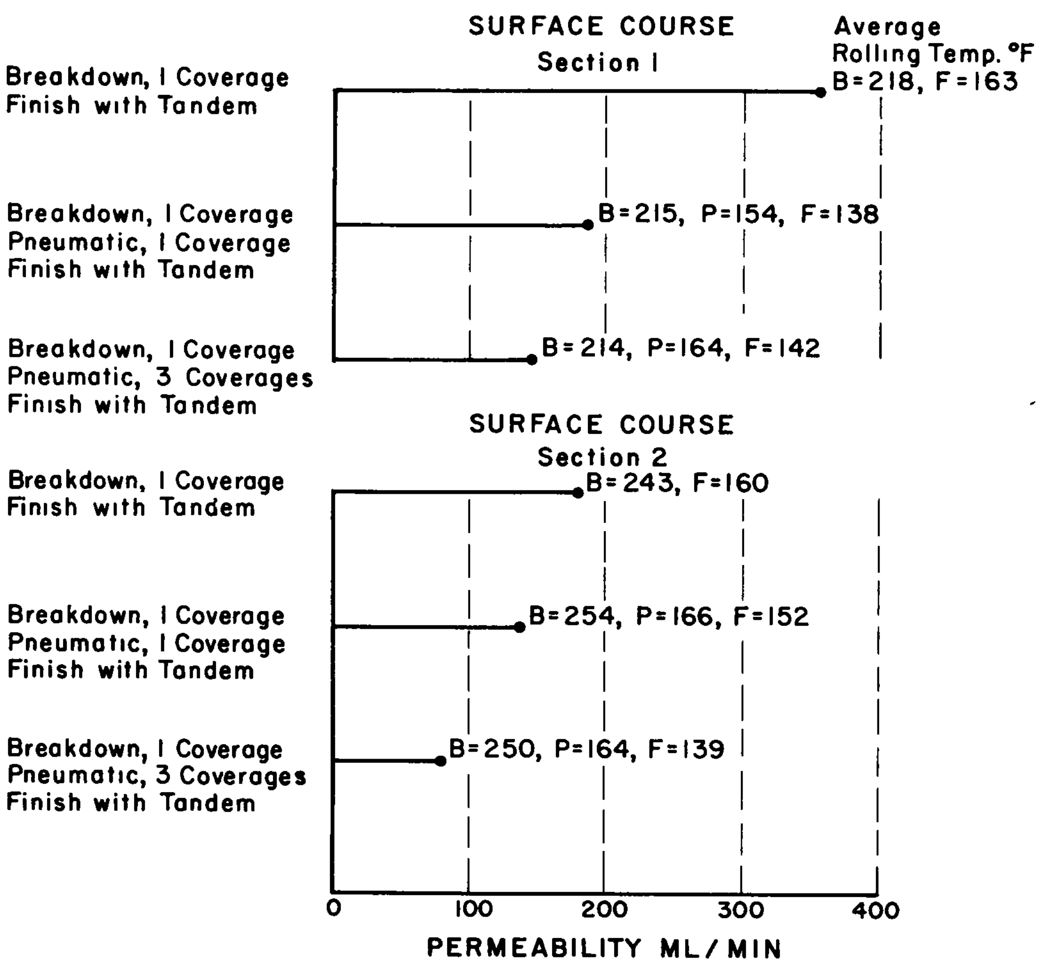


Figure 7. Effect of breakdown temperature on permeability, Project E. B=breakdown rolling; P=pneumatic rolling; and F=finish rolling.

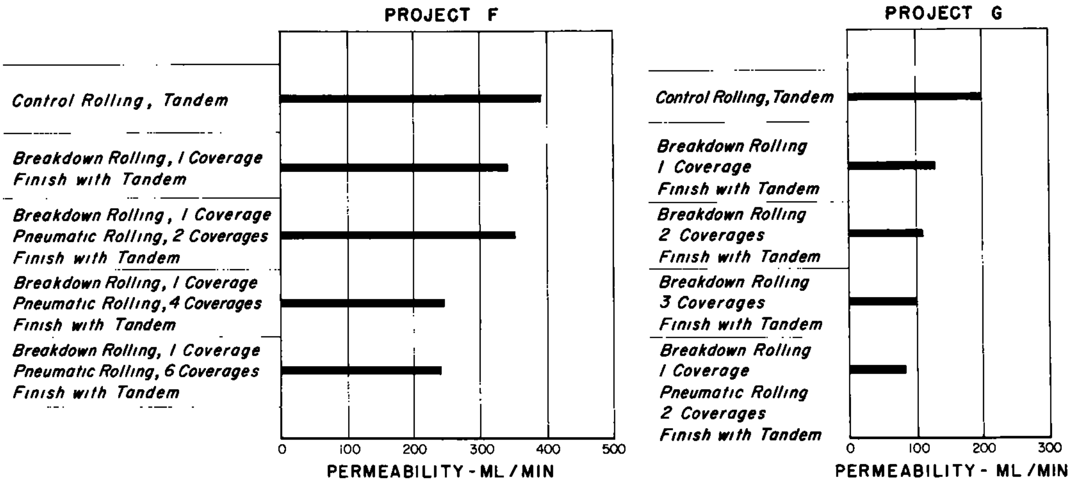


Figure 8. Effect of compaction procedures on permeability values.

Table 3 gives more recent permeability values obtained with varying pneumatic contact pressures and varying breakdown and pneumatic rolling temperatures. It appears that the most benefit from pneumatic rolling is obtained when the contact pressures and temperature of the mix are fairly high and when the permeability after breakdown is in the 100- to 400-ml per min range.

The California Division of Highways has been concerned during the past two paving seasons with the problem of "pick up" and "sticking" of the mix to the tires of the pneumatic roller and has found it necessary on a number of jobs to reduce rolling temperatures in order to avoid this problem and the resulting unsightly appearance of the surface. The addition of small quantities of water-soluble oil to the roller water has somewhat alleviated this condition but it seems imperative, that some means be found for preventing this problem with pneumatic rollers if the maximum benefit is to be attained from this method of compaction.

The workability of the mix and degree of compaction will depend not only on field conditions but also on mix design variables such as asphalt content and aggregate grading. Both will influence the permeability values. The differences between 4.5 and 5.5 percent asphalt on the same grading are shown in Figure 9. Of course, the proper

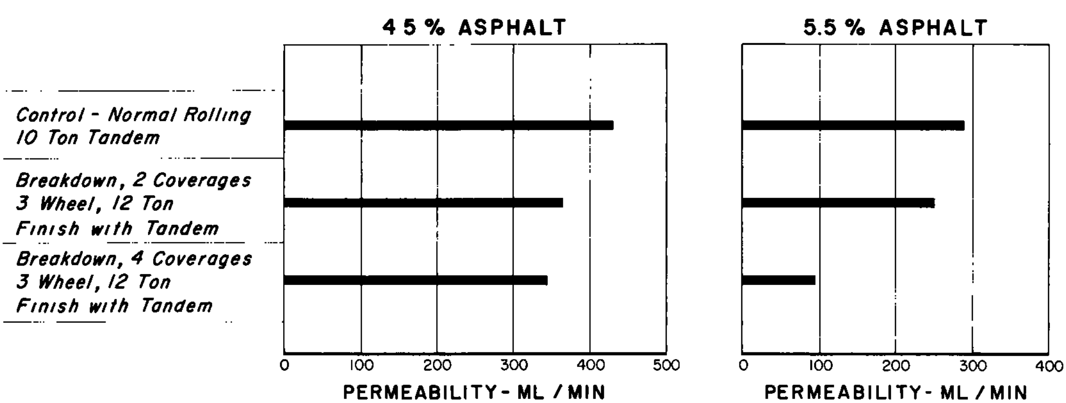


Figure 9. Average permeability values of section containing different asphalt contents, Project N.

TABLE 3

PERMEABILITY VALUES

Project	Section	Rolling Pattern	Temperature (°F) Breakdown	Temperature (°F) Pneumatic	Temperature (°F) Finish	Pneumatic Contact Press.[a] (psi)	Perm. 1-3 Hr After Rolling (ml/min)	Reduction by Pneumatic Rolling (%)	Remarks
H	1	Breakdown + finish	187	--	107	--	156	--	
	2	Breakdown + pneumatic + finish	220	128	115	68	113	28	Breakdown and pneumatic temperatures low, pneumatic contact pressure high
			220	178	115	68	77	51	Breakdown temperature low, pneumatic temperature and contact pressure high
I	1	Breakdown + finish	278	--	140	--	64	--	
	2	Breakdown + pneumatic + finish	263	188	131	62	46	28	Low permeability after breakdown
J	1	Breakdown + finish	253	--	130	--	386	--	
	2	Breakdown + pneumatic + finish	245	163	130	76	143	63	Breakdown and pneumatic temperatures high, pneumatic contact pressure high
K	1	Breakdown + finish	235	--	135	--	628	--	
	2	Breakdown + pneumatic + finish	235	135	135	34	507	19	Breakdown and pneumatic temperature low, pneumatic contact pressure low; permeability high
L	1	Breakdown + finish	250	--	140	--	288	--	
	2	Breakdown + pneumatic + finish	250	143	140	39	228	21	Breakdown temperature high; pneumatic temperature and contact pressure low
M	1	Breakdown + finish	243	--	140	--	149	--	
	2	Breakdown + pneumatic + finish	255	140	115	64	59	60	Breakdown temperature high; pneumatic temperature low; contact pressure high

[a]From Bros chart.

asphalt content must be based on consideration of a number of factors and is limited by the stability requirements. In the case of noncritical mixes the asphalt content is limited by necessary safeguards against possible future "flushing" of excess asphalt to the surface, thus providing a skid hazard.

It is logical to assume that the percentage of voids and the relative compaction should be related to the permeability values immediately or shortly after construction. As pointed out earlier, the permeability is greatly influenced by the number of interconnected passageways in the pavement and these will vary depending on factors involved in design and construction. Further, the "sealing" of the surface by pneumatic rolling and traffic action may markedly reduce the permeability measured after the breakdown pass while not materially reducing the total void content of the pavement.

The relation between void content and permeability was measured on a series of jobs by determining the permeability 24 hr after completion of rolling. Cores were then removed from areas of different permeability values and the density and percentage of relative compaction were determined, with 100 percent relative compaction assigned to a laboratory compacted specimen. Results are shown in Table 4 and Figures 10 and 11. The curve for the void-permeability relation indicates that there is no serious

TABLE 4

FIELD PERMEABILITY, VOID RELATIONSHIP FOR SOME INDIVIDUAL PROJECTS

Project	Type of Mix	Perm. (ml/min)	% Relative Compaction	% Voids
N	Type A-3/4" Max.	200	97.4	8.8
	4.5% Asphalt	230	97.0	9.0
		510	93.5	12 3
	Type A-3/4" Max.	30	98.3	4.5
	5.5% Asphalt	70	97.0	5 8
		250	93.2	9.5
		250	94.5	8.2
		550	92.8	10.1
G	Type A-3/4" Max.	15	98.3	5 0
	5.2% Asphalt	15	99.6	3.5
		35	98.3	5.0
		35	100.0	3.3
		40	98.3	5.0
		50	98.8	4.5
		55	97.4	5.8
		70	97.4	5.8
		80	97.9	5.4
O	Type B-3/4" Max.	150	96.4	9 6
	4.3% Asphalt	175	96.0	10.0
		175	94.2	11.6
		195	93.8	12.1
		210	94.2	11.6
		300	94.2	11.6
H	Type B-3/4" Max.	55	95.0	8.9
	5.0% Asphalt	150	93.3	10 6
		190	93.7	10.2
		265	91.1	12.6
		340	93.4	10.1
		520	90.3	13.4

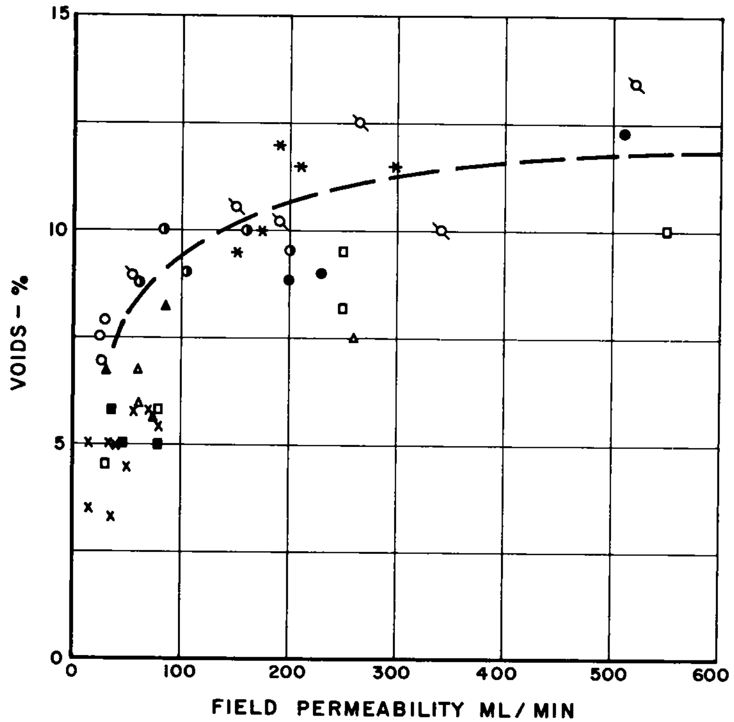


Figure 10. Permeability—voids relation for ten different projects.

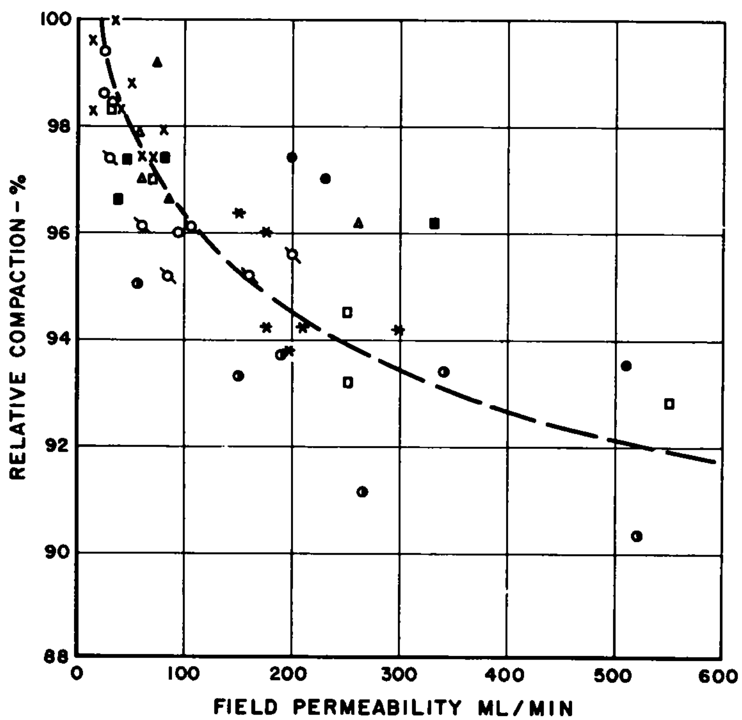


Figure 11. Permeability—relative compaction relation for ten different projects.

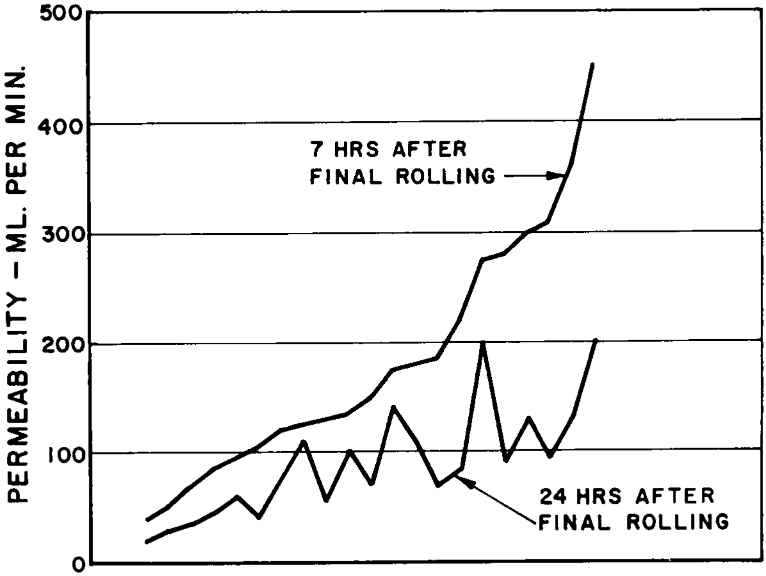


Figure 12. Change in permeability values following final rolling, no traffic, Project G.

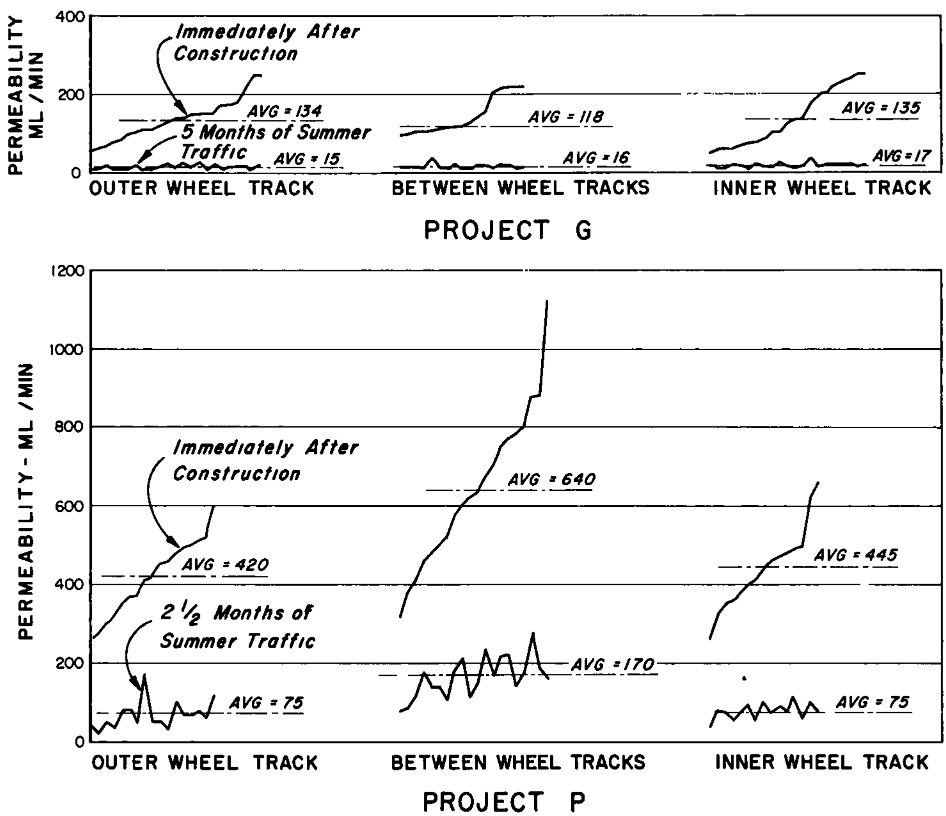


Figure 13. Change in permeability values after summer traffic, travel lane.

change in permeability up to about 10 percent voids. However, even small increases in void-content above this figure show a marked increase in permeability. A similar relationship is found when the percentage of relative compaction, falls below 94 percent (Fig. 11).

As shown later, the permeability of pavements laid during the summer paving season show a marked decrease due to traffic action. This decrease is not accompanied by any pronounced reduction in void content since only the uppermost portion of the surface course is "sealed" by this action. However, pavements laid in the late fall cannot be expected to "seal" before winter rains. The void-permeability curve clearly indicates that excessive water may enter the pavement if compaction procedures during construction are not effective in reducing the void content to a safe level.

There is a reduction in permeability during at least the first 24 hr after completion of rolling (Fig. 12). This is best accounted for on the basis of "cold-flow" of the binder because the test section was not subjected to any traffic. It is reasonable to infer that a number of original interconnected passageways are sealed at different points by the slowly continuing movement and adjustment of the asphalt binder.

The importance of traffic action is shown in Figure 13. This striking reduction in the permeability of all areas of the roadway to a very uniform and low level has been found on a number of jobs paved during the early summer and subjected to traffic during warm weather. In contrast on another job, constructed in December, no reduction was found in the initially high permeability values until the following summer.

During the late fall and winter paving, the lower atmospheric temperatures are a definite handicap in attaining proper compaction. Even elevated mixture temperatures and immediate traffic action will not satisfactorily knead or seal the surface of the pavement to prevent entrance of water. Increasing the mixing temperature may have, in some cases, an immediate effect on the compaction, but at the same time may harden the bituminous binder sufficiently to effect a marked lowering of the service life of the pavement. Table 5 shows permeabilities obtained during paving operations in September and October-November on the same project. The September permeabilities average about 47 ml per min against 371 ml per min for the October-November values.

An interesting illustration of the change in permeability of a pavement laid in the early winter season is shown in Figure 14. This pavement was laid during low atmospheric temperatures and was not subjected to traffic until the following spring. The pavement was laid over a virtually impermeable cement-treated base. In February 1958 after a series of storms an over-all drop in permeability was noted from that found after fog sealing. This was most likely caused by entrance and entrapment of rain water within the pavement and was further confirmed by the gain in permeability values after a period of dry weather. The increase was probably caused by evaporation of pavement moisture. A decrease in permeability values occurred after opening of the pavement to traffic.

TABLE 5

AVERAGE PERMEABILITY VALUES FOR A PAVEMENT CONSTRUCTED DURING CHANGING CLIMATIC CONDITIONS

Paving Date	Atmospheric Temperature Range During Paving Max.	Min.	Permeability (ml/min)
Sept	87	51	47
Oct. Nov.	56	35	371

The Materials and Research Department of the California Division of Highways has consistently maintained that cold and inclement weather is the most adverse factor affecting success or failure during the placing of any type of bituminous pavement or seal coat.

Based on California weather conditions, particularly in the northern part of the State, the following tentative schedule has been suggested for placing bituminous pavements or seal coats:

1. Seal coat construction using emulsified asphalts should be terminated by September 15.
2. Seal coat construction using cutback

asphalts of the rapid curing type may be extended until October 15.

3. Asphaltic concrete mixes, both open and dense graded may be placed until December 1, although a November 15 deadline would be preferable.

The studies have shown that the normal fog seal (application of 0.05- to 0.10-gal per sq yd mixing emulsion diluted one to one with water, on completion of paving operations) will only be effective in sealing a pavement if the original permeability is fairly low. Figure 15 shows the reduction in permeability by the application of a fog seal. (Readings were obtained before and after fog sealing at identical spots.) The permeability after fog sealing tends to parallel the original curve. It is logical to assume that passageways with relatively large diameters will not be sealed by the application of a small amount of asphalt; therefore, in areas of high permeability no real improvement will be noted.

