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Moyie River Bridge, Moyie Springs, Idaho (April 1964).

Completion of construction of the Moyie River Bridge, part of a Federal-Aid Forest Highway Project, will eliminate hazardous driving conditions in this area by a relocation of U.S. 2. The old route of U.S. 2 is shown in foreground and along the side of the canyon on the right. Note the safety nets in use during construction.

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IN THIS ISSUE

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U.S. DEPARTMENT OF COMMERCE
LUTHER H. HODGES, Secretary

BUREAU OF PUBLIC ROADS
REX M. WHITTON, Administrator

Selecting the Best Scale for In-Motion Weighing

by **RUSSELL E. PUCKETT, P.E., Assistant Professor of Electrical Engineering, College of Engineering, Engineering Experiment Station, University of Kentucky**

Factors that might affect the choice of the type of scale to be used in a dynamic weighing system have been considered in the study of the characteristics of two dissimilar scale platforms reported in this article. Each of the two types of scales has been deemed adequate for the purpose for which it was designed. The determining factor in the selection of the type of scale is the desired form of the ultimate recorded output from the in-motion weighing system, in which the platform will be the weight sensor. One of the scales yields an output that is a discrete value of a rapidly varying applied weight; the output of the other occurs over a relatively long period of time so that its output varies according to the variation in the load on the platform.

Introduction

THE PROBLEM of selecting the best type scale for in-motion weighing has been the basis for an active research program at the University of Kentucky during the past 4 years. The research project is sponsored jointly by the U.S. Bureau of Public Roads and the Kentucky Department of Highways. The primary objective of the research has been to determine the best mechanical configuration of a weighing platform to be used in a system to measure the axle weights of moving vehicles. This project was conducted as a part of the overall investigation of problems related to the development of a dependable, automatic, electronic weighing system and traffic data collection system.

A thorough investigation of the operating characteristics of two dissimilar configurations of scale platforms was made in this testing program, which was initiated by the research group and devised to satisfy the objective of the project. An experimental test site was set up on the approach to a static-weighing station. This location was chosen because it would permit rapid correlation of dynamic weight and static weight of each measured axle of passing vehicles. The primary objective was to assure that each of the platforms would yield an output that was a direct analog of any load applied to it. Lesser objectives were studies of preloading methods and their effects on the outputs of the scales. Some of the findings of the research group have been reported previously (*1*, *2*).[1]

Conclusions

On the basis of the research reported here, the conclusion has been reached that the choice of the type of scale for an electronic in-motion weighing system should be determined by the ultimate form desired for the recorded weight measurements and the end-use to be made of these measurements. From the comparison of the characteristics and capabilities of the two types of platform scales, it has been concluded that the broken bridge type of scale is the better choice when measurements of discrete values of weight are necessary, as in the classification of loads into weight groups. But the rigid platform type of scale was concluded to be the better choice when in-motion measurements are to be made and recorded and no manual manipulation is to be made of the recordings, as in an automatic data collection system.

The Scale Platforms

Two scale platforms were installed at the test site on Interstate 64, near Shelbyville, Ky., on the approach lane to a static-weighing station. One of the scales was a commercial design of the Taller-Cooper Co., a 4-point supported platform; the other scale was an experimental design based on the German broken bridge type of scale (*3*). The mechanical arrangements of the two platforms are sketched in figures 1 and 2. For the purposes of the testing program, pretensioning of the platforms on their supporting load cells, called *preload*, was accomplished by steel rods and turnbuckles for the Taller-Cooper scale and by heavy coil springs for the broken bridge scale.

Each scale platform uses strain-gage load cells as the weight-sensing transducers in the measuring system. The Taller-Cooper platform uses four 20,000-pound load cells, and the broken-bridge scale uses two 50,000-pound load cells. The Taller-Cooper platform is supported on the four cells, one at each corner of the platform; the broken-bridge platform is hinged at its outer edges and supported in the center by the two load cells. Each load cell is electrically driven at its input by transformer coupling from a 600-c.p.s. master

[1] References indicated by italic numbers in parentheses are listed on p. 47.

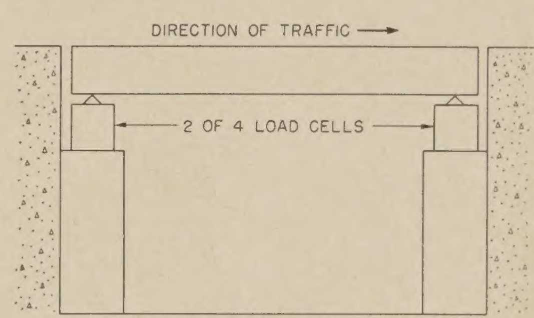

Figure 1.—Taller-Cooper scale.

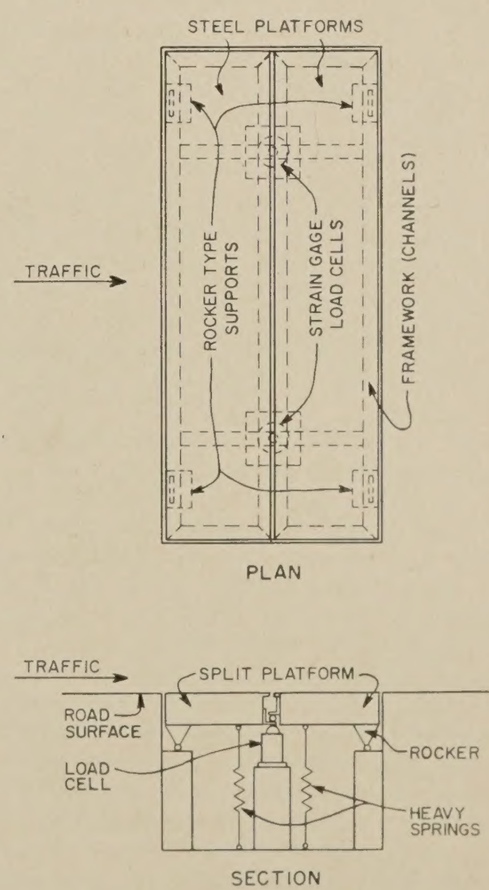

Figure 2.—Broken bridge scale.

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oscillator power amplifier (MOPA). The input signal is common to all load cells in both platforms. Transformer coupling of the input driving signals precludes interaction of the load cells when a load is applied. The MOPA unit was installed at the static scale house with the analog recording equipment. About 600 feet of interconnecting cable was used between the pits containing the load cells and the scale house. The output signals of all load cells in each of the platform pits were electronically added together so that the total output of each scale would be directly proportional to any vertical load applied to it. In this way, the output from the scale would be a direct analog of the axle weight of a passing vehicle.

The output signals from both scales were received in the static scale house and, after necessary signal conditioning had occurred, the signals were recorded on a 2-channel Sanborn paper chart recorder. When the chart of the recorder had been calibrated, the recorded traces could be analyzed to determine several important facts concerning the passing vehicle, including its speed, the weight of each axle, and its axle spacing. The theoretical shapes of the expected output signals from each scale are shown in figure 3, and some actual recorded traces from the two scales are shown in figure 4.

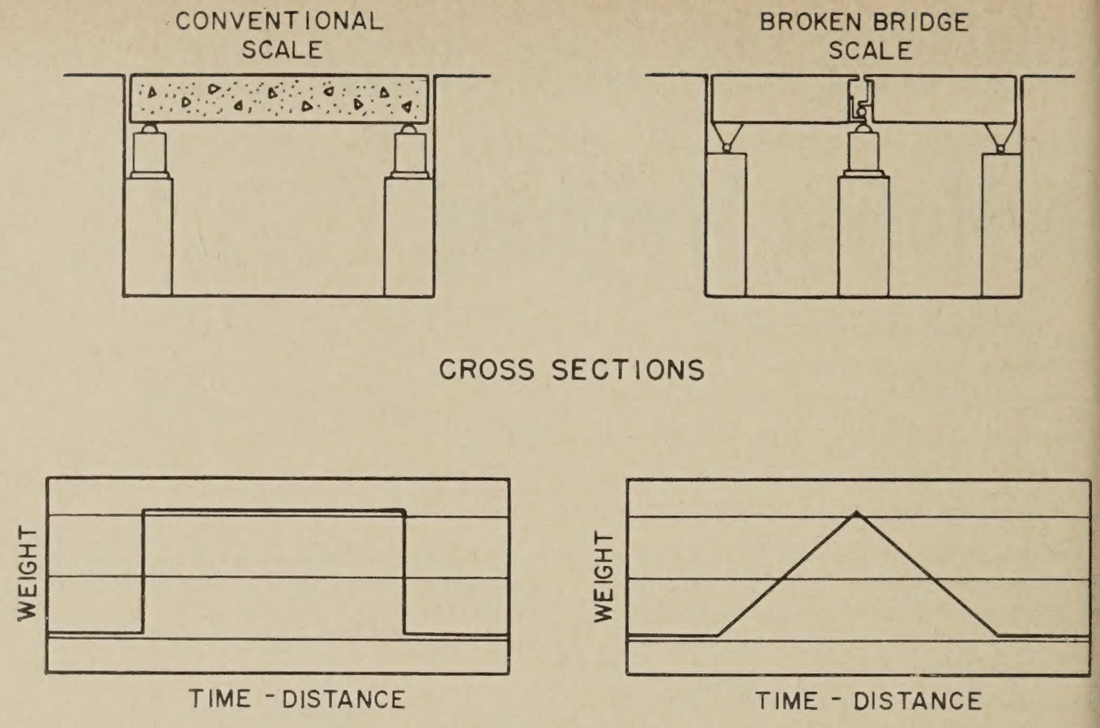

Figure 3.—Theoretical waveforms of output signals from two dissimilar scales.

