Effect of Moisture on Bituminous Pavement In Rocky Mountain Areas

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The paper deals with problems in Colorado, New Mexico, Utah and Wyoming resulting from the preferential affinity of many locally available aggregates for water rather than for asphalt. Surface raveling and, in extreme cases, softening and complete disintegration of the bituminous pavement result. The characteristic is commonly referred to as "stripping," meaning that the asphalt coating comes off the aggregate in the presence of water and is replaced by the water. It is more pronounced with the lighter types of asphaltic materials, such as cutbacks, used in road mixes and has been one of the principal reasons for the general replacement of road mixes by hot plant mixes and asphaltic concrete using asphaltic cements. Even with higher types of pavements the problem is common enough and serious enough to require the general use of seal coats, either immediately or after a period of several years. Types of seal coats used are described. Another common approach to the problem has been the use of chemical additives and, more recently, hydrated lime. Their effect and methods of specifying and use are described.

Early realization of the stripping problem led to the development of the immersion-compression test (AASHO Designation T-165) for measuring the effect of water on compacted bituminous mixtures and of the static-immersion test (AASHO Designation T-182) for determining the effect of water on coated coarse aggregate particles (used in surface treatment and seal coats).

The paper is not intended to be a technical analysis of the chemistry or mechanics of the stripping action, nor does it present any guaranteed solutions. It does, however, point up the extent and seriousness of the problem with the hope that it will stimulate further research leading to more consistently water-resistant pavements.

•THE PROBLEM of the effect of water on bituminous paving mixtures is certainly not limited to the Rocky Mountain area (1, 2). Ideally, water should have no effect on pavement, but all too often adhesion has been reduced between the asphalt and aggregate to the point that serious "stripping" has occurred with resultant loss of mat stability and often severe "raveling" of aggregate from the surface. Stripping is defined as the loss of asphalt films from aggregate surfaces in the presence of moisture (9), and raveling as the loss of aggregate particles in 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 has even occurred to a serious extent in the hot plant-mix and asphaltic concrete types. The Rocky Mountain area is generally dryer than other parts of the country so it might be expected the problem would be less severe than elsewhere. However, this lack of moisture may well make the effect of water more pronounced when it does become available. Water has not been available to leach out deleterious

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fines or disintegrate soft particles in natural deposits of aggregates, as is the case in areas of greater rainfall. Possibly because of this, degradation is common with many of the aggregates available in the area. Although this degradation seldom increases plasticity significantly, it undoubtedly helps to reduce resistance of the pavement to the effects of water.

This problem may be common because of the widespread use of local, on-the-job aggregate deposits rather than those commercially produced. Specifications have commonly been written to fit these local aggregate sources rather than to require a high-quality standard aggregate.

In recent years, especially since the start of the Interstate program, there has been a general upgrading of specification requirements with the result that some aggregates formerly permitted are now excluded or, if permitted, are upgraded by improved processing (as pre-wasting of fines and use of screening equipment which removes and wastes coatings on the aggregate and breaks down and wastes softer aggregate particles) or by the addition of mineral filler, asphalt, cement, or hydrated lime. Washing of aggregates for bituminous construction has not yet come into general use, but as specification standards become higher and the availability of naturally high-grade aggregate becomes less, this will be a logical development.

EARLY EXPERIENCE

The early bituminous pavements in rural areas of the Rocky Mountains were nearly all of the road-mix type using slow-curing road oils and then cutbacks of the MC-types. It was commonly recognized that these surfaces lacked resistance to the effects of moisture, and seal coats were used as deterrents. In many cases, however, water still penetrated the pavement—probably by capillary action or as water vapor. Excess water in the mat was a common cause of early deterioration. Observations of certain pavements in Utah and Colorado in 1949 (3) indicated that all bituminous pavements containing more than 2 percent moisture failed and that the moisture content increased as the percentage of minus No. 200 aggregate increased. The maximum allowable minus No. 200 size appeared to be 12 percent. The tests did not indicate the Plasticity Index (PI) of the mat aggregate, higher PI values were probably associated with higher moisture contents with pavement failures.