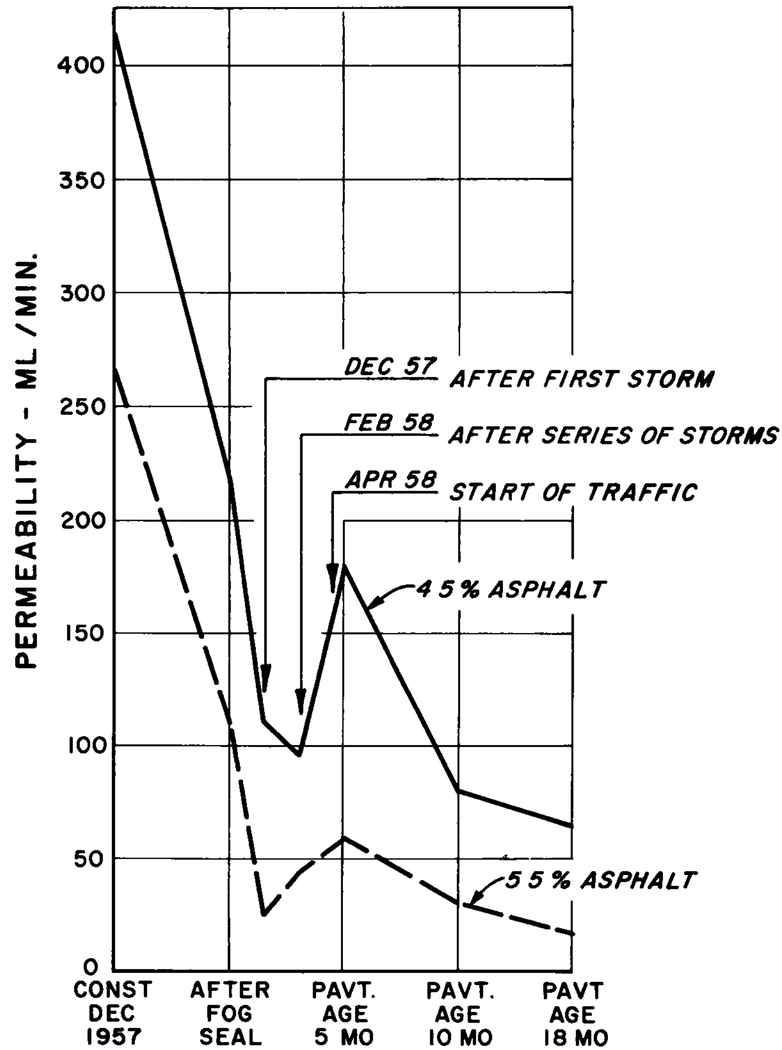


Figure 14. Change in permeability values caused by increase and decrease of pavement moisture and traffic compaction, Project N.

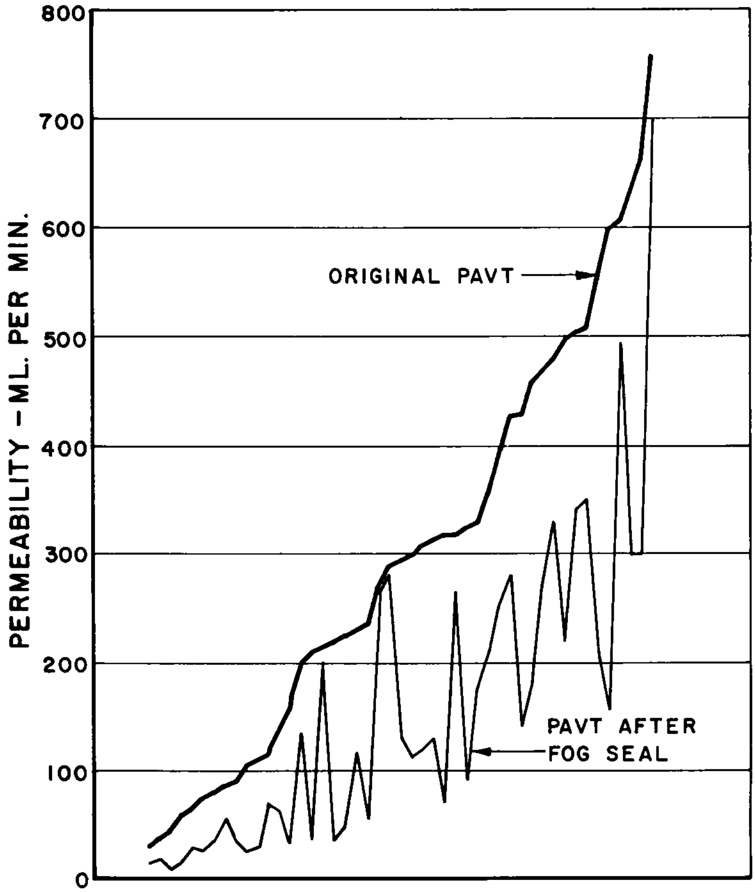


Figure 15. Change in permeability values after application of fog seal, Project N.

On the other hand, a slurry seal or screening seal coat reduces the water permeability sufficiently and virtually renders the pavement impermeable (see Table 6 and Fig. 16). Present data indicate that such seals completely prevent the entrance of surface moisture. Unfortunately, it is very difficult to attain a satisfactory job with either of these types of seal coats during cold or rainy weather although during this paving period newly laid pavements are most in need of some form of sealing.

TENTATIVE LIMITS FOR PERMEABILITY

A tentative average water permeability value not exceeding 150 ml per min for a 6-in. diameter area will be low enough to prevent the entrance of excessive moisture into the pavement from the surface. On the basis of the studies it is concluded that it is not feasible or even advisable in all cases to attempt construction of a completely impermeable asphalt pavement because to do so would in many instances require sacrificing other qualities of importance equal to the water problem. The objective is to reduce the potential for water infiltration to a minimum through properly designed mixes and practical construction methods. The permeability test is a useful tool in attaining this desirable end result.

The figure of 150 ml per min is a relative test figure only and indicates the ability of the newly compacted or existing pavement to accept water. Once the voids in the pavement are filled with water the amount of any additional water admitted depends on the permeability or porosity of the base material.

TABLE 6

REDUCTION IN PERMEABILITY VALUES FOLLOWING APPLICATION OF SLURRY SEAL, PROJECT Q

Test Condition	Station	New Pvt. Thickness	Permeability (ml/min)				
			Travel			Passing	
			Shldr.	O W. T.	B. W T.	B W. T.	O W T.
New Pvt. immediately after const Nov. 1957	603+00	2"	310	70	110	270	--
	603+25		--	100	130	140	--
	604+00		260	90	90	320	--
	606+50		230	90	100	270	--
	608+00		400	270	340	320	--
	Avg.		300	124	154	264	--
	589+00	3"	--	--	--	700	--
	590+00		--	--	500	480	--
	591+00		--	--	320	440	--
	591+89		--	--	400	500	--
	592+00		--	--	420	490	--
	593+00		--	--	360	--	--
	593+50		--	--	--	500	--
	594+00		--	--	360	350	--
	Avg.		--	--	393	494	--
	596+00	4"	--	--	250	--	--
	597+00		--	--	230	--	--
	598+00		--	--	500	--	--
	599+00		--	--	350	--	--
	600+00		--	--	750	--	--
	601+00		--	--	440	--	--
	Avg.		--	--	420	--	--
New Pvt plus Slurry Seal immediately after completion Nov 1957	604+00	2"	--	10	10	10	--
	604+25		--	10	10	10	--
	606+00		--	10	10	10	--
	606+50		--	10	10	10	--
	Avg.		--	10	10	10	--
	593+50	3"	--	10	10	10	--
	593+75		--	10	10	10	--
	Avg.		--	10	10	10	--
	598+00	4"	--	10	10	10	--
	598+25		--	10	10	10	--
	Avg.		--	10	10	10	--
New Pvt plus Slurry Seal, Feb. 27, 1958 18" rain	604+00	2"	--	--	--	--	--
	604+25		--	--	--	--	--
	606+00		--	10	10	20	10
	606+50		--	5	10	15	10
	Avg.		--	7.5	10	17.5	10
	593+50	3"	--	10	10	20	15
	593+75		--	15	10	10	15
	Avg		--	12 5	10	15	15
	598+00	4"	--	10	20	20	15
	598+25		--	10	10	10	15
	Avg.		--	10	15	15	15
New Pvt plus Slurry Seal May 28, 1958	593+50	3"	--	10	10	10	10
	593+75		--	10	15	15	10
	Avg.		--	10	12.5	12.5	10
	606+50	2"	--	10	10	10	--
	606+75		--	5	10	15	--
	Avg.		--	7.5	10	12.5	--

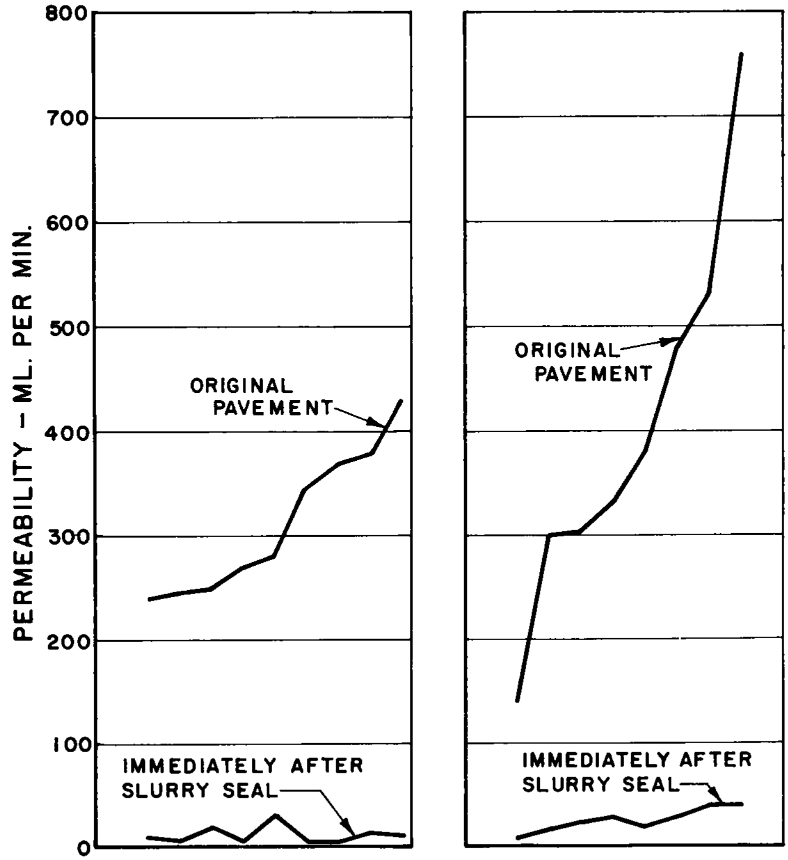


Figure 16. Reduction in permeability values following application of slurry seal coat, Project Q.

CONCLUSIONS

A simple and rapid test method for measuring the tendency of surface water to enter an asphaltic pavement has been developed. This test can be used during actual construction to give an indication as to the effectiveness of compaction operations.

The results of field studies clearly indicate that pavements, even of the so-called dense-graded mixtures, have been constructed that are quite permeable to the entrance of surface water. This water may contribute to possible failure of the pavement by acting as the agent for transporting base dust and clay fines into the interstices of the pavement mixture, and this action may contribute to the rapid hardening of the binder, especially in the lower part of the pavement.

Field tests indicate that adequate compaction, together with some form of pneumatic rolling, are very important factors in reducing pavement permeability. Also, permeability may continue to decrease immediately after construction and will definitely decrease for pavements laid during the normal paving season when subjected to traffic during the summer months. On the other hand, pavements laid during the late fall or winter must rely on adequate initial compaction because no further decrease in permeability may be expected before the following summer. Bituminous pavements or seal coats should not be placed in the late fall or during the winter months.

Fog seals will decrease the permeability but will not prove effective if the initial permeability is very high. Slurry seals and screening seal coats effectively reduce the permeability value to a very low figure.

Some of the early studies involving relatively permeable surfaces were conducted on pavements constructed under the 1954 California standard specifications. As the result of these studies the 1960 standard specifications carry more rigid requirements for temperature control and additional compaction equipment.

The 1960 and 1961 studies on a considerable number of projects show a marked decrease in permeability values and void content of the mix. This of course, should provide better durability for the bituminous binder with a resulting longer service life for the pavement.

ACKNOWLEDGMENTS

The investigations described herein were conducted under the general direction of F. N. Hveem, Materials and Research Engineer, California Division of Highways.

The writer wishes to acknowledge the help and full cooperation of the many resident engineers on whose jobs these investigations were carried out. Special acknowledgment is due John Skog, Merle Nelson, Glenn Kemp, and Rufus Hammond, who were active in the field and collected much of the data.

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Appendix A

Test Method No. Calif. 341-A
January, 1960
(6 pages)

State of California
Department of Public Works
Division of Highways

MATERIALS AND RESEARCH DEPARTMENT

METHOD OF TEST FOR MEASURING THE PERMEABILITY OF BITUMINOUS PAVEMENTS

Scope:

This method describes the procedure for determining the relative permeability of bituminous pavements.

Procedure:

A. Apparatus

1. 1 - 250 Ml special plastic graduated cylinder with valve.
2. 1 - 500 Ml special plastic graduated cylinder with valve.
3. 1 - 250 Ml Polyethylene graduated cylinder.
4. 1 - 500 Ml Polyethylene graduated cylinder.
5. 1 - Calking gun with a 5 inch piece of ¼-inch copper tubing with cap.
6. 1 - gal. Polyethylene bottle with handle and pouring spout.
7. 2 - 1 gal. friction top cans, one containing medium weight chassis grease.
8. 1 - Polyethylene funnel 4½-inch diameter top.
9. 1 - Stop watch with 60 second dial.
10. 1 - Aluminum template, 6-inch diameter with handle.
11. 2 - pieces yellow lumber crayon.
12. 1 - 6 foot folding Zig Zag Ruler.
13. 1 - 5-inch trowel.
14. 1 - 8-inch spatula.
15. 1 - 100 Ml glass graduated cylinder.
16. 1 - 1 qt. Polyethylene bottle for Aerosol Concentrate.
17. 1 - 5 gal. Polyethylene Carboy container for storage of test solution.
    The above items are furnished in a kit box.
18. 1 - 2 pound hammer.

19. 1 - 1 inch wide steel chisel.

20. 1 - Face shield.

B. Materials

1. Medium weight chassis grease. One gallon is furnished with kit.

2. Wetting agent known as Aerosol OT 75% liquid. 1 quart is furnished with kit.

3. Supply of distilled water.

4. Supply of Premix Patching Material.

C. Preparation of Test Solution

Prepare test solution by mixing 95 ml. of Aerosol OT 75% liquid per 5 gallons of distilled water.

D. Method for Filling Calking Gun with Grease

1. Remove front cover of calking gun by turning counter clockwise.

2. Turn handle at rear of calking gun one half turn so that notched teeth on the rod are in an upward position and pull handle all the way out.

3. Fill gun with grease by using spatula and work as many air bubbles out of the grease as possible with the spatula.

4. Replace front cover and turn rear handle so that the notched teeth are in a downward position.

5. Pump calking gun handle until grease extrudes from copper tubing.

6. Always store calking gun in test kit with notches in an upward position and cap on copper tubing tip; this will prevent grease from being extruded from the gun during storage.

E. Test Procedure

The procedure for Dense Graded Asphalt Concrete Pavements and Various Types of Seal Coats is as follows:

1. With the crayon and template, draw a 6 inch diameter circle on the pavement.

2. Extrude grease from the calking gun on the circle. The diameter of the grease on the ring should be about 1/4 of an inch; see Figure I.

3. Run the finger around the outside edge of the grease ring, pushing a small amount of grease into the pavement. This will form a sealed reservoir for the test solution.

4. Fill the special plastic graduated cylinder and Polyethylene graduated cylinder with the test solution. The Polyethylene cylinder is used for refilling the special cylinder when more solution is needed during test.

   Note: In areas where the permeability of the pavement is below 250 ml/min. the 250 ml. graduated cylinders shall be used.

The 500 ml. graduated cylinders are used in areas where the permeability of the pavement is greater than 250 ml./min.

5. Release valve at base of special plastic graduated cylinder, start stop watch and run solution from the special plastic graduated cylinder onto the area within the the grease ring, keeping this area covered constantly with the solution for two minutes; see Figure II. Refill the special plastic cylinder from the polyethylene graduate if more solution is needed during test.

   Note: At the end of the test the pavement in the grease ring should have an unflooded wet appearance.

6. At the end of the 2 minute test period, determine the total amount of solution used.

7. Pick up grease with trowel and place in gallon can. Do not mix used grease with the new grease furnished with kit.

The procedure for pavements surfaced with Open Graded Asphalt Concrete is as follows:

1. With the crayon and template, draw a 6 inch diameter circle on the pavement.

2. Put on a face shield.

3. Use the hammer and chisel to chip away the open graded surfacing from around the 6 inch diameter circle forming a trough around the permeability test area; see Figure III. The trough around the ring shall be about 1 inch wide and shall extend into the dense graded portion of the pavement about $\frac{1}{4}$-inch.

4. Use trowel to fill trough with chassis grease. The grease shall extend above the surface of the test area about $\frac{1}{4}$-inch; see Figure IV. This will form a sealed reservoir for the test solution.

5. The test is then run in the normal manner as previously described; see Figure V.

6. After test is completed remove grease, fill trough with Premix patching material and compact with hammer; see Figure VI.

F. Calculations

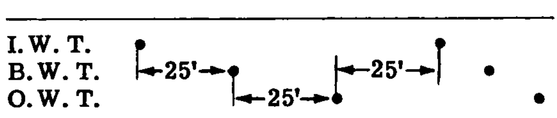
Divide the total quantity of solution used during the test period by two and record the relative permeability in mls/min.

G. Hazards

1. The operator should always wear a suitable face shield when chipping open graded mix in preparation for the test.

H. Tentative Procedure for New Pavements

1. The following tentative procedure is recommended for obtaining an average permeability result on a given section of new pavement. In any travel lane, determine the permeability at 25 foot intervals in the outer wheel track, inner wheel track and between the wheel tracks for a total of six readings. A diagram is shown below:



The six readings should be averaged to obtain the reading for the test area. This procedure should then be repeated at intervals of approximately 1000 feet.

2. In mountainous areas the above noted plan may have to be modified in order to provide a relatively flat area for testing.

3. When permeability studies are required after traffic action, it is advisable to test the passing lane in order to obtain the best indication of the relative permeability of the pavement.

4. When the test is performed on an open graded surface, about 30 ml./min. of solution will be held by the open graded mix, even though no solution is entering the dense graded mixture. Therefore, a reading in this range would not be indicative of any movement into the dense graded pavement.

REFERENCE
A California Method

End of Text on Calif. 341-A

PROCEDURE FOR DENSE GRADED ASPHALT CONCRETE PAVEMENTS AND VARIOUS TYPES OF SEAL COATS

Figure I. Forming grease ring.

Figure II. Applying test solution to pavement surface.

PROCEDURE FOR PAVEMENTS SURFACED WITH AN OPEN GRADED MIXTURE

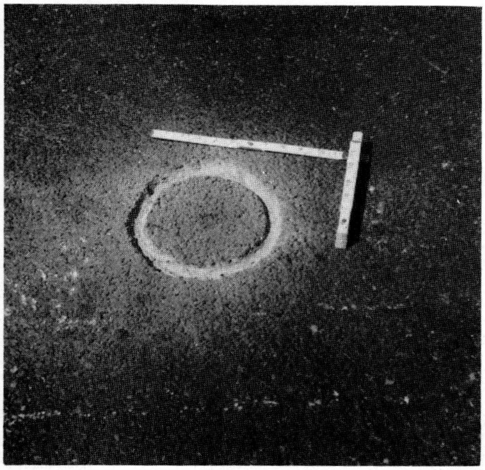


Figure III. Trough formed by removal of open graded mix. Note: Intact open graded mix within 6 in. diameter test area.

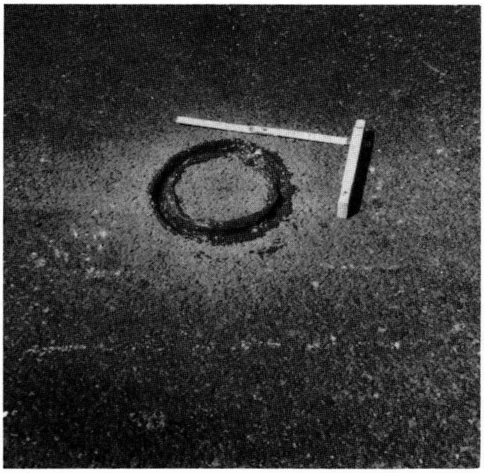


Figure IV. Grease ring formed around test area.

Figure V. Applying test solution to open graded surface within 6 in. diameter test area.

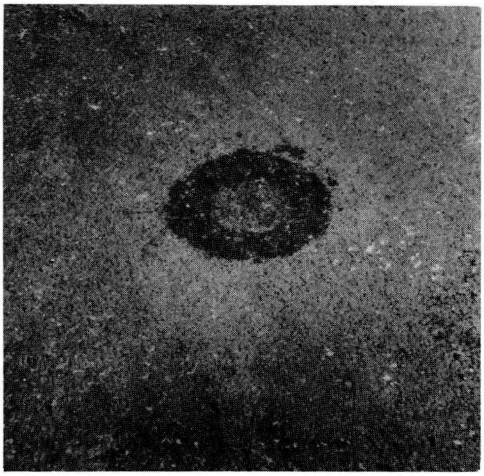


Figure VI. Trough area filled with Premix patching material.

Appendix B

Compaction Requirements for Asphaltic Concrete
Excerpts from California Standard
Specifications, January 1960

The Contractor will be required to furnish a minimum of one 12 ton 3-wheel roller or tandem roller, one pneumatic-tired roller, and one 8 ton 2-axle tandem roller for each asphalt paver. All mixtures, except open graded mixture, shall be spread at a temperature of not less than 225°F. and all initial rolling or tamping shall be performed when the temperature of the mixture is such that the sum of the air temperature plus the temperature of the mixture is between 300°F. and 375°F. Initial or breakdown rolling shall consist of one complete coverage of asphalt mixtures and shall be performed with a tandem or a 3-wheel roller. Such rollers shall weight not less than 12 tons.

Rolling shall commence at the lower edge and shall progress toward the highest portion. Under no circumstances shall the center be rolled first. Rolling shall be performed with the drive wheel of the tandem roller forward with respect to the direction of spreading operations, unless otherwise permitted.

The initial or breakdown rolling shall be followed by additional rolling consisting of 3 complete coverages with a pneumatic-tired roller while the temperature of the mixture is at or above 150°F.

The final rolling of the uppermost layer of asphalt concrete or the top of the layer immediately below a layer of Open Graded asphalt concrete shall be performed with an 8-ton 2-axle tandem roller.

Field Compaction Studies on Asphaltic Concrete

W. GARTNER, JR., D.A. COBB, and R.W. LINDLEY, JR., respectively, Assistant Engineer of Research and In-Service Training, Assistant Research Engineer, and Civil Engineer Trainee III, State Road Department of Florida

This paper presents the results obtained from a project undertaken to evaluate the effect of varying the compactive effort of the intermediate rolling on asphaltic concrete. Both pneumatic-tired and steel-wheel vibratory rollers were used on a total of 26 test section.

Results of the tests made with the pneumatic-tired intermediate roller indicated that maximum compaction is attained with six coverages, and a slight tendency for the material to decompact with additional coverages was noted. Results from the three sections in which no intermediate rolling was used showed that the average density attained exceeded minimum specification requirements. When the intermediate rolling was omitted but two extra coverages of the final steel-wheel rolling were applied instead, the average density attained was nearly as great as the average maximum density attained with the optimum number of coverages.

The results of permeability tests indicate no detrimental effect resulting from omission of the intermediate rolling. The results obtained with the vibratory compactor were inconsistent and due to the lack of replication no evaluation of these results was attempted.

• THE PROBLEMS of pushing or shoving, and wheelpath rutting in asphaltic concrete pavements have been the subject of many studies by asphalt paving technologists and highway engineers in general. In any study undertaken with the objective of minimizing the amount of this type of deformation, all elements or variables connected with the construction of asphaltic concrete pavement must be considered. Some of the variables encountered include design mix, mix temperature, compaction temperature, and the compactive effort. A considerable amount of work has been reported on studies involving the design of the mix and the importance of temperature control. Lately, additional emphasis has been given to field studies of the effect of varying the compactive effort applied.