Dynamic Weight Variations

The motion of a vehicle as it rolls along a modern highway will be affected by many factors. The condition and type of its suspension system, the amount and placement of its load, its speed, the type of body construction and its shape, wind velocity vectors, and other factors will influence its motion. Its motion in a vertical plane will become apparent on the road surface as variations of the weight applied to the surface by the wheels. As the vehicle's weight is the important factor in an in-motion weighing system, any variations of its weight from some static weight must be recognized. The dynamic weight variations will occur in some sinusoidal form, and in a rather narrow range of frequencies. Its frequency of variation will depend upon the physical limits of the suspension system and upon other physical factors related to the construction and loading of the vehicle. Typical values of the sinusoidal variations of dynamic weight of commercial trucks are in the range from 2 to 10 c.p.s. (*4*). The maximum variation of the dynamic weight of a given axle from its static value is as much as 100 percent heavy for lightly loaded vehicles and as much as 40 percent, heavy and light, for truck axles loaded at 14,000 pounds (*5*).

Assuming a pure sinusoidal variation of dynamic weight for a moving vehicle and as much as 50 percent variation from its static weight, the data presented in figure 5 show how the dynamic weight will vary with distance as the vehicle moves at different speeds. This result suggests the need for a weight-measuring system that consists of more than one weight-sensor platform, in order to increase the correlation of measured dynamic and static axle weights. Of course, whether the need exists will depend upon the use of the system. If the system is to be used to measure an actual dynamic weight, regardless of how this weight may vary from the static weight, then a single measurement will provide the desired result. However, if a weight is desired from the system that can be interpreted so as to obtain the actual static weight, then a more elaborate system containing more than one weight sensor must be used.

The output of the Taller-Cooper scale (fig. 4) is a radical departure from the theoretical waveform. It is recognized as a measurement of the variation of weight during the time the vehicle was on the platform. As the platform is about 3 feet wide, a vehicle having a high frequency of variation in weight, and moving at slow speed across the platform, yielded a rapidly varying output from the scale. The broken bridge, however, yielded an output at its maximum response point that is representative of the dynamic weight at that point on the platform. The Kentucky project has determined that each platform will measure equally well the weight applied to it.

Potential Uses of Each Platform

To satisfy the need to collect and analyze large volumes of traffic data, highway engineers are using electronic methods. Electronic in-motion weighing systems have been studied for several years, and many agencies are concerned with the development of a model station that could be used to collect the necessary data for load studies and planning purposes.

The Research Organization for Roadbuilding in West Germany devised the broken-bridge platform for weighing moving vehicles. This group was interested in classifying the measured axle weights into weight-groups of about 1,000 pounds. The recorded data showed how many axles weighed between 7,000 and 8,000 pounds, 8,000 and 9,000 pounds, etc. The output waveform has a gradually sloping excursion to its maximum; an electronic counter can be arranged to sum all weight signals in each of the preselected weight-groups (*6*).

A similar scheme for classifying axle weights has been developed by the Department of Scientific and Industrial Research, Road Research Laboratory, in England. Their system consists of a platform mounted in the road and supported on four load cells. The output of the cells is fed to a reflecting galvanometer whose motion is proportional to the load applied to the platform. The optical system of the galvanometer is arranged to cause a light beam to traverse a linear array of photocells. The placement of the photocells determines the interval of a weight-group, so that any applied load is classified into predetermined weight-groups. Each photocell is connected to a switching circuit that controls a mechanical counter

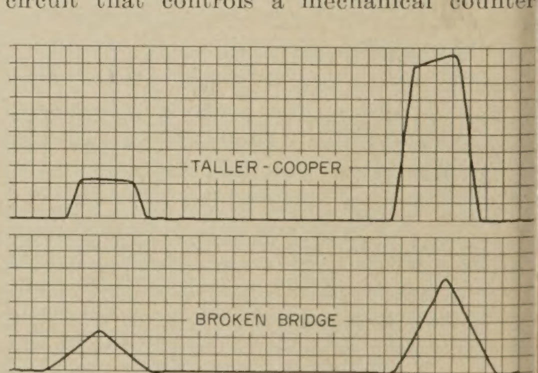

Figure 4.—Recorded output traces from Taller-Cooper and broken bridge scales.

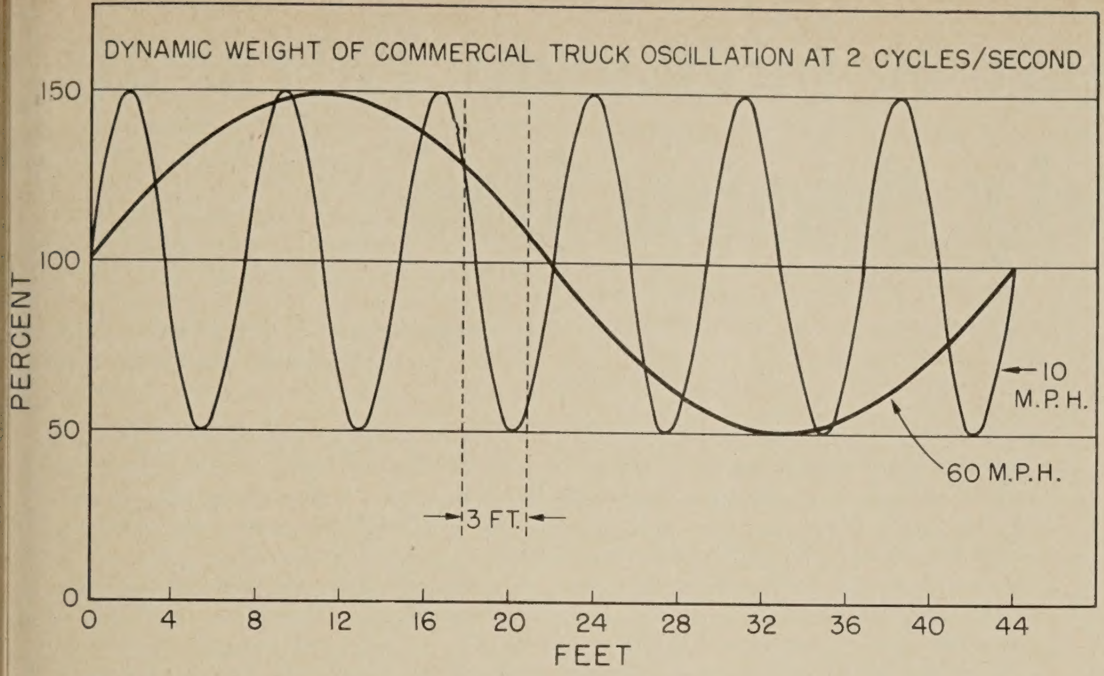

Figure 5.—Variation of dynamic weight of moving vehicle having 50 percent maximum variation from static weight, related to distance and speed of vehicle.

An excursion of the light beam past a given photocell causes its counter to register one count. Thus, the number of axle loads in any class is given by the difference of the readings of the two counters corresponding to the weight interval (*7*).

Each of these systems recorded data in digital form, but these data merely indicated how many axle loads occurred in a particular weight class. No information on actual dynamic weight variation of the axles was provided. To do this, the recording system must sample the load on an instantaneous basis rather than in discrete classification groups. An examination of the theoretical output waveforms of the Taller-Cooper and broken bridge scales shows that either type will permit such a sampling of loads applied to them. The rigid platform type of scale, however, is better suited for measuring actual dynamic weight variations of axles than the broken bridge type of scale because its output occurs over a relatively long period of time during which sampling could be done. The broken bridge type of scale yields a maximum value as an output, and this output will be representative of the dynamic load at that instant only. An electronic system to sample and hold this maximum output would require complex circuitry, and the cost would probably be prohibitive. But as the Taller-Cooper scale yields a varying output corresponding to the variations in load on the platform, the selection of one particular output value during this time as the dynamic weight of the axle is invalid. Perhaps an average of all output values occurring during the time the axle is on the scale would yield an acceptable weight result.

The Taller-Cooper scale was designed to measure loads applied to it in a nearly static state or while they moved slowly across the scale. Its response to fast moving loads can be improved through the use of large amounts of preload on the platform. The broken bridge scale was designed to be used in a measuring system where the loads would be moving at high speeds. Each of the two types of platforms have certain advantages and disadvantages and each has proved to be capable of doing the job for which it was designed.