Although seal coats prevent or retard surface raveling, they have not been effective, and occasionally have been detrimental, in preventing stripping within the mat. The general use of seal coats on road-mix mats seems to have carried over into use on hot plant-mix and bituminous concrete surfaces. Only in recent years has there been any tendency to leave seal coats off these higher type pavements and often then with reluctance and a common feeling that they will need seal coats within a short time. Too often this feeling has been borne out.

The seriousness of the problem was first brought forcibly to many people's attention by an experimental project in Colorado in 1941. Sections of road-mix bituminous surfacing using different sources and types of asphaltic materials were constructed with a local aggregate of quality adequate by the normally accepted standards at that time. However, all sections quickly showed serious distress with raveling taking place with the first rain. This necessitated prompt seal coating of all sections, thus obscuring any differences in the sections. A subsequent laboratory study of the aggregate by the Bureau of Public Roads Materials Research Division (4) led to the development of the immersion-compression test (AASHO Designation T-165 and ASTM Designation D-1075).

Even before the Colorado experiment there were some very unhappy experiences with asphalt stripping. At one time in the mid 30's, open-graded bituminous mat was tried on direct Federal construction projects. Because most aggregate sources were gravel, to obtain the 100 percent crushed aggregate required, the natural pit fines were wasted and the required aggregate was produced by crushing the oversized gravel. This gave a mat of high mechanical stability, but with high void space. To keep surface water out, the surface was choked with fines and seal coated. The aggregates were mainly granitic and although surface water may have been kept out and the surface looked good, the asphalt soon stripped badly from the coarse crushed aggregate in the lower parts of the mat, and these aggregate particles became coated with water instead of asphalt. The general opinion at the time was that the seal was ineffective and allowed surface water to enter and soften the mat. This type of construction was discontinued in favor of a return to the dense-graded type. However, there is no assurance that free water actually entered the mat from the surface. The open grading and the much greater affinity of the aggregate for water than for asphalt created conditions favorable to the development of stripping. The extreme temperature differentials in the high mountains could well have led to condensation of moisture in the large void space of the open-graded mat.

On one of the projects referred to previously, the State used the wasted pit fines to build a dense-graded road-mix mat on an adjacent project with excellent results. This does not prove that open-graded crushed aggregate mats are necessarily all bad and dense-graded ones all good, but rather that with hydrophilic aggregates the chances for moisture to penetrate and cause stripping and softening of the mat are much less with the denser mixture.

These projects were all built before the days of the immersion-compression test (AASHO T-165), the static-immersion test (AASHO T-182), or any other commonly accepted test to measure the effect of water on the aggregate-asphalt combination. In addition, commercial chemical anti-stripping agents, proprietary products that under some conditions greatly increase the affinity between asphalt and aggregate in the presence of water, had not yet come into general use.

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 having a wet stability of 70 or 75 percent of the dry stability. This ratio generally seems to be a good one for hot-mix or asphaltic concrete, although it is often difficult to achieve without some stabilizing admixture. Swanberg and Hindermann (8) recommended a minimum immersion-compression wet-dry stability of 75 percent. Utah considers the actual wet stability more significant than the wet-dry stability ratio and specifies a minimum wet stability of 150 psi. Wyoming accomplishes this by requiring a minimum dry stability of 250 psi and a wet-dry stability ratio of 70 percent which gives a minimum wet stability of 187 psi. This approach has considerable merit because mixtures of rather clean, coarse-graded aggregates often show a good immersion-compression ratio but may be too low in either dry or wet stability or density to make the best pavement. In addition, chemical additives commonly reduce the actual dry stability value and increase the wet stability; that is, they bring the two values closer together, thereby increasing the wet-dry stability ratio. In this case, the ratio alone does not give a true measure of the nature of the mixture or of the effect of the additives.