This latter variable has been investigated by Louisiana (1) and Ohio (2) highway departments among others. Their results indicate asphaltic concrete tends to reach a point of maximum density under optimum compactive effort, with any additional compaction tending to reduce the density or "decompact" the material. Louisiana found that this optimum compaction occurred with six to eight coverages of the same type of rollers used in Florida. After a thorough study of these results it was decided that the Florida State Road Department would benefit from a similar study. Recent literature also points out the need for using higher tire pressures than is the current practice and recommends pressures closer to the contact pressure expected from the more common type of truck tires (3). This seems reasonable and presents a further area of study beneficial to the Department.

Several manufacturers of compaction equipment have recently advocated the use of vibratory rollers for use on asphaltic concrete surface courses. It was though desirable to study the effectiveness of this type of roller as well as conventional rollers.

To accomplish these objectives the Division of Research and In-Service Training undertook an investigation of the effect of varying the amount of intermediate rolling with self-propelled pneumatic-tired and self-propelled vibratory rollers on the pavement density of asphaltic concrete.

PURPOSE

The objectives of this project were to investigate the following effects on the pavement density of Florida's type I asphaltic concrete surface course:

1. Varying the number of coverages with a pneumatic-tired roller meeting current Florida specification requirements.
2. Varying contact pressure exerted by pneumatic-tired rollers on the pavement.
3. Varying the number of coverages with one type of vibratory roller.
4. Ascertaining traffic for the period of one year following construction.
5. From the preceding studies, determining the optimum compactive effort required to produce a maximum density for this type of mix.

It was assumed that maximum density would produce the maximum stability of the asphaltic concrete mix. This was to be determined by studying the amount of distortion or additional compaction incurred by one year of traffic.

The study included a total of 26 test sections from which six density samples each were taken, two in each wheelpath and two inbetween the wheelpaths. One density sample in each location was used to study the construction compactive effort, the remaining sample will be used to study the effect of traffic.

The test sites were on a type I asphaltic concrete pavement located on Interstate 4, Project 92130-3402, just south of its intersection with Florida 530 fifteen to twenty miles West of Kissimmee, Florida. Construction of the test sections was started on January 24, 1961, and completed February 2, 1961. The test sections were located in both the traffic and passing lanes of the eastbound roadway on straight sections of the road, well clear of intersections. The design thickness of the surface course was 1.5 in. overlaying an open type of binder course with an average thickness of 3 in.

PROCEDURE

The equipment used for the test site construction was typical of that normally used for construction of asphaltic concrete pavements and included a portable continuous mix type of asphalt plant with two driers in series, a Barber-Greene model SA-40 paver, a Buffalo 5-ton tandem steel-wheel breakdown roller, three intermediate rollers (Bros SP-730B pneumatic-tired, Bros SP-54B pneumatic-tired, and Bros SP-54B rigged with two vibra pactors), and a Buffalo 9-ton tandem steel-wheel finish roller.

The equipment was all thoroughly checked for proper operation and adherence to Florida State Road Department specifications where applicable. Paving operations were stabilized by allowing several days of paving before the commencement of test site construction.

The 26 test sections were laid out before paving according to the diagrams in Figures 1 and 2. Each test section was constructed with material from a single truck load of asphalt mix, with adequate allowance made for the transition from the previous material in the spreader. For easy removal of the density samples, a double layer of aluminum foil was placed on the binder course on the spot from which the density samples were cut later (Fig. 3). This method was found to eliminate most of the troubles encountered in separating the surface from the binder course.

Each morning the asphalt plant was allowed to operate for a short time and checked for uniformity of production. The asphalt mix temperature was checked as it arrived

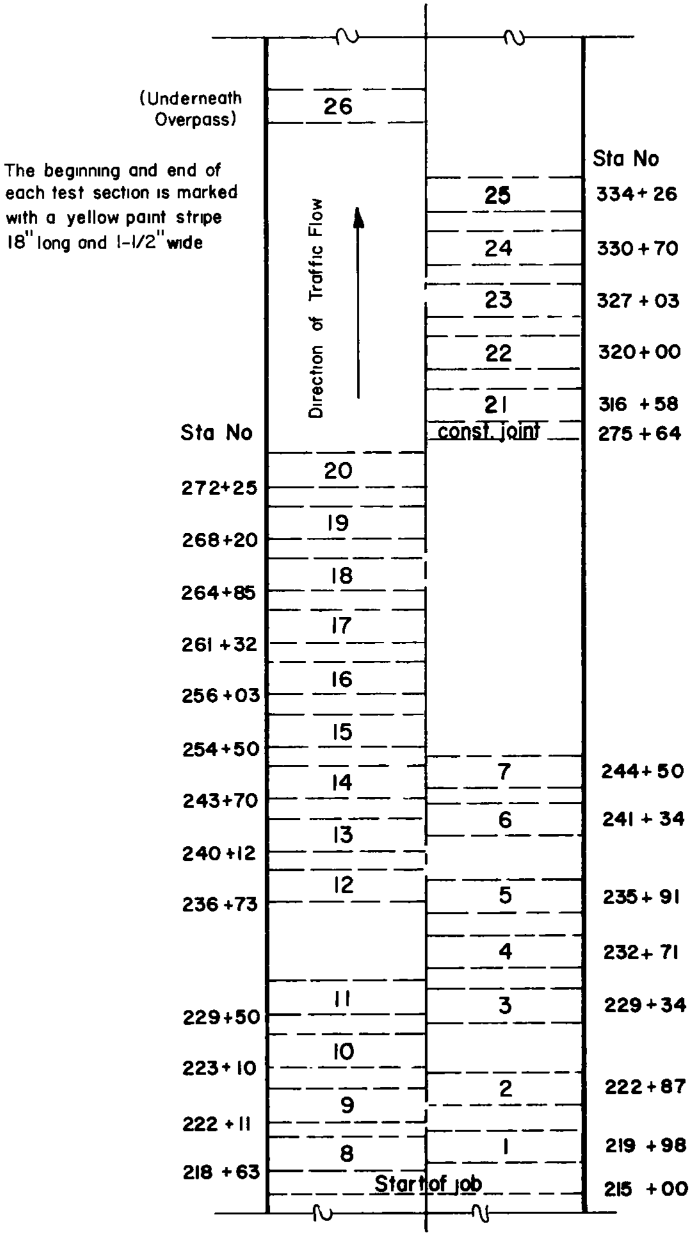


Figure 1. Location of test section.

at the spreader and only those truck loads with temperature in the range of 300 to 330 F were used in the construction of the test sections. As the mix was being spread a sample representative of the material in each experimental test section was obtained by continuous sampling of the load during the entire operation of unloading. These samples, consisting of approximately 50 lb of material packed in cardboard boxes, were taken to the laboratory and the material checked for uniformity using the following tests:

1. Density using Hubbard field procedure AASHO T-169, ASTM D-1138.
2. Density using Marshall procedure ASTM D-1559.

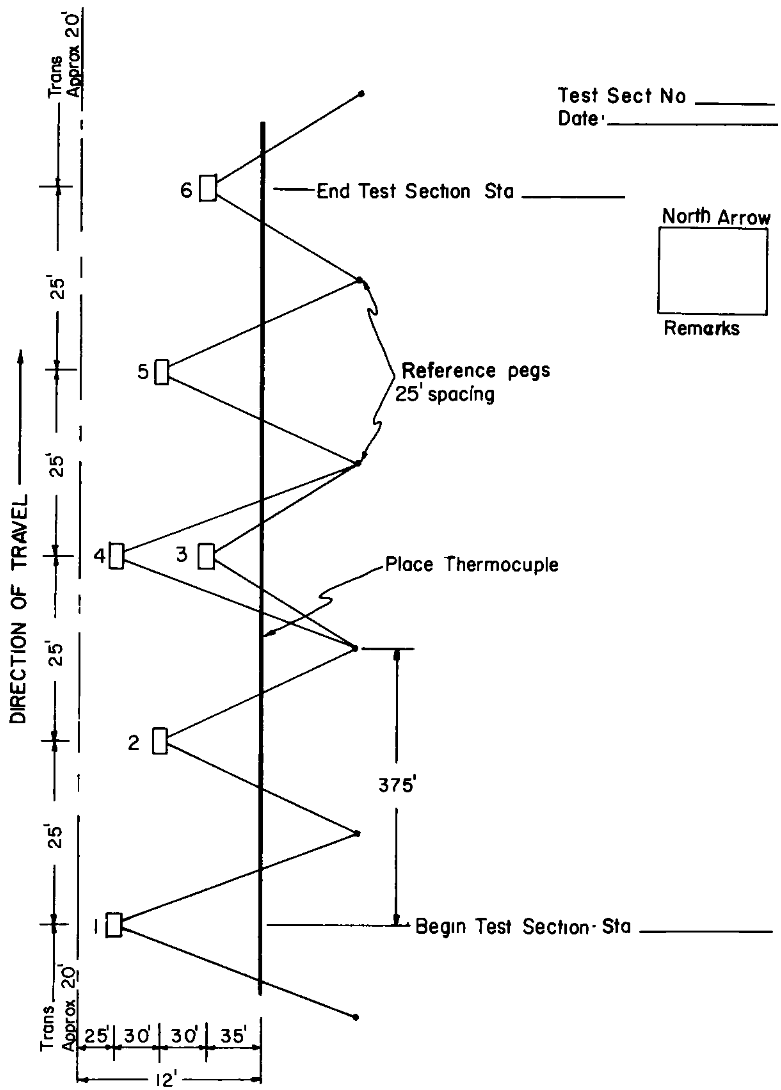


Figure 2. Layout of test section.

3. Asphalt content AASHO T-164, ASTM D-1097.
4. Gradation AASHO T-30.

Immediately after the mix was spread, a thermocouple was inserted midway in the layer of asphalt mix and the temperature recorded (Fig. 4). This entire operation was usually accomplished within 1 min and additional temperature readings were made at 3-min intervals thereafter, until finish rolling had been accomplished. The time of the initial and final pass of each type of rolling was also recorded on the same chart as the temperature records, so that the two could be related.

The procedures for initial or breakdown, and final rolling were conducted according to existing Florida specifications except that no finish roller was used in the sections

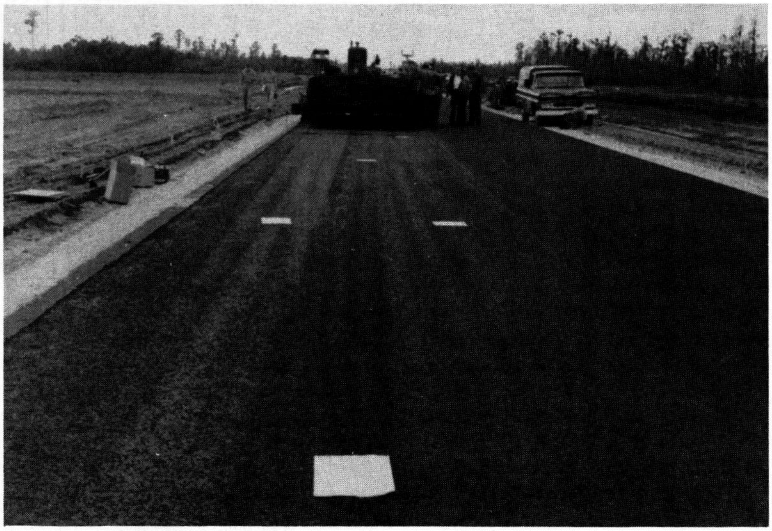


Figure 3. Location of aluminum foil under areas to be used as density samples.

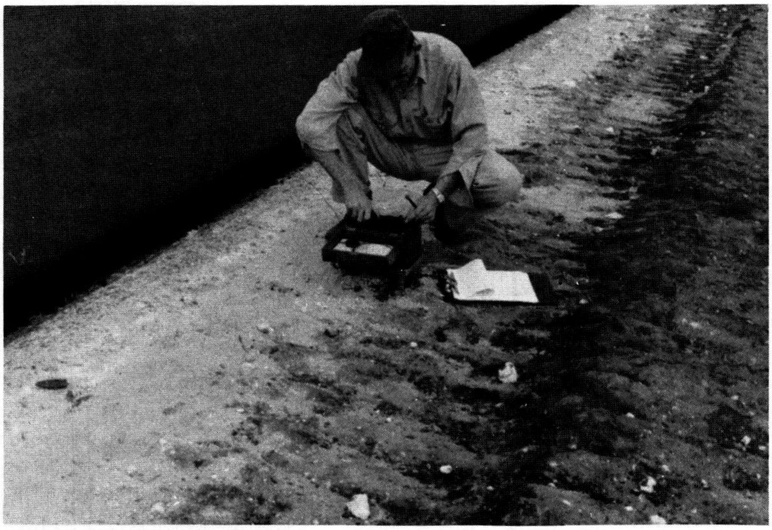


Figure 4. Thermocouple reading via a potentiometer calibrated to temperature in degrees fahrenheit directly.

with vibratory intermediate rolling and the temperature for finish rolling was between 140 and 130 F instead of 150+F. The existing temperature specifications for intermediate rolling were found to be unrealistic, as even with the lowest contact pressure used in the test, (40 psi) it was impossible to roll the asphalt mix at a temperature higher than 150 F without excessive displacement and pickup of the material. At the higher contact pressures (70 psi) an even lower temperature (140 F) was required to prevent this phenomenon from occurring. In an effort to acquire more uniform results each type of rolling was started at approximately the same temperature for each test section. The temperatures used are given in Table 1.

These temperatures were the maximum at which the mix was stable enough to support the rollers without excessive displacement or pickup occurring (Fig. 5).

Turning, parking, and the reversing of directions of all rollers was conducted on areas well clear of the test sections.

In the sections in which the effects of varying the compactive effort imparted by the self-propelled pneumatic-tired rollers (Fig. 6) were studied, the contact pressure and the number of coverages were varied as given in Table 2. Except for Section 16, where five extra passes of the final 8-ton steel-wheel roller were applied, initial and final rolling procedures were kept constant in all of the sections.

TABLE 1

TEMPERATURES USED FOR TEST SECTIONS

Type	Temperature (°F)
Initial	210
Intermediate:	
40 psi	150
50 psi	150
70 psi	140
Vibratory	150
Finish	140-130

In the sections in which the vibratory roller (Fig. 7) was evaluated, the vibratory roller was substituted for both the intermediate and final rollers. Sections were constructed with two, three, and four coverages with this equipment. Only one replicate section (with three coverages) was conducted as there was considerable trouble getting the equipment to function which caused extended delays in the paving operations.

On completion of the construction, the sections were surveyed and marked for future reference. The survey included measurement of the transverse profile by visually checking with a 10-ft straightedge and measurement of transverse displacement of the pavement incurred during rolling operations. This was accomplished by locating pins near the edge of the pavement in the mix immediately after spreading and checking the amount of transverse displacement during rolling operations.

No traffic was permitted on the test sections until the density samples were obtained on the day following construction. The density samples were removed from the pavement by sawing them out with a portable power saw using a carborundum blade. These samples were marked, individually boxed, and stored in such a manner as to

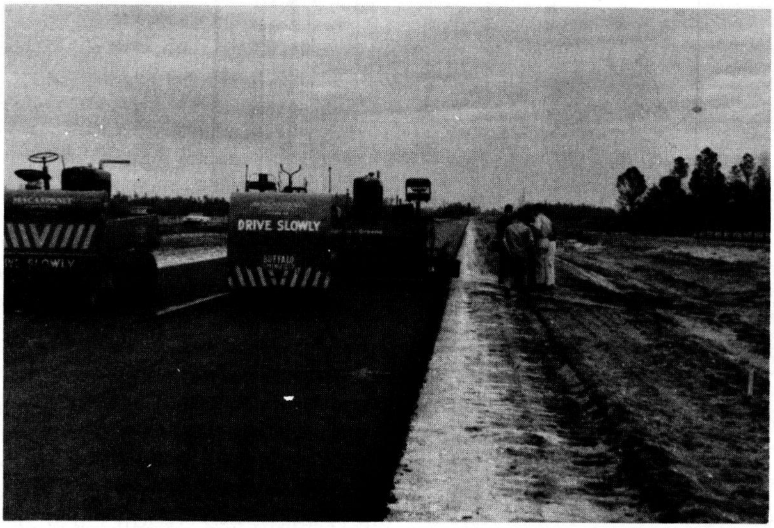


Figure 5. Barber Greene spreader, 5-ton tandem wheel roller, and Bros SP-54B pneumatic-tired roller parked between test sections during break in construction operations.

Figure 6. Bros SP-730B pneumatic-tired roller in operation.

Figure 7. Vibratory roller in operation.

TABLE 2

COMPACTIVE EFFORT APPLIED IN EACH TEST SECTION

No. of Coverages	Test Section[a] 0 Psi	40 Psi	50 Psi	70 Psi
0	5, 11, 16[b]			
4		4, 9		
6		3, 8, 18	13, 15	20, 25
8		2, 7		
10		1, 6, 10, 7	12, 14	24, 26

[a]At four contact pressures.
[b]Five extra passes of final 8-ton steel-wheel roller applied in Section 16.

prevent any damage while being transported to the laboratory. In the laboratory the samples were measured, weighed, waxed, and weighed in water at room temperature and the specific gravity of each computed.

Additional field tests conducted on each test section included tests for skid resistance using the Tapley decelerometer in accordance with procedure described in Bulletin 29, "Skid Characteristics of Florida Pavements" and permeability tests as described in Bulletin 11 "Procedure

TABLE 3

SUMMARY OF FIELD DATA

Section Number	No. of Coverages	Tire Contact Pressure (psi)	Field Density (% lab avg. of 3)	Avg. Thickness (in.)	As Mixed	As Placed	Initial Rolling		Intermediate Rolling		Final Rolling	
							Begin	End	Begin	End	Begin	End
1	10	40	97.6	1.8	305	300	247	213	148	132	132	128
2	8	40	96.8	1 8	335	290	220	210	151	133	129	120
3	6	40	97.1	1.65	305	290	210	190	150	142	130	124
4	4	40	97.1	1.7	305	269	204	195	150	142	132	127
5	0	0	95.9	1.6	305	252	202	169			132	132
6	10	40	96.5	1.65	320	265	207	175	150	134	132	125
7	8	40	96.9	1.65	307	266	202	166	146	136	132	122
8	6	40	97.6	1.6	310	284	204	182	150	144	130	121
9	4	40	96.4	1.45	300	285	208	189	148	Thermocouple failed		
10	10	40H	96.3	1.575	335	290	207	190	142	130	130	122
11	0		94.7	1.5	285	266	201	194			144	133
12	10	50	96.4	1.55	300	275	210	191	147	130	130	126
13	6	50	96.2	1.5	305	283	205	191	140	128	128	122
14	10	50	96.7	1.6	290	270	207	194	148	126	126	122
15	6	50	97.3	1.6	310	285	211	200	151	134	129	125
16	0[a]	0[a]	96.9	1.6	330	285	208	194			150	137
17	10	40	96.0	1.7	320	285	210	198	150	124	120	114
18	6	40	97.1	1.55	295	272	210	193	145	130	127	121
19	3	V	96.4	1 45	300	272	210	198			174	139
20	6	70	96.9[b]	1.575	290	270	208	196	140	125	123	117
21	4	V[c]	96.2	1.6	315	298	213	184			150	139
22	2	V	95.9	1.6	315	298	205	183			149	144
23	3	V	94.6	1.6	305	288	194	177			150	141
24	10	70	96.2	1.6	305	290	201	176	140	117	115	111
25	6	70	97.3	1.6	305	280	216	200	142	132	126	122
26	10	70	96.4[b]	1.5	300	267	209	188	140	131	128	126

[a] Five extra passes of final 8-ton steel-wheel roller applied on Section 16.

[b] Average of two.

[c] Rear vibratory motor quit—third pass only.

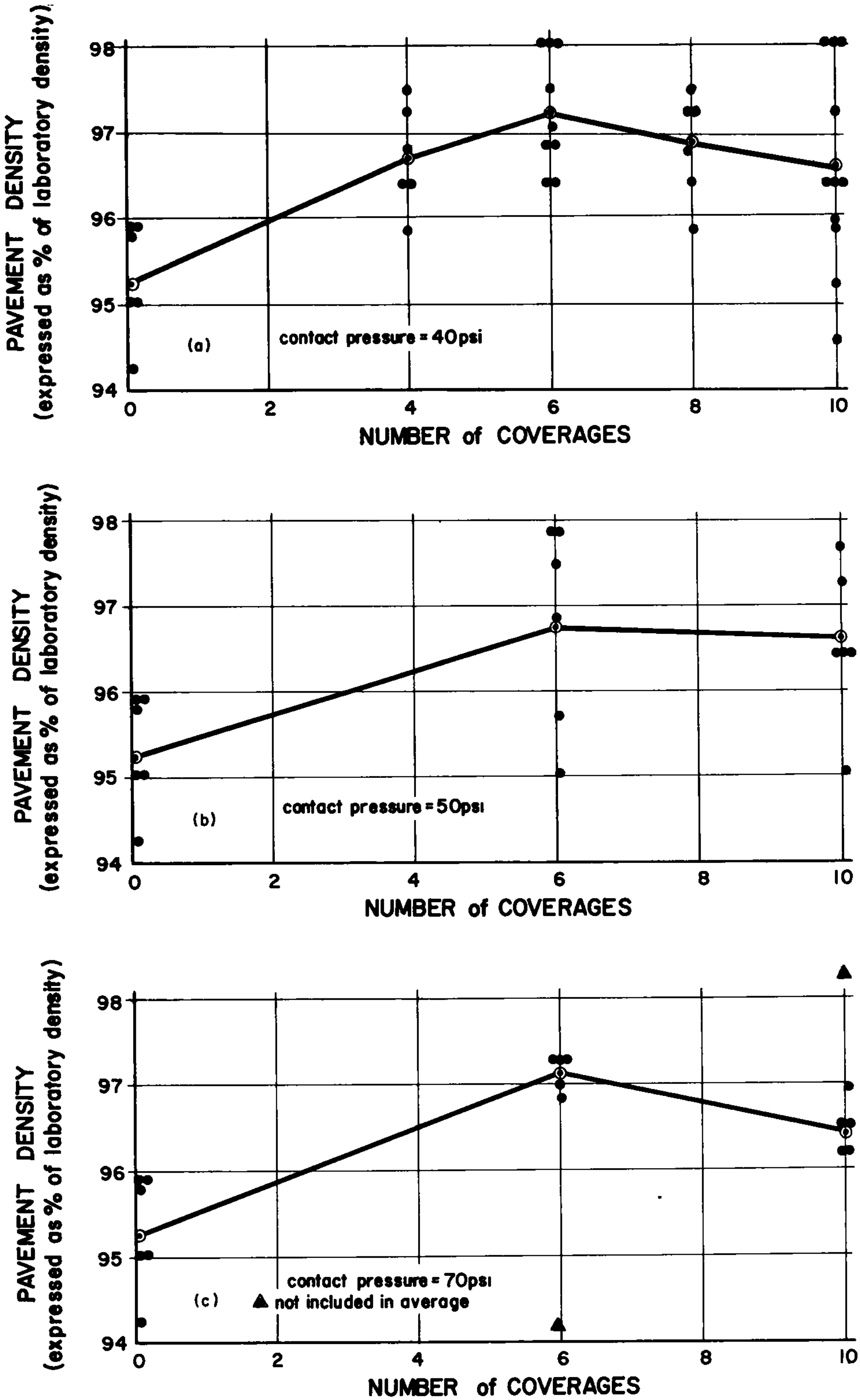


Figure 8. Pneumatic-tired rollers, pavement density vs number of coverages: (a) 40-psi contact pressure; (b) 50-psi contact pressure; and (c) 70-psi contact pressure.