The output waveform from the Taller-Cooper scale is suited to automatic measurement and recording of loads applied to it. However, some interval of time must be provided in the use of this type of scale during which a weight measurement may be made and sampled. The broken bridge type of scale, because of its single discrete measurement of the load applied to it, has not proved to be very well suited for the automatic recording of weights. But the output of a broken bridge type of scale can be more useful for in-motion weighing than the other type if a human operator can scan its record of weight measurements in the form of the Sanborn paper charts that were used for the test reported in this article. The peak of the recorded trace can be easily recognized from these charts, but the actual variation of dynamic weight of a passing vehicle is not apparent on the broken bridge recording because of this scale's mechanical construction.

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(*1*) *Dynamic Weighing of Vehicles,* by J. A. Dearinger, Public Roads, vol. 31, No. 10, October 1961, pp. 200–204, 210.

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(*3*) *Die Automatische Aachslastwaage bei Grunbach (Remstal), (The Automatic Axle-Load Scale at Grunbach),* by W. Schwaderer and W. Reimund, Strasse und Autobahn, No. 2, 1959.

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(*5*) Unpublished annual report of HPS 1(22), Kentucky Research Foundation, University of Kentucky Project "Weighing Vehicles in Motion," January 1963.

(*6*) *Weighing Moving Motor Vehicles,* by Wolfgang Bachmann, Trends, German Construction Engineering, June 1959, p. 870.

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Effect of Moisture on Bituminous Pavements in Rocky Mountain Areas

REGION 9
BUREAU OF PUBLIC ROADS

Reported by [1,2] WILLIAM L. EAGER
Regional Materials Enginee

Determination of the quality of the materials and type of mix that will produce a bituminous pavement that will be serviceable and that will reasonably withstand the effect of moisture, especially in areas having a climate similar to that in the Rocky Mountain area, is the ultimate goal of highway engineers. Mindful of the disintegration of bituminous pavements caused by the effects of water or moisture in the Rocky Mountain area, test data on different types of asphalts, aggregates, and additivies used in pavement construction have been summarized and analyzed. Additional information is presented in the companion article, "Stability of Bituminous Pavements Related to Aggregate Characteristics," by R. E. Olsen. The data analyzed for this article were obtained from tests performed in Colorado, Utah, and Wyoming, and in the Bureau of Public Roads Laboratory, Denver, Colo.

The study reported here was made in an attempt to forecast the serviceability of materials and correlate laboratory test results with actual performance of bituminous pavements. The analyses were made from the results of immersion-compression tests conducted on different bituminous paving mixtures.

Road failures have been pronounced in road-mix paving materials but they have also occurred to a serious extent in roads constructed of hot plant-mix and asphalt concrete materials. It is suggested that the immersion-compression test be used for determining in advance the probable effect of water on compacted paving mixtures. The determinations reached in this study indicate that further studies to correlate laboratory data and pavement performance would be worthwhile in other areas where the effect of water on bituminous pavement is noticeable.

Nature of the Problem

MUCH HAS BEEN written on the effect of water on bituminous paving mixtures (*1*, *2*,[3]) and this problem is certainly not limited to the Rocky Mountain area. Ideally, water would have no effect on the pavement, but all too often it has reduced adhesion between the asphalt and aggregate to the point that serious stripping has occurred, causing loss of pavement stability and often severe raveling of aggregate from the surface. Stripping has been defined as the loss of asphalt films from aggregate surfaces in the presence of moisture (*3*); and raveling, as the loss of aggregate particles from the surface of the pavement—usually caused by loss of adhesion between aggregate and asphalt. This action is more pronounced in the road-mix types, but it has occurred to a serious extent in the hot-plant mix and asphaltic concrete types of pavement.

The Rocky Mountain area is generally dryer than other parts of the country. It might, therefore, be expected that the problem would be less severe there than elsewhere. However, this normal lack of moisture may well be a reason that the effect of water is more pronounced when it does become available. Water has not been available to leach out deleterious fines or disintegrate soft particles in natural deposits of aggregate, as in areas of greater rainfall. Possibly because of this, degradation is common for many of the aggregates available in the area. Although this degradation seldom causes any significant increase in plasticity, it does undoubtedly contribute to the reduction in resistance of the pavement to the effects of water.

A possible cause of the problem has been the widespread use of local, on-the-job aggregate deposits, rather than commercially produced aggregates. Specifications have commonly been written to fit these local aggregate sources rather than to require a high quality standard aggregate.

[1] Presented at the 43d annual meeting of the Highway Research Board, Washington, D.C., January 1964.

[2] E. G. Swanson and M. S. Tilzey of the Colorado Department of Highways; Travis Cole of the New Mexico State Highway Department; J. M. Desmond of the Wyoming Highway Commission; W. J. Liddle of the Utah State Department of Highways; and W. F. Fitzer of the Bureau of Public Roads, Region 9 Materials Testing Laboratory, cooperated with the author in providing information used in the preparation of this report.

[3] Figures in parentheses indicate references listed on page 54.

In recent years, more specifically since the start of the Interstate program, a general upgrading of specification requirements excludes some aggregates that were formerly permitted. If the use of these is permitted, they are upgraded by improved processing—prewasting of fines and use of screening equipment that removes and wastes coatings on the aggregate and breaks down and wastes the softer aggregate particles—or by the addition of mineral filler or hydrated lime. Washing of aggregates for bituminous construction has not yet come into general use, but as specification standards become higher and high-grade aggregates become less available, this will be a logical development.

Early Experience

The first bituminous pavements constructed in rural areas of the Rocky Mountains were nearly all of the road-mix type; slow curing road oils and subsequently cutbacks of the medium curing (MC) type were used. These surfaces were commonly recognized to lack resistance to the effects of moisture, so seal coats were used as a deterrent. Many times, however, water got into the pavement—probably through capillary action or vaporization. Excess water in the mat was a common cause of its early failure. A study (*4*) of bituminous pavements in Utah and Colorado in 1949 showed that pavements containing moisture in excess of 2 percent failed, and that the amount of moisture was proportion to the percentage of aggregate passing the N 200 sieve. It was concluded that 12 perce was the maximum amount of aggregate th should be allowed to pass through this siev The tests did not indicate the plasticity ind (PI) of the mat aggregate, but it is probab that higher PI values were associated wi higher moisture content and with paveme failures.

Effectiveness of seal coats

Although seal coats have been effective preventing or retarding surface ravelin their use has not always been effective in pr venting stripping within the mat and, in fa occasionally has been detrimental. Becau of the general use of seal coats on road-m mats, this practice has been carried over in hot plant-mix and bituminous concrete surfa as they came into general use. Only in r cent years has there been any tendency leave seal coats off these higher type pa ments and often then with reluctance and common feeling that they would be need within a short time. All too often this feeli was borne out.

The seriousness of the problem was fi brought forcibly to attention by the resu of an experimental project constructed Colorado in 1941. A number of experimen sections of road-mix bituminous surfaci were constructed, in which different sour and types of asphaltic materials and a lo gregate of adequate quality (by the rmally accepted standards at that time) re used. However, all sections quickly owed serious distress; rapid deterioration of e surface (raveling) occurred with the first in. This necessitated prompt seal coating all sections, which obscured any differences the experimental sections. A subsequent boratory study of the aggregate by the ureau of Public Roads Materials Research ivision in Washington (*5*) lead to the velopment of the immersion-compression st as now provided by *The Standard Method Test for Effect of Water on Cohesion of Comcted Bituminous Mixtures* (AASHO Designtion: T 165 and ASTM Designation: 1075).

sphalt stripping

Even prior to the Colorado experiment, me very unhappy experiences occurred with phalt stripping. In the middle thirties, e use of open-graded bituminous mat was ed on direct Federal construction projects. ecause most of the aggregate sources were avel, to obtain the 100 percent crushed gregate required, the natural pit fines were sted and the required aggregate was proced by crushing the oversized gravel. his produced a mat of high mechanical ability and high air voids. To keep surface ater out, the surface was choked with fines d seal coated. The aggregates were from granitic source and, although surface water ay have been kept out and the surface looked od, the asphalt was soon stripping badly om the coarse crushed aggregate fragments the lower parts of the mat, and these aggrete particles were coated with water instead asphalt. The general opinion then was at the seal was ineffective and allowed rface water to enter and soften the mat. his type of construction, therefore, was disntinued in favor of a return to use of the nse-graded type of aggregate. However, ere was no assurance that free water actuy entered the mat from the surface. The en grading, in combination with the much eater affinity of the aggregate for water an for asphalt, permitted conditions favorle to the development of stripping. Temrature differentials are extreme in this ountainous area, and these differentials uld well have led to condensation of moisture times in the large void space of the openaded mat.