In comparing wet-dry stability ratios, the method of compacting and testing the specimens must also be considered. For example, Colorado compacts the specimens with a kneading compactor as used for Hveem stabilometer tests. Specimens so compacted will probably show significantly higher actual stabilities and higher immersion-compression ratios than specimens compacted by the standard double-plunger direct compression method. Some unpublished tests by the Bureau of Public Roads Materials Research Division show that specimens compacted and tested by the standard Marshall method have a wet-dry stability ratio of 112 percent of specimens compacted and tested by the standard T-165 method.

IMPROVED DESIGN AND CONSTRUCTION STANDARDS

In recent years, standards of design and construction of bituminous plant mix and asphaltic cement have been upgraded in this region so that it is believed they are now comparable to standards used elsewhere. Minimum compacted densities are 95 percent of laboratory density (Marshall standard or kneading compactor density) and include air voids of 2 to 6 percent. Sufficient asphalt is used to fill at least 75 percent of the voids in the compacted aggregate. Generally, this requires about 6 percent of 85 to 100

TABLE 1

IMMERSION-COMPRESSION TEST RESULTS[a]

Project or Sample	Additive		Asphalt		Compressive Strength at 77 F		Wet-Dry Stab. Ratio (x 100)
	Type[b]	%[c]	Grade	%	Dry	Wet	
Colo. 2721	None	-	85-100	5.3	456	158	30
Colo. 2721	PC	2	85-100	5.7	444	570	128
Colo. 2721	HL	1	85-100	6.0	378	515	138
Colo. 43076	None	-	85-100	5.8	293	56	16
Colo. 43076	HL	1	85-100	6.3	284	286	101
Utah 169-SA-63	None	-	85-100	5.2	-	195	-
Utah 169-SA-63	HL	1	85-100	5.2	-	283	-
Utah 97-SA-63	None	-	85-100	5.3	-	45	-
Utah 97-SA-63	HL	1	85-100	5.3	-	154	-
Wyo. 2938	None	-	85-100	6.0	302	188	62
Wyo. 2938	None	-	85-100	6.5	309	223	72
Wyo. 2938	HL	2	85-100[d]	6.0	404	404	100
Wyo. 2938	HL	2	85-100[d]	6.5	313	363	116
Wyo. 2938	None	-	85-100[d]	6.0	332	272	82
Wyo. 2938	None	-	85-100[d]	6.5	315	296	94
Wyo. 5120	None	-	85-100[d]	6.0	234	89	38
Wyo. 5120	None	-	85-100[d]	6.5	258	95	37
Wyo. 5120	HL	2	85-100[d]	6.0	285	208	73
Wyo. 5120	HL	2	85-100[d]	6.5	312	204	65
Wyo. 5120	HL	3	85-100[d]	6.0	430	234	54
Wyo. 5120	HL	3	85-100[d]	6.5	425	225	53
Wyo. 5120	HL	4	85-100[d]	6.0	453	275	61
Wyo. 5120	HL	4	85-100[d]	6.5	442	309	70
Wyo. 5120	PC	3	85-100[d]	6.0	280	131	47
Wyo. 5120	PC	3	85-100[d]	6.5	290	143	49
Wyo. 5120	AF	1	85-100[d]	6.5	278	110	39
Wyo. 5120	AF	2	85-100[d]	6.5	283	124	44
Wyo. 5120	FA	2	85-100[d]	6.5	310	80	26