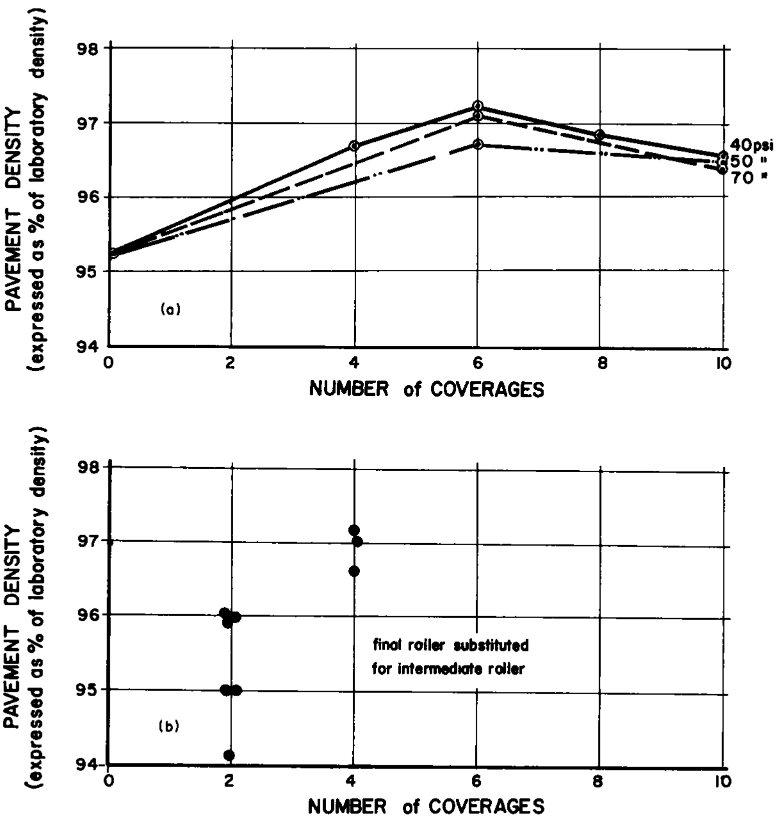


Figure 9. Pneumatic-tired rollers, pavement density vs number of coverages: (a) combined results; and (b) results from increasing number of coverages in section in which intermediate rolling was omitted.

for Conducting Permeability Tests." This was done to check the different test sections for honeycombing or flushing of asphalt from the mix caused by insufficient or excessive amounts of intermediate rolling during construction.

RESULTS

The results of the field and laboratory tests are summarized in this section. Details of the design mix and results of the extraction tests and laboratory compaction study are included in the Appendix.

The average field densities obtained, the compactive effort applied, and the temperature at which compaction was accomplished are given in Table 3. Relationships between the compactive effort applied and the densities obtained for pneumatic-tired rollers at the various contact pressures are shown in Figures 8a, 8b, and 8c. This same information is combined in Figure 9a to permit a direct comparison of the results. Figure 9b shows the results obtained by increasing the number of coverages with the final roller in the sections in which the intermediate rolling was omitted. Results of the skid tests, permeability tests, and the transverse profile measurements are summarized in Table 4.

Table 5 shows the laboratory density and the results of the extraction tests on the 26 truck samples obtained during the paving operations. A comparison of the individual density samples and the design mix density is also given in this table.

TABLE 4

PROPERTIES OF MIX

Section Number	Avg (2) Lab Density (% 2 21)	Specific Gravity Avg (2) Hubbard Field	Avg (8) Marshall	Theoretical	Bitumen (%)
1	100 8	2 23		2 26	5 5
2	100 4	2 22		2 26	5 1
3	99 9	2 21		2 26	4 9
4	100 6	2 22	2 226	2 26	5 5
5	99 1	2 19		2 26	5 0
6	99 6	2 20		2 27	5 3
7	100 8	2.23		2 27	5 3
8	101 3	2 24		2 26	5 5
9	99 0	2 19	2 214	2 26	5 1
10	97 8	2 16		2 27	5 3
11	100 6	2 22		2 26	5 6
12	100 4	2 22		2 26	5 5
13	101 4	2 24		2 25	5 8
14	99.8	2 19		2 26	5 5
15	100.4	2 22		2 26	5 6
16	102 2	2 26	2 229	2 26	5 7
17	101 4	2 24		2 26	5 6
18	100 9	2 23		2.26	5 4
19	100 8	2 23		2 26	5 6
20	100 2	2 22		2 27	5 4
21	100 9	2 23		2 27	5 4
22	101 0	2 23		2 26	5 4
23	99 6	2 21		2 26	5 4
24	99.8	2 21		2 26	5 3
25	100 1	2 22		2 26	6 0
26	100 9	2 23	2 21	2 27	5 4
$\bar{x}$		2 22			5 4
Max		2 26			5 0
Min		2 16			4 9
		0 03			0 025
Limits					
Max		2 28			5 9
Min		2 16			4 9

TABLE 5

PROPERTIES OF SURFACE

Transverse Profile Check	Avg Skid Resistance Decelerometer Reading	Permeability of Surface
OK	63	Impervious
OK	69	"
OK	67	"
OK	71	"
OK	72	"
OK	71	"
OK	69	"
OK	65	"
OK	64	"
OK	70	"
OK	73	"
OK	68	"
OK	66	"
OK	65	"
OK	64	"
OK	66	"
OK	67	"
OK	69	"
OK	65	"
OK	66	"
Humped slightly[1]	67	"
Humped slightly[1]	63	"
Humped slightly[1]	65	"
Very slight rutting[2]	66	"
Humped, slight rutting[2]	64	"
OK	64	"

[1] Sections "humped slightly" were about 1/32 in high at centerline

[2] Slight rutting refers to visible but unmeasurable tire prints of traffic roller remaining after finish rolling

Density vs Compactive Effort

When the results of the field density tests are averaged and plotted against the compactive effort applied, they indicate that maximum compaction is obtained at approximately six coverages regardless of the contact pressure employed but that the mix apparently decompacts when additional coverages are applied. This tendency to decompact is not very pronounced and it should be recognized that individual density samples taken from sections in which ten coverages were made were found to have densities equal to the highest densities found in the sections compacted with only six coverages. However, the fact cannot be ignored that the average densities obtained in the various sections show a reduction when more than six coverages are applied. The same phenomenon was reported by Louisiana.

Changing the contact pressure apparently had no significant effect on the densities attained for a given number of coverages. Figure 9a shows that the maximum average density attained with six coverages varied only between 97.3 and 96.7 percent of laboratory density and the maximum average density attained with ten coverages ranged only from 96.6 to 96.3 percent. However, in every instance the highest densities were attained with the lower contact pressure. This fact coupled with the results of the Louisiana tests indicates that the 40-psi contact pressure (attained with a tire pressure of 55 psi) is probably an optimum.

It must be kept in mind that whenever reference is made to variation in compactive effort in the preceding discussion it refers only to the intermediate roller. As was previously stated, the initial rolling was held constant in all 26 test sections and the final rolling was kept constant in all sections except Nos. 16, 19, 21, 22, and 23. Sections 19, 21, 22, and 23 were compacted with a vibratory roller and discussion of these sections will be treated later.

No intermediate rolling was done in Section 16 but two extra coverages were made with the finish roller for comparison with the results obtained in Sections 5 and 11. Figure 9b shows that this extra finish rolling provided an increase in density of 1.6 percent (from 95.3 to 96.9 percent of laboratory density). This appears significant, but due to the variation observed throughout the test in this project and the lack of replicate sections to provide a measure of the significance of variation, an accurate evaluation cannot be made of these data. Based purely on general observations, however, this writer feels that this increase obtained with the two additional coverages with the finish roller is significant.

Variation in Results

The variation in the results obtained seemed excessive—particularly with respect to individual test sections. Density samples taken from the same section varied as much as 2.9 percent and two samples taken within 25 ft of each other varied by 2.7 percent (Section 20). The average variation was 1.4 percent but variations greater than this occurred in ten sections. This variation (shown in Fig. 8) appears to be significant and should be recognized when preparing specifications for the compaction of asphaltic concrete because it represents two-thirds of the maximum average net increase in density attributable to the intermediate roller. Moreover, in computing the average density in Figure 8c two density samples were omitted due to their great deviation from the other data in their group.

When the results of density tests conducted on individual samples from within a given section are averaged and compared with the average of results of other sections submitted to a like compactive effort the average variation is greatly reduced, most of the overlapping of results disappears, and some definite trends become apparent. For instance, the least average density provided by six coverages (40 psi contact pressure) is greater than the maximum density provided by either four or eight coverages. Likewise, the least density provided by four coverages is greater than the maximum provided by zero coverages and the least density provided by eight coverages is greater than the maximum provided in three of the four sections submitted to ten coverages—Section 1 being the exception.

In an effort to account for some of the variation just shown, an analysis was made of results of the extraction tests and the results of the laboratory compaction study. None of these test results showed any correlation with the field densities. The variation in the gradation of the samples taken from the trucks for each of the 26 test sections is shown in the Appendix and the variations found in the percent bitumen and the densities of the specimens compacted in the laboratory from the same samples are shown in Table 4. The distribution of results obtained in each of these studies indicates that the variation is due more to chance error than to any induced bias. The distribution appears normal for the small number of data available and every indication is that the same errors would recur in the same approximate proportions if the sample population was expanded. This indicates a need for a quality control study to establish specification limits and to determine the number of samples required to ascertain that specifications are being met.

Temperature

Because it is well-recognized that the temperature at which an asphaltic concrete is mixed and compacted has an important effect on the densities attained, every effort was made to hold the temperature of the mix within as small a range as possible for each phase of the construction operation. The temperature data given in Table 3 show the success of this endeavor.

The temperature distribution appears to be both normal and random just as does the density and inasmuch as no correlation was found to exist between these variables it is assumed that the close control of the temperature was apparently successful in mitigating the effect of this variable.

The temperature data collected on this project revealed that the temperature requirements listed in the standard specifications are unrealistic with respect to compaction. The standard specifications require that the intermediate rolling with pneumatic-tired rollers "shall be done while the pavement temperature is between 175 F and 240 F..." and that final rolling "...shall be done before the pavement temperature is lower than 150 F...". Trials made on this project showed the maximum temperature at which this particular mix would support the pneumatic-tired roller without undue distortion or pick-up was 150 F for 40-psi contact pressure and 140 F for 70-psi contact pressure. The maximum temperature at which this mix would tolerate the 8-ton steel-wheel finish roller was 150 F. This information indicates the need for further study to determine the maximum temperature at which other mixes will properly support the compaction equipment. If future studies confirm the observations made on this project, the standard specifications should be revised accordingly.

Skid Resistance

The results of the skid tests summarized in Table 5 show that there apparently was no flushing of the asphalt in any section. The minimum skid resistance recorded was 63—well above the average for an asphaltic concrete. This can be expected to decrease with time due to the abrasive action of traffic and the accumulation of rubber, oil dripping, etc. Further tests will be made after traffic has used this section one year.

There was a very slight correlation between skid resistance and density with the lower skid resistance being generally associated with the higher densities. However, because the total range in density was so small, this correlation was not strong. In general, sections compacted with the higher contact pressures and the vibratory roller provided the poorest skid resistance with the results from the sections compacted with the vibratory roller going against the trend relating density and skid resistance as already noted.

Permeability

All sections were impermeable, showing that pneumatic-tired rolling is not necessary to achieve impermeability so long as 95 percent of the laboratory density is achieved.

Surface Irregularities

The only surface irregularities in any of the test sections were the slight tire marks noted in Sections 24 and 25 where 70-psi contact pressure was applied, and the slight "humping" noted in Sections 21, 22, and 23 where the vibratory roller was used. These distortions could be easily discerned immediately after construction but actually deviated from a straight plane by less than $\frac{1}{32}$ in. Most of the distortion noted is probably due more to construction procedure than to the particular equipment involved. Because both the rollers used in these sections were approximately $\frac{1}{2}$ the width of the pavement being placed, and because the compactive effort was carefully controlled in an attempt to provide the same compactive effort across each section of the pavement, no overlapping by the rollers was permitted and as a consequence rolling was highly channelized. All initial and finish rolling was lapped normally, but no finish rolling was done in the sections in which the vibratory roller was used.

Vibratory Roller

The results obtained with the vibratory roller in any one section were no more variable than those in any of the other sections, but the replicate sections with three coverages showed poor reproducibility of results. This particular equipment provided no direct control of the vibrating frequency and the change in the frequency-forward motion relationship may have changed between these sections. Due to the poor reproducibility of results demonstrated and the numerous mechanical difficulties that delayed paving operations and prevented the construction of additional test sites for further replication, no evaluation of the results obtained was attempted. Further study of vibratory rolling of asphaltic concrete will be undertaken when it has been demonstrated that the equipment available is suitable for this type of work and that positive control of the vibrating frequency and forward speed of the equipment can be maintained.

SUMMARY AND CONCLUSIONS

This project was undertaken to evaluate the effect of varying the compactive effort of the intermediate rolling on asphaltic concrete. Both pneumatic-tired and steel-wheel vibratory rollers were used. The number of coverages and the contact pressure was varied for the pneumatic-tired rollers used in 22 test sections. The number of coverages was varied in the four sections constructed with the vibratory compactor. The results obtained with the vibratory compactor were inconsistent and due to the lack of replication no evaluation of these results was attempted. The average compactive effort attained with the vibratory roller was considerably less than the average attained with the rubber-tired intermediate roller. In fact, the maximum compaction attained with the vibratory roller was less than the average compaction attained with no intermediate rolling at all but with two extra coverages of the 8-ton steel-wheel final roller.

Results of the tests made with the pneumatic-tired intermediate roller indicated that maximum compaction is attained with six coverages, and a slight tendency for the material to decompact with additional coverages was noted. Results from the three sections in which no intermediate rolling was used showed that the average density attained exceeded minimum specification requirements. When the intermediate rolling was omitted but two extra coverages of the final steel-wheel rolling was applied instead, the average density attained was nearly as great as the average maximum density attained with the optimum number of coverages and exceeded the average densities attained with eight and ten coverages of the intermediate roller.

Based on the results obtained in this study the following are concluded:

1. The optimum number of coverages with the pneumatic tired roller is six and that asphaltic concrete will tend to decompact with additional coverages beyond this optimum.
2. The optimum contact pressure that should be exerted by the pneumatic tired intermediate roller is 40 psi.
3. The densities exceeding the minimum specification requirements can be achieved without any intermediate rolling. The results of the permeability tests indicate no detrimental effect resulting from omission of the intermediate rolling.
4. The temperature requirements listed in the Florida standard specifications are unrealistic with respect to compaction. Further study should be made to determine the maximum temperature at which other mixes will properly support the compaction equipment, and, if these studies confirm the observations made on this project, the standard specifications should be revised accordingly.

REFERENCES

1. "The Use of Self-Propelled Pneumatic Tired Rollers in Bituminous Construction and Recommended Procedures." Special Report, Testing and Research Section, Louisiana Department of Highways, State Project 7-07-13 (Jan. 1958).
2. Kimble, F.W., "Compaction Pressures of Steel Wheel and Pneumatic Rollers with Special Reference to Hot-Mix Construction." Rural Roads (Jan. 1961).
3. Public Works, 28:No. 22 (Feb. 1958).

Appendix

TABLE 6

ASPHALT DENSITIES AS COMPUTED FROM SAMPLES REMOVED FROM ROADWAY

Section Number	Density (% of lab roadway) Inner Wheel-path	Between Wheel-path	Outer Wheel-path	Average	Contact Pressure[a] (psi)	Number Intermediate Roller
1	96 4	98 2	98 2	97.6	40(L)	10
2	95.9	97.3	97 3	96.8	40(L)	8
3	96.8	96.4	98.2	97 1	40(L)	6
4	96.4	97.7	97 3	97.1	40(L)	4
5	95.9	95.9	95 8	95 9		0
6	95 9	97 3	96.4	96.5	40(L)	10
7	96.4	97 7	96.8	96.9	40(L)	8
8	97 0	98.2	97.7	97 6	40(L)	6
9	96 4	95 9	96.8	96 4	40(L)	4
10	96.4	96 6	96.0	96.3	40(H)	10
11	95 0	95 0	94 2	94 7		0
12	97.7	96.4	95.0	96.4	50(H)	10
13	97 7	95 9	95 0	96.2	50(H)	6
14	97 2	96.4	96.4	96 7	50(H)	10
15	97.4	97 7	96 8	97.3	50(H)	6
16	97 3	96 4	97 2	96.9		0
17	98 2	95 3	94 6	96.0	40(H)	10
18	98.2	96.8	96 4	97.1	40(L)	6
19	95.5	96.8	96 8	96 4	Vibratory	3
20	97 0	96 8	94 1	96.0	70(H)	6
21	96.4	95 9	96 4	96 2	Vibratory	4
22	96.8	95 0	95.9	95 9	Vibratory	2
23	94.6	93 7	95.5	94.6	Vibratory	3
24	96.4	96.4	95 9	96.2	70(H)	10
25	97 3	97.3	97 3	97.3	70(H)	6
26	95.9	98 2	96 8	97 0	70(H)	10

[a]L = light roller, Bros SP-54B.
H = heavy roller, Bros SP-730B.

TABLE 7

DESIGN MIX DATA

Property	Avg (%)	Limit (%) Max.	Min.	Spec. Grav.
Percent passing				
1/2-in.	100	100	100	
3/8-in.	98	100	93	
No 4	63	68	58	
No. 10	43	47	39	
No. 40	33	36	30	
No. 80	16	19	13	
No 200	4	6	2	
Bitumen	5.7	6.1	5.3	1.02
Coarse aggregate	53.8			2 35
Fine aggregate	37 7			2 55
Mineral filler	2 8			2 71
Spec gravity·				
Theoretical				2 26
Laboratory				2 21
Lab. dens [a]	97.8			

[a]As percent of theoretical density.

TABLE 8

SUMMARY OF EXTRACTION GRADATIONS

Property	Avg.	Max.	Min.	Limit Max.	Min.
Cum. % Passing					
1/2 in	100	100	99	100	99
3/8 in.	98 5	99 8	96.7	100	96.7
No 4	64 2	76 2	57.2	71.2	57.2
No 10	41 4	49.0	35.8	47 2	35.6
No. 40	32 7	37.6	28.5	37.1	28.3
No 80	13 8	16 0	11.6	16.2	11 4
No. 200	3 2	4 2	2.3	4 2	2 2
Bitumen (5)	5.4[a]	6 0	4 9	5 9	4 9

[a]Four of 26 samples deficient in bitumen content

TABLE 9

GRADATION OF EXTRACTION SAMPLES TAKEN FROM TRUCK DURING PAVING OPERATIONS

Sieve Size	Cumulative Percent												
	Sec 1	Sec 2	Sec 3	Sec 4	Sec 5	Sec 6	Sec 7	Sec 8	Sec 9	Sec 10	Sec 11	Sec 12	Sec 13
3/4 in.						100.0							
1/2 in.	100.0	100.0	100.0	100.0	100.0	99.2	100.0	100.0	100.0	100.0	100.0	100.0	100.0
3/8 in.	99.4	99.2	97.8	98.6	96.8	96.7	99.6	98.4	98.7	99.2	99.5	99.5	98.8
No. 4	74.0	66.7	65.0	64.0	57.2	62.9	67.7	64.8	64.9	76.2	64.7	64.8	58.5
No. 10	49.0	45.1	41.3	40.0	35.8	39.2	43.3	40.2	38.3	47.8	39.7	38.9	37.3
No. 40	37.6	36.1	33.5	31.3	28.5	30.5	33.9	30.9	29.2	36.5	30.9	30.5	29.1
No. 80	15.4	16.0	14.9	13.4	13.1	13.8	13.6	12.7	11.6	14.4	12.8	12.8	11.4
No. 200	4.1	3.6	3.7	2.6	3.5	3.6	3.3	3.2	2.8	3.4	3.4	2.3	2.3

	Sec 14	Sec 15	Sec 16	Sec 17	Sec 18	Sec 19	Sec 20	Sec 21	Sec 22	Sec 23	Sec 24	Sec 25	Sec 26
3/4 in.						100.0	100.0						
1/2 in.	100.0	100.0	100.0	100.0	100.0	99.2	99.2	100.0	100.0	100.0	100.0	100.0	100.0
3/8 in.	98.3	97.0	98.7	98.7	98.8	97.4	97.4	99.2	99.4	98.6	98.2	99.4	98.2
No. 4	62.3	60.7	64.5	62.5	63.8	61.9	63.1	66.2	65.4	59.5	63.9	69.1	56.2
No. 10	40.8	42.7	42.5	40.3	41.5	41.3	43.6	42.1	40.4	39.7	41.6	44.2	38.5
No 40	32.2	34.3	33.3	31.7	32.7	32.9	34.8	33.0	32.6	32.5	33.7	35.4	31.7
No. 80	12.3	14.9	13.5	13.2	13.0	14.0	14.3	13.2	15.0	14.8	15.0	15.8	14.9
No. 200	3.3	3.4	3.1	3.0	2.3	4.2	2.5	2.9	3.3	3.7	2.9	3.1	3.5

An Examination of Mixing Times as Determined by The Ross Count Method

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Seventeen batch plants including a variety of types and equipment were tested by two procedures to establish minimum wet mixing times. Ross count determinations were made and extractions were run to determine the change in asphalt content of the coarse fraction with mixing time. Samples were taken at the plant and from the road.

Specifically, this study has shown that an appreciable amount of coating takes place in transit and while the mix is passing through the paver; the wet mixing time required to yield 98+ percent completely coated particles on the road was generally less than 20 sec; the use of a 15-sec dry-mixing interval did not appreciably improve the coating that was accomplished by a specific wet mixing time; the time required to yield 90+ percent coating on the road was in many cases close to the minimum time required to load the mixing chamber with aggregate and asphalt. The measurement of the asphalt content of the +3/8 in. fraction did not yield results that could be used to establish mixing times.

• THE RATE of production of an asphalt plant is influenced by many factors, one of the most significant being mixing time. The most appropriate mixing time is dependent on several considerations. For economic reasons a short mixing time is preferable; however, the necessity for well-coated particles makes a long mixing time desirable. A complicating factor is that long mixing periods may oxidize the asphalt by exposing the thin film of hot asphalt to air. It is necessary therefore to select a mixing time that is adequate from a coating standpoint but as brief as possible for economical production and low oxidation. The following are the most important variables influencing mixing times:

(a) aggregate (gradation, type),
(b) asphalt (content, viscosity), and
(c) type and use of equipment (paddle arrangement, method of introducing material, etc.).

Tunnicliff, (1) after an extensive laboratory study, concluded that the equipment is the most significant factor of the three.

Chastain (2) in a survey of State practices reports that wet minimum mixing times for batch plants range from 20 to 55 sec. The current Virginia Department of Highways minimum is 45 sec. In dry mixing, the minimum mixing time varies from 15 to 20 sec, with 19 agencies including Virginia, specifying a 15-sec dry-mix period. Twelve agencies require only that the dry mixing be "thorough."

For continuous mixing operations minimum mixing periods were reported by only 22 of the 50 agencies surveyed. These 22 agencies reported that the following formula is used to determine the mixing period:

$$\text{Mixing period (sec)} = \frac{\text{Pugmill dead capacity (lb)}}{\text{Pugmill output (lb/sec)}}$$

The minimum mixing times resulting from this formula vary from 30 to 60 sec.

PURPOSE OF STUDY

The problem of selecting an adequate mixing time, though not insurmountable, is difficult. This is partially due to the fact that there is no clear-cut concept of what constitutes the ideal asphalt-aggregate relationship. However from a practical standpoint it does not seem necessary to digress into that question, because several reasonable approaches are possible. Ward and Warren (3) lists seven methods for determining when the blend has been adequately mixed:

1. Analysis of variance.
2. Ratio of asphalt content in fine and coarse aggregate.
3. Measurement of stripping tendencies.
4. Measurement of water absorption.
5. Measurement of freeze-thaw resistance.
6. Correlation with energy input.
7. Measurement of coating of calcareous aggregates in a mix by acid neutralization.