On one of the projects referred to above, e State used the wasted pit fines to build a nse-graded road-mix on an adjacent project. esults were excellent. This does not prove at open-graded, crushed aggregate mats are cessarily all bad and that dense-graded ones e all good, but rather that when hydrophilic gregates are used the chances for moisture enter and cause stripping and softening of e mat are much less as the density of the mpacted mixture is increased.

The projects referred to above were all ilt before the development of the immersionmpression test (AASHO T 165) or the atic-immersion test, *Standard Method of Test Stripping Test for Bitumen-Aggregate Mixtures* (AASHO Designation: T 182), or any other commonly accepted test to measure the effect of water on the aggregate-asphalt combination. In addition, commercial chemical antistripping agents had not come into general use.

Summary and Conclusions

In summary, the bituminous pavements in areas of the Rocky Mountains have always lacked resistance to the disintegrating effect of water. When road mixes were replaced by plant mixes, the problem was reduced but still existed. Additives—both a chemical type and hydrated lime—have alleviated the problem, as have better construction procedures and closer quality control of the materials. Seal coats have been necessary and have helped to compensate for undesirable stripping and raveling, but the goal is to so design and construct asphaltic concrete pavements that they will not need seal coats. Before that goal is reached it will be necessary to know just what further changes can be made in design, materials, or construction procedures. Considerable improvement in reducing the stripping and raveling problems may be expected if close attention is applied to the following listed details:

- Determination should be made by laboratory tests of the probable action of the compacted paving mixture in the presence of water. On the basis of the study reported here, the immersion-compression test has been concluded to be the best laboratory test for this determination.
- On the basis of the results of the laboratory tests, it has been concluded that the unsatisfactory aggregates should be eliminated, improved by screening or washing, or the differences compensated for by the addition of suitable admixtures (chemical antistripping additives, hydrated lime, filler, or other aggregate sizes to improve gradation).
- It has been concluded that good procedures for the design of paving mixtures and close control of construction (particularly mixing times, temperatures, and compaction procedures—more concentrated rolling when the mixture is still warm enough for the rolling to be effective) are essential for construction of bituminous highways that will withstand the effect of water.

Conclusions drawn from test

Conclusions drawn from analysis of test results compiled in table 1 are, as follows:

- Asphalt cement pavements have higher stabilities and higher indices of retained stabilities than cutbacks. Part of this is, no doubt, because a higher percentage of asphalt cement than of cutback can be used.
- Use of hydrated lime as an additive also permits use of a higher percentage of asphalt, this probably accounts for some of the higher index of retained stability of bituminous pavements when the hydrated lime is used. Hydrated lime adds to both dry and wet stabilities. Wet stabilities, when lime is added, are often higher than dry stabilities, probably because of continued reaction between lime and aggregate fines during the wet soaking period.
- A higher percentage of asphalt used in a mixture produces a higher index of retained stability although dry stability may be reduced by the higher percentage of asphalt.

Immersion-Compression Test

For laboratory determination of the effect of moisture on dense-graded, hot plant-mixed, bituminous mixtures, many States rely heavily on the immersion-compression test and design mixtures that will have a wet stability of 70 or 75 percent of the dry stability. The 70 or 75 minimum index of retained stability [4] generally seems to be satisfactory for hot-mix or asphaltic concrete, although it is often difficult to achieve without some stabilizing admixture. Swanberg and Hindermann (*6*) recommended a minimum index of retained stability of 75.

Methods of making test

Results of immersion-compression tests performed by Colorado, Utah, Wyoming, and the Bureau of Public Roads laboratories are shown in table 1. The test specimens made in the Colorado laboratory were prepared with a kneading compactor—125 blows at 500 p.s.i. foot pressure. The other laboratories prepared test specimens by using the double plunger method in accordance with the *Standard Method of Test for Compressive Strength of Bituminous Mixtures* (AASHO: T 167).

For water-immersed specimens (24 hours at 140° F.), the three States and Public Roads determined strength according to the *Standard Method of Test for Effect of Water on Cohesion of Compacted Bituminous Mixtures* (AASHO: T 165).

In Utah the actual wet stability is considered to be of greater significance than the index of retained stability and, therefore, a minimum wet stability of 150 p.s.i. is specified. In Wyoming the same is accomplished by requiring a minimum dry stability of 250 p.s.i. and an index of retained stability of 70 (70 percent of 250 is 187 p.s.i., which is, therefore, the minimum wet stability). This approach has considerable merit because mixtures of rather clean, coarse-graded aggregates often have a good index of retained stability but may be too low in either dry or wet stability or density to make the best pavement. In addition, chemical additives sometimes reduce the actual dry stability value and increase the wet stability value. In other words, they bring the two values closer together, thereby increasing the index of retained stability. When this is so, the index alone does not represent a true measure of the nature of the mixture or of the effect of the additives.

In comparing indices of retained stability, the method of compacting and testing the specimens must also be considered. For example, in Colorado the specimens are compacted with a kneading compactor, such as is used for Hveem stabilometer tests. Specimens so compacted show significantly higher stabilities, as well as higher indices of

[4] The term index of retained stability is synonymous with the term index of retained strength.

Table 1.—Immersion-compression test

Source of data	Asphalt: Penetration grade	Asphalt: Amount, percent	Additive: Type	Additive: Amount,[1] percent	Compressive strength, 77° F.: Dry	Compressive strength, 77° F.: Wet	Index of retained strength
Colo. Projects:							
2721	85-100	5.3	None	0	456	158	30
2721	85-100	5.7	PC [2]	2	444	570	128
2721	85-100	6.0	HL [3]	1	378	515	138
43076	85-100	5.8	None	0	293	56	16
43076	85-100	6.3	HL [3]	1	284	286	101
Utah projects:							
169-SA-63	85-100	5.2	None	0	------------	195	------------
169-SA-63	85-100	5.2	HL [3]	1	------------	283	------------
97-SA-63	85-100	5.3	None	0	------------	45	------------
97-SA-63	85-100	5.3	HL [3]	1	------------	154	------------
Wyo. projects:							
2938	85-100	6.0	None	0	302	188	62
2938	85-100	6.5	do	0	309	223	72
2938	[4] 85-100	6.0	HL [3]	2	404	404	100
2938	[4] 85-100	6.5	HL [3]	2	313	363	116
2938	[4] 85-100	6.0	None	0	332	272	82
2938	[4] 85-100	6.5	do	0	315	296	94
5120	[4] 85-100	6.0	None	0	234	89	38
5120	[4] 85-100	6.5	do	0	258	95	37
5120	[4] 85-100	6.0	HL [3]	2	285	208	73
5120	[4] 85-100	6.5	HL [3]	2	312	204	65
5120	[4] 85-100	6.0	HL [3]	3	430	234	54
5120	[4] 85-100	6.5	HL [3]	3	425	225	53
5120	[4] 85-100	6.0	HL [3]	4	453	275	61
5120	[4] 85-100	6.5	HL [3]	4	442	309	70
5120	[4] 85-100	6.0	PC [2]	3	280	131	47
5120	[4] 85-100	6.5	PC [2]	3	290	143	49
5120	[4] 85-100	6.5	AF [5]	1	278	110	39
5120	[4] 85-100	6.5	AF [5]	2	283	124	44
5120	[4] 85-100	6.5	FA [6]	2	310	80	26
5120	[4] 85-100	6.5	FA [6]	3	307	94	31
5120	[4] 85-100	6.5	LWA [7]	2	248	67	27
5120	[4] 85-100	6.5	LWA [7]	3	243	86	35
62-521	[4] 85-100	6.0	None	0	413	102	25
62-521	[4] 85-100	6.5	do	0	418	159	38
62-521	[4] 85-100	6.0	HL [3]	2	366	255	70
62-521	[4] 85-100	6.5	HL [3]	2	344	235	69
62-313-317	[4] 85-100	6.5	None	0	296	177	60
62-313-317	[4] 85-100	7.0	do	0	305	204	67
62-313-317	[4] 85-100	6.5	HL [3]	2	342	310	91
62-313-317	[4] 85-100	7.0	HL [3]	2	334	313	94
62-313-317	[4] 85-100	6.5	Cement	2	331	283	86
62-313-317	[4] 85-100	7.0	do	2	305	253	83
62-313-317	85-100	6.5	None	0	299	194	65
62-2778	[4] 85-100	5.25	None	0	196	68	35
62-2778	[4] 85-100	6.25	do	0	193	105	55
62-2778	[4] 85-100	6.75	do	0	190	157	82
62-2778	85-100	6.25	do	0	195	82	42
62-2778	85-100	6.75	do	0	207	115	56
62-2778	[4] 85-100	6.25	HL [3]	2	222	179	80
62-2778	[4] 85-100	6.75	HL [3]	2	213	231	108

[1] Additives incorporated in the mixture in the laboratory test, as contrasted to those added to the asphalt at the refinery, which are designated by footnote 4.
[2] Portland cement—used in mix at percentage shown (by weight of aggregate).
[3] Hydrated lime—used in mix at percentage shown (by weight of aggregate).
[4] Asphalt fortified at refinery with antistripping additive, amount and type not known. Other asphalts contained no refinery-added additive.

retained stability, than specimens compacted by the standard, double plunger, direct compression method.