Wyo. 5120	FA	3	85-100[d]	6.5	307	94	31
Wyo. 5120	LWA	2	85-100[d]	6.5	248	67	27
Wyo. 5120	LWA	3	85-100[d]	6.5	243	86	35
Wyo. 62-521	None	-	85-100[d]	6.0	413	102	25
Wyo. 62-521	None	-	85-100[d]	6.5	418	159	38
Wyo. 62-521	HL	2	85-100[d]	6.0	366	255	70
Wyo. 62-521	HL	2	85-100[d]	6.5	344	235	69
Wyo. 62-313-317	None	-	85-100[d]	6.5	296	177	60
Wyo. 62-313-317	None	-	85-100[d]	7.0	305	204	67
Wyo. 62-313-317	HL	2	85-100[d]	6.5	342	310	91
Wyo. 62-313-317	HL	2	85-100[d]	7.0	334	313	94
Wyo. 62-313-317	Cement	2	85-100[d]	6.5	331	283	86
Wyo. 62-313-317	Cement	2	85-100[d]	7.0	305	253	83
Wyo. 62-313-317	None	-	85-100[d]	6.5	299	194	65
Wyo. 62-2778	None	-	85-100[d]	5.25	196	68	35
Wyo. 62-2778	None	-	85-100[d]	6.25	193	105	55
Wyo. 62-2778	None	-	85-100[d]	6.75	190	157	82
Wyo. 62-2778	None	-	85-100	6.25	195	82	42
Wyo. 62-2778	None	-	85-100	6.75	207	115	56
Wyo. 62-2778	HL	2	85-100[d]	6.25	222	179	80
Wyo. 62-2778	HL	2	85-100[d]	6.75	213	231	108
BPR C	None	-	60-70	5.8	183	84	46
BPR C	None	-	85-100	4.9	206	70	34
BPR C	HSA	1	85-100	4.9	143	70	49
BPR C	HL	1	85-100	4.9	277	130	47
BPR C	None	-	85-100	5.6	276	132	47
BPR C	HSA	1	85-100	5.6	250	109	44
BPR C	HSA	1	85-100	6.2	268	120	45
BPR C	HL	1.5	85-100	6.3	296	296	100
BPR D	None	-	85-100	5.6	227	226	100
BPR D	HSA	1	85-100	5.6	212	238	112
BPR D	HL	1	85-100	5.6	225	252	110
BPR E	None	-	MC-3	3.9	70	0	0
BPR E	HSA	1	MC-3	3.9	99	40	40
BPR E	HL	1	MC-3	5.0	95	46	48
BPR E	None	-	120-150 AC	5.8	291	0	0
BPR E	HSA	1	120-150 AC	5.8	290	327	113
BPR E	HL	1	120-150 AC	6.7	315	235	75
BPR F	None	-	MC-3	5.2	62	13	29
BPR F	HSA	1	MC-3	5.2	35	25	45
BPR F	HSA	1	MC-3	5.6	65	27	42
BPR F	None	-	120-150 AC	5.2	211	152	72
BPR F	HSA	1	120-150 AC	5.2	234	194	83
BPR F	HL	1	120-150 AC	5.6	262	219	84
BPR H	None	-	120-150 AC	5.3	374	139	37
BPR H	HSA	1	120-150 AC	5.3	405	307	76
BPR H	HL	1	120-150 AC	6.1	387	367	95
BPR I	None	-	MC-3	4.0	148	33	22
BPR I	HSA	1	MC-3	4.0	111	67	60
BPR I	HL	1.5	MC-3	4.0	184	130	72
BPR I	None	-	120-150 AC	5.2	366	298	81
BPR J	None	-	MC-3	4.7	-	0	0
BPR J	HSA	1	MC-3	4.7	102	55	54
BPR J	HL	1	MC-3	4.7	118	64	54
BPR J	HL	1.5	MC-3	4.7	147	94	64
BPR K	None	-	MC-3	3.7	126	33	26
BPR K	HL	1	MC-3	3.7	187	130	70
BPR K	None	-	120-150 AC	5.7	259	131	51
BPR K	HL	1	120-150 AC	5.7	286	225	79
BPR L	None	-	MC-3	4.4	175	15	9
BPR L	HSA	1	MC-3	4.4	192	35	18
BPR L	None	-	120-150 AC	5.4	297	165	56
BPR L	HSA	1	120-150 AC	5.4	256	181	21
BPR L	HL	1	120-150 AC	5.4	293	192	66
BPR M	None	-	120-150 AC	4.9	553	443	80
BPR M	HSA	1	120-150 AC	4.9	472	443	94
BPR M	HL	1	120-150 AC	4.9	518	468	90