In the same paper, the suitability of the Ross count method was reported. It was concluded by the authors that this method offered the greatest promise for determining the most appropriate mixing time. In the Ross count method, the complete coating of the coarse particles is considered indicative of a properly mixed blend; therefore, the shortest mixing interval yielding complete coating of the coarsest particles is selected as the proper mixing time. This is postulated on the basis of the visual observation that the coarsest particles are the last to be coated.

One further approach is to measure the change in the asphalt content of the coarse fraction as the mixing time is varied, here termed asphalt content equilibrium method. When increased mixing time no longer produces an increase in the asphalt content of the coarse fraction, the blend could be considered in equilibrium. Although this approach is also based on the principle that the coarsest fraction is the last to be coated, it does not depend on visual observation as does the Ross Count method. The determination of the asphalt content would yield a numerical value.

The moisture resistance of a blend has also been suggested as a measure of adequate mixing. That is, it was supposed that the moisture resistance would vary with completeness of mixing but this does not appear to be true. Tunnicliff (1) investigated the effect of mixing time on stripping but found that no significant influence was evident in the immersion-compression test. Also, he found that the stability of bituminous concrete as measured by the Marshall machine, the Hveem stabilometer, the cohesiometer, and unconfined compression was not highly sensitive to the degree of mixing. Tunnicliff suggests the method of "batch rating" (entire batch, visually examined) or a random sampling technique (many small samples, visually examined) to evaluate mixing efficiency.

Both the Ross count method and the asphalt content equilibrium method were selected for inclusion in a statewide study in Virginia. Broadly speaking the purpose of the study was to compare the mixing times at batch plants resulting from the use of these two methods with the current specifications of the Virginia Department of Highways. Other useful information collected during the study is given in the results section.

The scope of the study was narrow. The intent was not to investigate the efficiency of such parameters of production as various paddle arrangements, methods of introducing asphalt, or temperature-viscosity-mixing efficiency relationships. The essential question was whether mixing times would be essentially lower than the Virginia 45-sec minimum if the two methods were used to establish mixing times.

Five types of mixes were included in the study:

H-3 (1)—100 percent passing 2-in. sieve; 25 to 85 pen. at 4 to 5 percent; used as base course.

H-3 (3)—100 percent passing 1 ½-in. sieve; 120 to 150 pen. at 4 to 5 percent; used for resurfacing in place of an application treatment.

H-2—100 percent passing 1-in. sieve; 85 to 100 pen. at 5 to 5.5 percent; used as binder course.

I-3—100 percent passing ½-in. sieve; 85 to 100 pen. at 6 to 6.5 percent; used as surface course.

F-1—100 percent passing ⅜-in. sieve; 85 to 100 pen. at 7 to 7.5 percent; used as surface course.

A variety of plant capacities, ages, paddle arrangements, and other elements were included in the plants tested. Also a variety of pavers were included, but no attempt was made to determine the most efficient equipment.

Temperature specifications in Virginia are 250 ± 25 F. Occasionally mixes were produced that were outside of this range but the great majority were within it.

SAMPLING PROCEDURE

For the investigation of the Ross count determination of wet mixing times the general procedure was to begin at the current specification mixing time (45 sec) and decrease the mixing times in regular 10- or 5-sec increments. Ross count determinations were made at each mixing time and the mixing times lessened until a Ross count of less than 90 percent was obtained. Generally three mixing times were used, and after the first few plants were tested the beginning time was set at 30 to 35 sec rather than 45 sec.

Six samples were taken at each wet mixing time—three from batches with a 15-sec dry mix and three from batches without a dry-mix interval. These six samples were taken from one or two trucks. Therefore, the truckload included only the results of one wet-mixing time but two dry-mixing intervals. These trucks were marked and the road samples were taken of the mix. Of course, the location of a particular truckload was not precisely determinable but a reasonably good estimate could be made of where the load began on the road and where it ended. The road samples were then taken from the center of the resulting test mix stretch.

For the Ross count determination the sample (about 5 to 8 lb) was sieved by hand through a ⅜-in. sieve (for the F-1 mix a No. 4 sieve was used). The particles that were completely coated were then recorded and the particles on which even a speck of stone showed through were recorded as incompletely coated. Using this procedure, the mixing time was lowered until a Ross count of less than 90 percent was obtained. The results were then plotted and the time at which 90 percent of the plus ⅜-in. particles were completely coated was determined graphically. This time was then used on "check runs." This was done to gain some idea about the reproducibility of the mixing system. The plant was then permitted to operate for at least several hours at the 90 percent time and additional samples taken at the plant and on the road. Generally six samples for Ross counts were taken with dry mix of 15 sec and six samples without a dry mix.

TEST RESULTS

Ross Count Determinations

A typical Ross count curve is shown in Figure 1. The curves are always relatively smooth and verify the fact that the coating of the plus No. 4 or plus ⅜-in. particles increases regularly with time.

Ross count determinations made at the plant and those taken from the road are given in Table 1. Much additional coating takes place in transit and while the mix is going through the paver. For example, at plants B and D the percent completely coated at the plant was 68 and 51 percent, respectively, at the lowest mixing times whereas the samples taken from the road were 99 and 100 percent completely coated. Even

TABLE 1

COMPARISON OF ROSS COUNT ON SAMPLES FROM PLANT WITH THOSE FROM BEHIND PAVER

Plant	Mix	Wet Mixing Times (sec)	Ross Count (percent)	
			Sample from Plant[a]	Sample from Road[b]
A	H-3 (1)	15	82	100
		25	96	99
		35	99	100
		45	100	100
B	H-3 (1)	15	68	99
		20	93	100
		35	100	100
C	H-3 (1)	15	79	99
		21	97	100
		25	98	100
D	I-3	20	51	99
		25	60	98
		30	68	96
E	I-3	20	84	100
		23	93	100
		25	96	100
F	H-3 (1)	20	74	100
		25	92	100
		30	97	100
G	I-3	25	99	--
H	I-3	25	100	--
I	I-3	25	81	99
		30	90	99
		25	95	--
J	I-3	20	86	100
		22	95	100
		25	97	99
K	F-1	13	92	96
		15	95	98
		20	99	98
L	F-1	15	78	
		18	96	
		20	100	
M	H-2	13	89	100
		15	94	100
		20	96	100
N	H-3 (1)	17	89	100
		20	92	100
		25	97	100
O	I-3	20	90	--
		24	98	--
		26	99	--
P	H-3 (3)	18	79	98
		20	98	100
		25	96	100
Q	H-3 (3)	13	68	95
		15	90	100
		20	97	100

[a]Mean of 6 to 8 determinations—one-half had 15-sec-dry mix; remainder, no dry mix.
[b]Mean of 6 determinations.

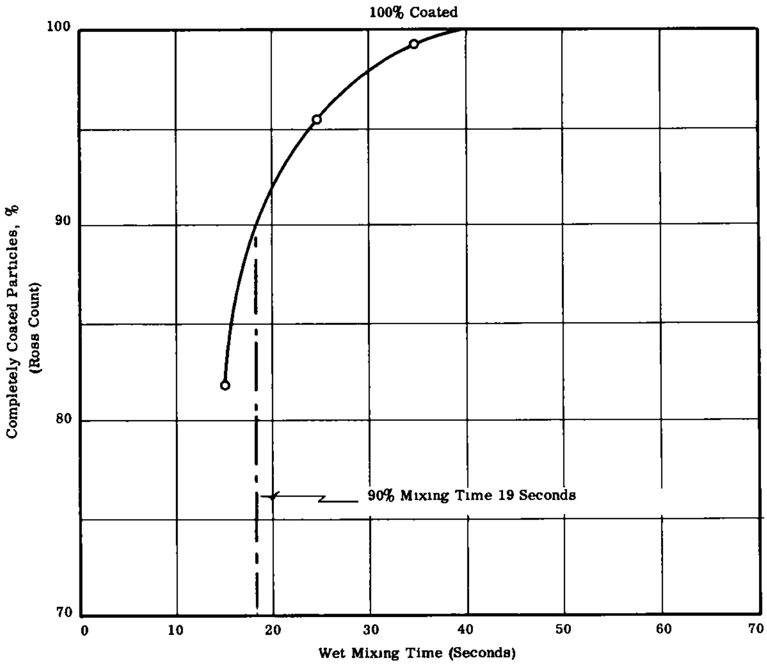

Figure 1. Typical Ross count curve.

TABLE 2

COMPARISON OF ROSS COUNT FROM SHORT INTERVAL TESTS AND LONGER "CHECK RUNS"

Plant	90% Mixing Time (sec)	Ross Count Check Runs	
		Sample from Plant[a]	Sample from Road[b]
A	19	93	99
B	20	90	98
C	18	90	99
D	35	91	100
E	23	93	100
F	24	86	99
G	25	99[c]	--
H	25	100[c]	--
I	30	86	99
J	21	92	100
K	--	--	--
L	17	90	--
M	17	94	--
N	18	87	100
O	22	96	--
P	20	91	100
Q	15	88	99

[a]Mean of 12 determinations—one-half had 15-sec dry mix, remainder no dry mix
[b]Mean of 6 determinations
[c]Charging required, 25 sec.

TABLE 3

INFLUENCE OF DRY MIX ON ROSS COUNT TIME (PLANT SAMPLES)

Plant	Mixing Time (sec)	Ross Count (%) Dry Mix[a]	Ross Count (%) Without Dry Mixing
A	25	95	96
	35	99	100
	45	100	100
	19[b]	90	94
B	15	79	58
	20	93	93
	35	100	100
	20[b]	90	91
C	15	77	79
	21	98	96
	25	99	98
	18[b]	86	94
D	20	53	49
	25	61	60
	30	71	65
	35[b]	84	97
E	20	82	86
	23	91	95
	27	93	99
	23[b]	91	95
F	20	81	67
	25	91	93
	30	95	98
	24[b]	87	85
G	25	100	100
H	25	100	--
I	25	81	--
	30	90	--
	35	95	--
	30[b]	87	85
J	20	86	
	22	95	
	25	97	
	21[b]	93	90
K	13	92	
	15	95	
	20	99	96
	15[b]	95	
L	15	78	
	18	96	
	20	100	
	17[b]	89	92
M	13	89	
	15	94	
	20	99	
	17[b]	99	90
N	17	89	
	20	92	
	25	97	
	18[b]	100	83
O	20	90	
	24	98	
	26	99	
	22[b]	96	97
P	18	75	82
	20	99	97
	25	95	91
	20[b]	94	88
Q	13	64	72
	15	87	92
	20	100	94
	15[b]	87	89

[a]With 15 sec

[b]90 percent Ross count time, results obtained during "check runs."

at the lowest wet-mixing time attempted (13 sec, plant M) the road samples were 100 percent coated.

The results on the check runs are summarized in Table 2. It can be seen that the 90 percent Ross count time was a stable parameter and was reproducible during the check runs. Also, the table shows that when the 90 percent Ross count time was used during the check runs that the coated particles were close to 90 percent. Also, the road samples were again close to 100 percent coated.

The influence of dry mixing is given in Table 3. In only a very few instances did the use of a dry mix interval result in improved coating. For instance, with plant K good coating resulted when a 15-sec dry mix was used with a 15-sec wet mix, but the use of a 15-sec wet mix alone resulted in very poor coating. In general, however, the data indicate that there was little benefit in coating to be gained from using a dry mix of 15 sec.

Equilibrium Results

Typical equilibrium results are shown in Figure 2. These results indicate that the asphalt content of the coarse fraction is relatively erratic and is not clearly related to mixing time. It does not permit the ready discernment of a point at which to select a mixing time. The fact that the coarse particles are the last to be coated led to the conjecture that the coarsest fraction might gain in asphalt content up to a point, and beyond this the asphalt content would remain constant. The time at which no further increase in asphalt content occurred would constitute an "optimum mixing time." This would probably hold true if only coarse aggregate were involved but the fact that the mixes contain fines complicates the rationale. The coarse particles are, at first, coated with globules of asphalt and fines. As mixing proceeds the globules break down and form films. The coarse fraction in all probability contains particles of various stages of coating– some with globules, some with films. This then leads to the seemingly erratic results. The equilibrium results were obtained at all plants except one but only typical results are shown. In Figure 2 a trend is discernible for plant B but not for the other three.

In any event, the asphalt equilibrium method is unsuitable for determining mixing times.

Influence of Dry Mix on Gradation Control

Only a very limited amount of data was collected on gradations, with and without a dry mix. The lack of time and relative inexperience of the crew in running gradations retarded the collection of these data. No test results are presented here

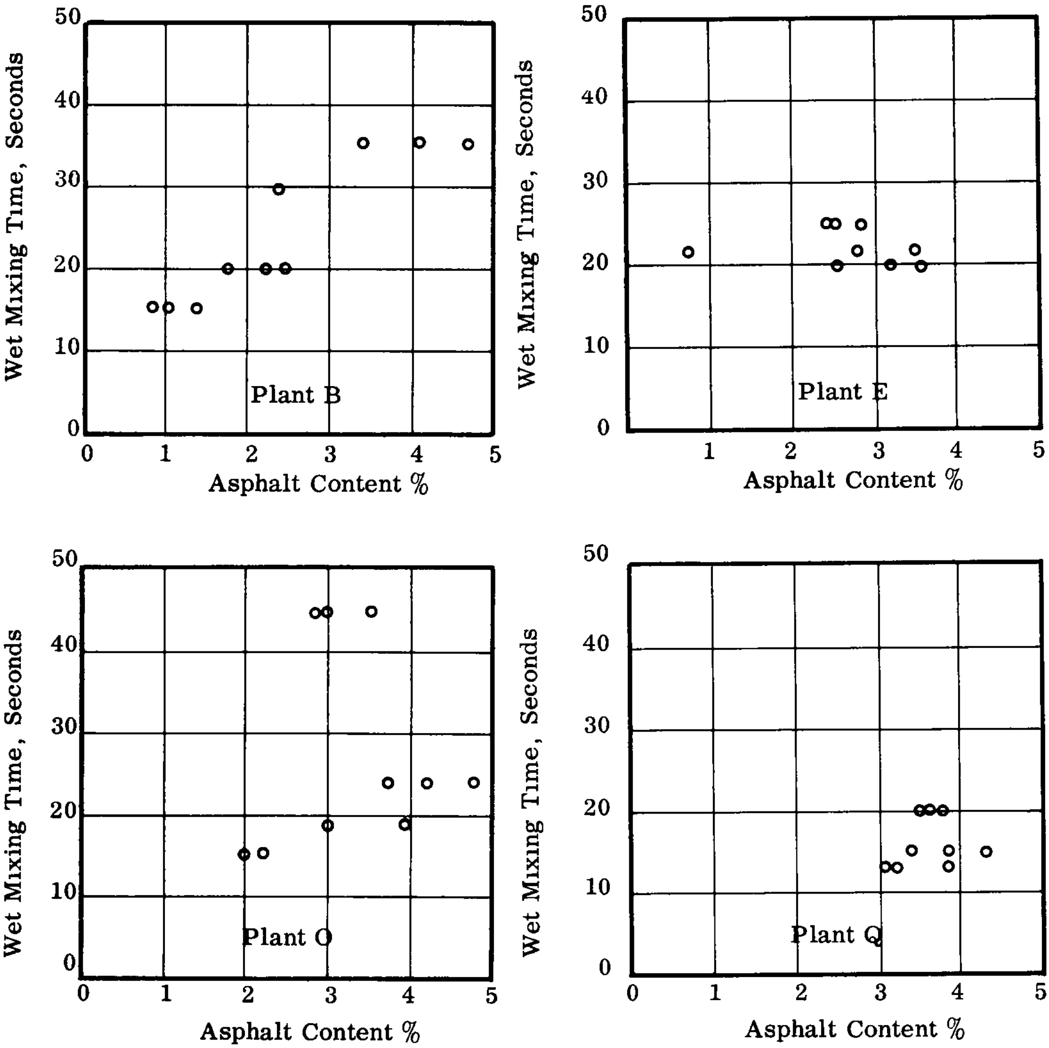

Figure 2. Typical plots of change in asphalt content of coarsest fraction with mixing time.

only a few observations are made. The requirement of a dry-mixing time is predicated on the basis that the aggregates are segregated prior to admission to the mixing chamber and that it is necessary to impart a nonsegregated condition to the total aggregate prior to the admission of the asphalt. That appreciable aggregate distribution takes place during wet mixing is undoubtedly true. Furthermore, the suitability of a mix from a gradation standpoint has, of course, been tested by a sieve analysis for many decades. That is, the contractor must meet a gradation specification or a job mix in any event and in this light the use of a minimum dry-mix requirement may be redundant. If the dry mix does not influence the coating with asphalt appreciably, and it does not appear to, then the sieve analysis will indicate whether the aggregate is uniformly distributed and a dry mix requirement need not be specified. If "dirty" or weakly conglomerated natural aggregates are used, the dry mix may be necessary to aid in breaking down the conglomerations. But in using clean strong aggregates the lack of proper distribution of the various sizes would exhibit itself in the routine sieve analysis.

SUMMARY AND CONCLUSIONS

The Ross count procedure is based on the assumption that the "proper" degree of mixing is a function of the complete coating of the coarsest aggregate. Although it does not take into consideration other factors such as film thickness, it would seem far superior to the arbitrary establishment of minimum mixing times. Until a more perfect method is advanced that takes the many factors into account, it appears to be a useful tool for selecting minimum mixing times. When used in conjunction with a sieve analysis it appears capable of providing a logical basis for specifying total mixing time for hot plant mixes. It is to be emphasized again that the conclusions are based on conditions and equipment prevalent in Virginia.

Specifically, this study has shown the following:

1. An appreciable amount of coating takes place in transit and while the mix is passing through the paver.
2. The wet mixing time required to yield 98+ percent completely coated particles on the road was generally less than 20 sec.
3. The use of a 15-sec dry-mixing interval did not appreciably improve the coating that was accomplished by a specific wet-mixing time.
4. The time required to yield 90+ percent coating on the road was in many cases close to the minimum time required to load the mixing chamber with aggregate and asphalt.
5. The measurement of the asphalt content of the plus $^3/_8$-in. fraction did not yield results that could be used to establish mixing times.

ACKNOWLEDGMENTS

This study was accorded the full cooperation of the asphalt paving contractors, the Virginia Asphalt Association, Inc., and personnel of the Virginia Department of Highways.

Also, the study received the guidance of an advisory subcommittee in the planning and conduct of the experiment. Many of the important decisions regarding the study were obtained from the subcommittee, to whom authors wish to express their thanks.

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Aggregate Temperature and Moisture Prediction From Asphalt Plant Data

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A method of combining more than 40 aggregate drying variables into three primary variables is outlined in this report. The three primary variables are (a) aggregate heat absorption (the heating rate of a given aggregate at a given temperature during drying), including secondary variables such as initial moisture content, saturation, pore structure, and solid thermal characteristics; (b) temperature gradient (the effective temperature profile of dryer gases from entrance to exit), including variables of burner rate, flame position, and veil density; and (c) exposure time (the time a given aggregate is exposed to the temperature gradient), including the variables of dryer slope, length, rotation, and aggregate size.

The control of these three variables during the drying operation makes aggregate temperature at discharge predictable. To arrive at this finding, the results of 250 laboratory tests on three aggregate types (sized 3/4 to sand) were coupled with the conclusions of previous aggregate drying research at the Ohio State University. Temperature gradient and exposure time had been isolated as significant variables in aggregate drying. The three primary variables were found by adding the results of lab tests on heat absorption rates to data from previous studies.

• THE PURPOSE of this study is to develop a method of predicting aggregate temperature and moisture during the drying operation under controllable asphalt plant operations. The relationships of aggregate-dryer variables would then be used to evaluate their effects on dryer operations. The study was undertaken in cooperation with the Bureau of Public Roads on aggregate heating and drying economics at asphalt plants.

Analyses of dryer operations are complicated by different dryer designs, sizes, and heating operations. As a result, complicated interactions exist between aggregates, hot gases, and dryer designs. Any analysis must reduce over 40 dryer-aggregate variables to some fundamental parameters in terms of the dryer function—aggregate temperature increase and moisture loss.

Dryer operations are considered to be controllable when the aggregate-dryer variations can be measured quantitatively. Some important variations are moisture, exposure temperature, and exposure time. During dryer operations these variations affect aggregate drying and heating.

Moisture content is not consistent in aggregates at dryer entrance. This is apparent in one aggregate source at one plant or over a number of plants where thousands of aggregate types and moisture combinations exist. Data illustrating moisture content at several dryers are shown in Figure 1 (Reprint from (9)). These data were obtained in

1960 in an earlier study with the Bureau of Public Roads. Each data point represents the average moisture content of 3 to 6 samples taken during one day of operation at each dryer. Other past data indicate that feed moisture contents for each aggregate size per dryer will vary, sometimes up to 2 to 3 percent, over the operational season. Because moisture content variations of 1 or 2 percent can effectively influence dryer production, dryer evaluation for each aggregate-moisture combination appears necessary throughout the operational season.

The time of aggregate exposure or passage through the dryer also varies. Even during steady operation, exposure times will vary within an aggregate size group; and each size group will vary. For example, prototype dryer data indicate that for average coarse aggregate exposure times of 2 min, sand exposure time is 2 ½ min. When increasing the average coarse aggregate exposure time to 4 ½ min, sand time increases to 6 min. Therefore, for the size groups of bituminous concrete mixtures, differences of exposure times will exist.

The third complexity is "temperature gradient." This is a function of burner rate, flame shape, aggregate moisture, veil resistance, and dryer design and airflow. The burner rate and aggregate veil usually vary during dryer operation. The temperature gradient as a function of two variables is shown in Figure 2. When the same burner rate is used (A, A', or A"), variations of veil density (A-A') or variations of hottest flame location (A-A") produce changes in temperature gradients. If the burner rate is increased from A to B, the temperature gradient increases in a parallel manner.

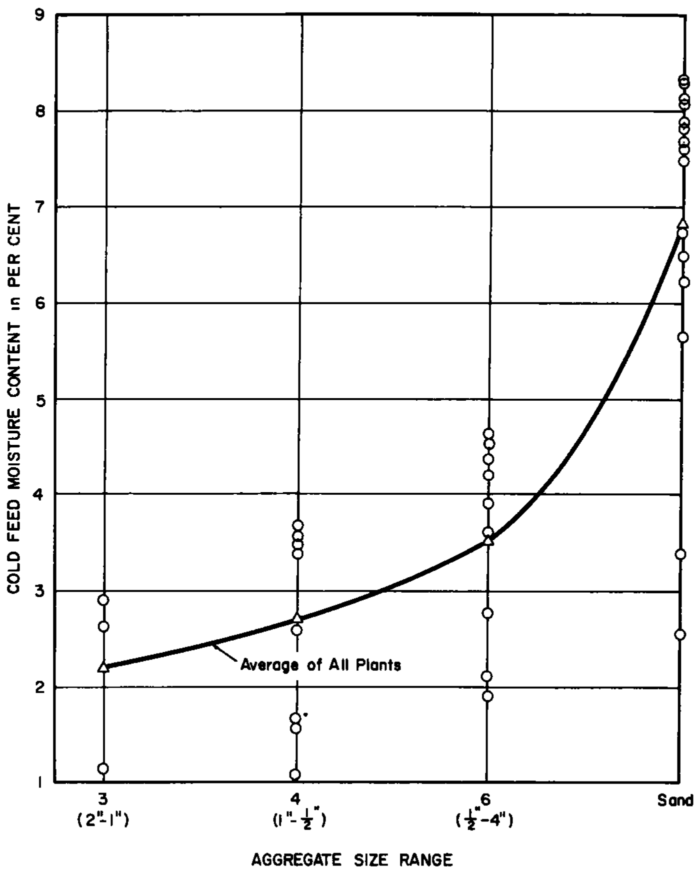

Figure 1. Aggregate size vs moisture content.