Improved Design and Construction Standards

In recent years, standards of design and construction of bituminous plant-mix and and asphaltic cement have been upgraded in the Rocky Mountain area and are now believed comparable to standards used elsewhere. Minimum densities are 95 percent of laboratory density (Marshall standard or kneading compactor density), which will include air voids of 2 to 6 percent. Sufficient asphalt is used to fill at least 75 percent of the mineral voids in the compacted mixture. Generally, this requires about 6 percent of 85–100 penetration grade asphalt. The index of retained stability in the immersion-compression test is 70 or 75 for Interstate and primary roads. Commonly, an additive is required to obtain these indices of retained stabilities, and both chemical additives and hydrated lime are being used for this purpose. The normal laboratory practice is to test the bituminous mixture without any additive and, if the minimum index of retained stability is not obtained, to use a chemical antistripping additive, hydrated lime, portland cement, or other additive. Portland cement as an additive generally is not as satisfactory as hydrated lime. In some mixtures, chemical additives produce better results than hydrated lime but, in others, the reverse is true. Results of many immersion-compression tests are listed in table 1 and are illustrated in figure 1.

Experience with Hydrated Lime and Other Treatments

Emil Swanson, Materials Engineer for the Colorado Department of Highways recently presented a paper *Use of Hydrated Lime in Asphalt Paving Mixtures* before the National Lime Association (*7*). Some of the test results were rather startling. For example, one mixture without hydrated lime in the admixture fell apart during immersion in water. The same mixture, containing 1 per cent hydrated lime, had a dry stability of 48[illegible] p.s.i. and a wet stability of 442 p.s.i., for a[illegible] index of retained stability of 92. This aggre gate was not cured after the hydrated lim[illegible] was added. When other aggregates wer[illegible] used, a curing period of 2 to 5 days was neede[illegible] for the lime to react with the aggregate befor[illegible] it was mixed with asphalt. On one such job the indices of retained stabilities were ob tained as follows:

Percent added		Index of retained stability
0	hydrated lime	43
1	hydrated lime (no curing period)	85
1	hydrated lime (48-hour curing period)	117

In the proportion normally used (1 to 1[illegible] percent of weight of aggregate), hydrated lim[illegible] is considerably more expensive than chemica[illegible] additives, which may be used in the proportio[illegible] of 0.5 to 1.0 percent of the weight of asphalt. In these proportions, the cost of the mixture i[illegible] increased about 45 cents a ton if hydrated lim[illegible] esults for different bituminous mixtures

Source of data	Asphalt		Additive		Compressive strength, 77° F.		Index of retained strength
	Penetration grade	Amount, percent	Type	Amount, [1] percent	Dry	Wet	
BPR:							
Sample C	60–70	5.8	None	0	183	84	46
Sample C	85–100	4.9	do	0	206	70	34
Sample C	85–100	4.9	HSA [8]	1	143	70	49
Sample C	85–100	4.9	HL [3]	1	277	130	47
Sample C	85–100	5.6	None	0	276	132	47
Sample C	85–100	5.6	HSA [8]	1	250	109	44
Sample C	85–100	6.2	HSA [8]	1	268	120	45
Sample C	85–100	6.3	HL [3]	1½	296	296	100
Sample D	85–100	5.6	None	0	227	226	100
Sample D	85–100	5.6	HSA [8]	1	212	238	112
Sample D	85–100	5.6	HL [3]	1	225	252	110
Sample E	MC-3	3.9	None	0	70	0	0
Sample E	MC-3	3.9	HSA [8]	1	99	40	40
Sample E	MC-3	5.0	HL [3]	1	95	46	48
Sample E	120–150	5.8	None	0	291	0	0
Sample E	120–150	5.8	HSA [8]	1	290	327	113
Sample E	120–150	6.7	HL [3]	1	315	235	75
Sample F	MC-3	5.2	None	0	62	13	29
Sample F	MC-3	5.2	HSA [8]	1	35	25	45
Sample F	MC-3	5.6	HSA [8]	1	65	27	42
Sample F	120–150	5.2	None	0	211	152	72
Sample F	120–150	5.2	HSA [8]	1	234	194	83
Sample F	120–150	5.6	HL [3]	1	262	219	84
Sample H	120–150	5.3	None	0	374	139	37
Sample H	120–150	5.3	HSA [8]	1	405	307	76
Sample H	120–150	6.1	HL [3]	1	387	267	95
Sample I	MC-3	4.0	None	0	148	33	22
Sample I	MC-3	4.0	HSA [8]	1	111	67	60
Sample I	MC-3	4.0	HL [3]	1½	184	130	72
Sample I	120–150	5.2	None	0	366	298	81
Sample J	MC-3	4.7	None	0	----	0	0
Sample J	MC-3	4.7	HSA [8]	1	102	55	54
Sample J	MC-3	4.7	HL [3]	1	118	64	54
Sample J	MC-3	4.7	HL [3]	1½	147	94	64
Sample K	MC-3	3.7	None	0	126	33	26
Sample K	MC-3	3.7	HL [3]	1	187	130	70
Sample K	120–150	5.7	None	0	259	131	51
Sample K	120–150	5.7	HL [3]	1	286	225	79
Sample L	MC-3	4.4	None	0	175	15	9
Sample L	MC-3	4.4	HSA [8]	1	192	35	18
Sample L	120–150	5.4	None	0	297	165	56
Sample L	120–150	5.4	HSA [8]	1	256	181	21
Sample L	120–150	5.4	HL [3]	1	293	192	66
Sample M	120–150	4.9	None	0	553	443	80
Sample M	120–150	4.9	HSA [8]	1	472	443	94
Sample M	120–150	4.9	HL [3]	1	518	468	90

[5] Asbestos fiber—used in mix at percentage shown (by weight of aggregate).
[6] Fly ash—used in mix at percentage shown (by weight of aggregate).
[7] Light weight aggregate—used in mix at percentage shown (by weight of aggregate).
[8] Concentrated heat stable additive—used in asphalt at percentage shown (by weight of asphalt).

added, contrasted to about 11 cents a ton if he chemical additive is used. However, if he hydrated lime performs more satisfactorily han the chemical additive then, obviously, the dditional expenditure is justified. In such a ituation the proper cost comparison is between he added cost of the hydrated lime and the dded cost of some other aggregate not needing he hydrated lime.

If a seal coat would definitely prevent the ndesirable characteristics of a hydrophilic ggregate from manifesting themselves, then t would be proper to compare the cost of he additive or other treatment with the cost f a seal coat. Seal coats cost about 11 cents er square yard or, on a 3-inch thick mat, n additional 1.7 cents per ton of mixture. This, of course, is significantly cheaper than ither the chemical additive at 11 cents per on of mixture or the hydrated lime at 45 ents per ton of mixture. The trouble is, he seal coat will not always solve the problem; lso so many uncertainties are involved in eal coat construction that the results are omewhat of a gamble.

Correlation of Test Results and Pavement Performance

Little correlation between pavement serviceability and laboratory mixture studies based on the immersion-compression tests was reported by Goldbeck (*8*). However, the Colorado experimental project (*4*) previously referred to did show good correlation. Although it is logical to expect a mixture that has a high index of retained stability to be better than one that has a low index, other factors affect the resistance of the mat to moisture and sometimes may be of greater significance and may obscure the action taking place in the immersion-compression test. If the mat is so dense, so well mixed, or is so well sealed over—either by compaction, traffic in warm weather, or by a seal coat—that water does not enter it, then stripping, swelling, and loss of stability cannot occur. If water could definitely be kept out of the mat and off its surface, there would be no need for concern about stripping or raveling or loss of stability. Obviously, there can be no such assurance and, therefore, it is proper to take all possible precautions to prevent mat damage by water.

Although no exact correlation exists between immersion-compression test results and the stability of the road (as stated previously, other factors affect the serviceability of the pavement), there are many examples of stripping, raveling, and even softening of the mat that are obviously caused by the effects of water in the mixture. The use of chemical additives or hydrated lime will not eliminate all these problems, as such materials cannot compensate for an inadequately designed or constructed pavement. Experience has shown, however, that such additives do help.