[a] Immersion-compression tests by following methods:

1. Colo.: specimens compacted by kneading compactor with blows at 500 psi; testing strength, AASHO T-165 with 24-hr soaking at 140 F.
2. Utah and Wyo.: specimens compacted by double plunger at 3,000 psi (AASHO T-167); testing strength same as Colo.
3. BPR: specimens compacted as for Utah and Wyo.; testing strength, AASHO T-164 with 24-hr soaking at 140 F.

[b] HSA = concentrated heat stable additive; HL = hydrated lime; PC = portland cement; AF = asbestos fiber; FA = fly ash; LWA = light wt aggregate.

[c] By wt of aggregate.

[d] Fortified at refinery with anti-stripping additive of unknown type and amount.

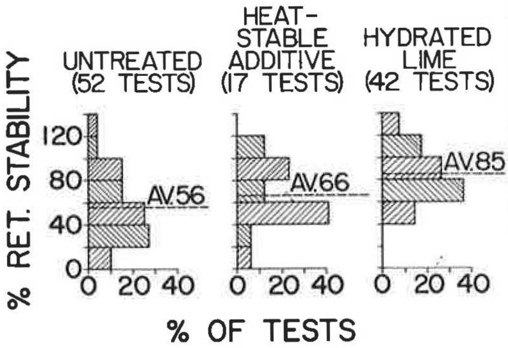

Figure 1. Effect of moisture on bituminous pavement in Rocky Mountain areas.

penetration grade. Retained stabilities in the immersion-compression test are 70 or 75 percent for Interstate and primary roads. Commonly, an additive, either chemical or hydrated lime, must be used to obtain these retained values. Normally, the bituminous mixture is tested without any additive and if the minimum wet-dry stability ratio is not obtained, different commercial chemical anti-stripping additives, hydrated lime, and in some cases, portland cement or other additives are tried. Generally, cement is not as effective as hydrated lime. In some cases, chemical additives give better results than hydrated lime, but in other cases the reverse is true (Table 1, Fig. 1).

EXPERIENCE WITH HYDRATED LIME AND OTHER TREATMENTS

A recent paper presented by Swanson (5) on the use of hydrated lime in asphalt paving mixtures contained rather startling results. For example, with one aggregate the wet stability without any lime was so low that the sample fell apart before it could be tested, but the same mix with 1 percent hydrated lime showed a dry stability of 482 psi and a wet stability of 442 psi for a a wet-dry stability ratio of 92 percent. There was no curing period after adding the hydrated lime. In other cases a curing period of 2 to 5 days was necessary for the lime to react with the aggregate before mixing with asphalt.

On one such job, wet-dry stabilities of 43, 85 and 117 percent, without hydrated lime, with 1 percent hydrated lime and no curing period, and 1 percent hydrated lime with a 48-hr curing period, respectively, were obtained.

The Bureau of Public Roads usually finds that some additive is necessary to obtain a wet-dry stability ratio of 70 percent or more. In some cases a commercial chemical additive is effective, but in others hydrated lime is found to be the more effective additive. Hydrated lime is also considerably more expensive in the proportion normally used (1 to 1.5 percent of weight of aggregate) than chemical additives where only 0.5 to 1.0 percent of the weight of asphalt is necessary. At these proportions, hydrated lime adds about $0.45 a ton to the cost of the mix, whereas the commercial chemical asphalt additive adds about $0.11 a ton. However, if the hydrated lime does the job and the chemical does not, then obviously the additional expenditure is justified. In such case, the proper cost comparison is between the added cost of the hydrated lime and the added cost of some other aggregate not needing the hydrated lime.