Sufficient aggregate exposure is necessary because moisture loss and temperature are dependent on exposure time. The time in the veil (maximum heat exposure) is relatively short. It is necessary for heat to penetrate to the aggregate centers during the periods of flight and cascading contact near the drum walls. Under these exposure conditions, the interaction of the dryer hot gases and aggregate becomes a function of the dryer and burner design. Dryer dimensions usually influence the amount of aggregate heated. However, irrespective of the dryer design, aggregate heating is accomplished by exposure time. For a given aggregate temperature, required exposure time is inversely proportional to the exposure temperature.

In his investigation, Pagen (8) found that all initially wet aggregates exposed to heat go through a drying cycle. Drying cycle changes were shown due to aggregate type and moisture differences. Fischmann (8) in the same paper pointed out that aggregate heating rate is directly dependent upon the thermal conductivity of the solid. Tests by other physical investigators have shown that thermal conductivity increases as moisture increases in porous materials. For aggregate heating aggregate temperature consists of aggregate solid temperature plus the temperature of the interior water in the aggregate pores. From this reasoning one would expect a decreasing thermal conductivity and thus a decreasing heating rate as aggregates dry under a given exposure temperature.

Baker and Lottman (1) conclude drying cycle phases are dependent on exposure temperature. Their data from several drying cycles of saturated aggregates show that

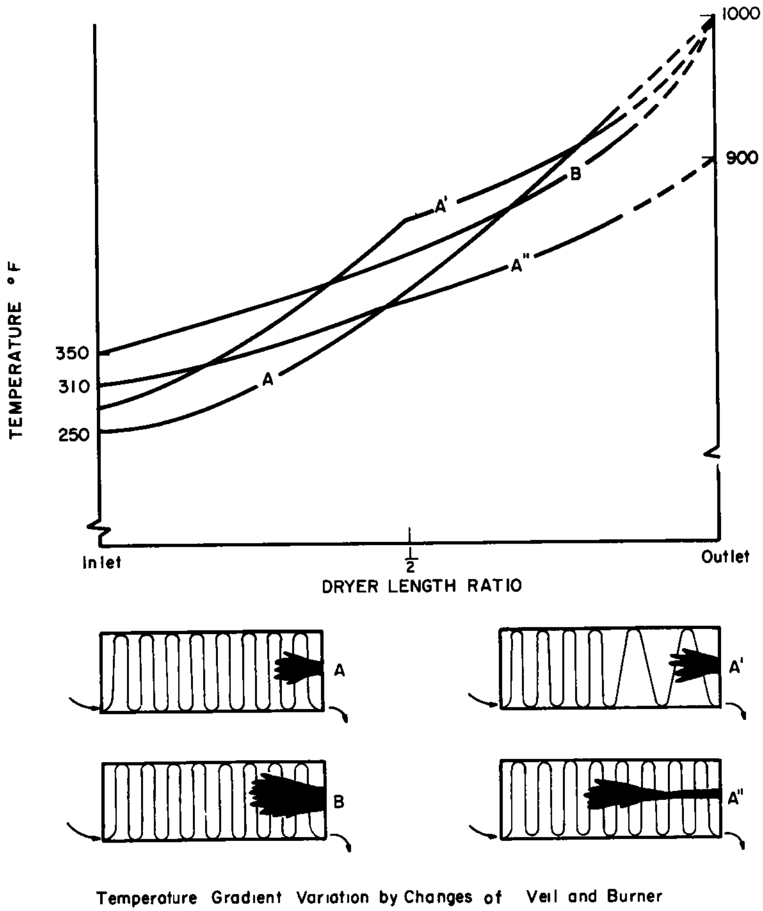

Figure 2. Dryer length vs effective temperature.

temperatures, time durations, and rates of moisture loss are related. Although their data relate to moisture loss, aggregate heating dependency on drying cycles is implied. In a later study, Deming (6) found aggregate heat absorption rate was related to the drying cycle.

Prototype dryer studies were later performed to evaluate aggregate characteristics, temperatures, and exposure times. Smeins (5) found changes in initial saturation levels produced different heating rates for aggregates. When varying initial moisture content, Smeins found greater aggregate discharge temperatures for lower initial moisture contents. The discharge temperatures of porous aggregates were found to be highly dependent on initial saturation. Also, for initial saturations over 50 percent, gravel aggregate was discharged from the dryer at greater temperatures than other aggregate types.

Smeins found also in the prototype that greater percentages of sand in the dryer aggregate lowered the temperature of the aggregate at discharge. He found the temperature of the aggregate combination dropped from 500 to 250 F when the percentage of sand increased from 25 to 75. This was due to the increased moisture to be removed and the greater veil density or resistance to heat flow from the increased surface area. The following are three implications derived from Smeins' study:

1. For heating a given aggregate, a greater initial moisture content will decrease its heat absorption rate regardless of the particular drying cycle phase under consideration.
2. The location of moisture content, surface or interior water, may be an important factor in aggregate heating.
3. For a given dryer set-up, a desirable gradation range may exist for maximum heating outside of which excessive aggregate surface areas or diameters produce less than maximum heating rates.

Lovejoy (2) checked prototype drum temperatures under varying air flow, burner rates, slopes, and feed rates. Drum temperatures were measured by thermocouples at points that divided the drum length into three equal parts. He found the ⅓-point temperature gradients usually reflected thermal efficiency. Relatively small temperature differentials between the ⅓-points proved efficient operation, although moisture removed and discharge temperatures were low. The second ⅓-point temperature varied more than the first with different dryer settings. As a result, the second ⅓-point temperature usually indicated aggregate discharge temperature. The discharge temperatures in turn were found to be directly proportional to the percentage of moisture removed. Providing drum temperatures reflect aggregate-hot gas temperatures along the dryer, one implication for current rotary dryer operation is that greater thermal efficiencies can be obtained when aggregate discharge temperature is decreased by proper dryer operational changes.

Exposure time was found to be important for utilizing available burner heat to heat and dry aggregate. Both Uehling (4) and Lovejoy (2) found greater exposure times, produced by flatter prototype drum slopes, resulted in greater thermal efficiencies. With a constant burner rate, their data implied increased thermal efficiency with increases in aggregate exposure times and weights in the drum.

The temperature-size distribution within aggregate samples at dryer discharge was observed by Uehling (4) and Middleton (3). Although temperature variation was apparent when measuring several samples from each size group, the over-all observation was that groups of the larger sizes were usually cooler than those with the smaller ones. Maximum differentials of 110 F from the 2-in. range to sand were found. Both investigators concluded that temperature differentials will usually exist in the discharged aggregate whenever aggregates of different sizes are heated.

The past conclusions imply aggregate size, type, and moisture content are several of the primary factors that determine rate of heat absorption. When considering the effects of exposure times and temperatures on aggregates, the conclusions indicate an aggregate heating solution is possible.

In conduction, convection, or radiation, the rate of heating is represented by a heat absorption coefficient. Aggregate heating involves all three methods of heat transfer. Aggregate size, type, and moisture characteristics, the limitations of instrumentation, and the critical sensitivity of thermal measurements needed to determine heat coefficients are difficult to collect and apply to the aggregate heating problem. As a result, a laboratory oven heating test has been used to determine over-all heat absorption rate coefficients. To some extent heat absorption plots describe aggregate ability to heat; they can be used to predict and analyze the heating of aggregates in asphalt plant dryers.

The study has been undertaken as a practical investigation of aggregate heating characteristics for use in field conditions. Aggregate temperature was evaluated in terms of generalized analytical approaches to dryer-aggregate combinations. Several quantitative solutions and heating considerations were attempted to predict aggregate temperature on the basis of fundamental characteristics of aggregate and dryers. These fundamental characteristics or variables are (a) aggregate heat absorption rate, (b) dryer temperature gradient, and (c) dryer exposure time.

The method used in this study includes a laboratory heating test to identify aggregate heating characteristics and a field test to measure exposure times and temperature gradients. These variables are then used as the factors for predicting aggregate temperature. Also, they may serve as a future basis for dryer efficiency and design.

GENERAL PROCEDURE

Laboratory Heating Test

The test work was based on the assumption that a heating test for aggregates could combine all the important aggregate variables. Test results would then represent quantitatively heat transfer during the drying cycle. A preliminary investigation of drying cycle changes during heating added two other variables to the test—oven temperature and initial saturation. The variables considered in the aggregate heat absorption experiments were (a) drying cycle phases, (b) oven temperature, and (c) initial aggregate saturation.

The following steps were used in the laboratory procedure:

1. Samples of stockpile aggregates representing each hot-bin size range were obtained for a laboratory oven-heating test. The moisture content of each aggregate size range was determined as it existed in the stockpiles.
2. Samples were heated to oven temperature. The "dry" specific heat values of the aggregates were then determined by a calorimeter.
3. Drying curves were plotted at constant temperature for each aggregate size at its stockpile moisture content.
4. Phases of the drying cycles were found and the corresponding oven times recorded. The oven temperature (300 F+) was also recorded.
5. Drying cycles were repeated for each aggregate size. The cycles were interrupted at the terminal point of each phase.
6. The aggregates were immediately placed in a calorimeter and a temperature rise noted. The moisture remaining in the aggregates at each terminal point was determined from the drying cycle and used to compute the "wet" specific heat of the aggregates. Heat absorption rates (Btu/min) were then calculated from heat exchange laws.
7. When heat absorption rates for all aggregate sizes were calculated at each phase of the drying cycle, they were plotted with oven temperature on a semi-log scale.

The data from laboratory heating tests are plotted in Figures 3, 4, and 5. In general, there is a straight-line relationship for Btu/min absorption and oven temperature at each drying cycle phase. Also, at lower oven temperatures sand heat absorption is less than stone heat absorption at each drying cycle phase. As oven temperature increases, the sand heat absorption rates approach the stone rates. As aggregates dry, heat absorption rates generally lower.

It is hypothesized that all heat-drying cycle lines form a dry aggregate temperature curve above the working temperature of the oven. The temperature at which this occurs

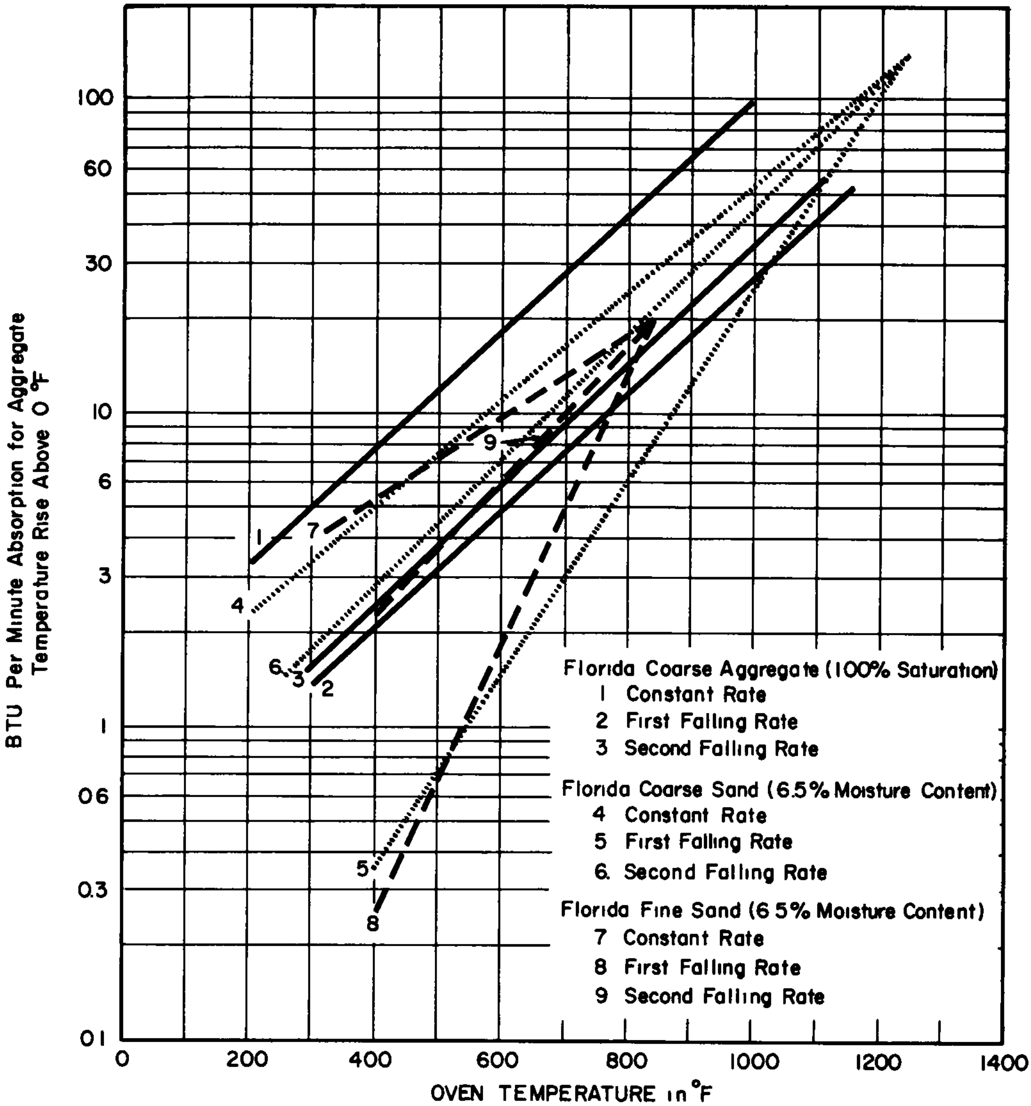

Figure 3. Oven temperature vs heat absorption for Lakeland, Fla., limestone.

apparently depends on aggregate size. For example, in Figure 3 fine sand lines join at 850 F, coarse sand at 1,250 F, and coarse aggregate probably at a much higher temperature outside the graph. At the temperatures beyond which aggregate drying lines join, heat absorption of wet aggregates may be independent of the drying cycle. In practice, these temperatures may never be reached, at least in the first portion of the dryer.

Figures 4 and 5 are heat absorption plots of a coarse limestone and a slag sand. In these figures the initial moisture content or saturation was varied. The limestone plot shows that some initial moisture (60 percent saturation) is required for the greatest heat absorption rate in each drying period. Observations of the slag sand plot show that initial moisture has a variable effect on heat absorption rate. At the lower oven temperatures, the greatest heat absorption rates occur in the constant rate periods (where most aggregate moisture exists during drying).

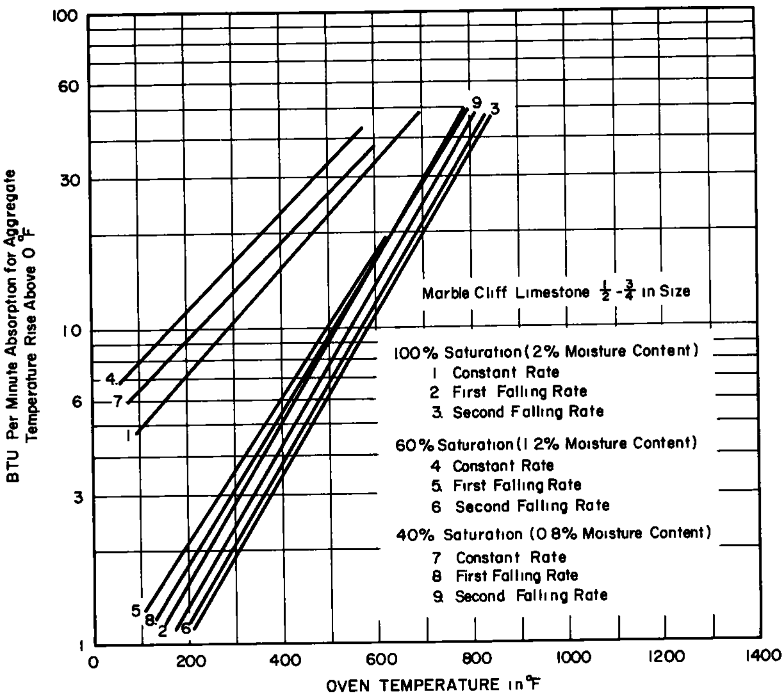

Figure 4. Oven temperature vs heat absorption for Marble Cliff limestone.

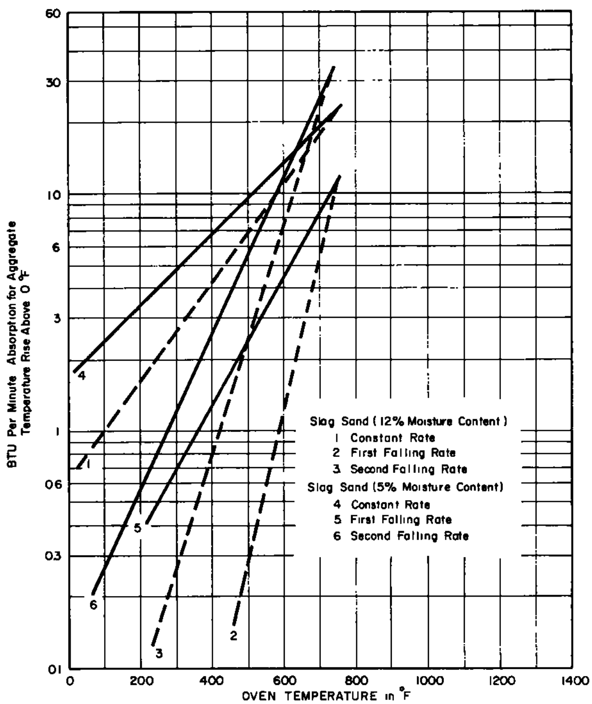

Figure 5. Oven temperature vs heat absorption for slag sand.

Field Test

Laboratory tests showed that drying phase and exposure temperature will determine aggregate heat absorption rates. Heat absorption is found from drying phase, exposure temperature, and aggregate exposure time at asphalt plant dryers. The field measurement of these three variables is described generally by the following procedure:

1. The average exposure time for each aggregate size is found by use of colored aggregate tracers representing each size.
2. Temperature measurements of the dryer exterior are taken at $\frac{1}{8}$-points from entrance to discharge. The profile of drum temperature indicates the aggregate mass heating curve and the drying cycle. Stack temperature, gas velocity and burner rates are recorded. Dryer heat balance or aggregate moisture loss tests will give gas temperature gradient along the drum interior.
3. It is assumed the temperature profile shows how much of dryer is taken up by each drying cycle phase.
4. The temperature gradient gives the exposure temperature for each drying phase.

Application of Lab and Field Test to Determine Aggregate Discharge Temperature

The application of aggregate heat absorption rates, exposure times, and temperature gradients is outlined in the following procedure:

1. Using each aggregate size's average exposure time, and assuming this exposure time is a uniform summation of exposure times along the dryer, the exposure time for each drying cycle phase can be calculated using the drum temperature profile.
2. With the laboratory heat absorption rates (Btu/min) for each drying phase, exposure time, and average gas temperature for each drying cycle phase, the aggregate heat absorption for each drying cycle phase is calculated. Total heat absorption, or Btu rise, for the given aggregate size is determined. The discharge temperature is then found for this aggregate size. (This step is repeated for the other aggregate sizes, to determine their temperature rise.)

Details of the method are outlined in the Appendix. Computations of aggregate heating characteristics, temperature gradients by both heat balance and average moisture loss methods, and exposure times for the drying zones are included for the Ohio State prototype dryer and for a field dryer. The heat balance method was not used for the field dryer due to the absence of burner-rate data.

Average values of exposure time, temperature gradient, and moisture content are used in the Appendix. The effects of their variations on aggregate discharge temperature and moisture content can be determined by the Appendix method when replacing average values by the variation extremes of each value. For greater practical meaning, statistical data of these variations could find the probability of different levels of each value. These levels could be used to find the normal or expected distributions of discharge temperature and moisture content.

Calorimeter Device

Temperature measurements of aggregates during drying were necessary in this study. Earlier cooperative studies at Ohio State with the Bureau of Public Roads made use of the calorimeter principle when special water containers and thermometers were used. Their accuracy was greater than thermometers—especially when finding coarse aggregate temperature. Rapid temperature measurement is needed to hold aggregate cooling effects to a minimum during drying sampling. The calorimeter principle was a significant improvement over other thermal sensing principles.

Subsequent development led to the design of the calorimeter used in this study. It is illustrated with a 2-lb sample of coarse aggregate in Figure 6. It utilizes the principle of heat exchange between aggregate and water. When a sample is placed in the screen basket (suspended inside the casing) and rotated by means of the crank handle, heat transfer occurs between aggregates and water. The tumbling action

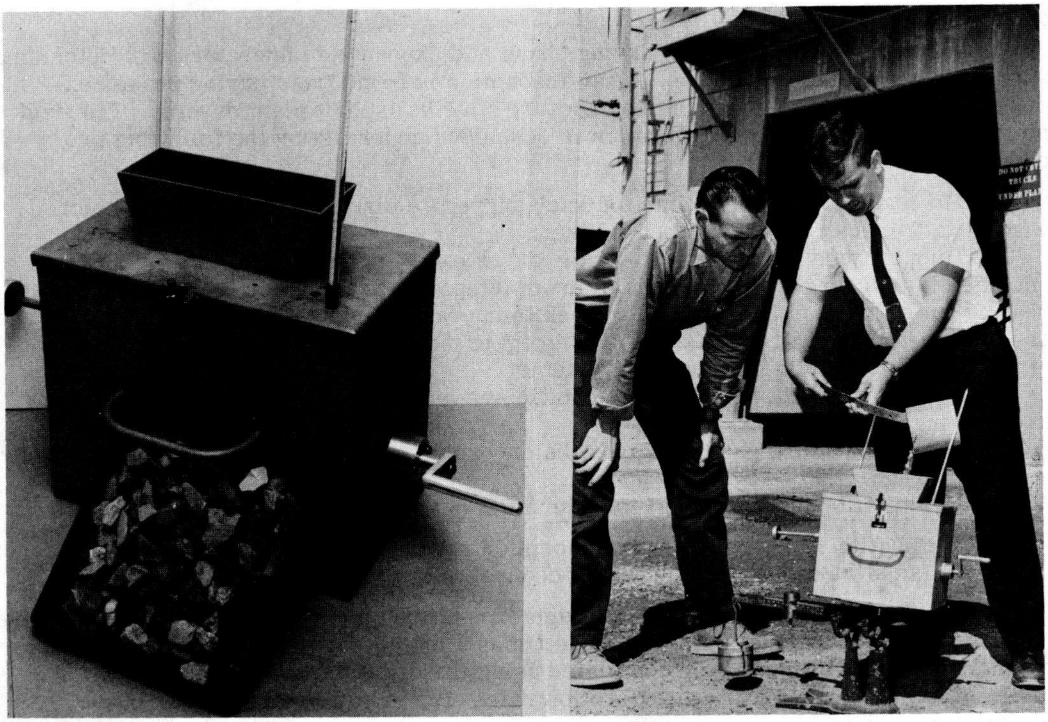

Figure 6. Aggregate calorimeter.

causes maximum surface exposure to churning water. In 20 to 30 sec, equilibrium temperature is reached. Initial aggregate temperature is calculated from water and aggregate weights, initial water temperature, final water temperature, and specific heat of the two media. Specific heat of aggregate can also be calculated by a similar procedure when initial aggregate temperature is known. This can be established by several hours of oven heating at a constant temperature. In addition, the calorimeter measures heat input to aggregate. This is usually measured during drying; the procedure is the same as aggregate temperature determination. The details of the laboratory heating test described previously refer to the use of this device.