Some of the reasons why bituminous pavements are or are not damaged by water are known. An increasing number of pavements are little damaged even though they are not sealed. However, they still seem to be in the minority, and the number that last as long as

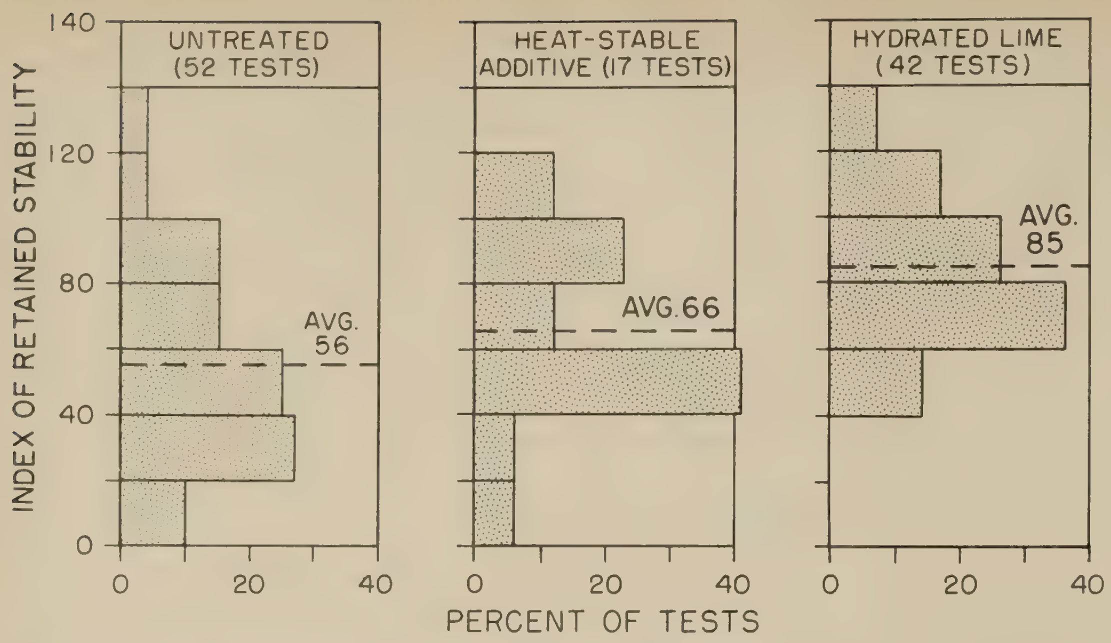

Figure 1.—Comparative results of tests in which additives were used in bituminous materials.

10 years, or even 5 years, without a seal coat does not constitute a large proportion of the bituminous pavements constructed.

Need for sealing

Whether all roads that are sealed actually need sealing is questionable. The need is often a matter of personal opinion, and some engineers and maintenance men might say a pavement needs sealing, whereas others, having a different background of experience, might not. Add the fact that the need for a seal coat usually develops at a time of the year when it is not possible to do anything about it (winter or spring), and it is understandable why some pavements are given precautionary seal coats when they might not need them. The first winter is usually the critical time; pavements seem to develop increased resistance to the effect of water as time passes, and pavement density is increased by traffic during warm weather. Thus, even if the value of an additive were lost over long periods of time, as has been argued, this might not matter so much if its value lasted through the critical early life of the pavement.

Late Season Construction

Generally, pavements placed late in the year require some form of sealing; but, those placed earlier, and therefore subjected to traffic during a season of warm weather, are much more likely to perform satisfactorily without being sealed. The possibility exists that compaction procedures need to be revised for pavement placed late in the year and that an effort should be made to duplicate, by rolling, the effect of traffic on the pavement during warm weather. Logically, this change in compaction procedure would be an increase in the amount of rubber-tired rolling while the mat is still warm. Although more rolling may be necessary to obtain the specified mat density, the specifications now do not require any different rolling procedures during cooler parts of the construction season. Density alone is not the criterion of a pavement that will adequately resist the effects of water. Rubber-tired traffic in warm weather tends to knead the pavement surface, working some asphalt mortar to the surface in much the same manner as it works fresh concrete and brings concrete mortar to the surface. It is doubtful whether the effect of traffic in warm weather can be duplicated on the surface of pavements laid late in the season. For example, the lower part of asphalt paving material placed on a cold base will cool quickly while the top remains so warm as to prohibit heavy rolling. It is believed that considerable improvement can be made in procedures for cold weather compaction.

Seal Coats

Not all pavements require seal coats b[e]cause of the effect of water on the aggrega[te-] asphalt combination. Raveling often sta[rts] because many mats become dry and brittle [as] time passes. Probable causes of this co[n]dition include weathering or hardening [of] the asphalt or selective absorption of t[he] asphalt into absorptive aggregates. Se[al] coats generally consist of a coarse-grad[ed] gravel or crushed rock cover aggrega[te] having a maximum seal of about thre[e-] eighths to one-half inch, applied at abo[ut] 20 pounds per square yard over a rapid curi[ng] (RC) cutback used at the rate of abo[ut] 0.20 gallon per square yard. In most plac[es] the results have been good, but there ha[ve] also been numerous exceptions where, b[e]cause of adverse weather, uncontrolled traff[ic] or a stripping type of cover aggregate, t[he] chips have failed to adhere to the binder, a[nd] the result has been a black, shiny, stic[ky] nonuniform surface, as shown in figure [2]. On some pavements, a sand or sand-grav[el] cover aggregate has been used, but the r[e]sults have generally been less uniform a[nd] satisfactory than a regular type chip se[al] although the sand or sand-gravel seals ha[ve] waterproofed the surface effectively.

A third type of seal frequently used wh[en] weather is not suitable for applying a chip [or] sand seal (as when a mat has been complet[ed] late in the construction season) is the so-call[ed] fog seal or black seal. A dilute emulsi[on] (SS–1) or a light RC or MC is used witho[ut] cover aggregate. The seal must, of cours[e,] be applied at a very light rate or the resu[lt] will be a slippery surface. The SS–1 has a[n] advantage in this regard because it may b[e] diluted with water to any desired extent, t[o] provide a complete yet very thin cover

Figure 2.—Unsatisfactory (excessively rich) seal coat.

sphalt. Rate of application is about 0.02 to 05 gallon of asphaltic residue per square ard.

ype of seal coat

Another type of seal that is coming into in-eased use is the plant-mixed seal in which a mi-open-graded, crushed cover aggregate aving a maximum size of three-eighths inch plant mixed with asphalt cement and spread y a regular paver at about 60 pounds per quare yard, as shown by figure 3. The mix very rich (6 to 7 percent asphalt) and must e mixed at a relatively low temperature to ermit retention of the required thick film of sphalt. As with the mat itself, it may be ecessary to use an antistripping asphalt lditive or hydrated lime in the mixture to rovide adequate resistance to stripping. If is is required, as in uncoated cover aggre-ates used in seal coats or surface treatments, e need for an antistripping agent is deter-ined by the static-immersion test (AASHO esignation: T 182), which requires 95 per-nt asphalt retention. The data in table 2 ow typical results in this test. Typical ading requirements for aggregates in plant-ixed seals are as follows:

Sieve size	*Percent passing*
⅜-in	100
No. 4	30–50
No. 8	15–30
No. 40	0–10
No. 200	0– 3

This type of seal, if a large quantity is re-iired, or if the contractor is already set up r hot plant-mix work, becomes less expen-ve than the chip seal and is far superior in ppearance and durability.

Another type of seal that is becoming in-easingly accepted is the so-called slurry seal that uses a sand, mineral filler and a dilute SS-1 emulsion. This seal is very inexpensive, but the results have not been as consistently good as those obtained by use of the plant-mixed seal previously described.

Table 2.—Static-immerson test results

Project number and source of data	Additive		Asphalt		
	Type	Amount, percentage	Type	Source	Percentage stripped from aggregate
Utah, No. 1	0	0	SC-3	Unknown	85–90
	A	1	SC-3	Unknown	45–50
	0	0	RC-4	A	85–90
	A	1	RC-4	A	5–10
	0	0	RC-4	B	85–90
	A	1	RC-4	B	0
	0	0	120–150[1]	Unknown	70–75
	A	1	120–150[1]	Unknown	0
Utah, No. 2	0	0	SC-3	Unknown	75–80
	A	1	SC-3	Unknown	25–30
	0	0	RC-4	A	60–65
	A	1	RC-4	A	5–10
	0	0	RC-4	B	15–20
	A	1	RC-4	B	0
Utah, No. 3	0	0	RC-4	A	70–75
	A	1	RC-4	A	2–5
	0	0	RC-4	B	0
	A	1	RC-4	B	0
Utah, No. 6	0	0	SC-3	Unknown	65–70
	A	1	SC-3	Unknown	5–10
	0	0	RC-4	A	70–75
	A	1	RC-4	A	2
	0	0	RC-4	B	2–5
	A	1	RC-4	B	0
	0	0	120–150[1]	B	5–10
	A	1	120–150[1]	B	0
Utah, 182-SA-61	0	0	MC-3	B	90–95
	C	1	MC-3	B	20–25
	0	0	RC-4	A	30–35
	B	1	RC-4	A	0–2
	0	0	120–150[1]	A	5–10
	B	1	120–150[1]	A	0–2
	0	0	RS-1 anionic.[2]	Unknown	10–15
	0	0	RS-2 cationic.[2]	Unknown	0–2
	0	0	RS-2 anionic.[2]	Unknown	10–15

[1] Penetration grade.
[2] Asphalt emulsions.