Aggregate Heating Considerations

The method for predicting aggregate temperature can be applied to the evaluation of dryer variables. There are many variables that influence these three primary variables. The following is a partial list of variables and the primary variable they influence:

1. Aggregate heat absorption.—initial moisture content, saturation, pore, structure, solid thermal characteristics, size.
2. Exposure time.-dryer slope, length, rotation, aggregate size.
3. Temperature gradient.—burner rate, flame position, fuel type, veil density.

With the method used in this report, the effects of aggregate heat absorption, exposure time, and temperature gradient appear to have equal influence on aggregate temperature. Figure 7 shows the extremes of presently available data. When seeking aggregate temperature, Figure 7 suggests equal emphasis be given all three variables. Temperature gradients and exposure times may need to be established for each aggregate type. Undoubtedly there are combinations that can minimize excessive heat losses, aggregate temperature differentials, discharge moisture contents, and low production rates.

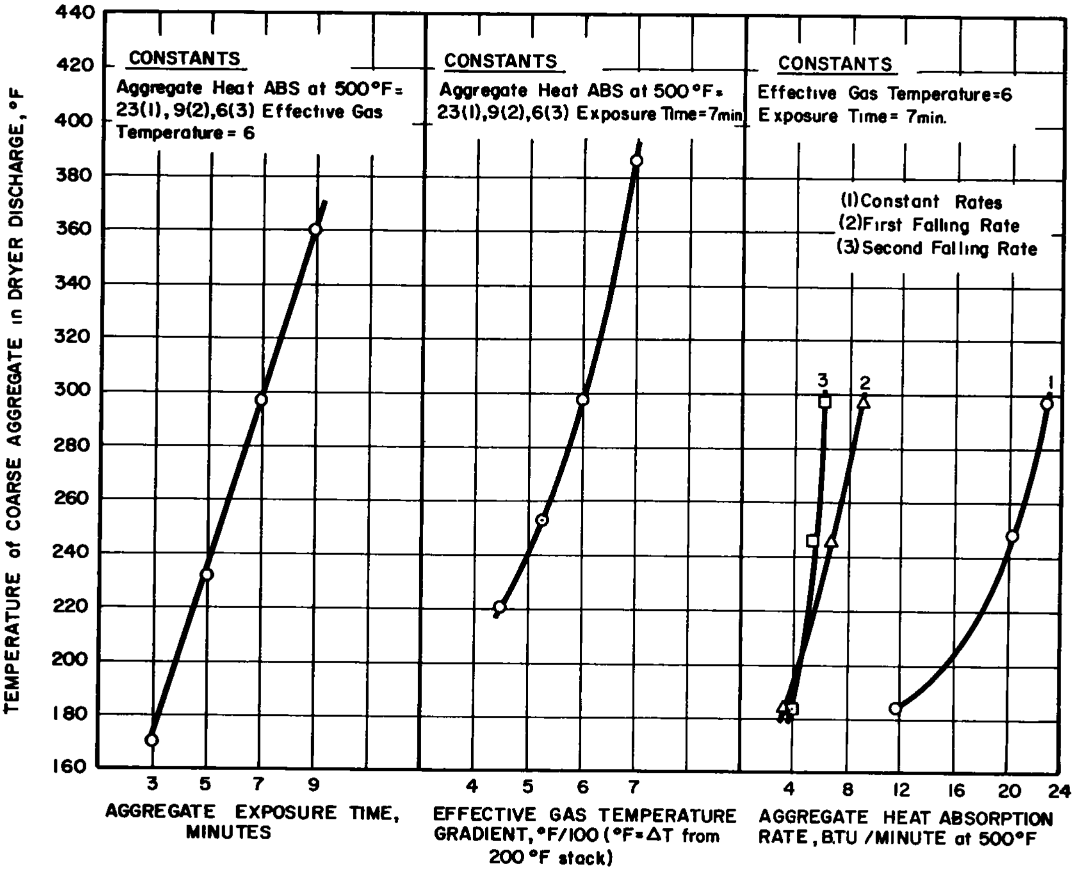

Figure 7. Effect of variables on coarse aggregate temperature.

The effect of temperature gradient and exposure time on aggregate discharge temperature for Marble Cliff limestone is shown in Figure 8. These plots were based on heat absorption curves using the three-value method, and for a constant aggregate weight in the drum. Figure 8 shows that high temperature gradients are required for aggregate temperatures of 300 F when exposure times are relatively small. Slopes of gradients show that changes of exposure time produce greater changes of aggregate temperature on high temperature gradients than on low temperature gradients. In Figure 8 the data indicate very high temperature gradients produce very high production rates. However, exposure time must be tightly controlled at high temperature gradients.

When considering thermal efficiency, past data on rotary dryers show long exposure times and low temperature gradients produce efficient conditions. The selection of economical combinations of variables is limited for present dryers. Exposure time controls and burner capacity should be considered with thermal efficiency when setting production rates.

Data from Figure 8 were re-plotted for Figure 9. Combinations of temperature gradients and exposure times were selected for each aggregate temperature. Plots obtained are a family of curves from which temperature gradients and exposure times can be found for the coarse aggregate temperature desired. As exposure time approaches zero, the required temperature gradient for the aggregate temperature approaches infinity. For aggregate temperatures over 200 F, exposure times approach infinity when temperature gradients of zero are used. This is based on a steady stack temperature of 200 F.

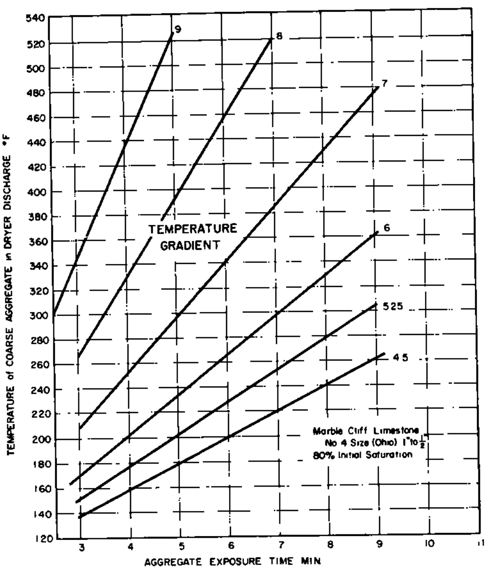

Figure 8. Effect of temperature gradient and exposure time for coarse aggregate discharge temperature.

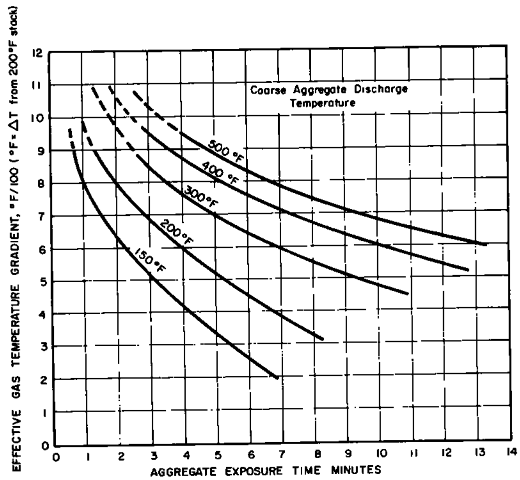

Figure 9. Effective temperature gradient vs exposure time for coarse aggregate discharge temperature.

A smaller increase in temperature gradient is required to increase already high aggregate temperatures at a given exposure time. However, when decreasing exposure time at a uniform rate, a greater rate of increase in temperature gradient is required to give the same aggregate discharge temperature.

The validity of the method used to aid in dryer evaluation requires extensive additional laboratory and field data. The preliminary studies show that the approach has merit; definite data required for dryer evaluation can be stipulated. This will be of great value to the consumer, producer, and research interest. The method will eventually allow selection of an economical combination of dryer-aggregate variables.

CONCLUSIONS

1. Aggregate temperature can be found by combining dryer exposure time, dryer temperature gradient, and aggregate heat absorption rate when aggregate moisture loss is known. Similarly, aggregate moisture loss can be found from aggregate discharge temperature under these conditions.
2. Many combinations of temperature gradients and exposure times for a given aggregate result in the same aggregate temperature.
3. Aggregate heat absorption rates change during drying for a given aggregate and initial moisture content.
4. For the same drying phase, different aggregates at the same initial saturation level have different heat absorption rates. Similarly, the same aggregates at different initial saturations have different heat absorption rates.
5. For some porous aggregates, initial moisture may increase heat absorption rates during the early stages of aggregate drying when aggregate temperatures are less than 300 F.
6. Aggregate heating comparisons can be found from the heat absorption test when changing aggregate type and/or initial moisture.

Appendix

DETAILS OF LAB AND FIELD TEST TO DETERMINE AGGREGATE DISCHARGE TEMPERATURE

A. Laboratory Heating Test for Aggregate Heat Absorption Rate

The general procedure for these tests has been outlined. The following, including Tables 1, 2, and 3, is a sample calculation of aggregate heat absorption values.

Aggregate:	Lakeland, Fla., limestone
Size:	$\frac{3}{4}$ - $\frac{1}{2}$ in.
Initial Saturation:	100 percent
Initial Moisture Content:	4.5 percent
Oven Temperature:	760 F

TABLE 1

CALORIMETER TEST DATA

Drying Cycle Interruption (min)	Wt. of Cal. + Water + Agg. (lb)	Wt. of Cal. + Water (lb)	Wt. of Cal. (lb)	Wt. of Agg.+ Water (lb)	Wt. of Water in Cal. (lb)	Temperature Initial (°C)	Temperature Final (°C)
1 ½	33.45	31.43	9.00	2.02	22.43	13.45	14.75
						13.30	14.60
4	33.86	31.85	9.00	2.01	22.85	14.20	16.70
						14.00	16.40
6	33.74	31.83	9.00	1.91	22.83	16.50	19.20
						16.25	18.95
10 ½	33.69	31.86	9.00	1.83	22.86	20.90	25.70
						20.60	25.45
15	33.63	31.73	9.00	1.90	22.73	19.00	25.35
						18.75	25.10

Figure 10 shows divisions of the drying cycle curve into the various zones for computing heat absorptions during drying.

TABLE 2

MOISTURE IN AGGREGATE AT POINTS OF INTERRUPTION IN DRYING CYCLE

Time (min)	Moisture per Unit Wt.	Aggregate Moisture (lb)
1 ½	41.74 × 0.0008 = 0.03339	0.03339 × 2.02 = 0.0674
4	22.99 × 0.0008 = 0.01839	0.01839 × 2.01 = 0.0370
6	11.64 × 0.0008 = 0.00931	0.00931 × 1.91 = 0.0178
10 ½	1.54 × 0.0008 = 0.00123	0.00123 × 1.83 = 0.0023
15	0	0

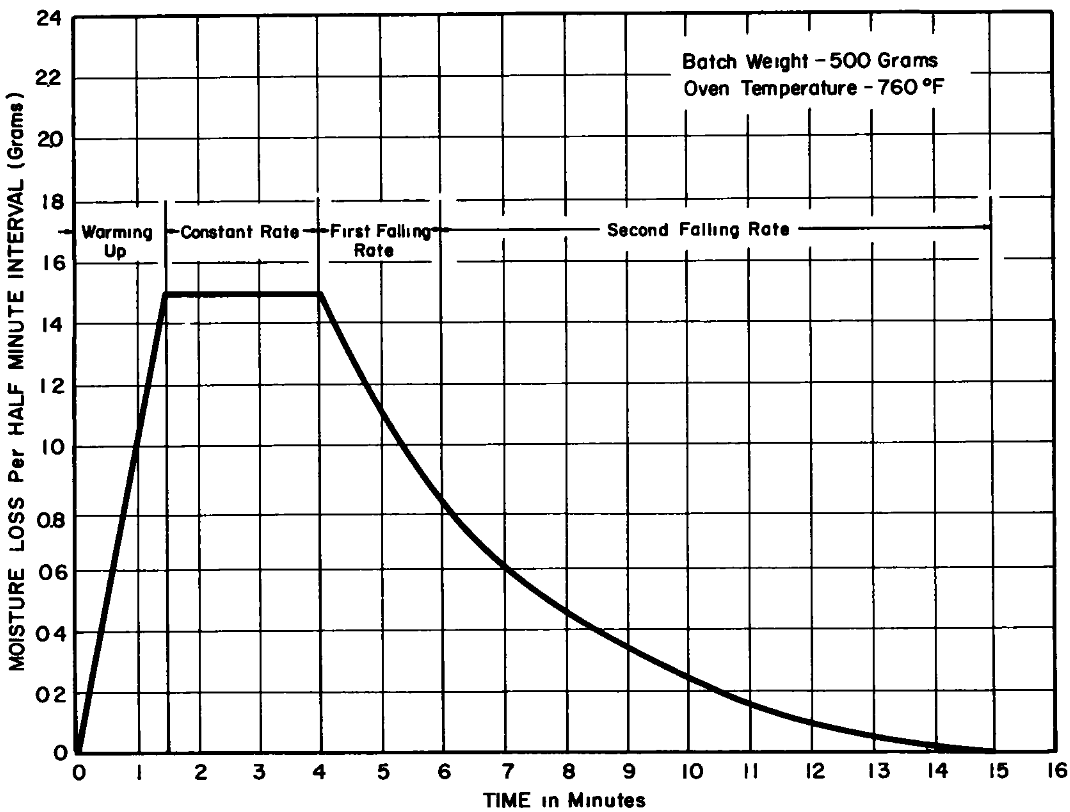

Figure 10. Aggregate moisture loss vs oven exposure time.

TABLE 3

BTU COMPUTATIONS

Time (min)	Wt. of Water in Cal. (lb)	T_f (°F)	T_i (°F)	W_{agg} + W_{moist}	W_{moist} (in Agg)	W_{agg} Dry	C_{Pagg}	Btu	Btu per Min. Last Reading	Btu per Min. Avg.
1 ½	22.43	58.41	56.07	2.02	0.0674	1.9526	0.218	81.29	54.19	
4	22.85	61.79	57.38	2.01	0.0370	1.9730	0.218	129.00	19.34	36.76(C.R.)
6	22.83	66.33	61.92	1.91	0.0178	1.8922	0.218	130.00	0.50	9.92(F.F.R.)
10 ½	22.86	78.03	69.35	1.83	0.0023	1.8277	0.218	229.69	22.33	12.05(S.F.R.)
15	22.73	77.40	65.97	1.90	0	1.9000	0.218	291.86	13.82	

The following formula is used for Btu absorption:

$$Btu = W_{w-cal}(T_f - T_i) + T_f(C_{Pagg} \times W_{agg} + C_{pw} \times W_{w-agg}) \tag{1}$$

in which

W_{w-cal} = weight of water in calorimeter (lb);
W_{agg} = weight of dry aggregate (lb);
W_{w-agg} = weight of moisture in aggregate (lb);
T_f = temperature, final of water in calorimeter (°F);

T_i = temperature, initial of water in calorimeter (°F);
C^1_{Pagg} = specific heat of dry aggregate; and
C_{Pw} = specific heat of water (assume unity).

Eq. 1 determines one point on each of the constant rate, first falling rate, and second falling rate period curves. One additional set of computations based on calorimeter test data at a different oven temperature determines the position of the CR, FFR, and SFR curves. These curves are shown in Figure 3.

Additional curves at different moisture contents for different aggregates (coarse and fine) are shown in Figures 4 and 5. The following examples will use the curves shown in Figure 4 for Marble Cliff limestone.

B. Computation of Temperature Gradient in Dryers

1. Prototype Dryer

(a) Moisture Method

Data:

Mix proportion	40 percent sand
	60 percent No. 46 stone
Moisture before	4.94 percent sand
	1.28 percent No. 46 stone
Moisture after	0.09 percent combination

Average exposure time in dryer for stone and sand = 3.7 min.

Drying curves for 60 percent saturated (1.28 percent) Marble Cliff limestone at 300, 400, and 550 F are obtained (see Fig. 4). Sample weight = 500 g.

Drying curves for sand with 4.94 percent moisture content at 400, 600, and 760 F can also be obtained. Sample weight = 500 g.

Temperature gradient by the moisture method is shown in the following steps:

Step 1. To find actual moisture removed in the average exposure time of 3.7 min.

Assuming a total batch weight of stone + sand = 500 g, stone = 0.6 × 500 = 300 g, and sand = 0.4 × 500 = 200 g, then initial moisture in stone 1.28/100 × 300 g = 3.84 g and initial moisture in sand 4.94/100 × 200 g = 9.88g, with an initial total = 13.72g. Final moisture (stone + sand) 0.09/100 × 500 g = 0.45 g, with moisture removed = 13.27 g.

Step 2. To find average dryer gas temperature.

Figure 11 shows the drying curves for the stone at temperatures 300, 400, and 550 F. Figure 12 shows the drying curves for the sand at temperatures 400, 600, and 760 F. Area under each curve gives moisture, in grams, removed per 500 g of aggregate. This area for each curve up to 3.7 min is given in Table 4.

These values are plotted into a graph (Fig. 13) of moisture removed against temperature. Values for sand for exposure time of 3.7 min (see Table 5) are also plotted in Figure 13.

TABLE 4

AREA UNDER CURVES IN FIGURES 11, 12, AND 13 FOR EXPOSURE TIME OF 3.7 MIN

Temp. (°F)	Area			
	No. 46 Stone		Sand	
	Per 500 G	Per 300 G[a]	Per 500 G	Per 200 G[a]
300	1.548	0.93		
400	2.31	1.39	5.56	2.22
550	3.33	2.00		
600			23.22	9.29
760			32.86	13.14

[a]By proportion.

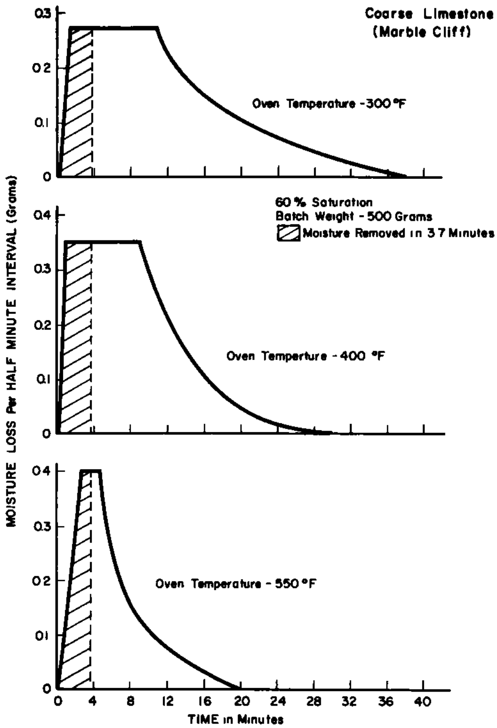

Figure 11. Coarse aggregate drying curves.

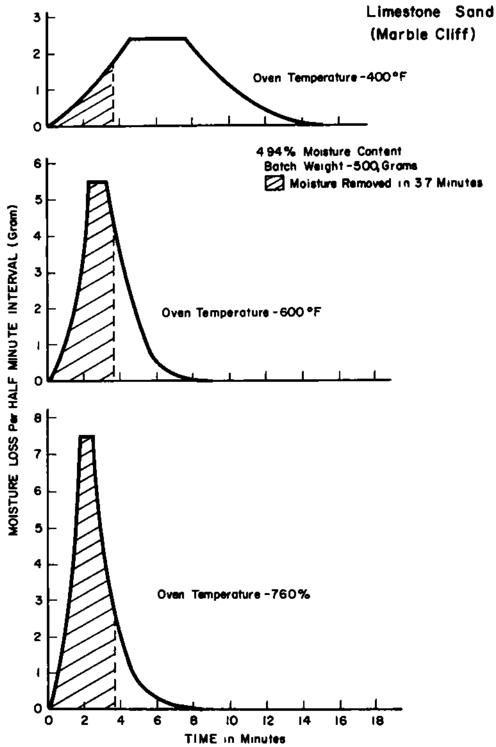

Figure 12. Fine aggregate drying curves.

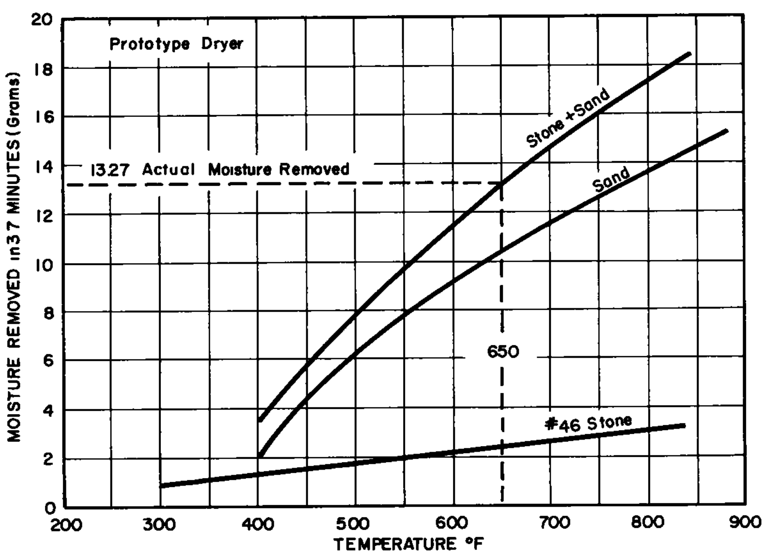

Figure 13. Oven temperature vs moisture loss.

The preceding graphs are compounded to obtain the curve of total moisture removed (in 3.7 min. from 500 g of stone plus sand) against effective dryer gas temperature. On this curve, the temperature corresponding to 13.27 g (actual moisture removed) is 650 F. This is an estimate of the average dryer gas temperature.

Step 3. To find the point along dryer where average drum temperature exists.

This point can be assumed to be located by the use of the drum shell temperature gradient. A vertical line through this point divides the area under the gradient curve into two equal parts. In this example, the average drum temperature point will be assumed to exist at 0.67L. This point also determines the location of the average dryer gas temperature. The location of this point P with coordinates (0.67L, 650 F) on a plot of effective dryer gas temperature is shown in Figure 14. A line is extended both ways from P to the stack temperature (260 F). This line indicates the temperature gradient of the dryer gases.

(b) Dryer Zone—Heat Balance Method as Shown by Parr (7)

Data:

Fuel gas consumption per hour at atmospheric pressure and 60 F = 12.50 cu ft.

Heat value of fuel gas	= 1,200 Btu per cu ft at 60 F
Rate of feed of aggregate	= 85 lb per min
Initial moisture content	= 5.30 percent
Final moisture content	= 0.75 percent
Air flow through the dryer	= 1,245 cfm
Stack gas temperature	= 250 F
Ambient temperature	= 75 F

Assumption:

1 mol of fuel gas	= 379 cu ft at 60 F
1 mol of water	= 18 lb
1 mol of stack gas	= 359 cu ft at 32 F

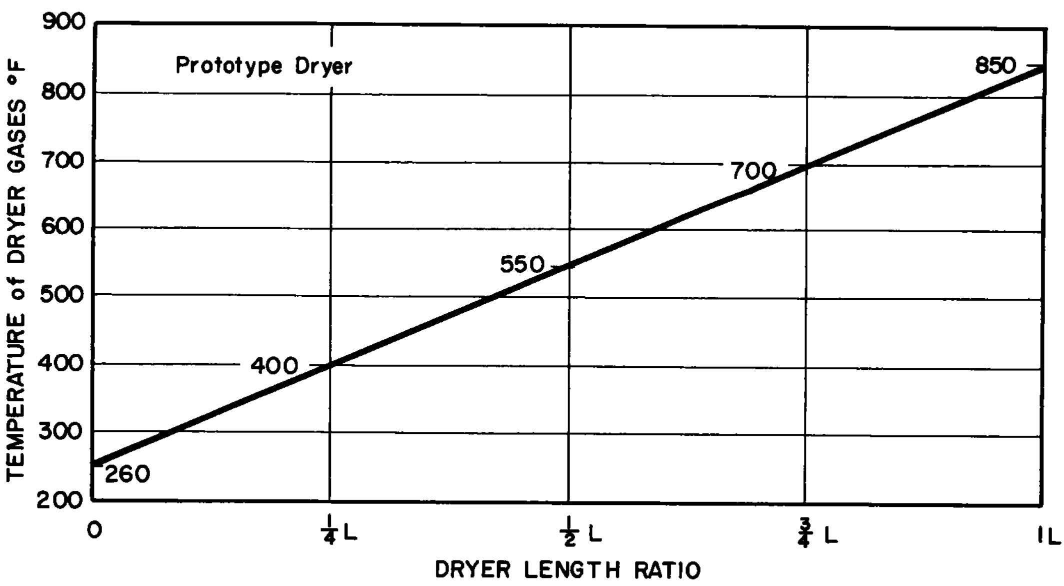

Figure 14. Prototype dryer temperature vs dryer length ratio—moisture method.