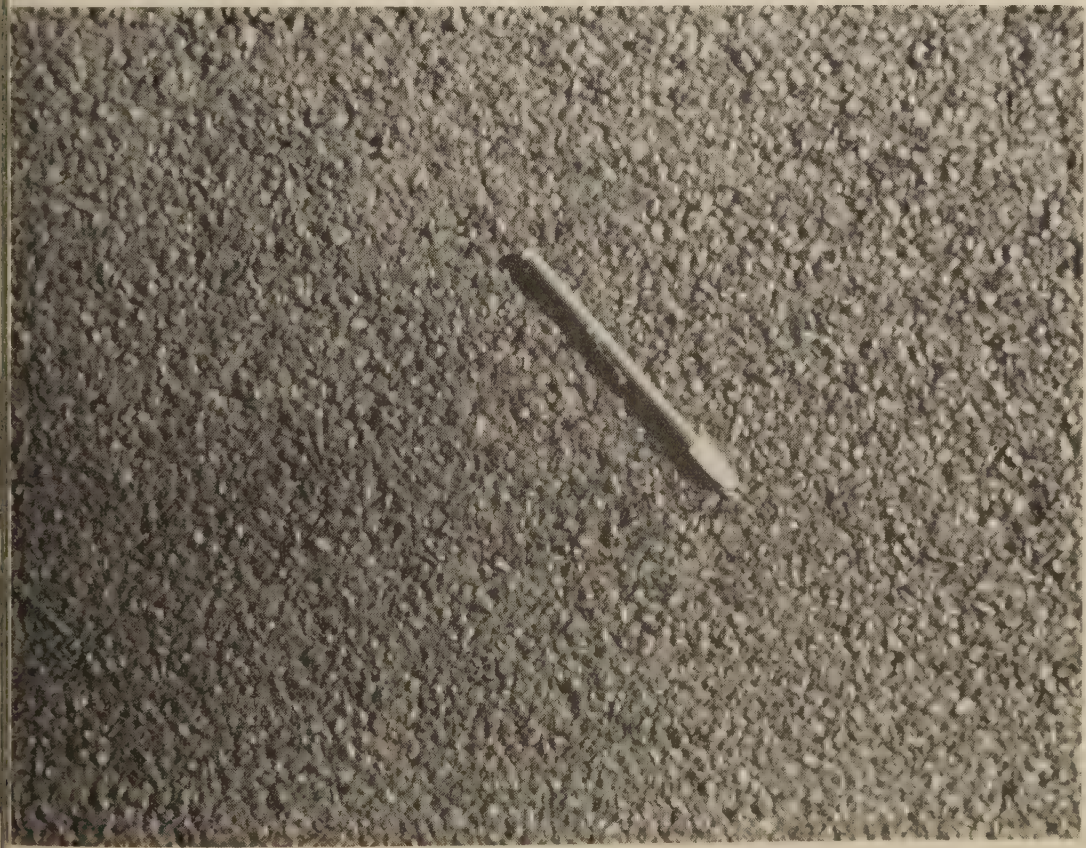

Figure 3.—Close-up of plant-mixed seal coat.

Other areas

Engineers from other parts of the country—particularly from farther east—often express surprise when mention is made of seal coating hot plant-mix or asphaltic concrete pavements. Their experience has lead them to expect that a properly constructed pavement will have adequate resistance to water and they, therefore, consider a seal coat to be entirely unnecessary. Even allowing for the possibility that some pavements get seal coated when they do not need it, a significant difference seems to remain between the water resistance of pavements in the Rocky Mountain area and those in some other areas. Just what causes this difference and what can be done about it poses a problem. Are the materials different, is something lacking in the construction procedures, or is the difference in climate a significant factor: Unquestionably, there are many hydrophilic aggregates in the study area, but this is probably true elsewhere as well.

Wyoming adopted new design criteria and specifications for plant-mixed surface for the 1962 season. The new specifications require: (1) the aggregate to be 100 percent crushed (including the fines), (2) a grading straddling a maximum density curve (0.45 exponential chart—(*9*)), and (3) a compacted density of at least 95 percent of Marshall specimens prepared according to the procedure outlined in the *Tentative Method of Test for Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus* (ASTM Designation: D 1559). The design criteria require a minimum index of retained stability of 70 for hightype plant mixture, 75 to 85 percent of voids filled with asphalt, and an air void content of 3 to 5 percent in the compacted pavement. Commonly, hydrated lime is needed as an additive to obtain an index of retained stability of the required 70 or more.

Some difficulty has been encountered in obtaining the specified 95 percent of Marshall density—probably because of the high mechanical stability of the 100 percent crushed aggregate—and, also for the same reason, it has often been necessary to waste a very high percentage of the pit material in gravel deposits. This is not only expensive but, in some areas, there just is not enough aggregate available to permit such extravagent use. Consequently, the specifications have been revised to require only 50 percent of the material retained on the No. 4 sieve to be crushed, so most of the pit fines will now be used. Admittedly, the standards have thereby been lowered. The pit fines frequently contain undesirable portions of the deposit, but any undesirable characteristics will normally be corrected by the need to comply with the index of retained stability of 70.

Effect of PI

In a series of tests run on four typical Wyoming aggregates, engineers in the Bureau of Public Roads determined that aggregates having PI values of 5 and 6 gave indices of retained stabilities of only 55 and 60 (using Marshall specimens); whereas, when these same aggregates had their plastic fines removed by washing and they were replaced by limestone fines, the indices increased to 77 and 86. They also learned that if the proper minimum index of retained stability were 70, using standard 4- × 4-inch, double plunger, compacted specimens tested for unconfined compression, then the corresponding minimum for Marshall specimens would be 75. Aggregates have seldom been washed for use in bituminous pavements in Wyoming but, from these tests, washing seems to be one method of improving pavement quality.

The normally accepted limit of a PI of 6 for the fine portion of the aggregate proved to be too high for a great many mixtures, and the general opinion is that the specifications for plant-mix aggregates should require the fines to be nonplastic. Even when specifications do not require nonplastic fines, the specifications for an index of retained stability of 70 requires the addition of hydrated lime to most plant-mix aggregates that contain plastic pit fines.

Factors Influencing Immersion-Compression Results

The immersion-compression tests generally show that the indices of retained stability are higher when heavier asphalts are used and when more asphalt is used. The addition of hydrated lime permits the use of more asphalt and increases dry and wet stabilities. Wet stabilities when lime has been used in the paving mixture are often higher than corresponding dry stabilities, possibly because of continued reaction between lime and aggregate fines during the wet soaking period. Chemical additives are just as effective at increasing the index of retained stability when used with some materials as is hydrated lime, [illegible] the chemical additives do tend to reduce [illegible] dry stabilities.

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Stability of Bituminous Pavements Related to Aggregate Characteristi[illegible]

Materials Research Division
Bureau of Public Roads

by[1] **ROBERT E. OLS[illegible]**
Highway Research Engin[illegible]

COMMENTS made by Mr. Olsen, in relation to the information on stability of bituminous pavements presented by Mr. Eager, were predicated upon results of tests made in the Bureau of Public Roads Laboratory, Washington, D.C. Mr. Olsen's comments were, as follows:

Mr. Eager has reviewed the problems associated with the use of local aggregates in bituminous construction in some of the Western States and reviewed several practices that are followed in their utilization. The Materials Research Division of the Bureau of Public Roads has been interested in and has followed State practices in bituminous construction in these States for several years and on a number of occasions has cooperated in studies.

A recent study was made to determine [illegible] relative quality characteristics of aggrega[illegible] from four sources in the State of Wyomi[illegible] Some of the data collected pertaining to [illegible] physical characteristics of these aggrega[illegible] are shown in table 1; these include aggreg[illegible] gradation, sand equivalent test results, liq[illegible] limit and plasticity index results on the mi[illegible] No. 40 sieve aggregate fraction and mi[illegible] No. 200 sieve fraction, and hydrometer ana[illegible] sis of the minus No. 200 sieve material.

[1] Presented at the 43d annual meeting of the Highway Research Board, Washington, D.C., January 1964.

ble 1.—Physical characteristics of aggregates from Wyoming

	Aggregate source			
	P	PR	R	GC
Gradation of aggregate:				
Passing sieve size:				
3/4-inch ... percent	100	100	100	100
1/2-inch ... do	90	88	94	90
3/8-inch ... do	84	71	85	78
No. 4 ... do	70	40	63	53
No. 10 ... do	55	25	48	37
No. 20 ... do	39	20	34	30
No. 40 ... do	28	17	25	21
No. 80 ... do	17	12	14	10
No. 200 ... do	7.6	8.0	9.4	5.9
Hydrometer analysis of aggregate:				
Passing No. 200 sieve ... percent	100	100	100	100
Smaller than:				
0.50 mm ... do	83	87	84	88
0.20 mm ... do	55	55	45	56
0.005 mm ... do	36	31	21	28
0.002 mm ... do	29	23	14	18
0.001 mm ... do	24	19	11	14
Clay in total aggregate [1] ... percent	2.7	2.5	2.0	1.6
Sand equivalent value	28	24	31	46
Plasticity index:				
Minus No. 40 sieve fraction	5	6	NP	NP
Minus No. 200 sieve fraction	21	13	8	8

[1] Calculated from percent finer than 0.005 mm.

)ne phase of the study included the determination of the effect of water on bituminous mixtures prepared with these aggregates. st specimens were prepared at previously ermined optimum asphalt contents and ted by the immersion-compression test for lex of retained strength following the procedure given in ASTM Methods D 1074 and D 1075. The immersion period was 4 days at 120° F. In addition, the effects of water on the stability of 50 blow Marshall specimens were determined for mixtures of the same composition and using an immersion period of one day at 140° F. The same asphalt, an 85–100 penetration grade, was used for all mixtures. The results of these tests and related physical characteristics of the molded specimens are shown in table 2.

Table 2.— Results of tests on bituminous mixtures made with aggregate from Wyoming

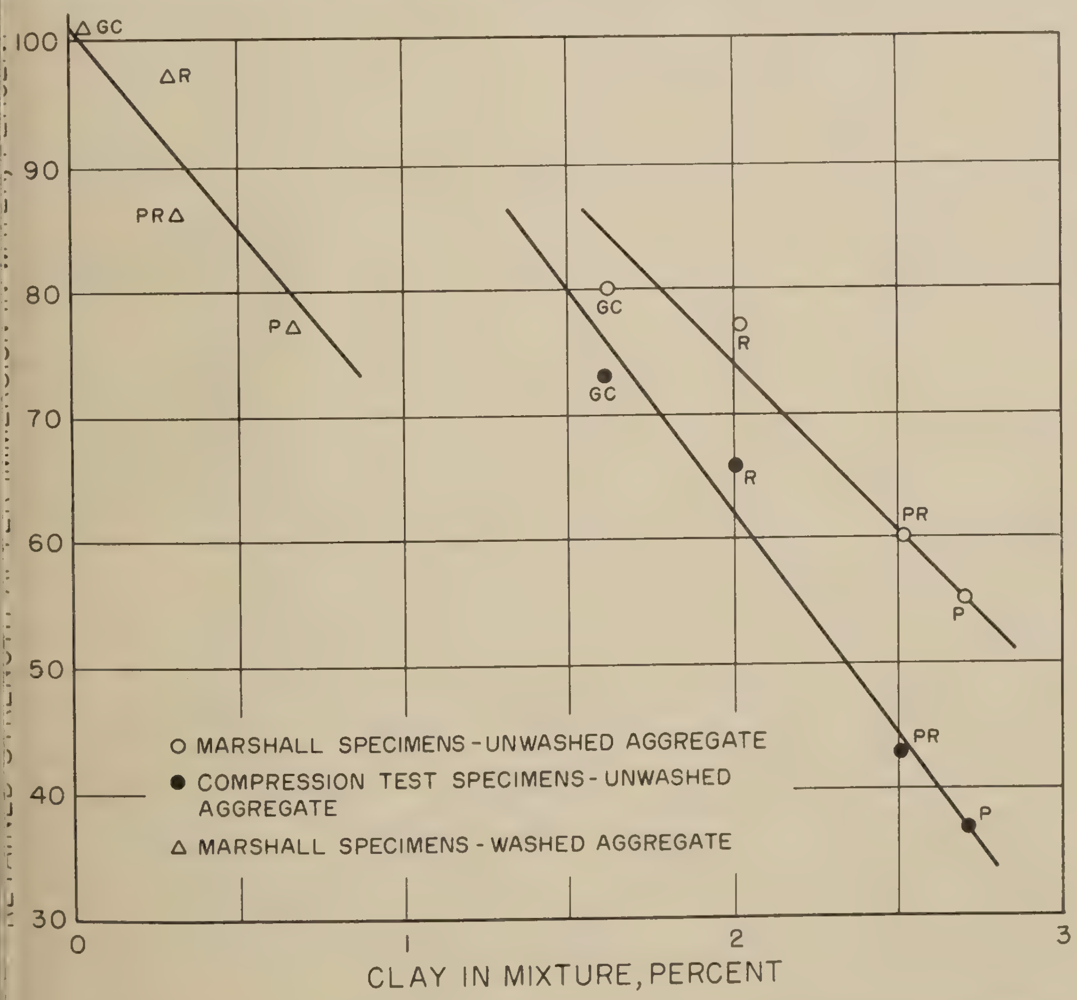
Characteristics of bituminous mixtures	Aggregate source			
	P	PR	R	GC
Aggregate not washed:				
Asphalt content, percent-mix basis	6.10	5.25	5.75	4.50
4-inch by 4-inch specimens:				
Compressive strength, dry ... p.s.i.	246	246	289	249
Compressive strength, wet ... p.s.i.	92	105	192	181
Index of retained strength	37	43	66	73
Volumetric swell ... percent	4.6	3.4	1.6	1.2
Air voids ... do	7.8	6.7	6.5	6.9
Marshall specimens:				
Stability, dry ... pounds	1,646	1,525	2,044	1,678
Stability, wet ... do	900	910	1,570	1,350
Index of retained stability	55	60	77	80
Air voids ... percent	4.3	4.3	3.9	3.7
Aggregate washed:				
Asphalt content, percent-mix basis	5.60	4.75	5.25	4.00
Percent dust after washing	1.8	1.0	1.3	0.8
Percent added limestone dust	2.0	3.0	3.4	2.2
Total passing No. 200 sieve ... percent	3.8	4.0	4.7	3.0
Marshall specimens:				
Stability, dry ... pounds	1,192	1,252	1,560	1,236
Stability, wet ... do	912	1,074	1,508	1,249
Index of retained stability	77	86	97	101
Air voids ... percent	6.3	4.8	4.3	4.8

A comparison of the data in tables 1 and 2 shows increasing indices of retained strength and decreasing percentages of volumetric swell with decreasing values of plasticity index of the material passing the No. 200 sieve and percent clay (material finer than 0.005 mm).

To further evaluate these aggregates and to isolate the effect of clay in the bituminous mixture, a series of tests was made with most of the naturally occurring dust, or material passing the No. 200 sieve, removed by washing. Limestone dust was then added in amounts required to bring the total percentage of minus No. 200 sieve material to equal one-half of that which the respective aggregates originally contained. The asphalt contents of these mixtures were reduced by one-half percent in order to ensure that the air voids would be high enough to allow water to enter the molded specimens and that the asphalt film thickness would not be so great, so as to protect the aggregate particles from the effect of water.

The results of tests of mixtures using the washed aggregates are also shown in table 2. It will be noted that the level of dry strengths is lower for the washed aggregate mixtures than the unwashed aggregate mixtures in each case; this, however, should be expected with the reduced dust and asphalt contents. The index of retained strength of each mixture, however, is from 25 to 50 percent higher than for the comparable mixtures using the unwashed aggregates. This supports Mr. Eager's statement that the washing of aggregate may be a logical step toward the up-grading of local aggregates.

Figure 1 shows graphically the relationship of percent retained strength of molded specimens to percent clay in the respective aggregates. The percent clay in the washed aggregates, as shown in the figure, is based on the percent passing the No. 200 sieve after washing and the percentage of material finer than 0.005 mm., as determined by the hydrometer analysis of the original fines. The percentages of clay in the washed aggregates are, therefore, not absolutely correct.



igure 1.—Relationship of percentage of clay in bituminous mixtures and index of retained strength.

THE NATIONAL SYSTEM OF INTERSTATE AND DEFENSE HIGHWAYS

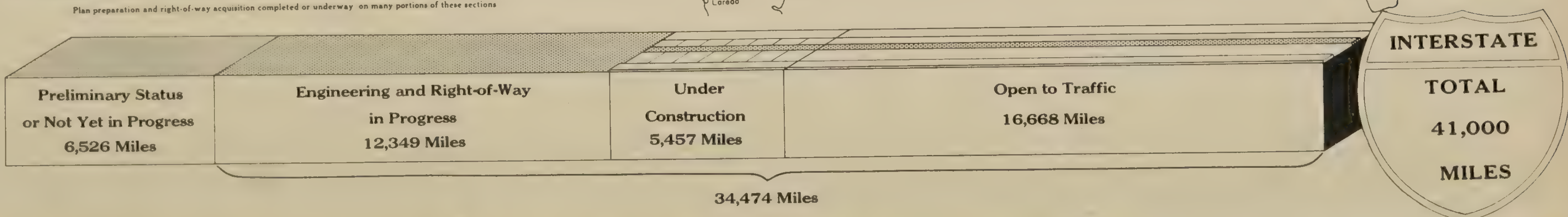
STATUS OF IMPROVEMENT AS OF MARCH 31, 1964

COMPLETED OR IMPROVED AND OPEN TO TRAFFIC
Completed to full or acceptable standards, or improved to standards adequate for present traffic; built with Interstate or other public funds

MAJOR TOLL ROADS
Incorporated in the Interstate System

UNDER CONSTRUCTION

PRELIMINARY STATUS OR NOT YET IN PROGRESS
Plan preparation and right-of-way acquisition completed or underway on many portions of these sections



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