For 1 mol of fuel gas:

1.995 mol O_2 required for combustion
1.022 mol CO_2 formed
1.952 mol H_2O formed
0.065 mol N_2 formed (from fuel gas)
7.50 mol N_2 formed (from combustion air)

Specific heat of stack gas analysis:

CO_2 = 9.7 Btu per mol per F
H_2O = 8.1 Btu per mol per F
O_2 = 7.2 Btu per mol per F
N_2 = 7.0 Btu per mol per F
Air = 7.04 Btu per mol per F

The assumption for first trial that over-all aggregate temperature at discharge is 350 F is subject to correction, if necessary, after the temperatures of the coarse and fine aggregate are individually determined. The solution is as follows: Fuel gas burned per hour = 12.50 × 60 = 750 cu ft at 60 F = 750/379 = 1.97 mol.

For 1.97 mol of fuel gas per hr:

1.97 × 1.995	=	3.93	mol O_2 required per hour for combustion.
1.97 × 1.022	=	2.01	mol CO_2 formed per hour
1.97 × 1.952	=	3.84	mol H_2O formed per hour
1.97 × 0.065	=	0.13	mol N_2 formed per hour (from gas fuel)
1.97 × 7.50	=	14.80	mol N_2 formed per hour (from combustion air)
		20.78	mol combustion gases formed per hour

Aggregate discharged = 85 × 60 = 5,100 lb per hr; and water evaporated = (5.30 - 0.75)/100 × 5,100 = 232 lb per hr = 232/18; i.e., 12.9 mol per hr. Total computed stack gas = 20.78 + 12.9 = 33.7 mol per hr. Stack gas temp. = 250 F = 710 F Abs. and 1 mol of stack gas = 359 cu ft at 32 F = 519 cu ft at 250 F.

Total computed stack gas volume = 519 × 33.7 = 17,460 cu ft per hr at 250 F
Actual stack gas volume = 1,245 × 60 = 74,700 cu ft per hr
Excess air in stack gas = 74,700 - 17,460 = 57,240 cu ft per hr
Actual air required for combustion = 14.80 + 3.93 = 18.73 mol per hr
= 18.73 × 519 = 9,700 cu ft per hr
Excess air percentage = 57,240/9,700 = 590 percent
Stack gas analysis:

CO_2		= 2.01	mol per hr
H_2O	= 12.9 + 3.84	= 16.74	mol per hr
N_2	= 14.80 + 0.13	= 14.93	mol per hr
Excess Air	$= \frac{57,240}{519}$	= 110.30	mol per hr
		143.98	mol per hr

Total air in = 110.3 + 18.73 = 129.03 mol per hr at 75 F

Heat Balance (Datum = 60 F):

Heat in

Heat in fuel = 750 × 12,000	=	900,000 Btu per hr
Heat in aggregate = 5,100 × 0.2 × (75 - 60)	=	15,300 Btu per hr
Heat in moisture = 232 × 1.0 × (75 - 60)	=	3,485 Btu per hr
Heat in air to dryer = 129.03 (7.04)(75 - 60)	=	13,630 Btu per hr
Total heat in		932,415 Btu per hr

Heat out

Heat in aggregate	$= 5,100 \times 0.2 \times (350-60) =$	296,000 Btu per hr
Heat in water	$= 16.74 \times 18 \times (1,164-28) =$	342,000 Btu per hr
Heat in CO_2	$= 2.01 \times 9.7 \times (250-60) =$	3,700 Btu per hr
Heat in N_2	$= 14.93 \times 7.0 \times (250-60) =$	19,850 Btu per hr
Heat in excess air	$= 110.3 \times 7.04 \times (250-60) =$	147,300 Btu per hr
Total heat out		808,850 Btu per hr

Heat loss = 932,415-808,850 = 123,565 Btu per hr or 125,000 Btu per hr

The dryer is divided into three zones to correspond approximately to the three stages in the drying cycle for constant rate, first falling rate and second falling rate periods. In this example 0 to ½ L, ½ L to ¾ L and ¾ L to L are suitably assumed as the three zones.

The heat loss of 125,000 Btu per hr at 25 percent in Zone I, 50 percent in Zone II, and 25 percent in Zone III is allocated. As Zone II aggregate temperature of 210 F is assumed. Aggregate feed = 5,100 lb per hr. Aggregate moisture = 232.0 lb per hr. Stack gas temperature = 250 F, over-all aggregate discharge temperature = 350 F, room temperature = 75 F. Stack gas = 143.98 mol per hr.

Zone I

Heat absorbed by aggregate $= 5,100 \times 0.2 \times (210-75)$	= 137,500 Btu per hr
Heat absorbed by moisture $= 323 \times 1 \times (210-75)$	= 31,300 Btu per hr
Heat loss = ¼ (125,000)	= 31,250 Btu per hr
	200,050 Btu per hr

Heat balance for Zone I

$$200,050 = 143.93 \times (7.04) \times \Delta T$$

$$\Delta T = 198 \text{ F}$$

Stack gas temperature = 250 F

ΔT = 198 F

Temperature of dryer gases at interface of Zones I and II = 448 F

Zone II

Assume 80 percent moisture evaporation in Zone II

Moisture evaporated $= 0.80 \times 232.0$	= 186 lb per hr
Total heat of saturated steam at 212 F, 14.7 psi	= 1,150 Btu per lb
Total heat in water at 210 F	= 178 Btu per lb
Heat absorbed in Zone II by moisture	= 972 Btu per lb
Heat absorbed by moisture $= 186 \times 972$	= 180,500 Btu per lb
Heat loss = ½ (125,000)	= 62,500 Btu per lb
	243,000 Btu per lb

Heat balance for Zone II

$$243,000 = 134 \times (7.04) \times \Delta T$$

$$\Delta T = 258 \text{ F}$$

Temperature of dryer gases at interface of Zones I and II = 448 F

ΔT = 258 F

Temperature of dryer gases at interface of Zones II and III = 706 F

Zone III

Moisture evaporated $= 0.20 \times 232.0$	= 46 lb per hr
Heat absorbed by moisture $= 46 \times 972$	= 44,700 Btu per hr
Heat absorbed by aggregate $= 5,100 \times 0.2 \times (350-210)$	= 143,000 Btu per hr
Heat loss = ¼ (125,000)	= 31,250 Btu per hr
	218,950 Btu per hr

Heat balance for Zone III

$$218{,}950 = 131.5 \times (7.04) \times \Delta T$$

$$\Delta T = 236 \text{ F}$$

Temperature of dryer gases at interface of Zones II and III	= 706 F
	ΔT = 236 F
Temperature of dryer gases at burner	= 942 F

The temperature gradient is plotted for each zone interface temperature in Figure 15.

2. Dryer No. 2 at Marble Cliff Quarries Co. by Moisture Method

Data:	Mix proportion	40 percent sand
		40 percent No. 6 stone
		15 percent No. 4 stone
		5 percent asphalt
	Moisture before	6.16 percent sand
		1.81 percent No. 46 stone
		0.18 percent combination

Average exposure time in dryer for stone and sand = 7.9 min.
Recorded drum temperature:

Stack end	= 175 F
1/8 L	= 175-200 F
1/4 L	= 175-200 F
3/8 L	= 175-200 F
1/2 L	= 200-225 F
5/8 L	= 250-275 F
3/4 L	= 400-450 F
7/8 L	= 550-600 F
Burner end	= 550-600 F

Drying curves for 60 and 100 percent saturated Marble Cliff limestone at 300, 400, and 550 F, are known (Fig. 4). Sample weight = 500 g. Drying curves for sand with 6.5 percent moisture content at 400, 600, and 760 F, are known. Sample weight = 500 g. The initial moisture content of stone (1.81 percent) was found to correspond to 80 percent saturation. Hence, data for 80 percent saturation are found by interpolation from known data of 60 and 100 percent saturations.

The solution is given by the following steps:

Step 1. To find actual moisture removed in the average exposure time of 7.9 min.

It is assumed that a total sample weight of stone + sand = 500 g. Stone = 55/95 $\times$ 500 = 290 g and sand = 40/95 $\times$ 500 = 210 g. Initial moisture in stone 1.81/100 $\times$ 290 = 5.25 g and initial moisture in sand 6.16/100 $\times$ 210 = 12.94 g. Therefore, total initial moisture = 18.19 g and total final moisture (stone and sand) = 0.18/100 $\times$ 500 = 0.90. Moisture removed = 17.29 g.

Step 2. To find average dryer gas temperature. Area under drying curve gives moisture, in grams, removed per 500 g of aggregate. This area, for the relevant curves up to 7.9 min, is given in Table 5.

The figures for moisture removed in 7.9 min for 290 g of stone and 210 g of sand are shown in Figure 16. The two plots are compounded to obtain curve for total moisture removed (from stone + sand). From this compounded curve the temperature corresponding to 17.29 g (actual moisture removed) is 590 F. This is assumed to be the average dryer gas temperature.

TABLE 5

AREA UNDER DRYING CURVE FOR EXPOSURE TIME OF 7.9 MIN

Temp. (°F)	Area: No. 46 Stone, Per 500 G, 100% Sat.	No. 46 Stone, Per 500 G, 60% Sat.	No. 46 Stone, Per 290 G[a], 100% Sat.[a]	No. 46 Stone, Per 290 G[a], 60% Sat.[a]	No. 46 Stone, Per 290 G[a], 80% Sat.[b]	Sand, Per 500 G	Sand, Per 210 G[a]
300	4.030	3.988	2.34	2.32	2.33		
400	5.120	5.320	2.97	2.32	3.02	25.04	10.50
500	6.212	4.412	3.61	2.56	3.08		
600						34.14	14.30
760						38.78	16.30

[a] By proportion.
[b] By interpolation.

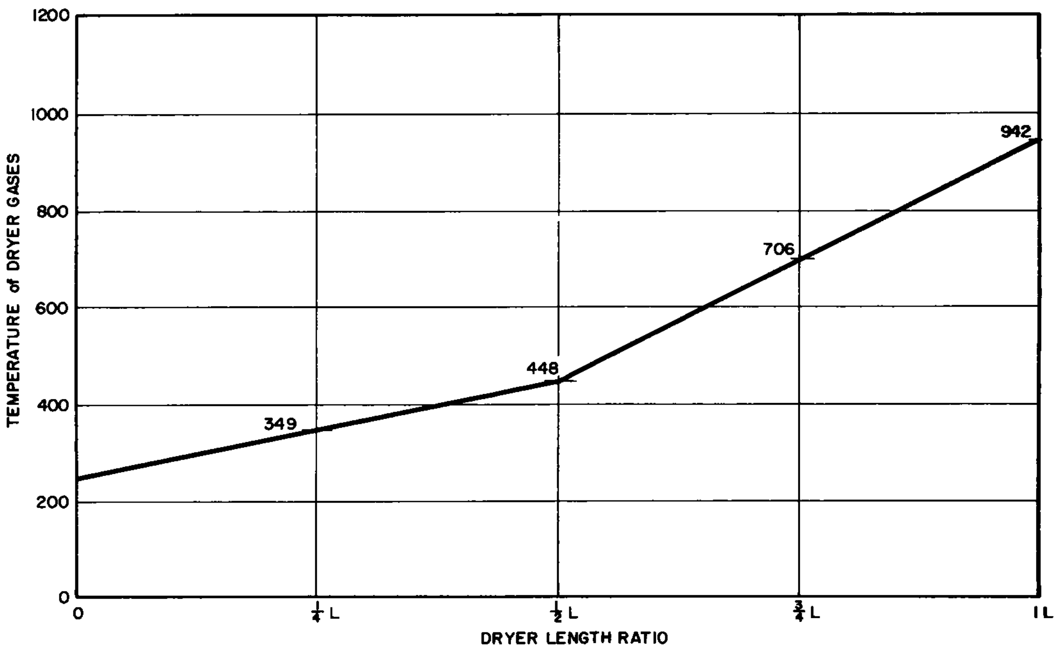

Figure 15. Prototype dryer temperature vs dryer length ratio—heat balance method.

Step 3. To find the point along dryer where average drum temperature exists.

The area under average drum shell temperature curve is found in Figure 17. This area is divided into two equal parts by a vertical line AB. This line is determined at 0.67 L for this case.

Step 4. Point C with coordinates (0.67 L, 590 F) is located in Figure 17. The stack temperature (200 F) is joined to 590 F. The line obtained represents the temperature gradient of the dryer gases.

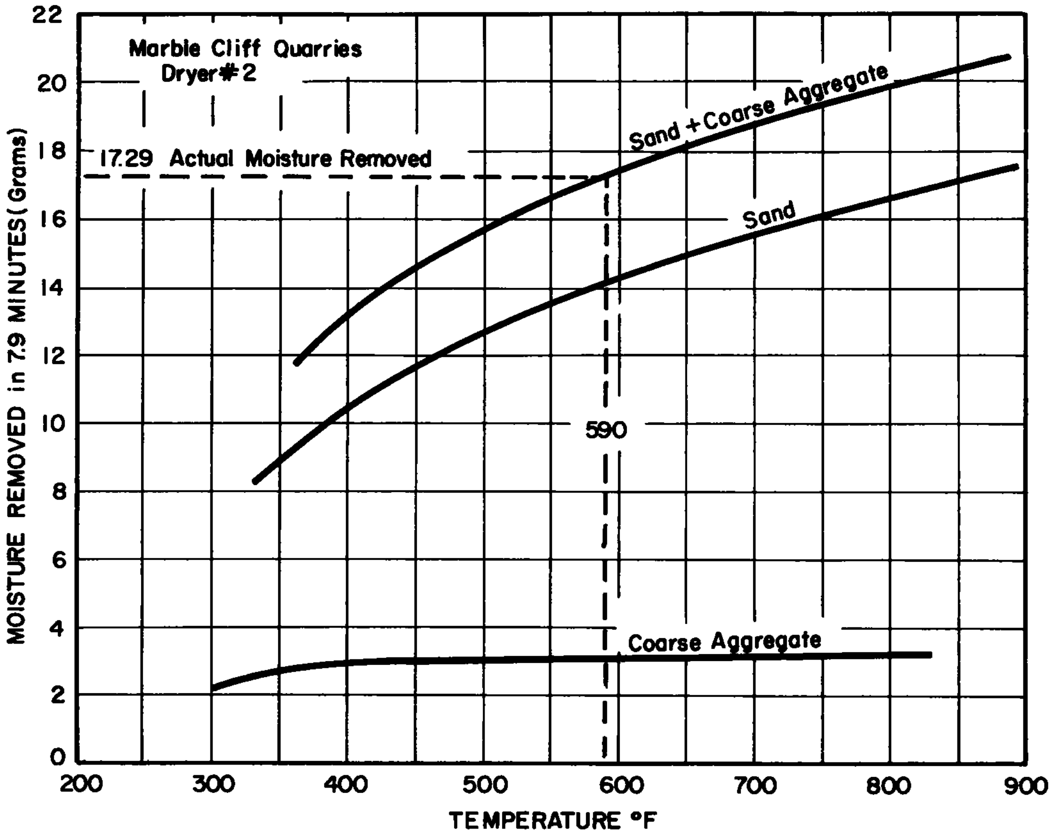

Figure 16. Moisture removed vs oven temperature.

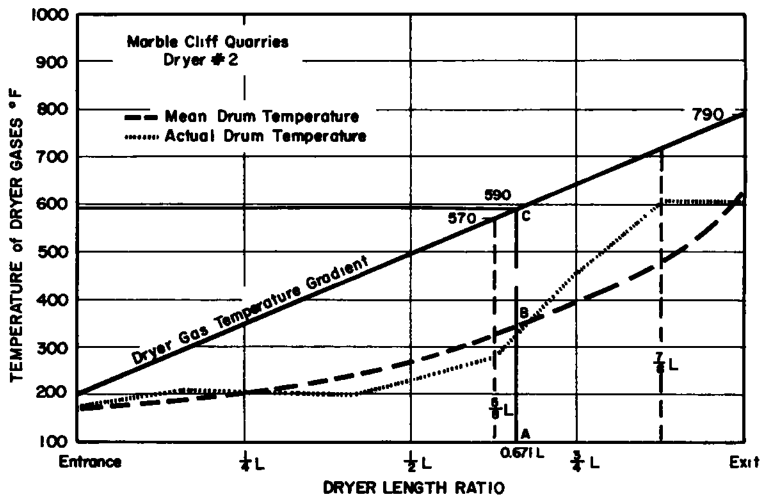

Figure 17. Dryer temperature vs dryer length ratio for Marble Cliff dryer 2.

C. Computation of Aggregate Temperature at Dryer Discharge

The computation of aggregate temperature at dryer discharge is given in Table 6. Temperatures of other sizes including sand may be computed in the same manner using oven heat absorption curves and dryer exposure times for each size. The same temperature gradient of dryer gases may be used for all other size temperature determinations.

The over-all temperature of the aggregate combination may be computed by the following relation when specific heat values of each size group are nearly equal:

$$\frac{P_1T_1 + P_2T_2 + P_3T_3 + \dots\dots\dots + P_nT_n}{100}$$

TABLE 6

COMPUTATION OF AGGREGATE TEMPERATURE AT DRYER DISCHARGE

Property	Prototype Dryer: Method A[a]	Prototype Dryer: Method B[b]	Dryer No. 2[c] Marble Cliff Quarries
Spec. heat	0.200	0.200	0.246
Avg. expos. time (min)	3.70	3.70	7.9
Oven sample wt. (lb)	2.0	2.0	2.0
Drying cycle (L):			
Constant rate	0 to $^2/_4$	0 to $^2/_4$	0 to $^5/_8$
1st falling rate	$^2/_4$ to $^3/_4$	$^2/_4$ to $^3/_4$	$^5/_8$ to $^7/_8$
2nd falling rate	$^3/_4$ to $^4/_4$	$^3/_4$ to $^4/_4$	$^7/_8$ to $^8/_8$
Drying cycle time (min):			
Constant rate	$^1/_2 \times 3.70 = 1.85$	$^1/_2 \times 3.70 = 1.85$	$^5/_8 \times 7.9 = 4.94$
1st falling rate	$^1/_4 \times 3.70 = 0.92$	$^1/_4 \times 3.70 = 0.92$	$(^7/_8 - {^5/_8}) \times 7.9 = 1.97$
2nd falling rate	$^1/_4 \times 3.70 = 0.92$	$^1/_4 \times 3.70 = 0.92$	$(^8/_8 - {^7/_8}) \times 7.9 = 0.98$
Avg. effec. gas temp. (°F):			
Constant rate	400	349	$200 + {^{370}/_2} = 385$
1st falling rate	625	577	$570 + {^{130}/_2} = 635$
2nd falling rate	775	824	$700 + {^{90}/_2} = 745$
Heat absorp. rate (Btu/min):			
Constant rate	25	21	16.5
1st falling rate	21	16	20.5
2nd falling rate	39	51	29.0
Total heat absorp. (Btu):			
Constant rate	$1.85 \times 25.0 = 46.2$	$1.85 \times 21.0 = 38.9$	$4.94 \times 16.5 = 81.7$
1st falling rate	$0.92 \times 21.0 = 19.4$	$0.92 \times 16.0 = 14.7$	$1.97 \times 20.5 = 40.4$
2nd falling rate	$0.92 \times 39.0 = 36.0$	$0.92 \times 51.0 = 46.4$	$0.98 \times 29.0 = 28.5$
Total	101.6	100.5	150.6
Total temp. rise[d] (°F)	$\frac{101.6}{2.0 \times 0.2} = 255$	$\frac{100.5}{2.0 \times 0.2} = 253$	$\frac{150.6}{2.0 \times 0.246} = 306$
Temp. meas. by calorimeter	250 to 276	250 to 276	301 to 320

[a]Based on temperature gradient obtained by moisture method.
[b]Based on temperature gradient obtained by heat balance method.
[c]Based on effective gas temperature gradient obtained by moisture method.
[d]ΔT, above 0 F.

in which

P_j = percent by weight of aggregate size group in gradation, and
T_j = temperature of aggregate size group in gradation.

D. Computation of Aggregate Moisture at Dryer Discharge

1. Dryer No. 2 at Marble Cliff Quarry Co. —The computation of aggregate moisture at dryer discharge is based on effective gas temperature gradient plot in Figure 8 and dryer data from Tables 5 and 6.

For aggregate exposure time of 7.9 min and stone temperature of 306 F, required temperature gradient (by interpolation) is 5.9, or 590 F (ΔT from 200 F stack). Temperature gradient is therefore 200 F at stack and 790 F at burner. A straightline gradient over dryer length is assumed (see Figure 17). Using these data, the average dryer temperature is found from steps 3 and 4 in section B2 to be 590 F.

The procedure now is the reverse as shown in section B2. Drying curve plots (from lab oven tests) are used to develop the curves in Figure 16 by step 2, section B2. The average dryer temperature of 590 F is used in Figure 16 to find the moisture removed in the stone (coarse aggregate), sand or stone and sand combination by known initial moisture contents and blend proportions.

Figure 16 and step 1, section B2, show the check of 17.29 g of moisture removed pe 500 g of aggregate (stone and sand), or 3.5 percent.

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THE NATIONAL ACADEMY OF SCIENCES—NATIONAL RESEARCH COUNCIL is a private, nonprofit organization of scientists, dedicated to the furtherance of science and to its use for the general welfare. The ACADEMY itself was established in 1863 under a congressional charter signed by President Lincoln. Empowered to provide for all activities appropriate to academies of science, it was also required by its charter to act as an adviser to the federal government in scientific matters. This provision accounts for the close ties that have always existed between the ACADEMY and the government, although the ACADEMY is not a governmental agency.

The NATIONAL RESEARCH COUNCIL was established by the ACADEMY in 1916, at the request of President Wilson, to enable scientists generally to associate their efforts with those of the limited membership of the ACADEMY in service to the nation, to society, and to science at home and abroad. Members of the NATIONAL RESEARCH COUNCIL receive their appointments from the president of the ACADEMY. They include representatives nominated by the major scientific and technical societies, representatives of the federal government, and a number of members at large. In addition, several thousand scientists and engineers take part in the activities of the research council through membership on its various boards and committees.

Receiving funds from both public and private sources, by contribution, grant, or contract, the ACADEMY and its RESEARCH COUNCIL thus work to stimulate research and its applications, to survey the broad possibilities of science, to promote effective utilization of the scientific and technical resources of the country, to serve the government, and to further the general interests of science.

The HIGHWAY RESEARCH BOARD was organized November 11, 1920, as an agency of the Division of Engineering and Industrial Research, one of the eight functional divisions of the NATIONAL RESEARCH COUNCIL. The BOARD is a cooperative organization of the highway technologists of America operating under the auspices of the ACADEMY-COUNCIL and with the support of the several highway departments, the Bureau of Public Roads, and many other organizations interested in the development of highway transportation. The purposes of the BOARD are to encourage research and to provide a national clearinghouse and correlation service for research activities and information on highway administration and technology.