Seal coats cost about $0.11/sq yd or on a 3-in. thick mat about $0.70/ton of mix. This is more expensive than either the chemical additive at $0.11/ton of mix or the hydrated lime at $0.45/ton of mix. There is, in addition, no assurance that the seal coat will always prevent the undesirable characteristics of a hydrophilic aggregate from manifesting themselves. Moreover, there are so many uncertainties involved in seal coat construction that the results are somewhat a gamble.

Assuming it has been found that measured by the immersion-compression test, a chemical additive, hydrated lime, or other additive is effective, field behavior should be compared with the laboratory tests. Goldbeck (6) did not find much correlation; however, the Colorado experiment (4) showed good correlation. Whereas it is logical to expect a mix having a high wet-dry stability ratio to be better than one with a low ratio, there are other factors affecting the resistance of the mat to moisture and in some cases they may be of the greater significance and may obscure the action taking place in the immersion-compression test. If the mat is so dense, so well mixed, or so well sealed over, either by compaction or warm weather traffic or by a seal coat, that water does not penetrate it, then stripping, swelling, and loss of stability cannot take place. If it were possible to be sure of keeping water out of the mat and off its surface, there would need be no concern with stripping, 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.

Whereas there is no exact correlation between immersion-compression values and behavior on the road, there are many cases of stripping, raveling, softening of the mat, and obviously inadequate resistance to the effects of water. The use of chemical additives or hydrated lime will certainly not eliminate these problems because such materials cannot compensate for an inadequately designed or constructed mix, but experience shows they do help.

There are an increasing number of pavements suffering little damage even though not sealed. However, the proportion of lasting as long as 10 yr or even 5 yr without a seal coat is not large. Whether all projects sealed actually need sealing is questionable. The need for sealing is often a matter of personal opinion and some engineers and maintenance men might say a pavement needs sealing, whereas others, with a different background of experience, might not. Because the need for a seal usually develops in winter or spring when it is not possible to do anything about it, it is understandable why some pavements get precautionary seal coats when they might get by without them.

The first winter is usually the critical time because pavements seem to develop increased resistance to the effect of water with time and warm weather traffic. Thus, the loss of an additive's value with time might not matter if its value lasted through the critical early life of the pavement.

LATE-SEASON CONSTRUCTION

As a general rule, pavements placed late in the season require some form of sealing, whereas those placed earlier and subjected to a season of warm-weather traffic before winter arrives are much more apt not to need sealing. Compaction procedures may need to be revised for pavement placed late in the season in order to duplicate by rolling what traffic does to the pavement during warm weather. Logically, this would be an increased amount of rubber-tired rolling while the mat is still warm. Whereas some increased rolling might be performed if necessary to obtain the specified mat density, the specifications do not require any different rolling procedures during cooler parts of the year. Density alone is not the criterion of a good pavement surface to adequately resist the effects of water. Warm-weather rubber-tired traffic tends to knead the pavement surface, resulting in working some asphalt mortar to the surface in much the same manner as working fresh concrete brings concrete mortar to the surface. It is doubtful if it is possible to duplicate the effect of warm-weather traffic on pavements laid late in the season. Asphalt paving material placed on a cold base will cool off quickly in its lower part, whereas the top might still be too warm to permit heavy rolling. However, considerable improvement could be made in cold-weather compaction procedures.

SEAL COATS

Not all seal coats are made necessary by the effect of water on the aggregate-asphalt combination. Many mats become dry and brittle with time, and raveling then starts. Probable causes of this include weathering or hardening of the asphalt or selective absorption of the asphalt into absorptive aggregates.

Seal coats used have generally been of the type using a coarse-graded gravel or crushed rock cover aggregate of about $\frac{3}{8}$- to $\frac{1}{2}$-in. maximum size applied at about 20 lb/sq yd over an RC cutback used at the rate of about 0.20 gal/sq yd. Most results have been good, but there have also been numerous exceptions when, because of adverse weather, uncontrolled traffic, or a stripping type of cover aggregate, the chips have failed to stick, producing a black, shiny, sticky nonuniform surface (Fig. 2).

Figure 2. Unsatisfactory (excessively rich) seal coat.

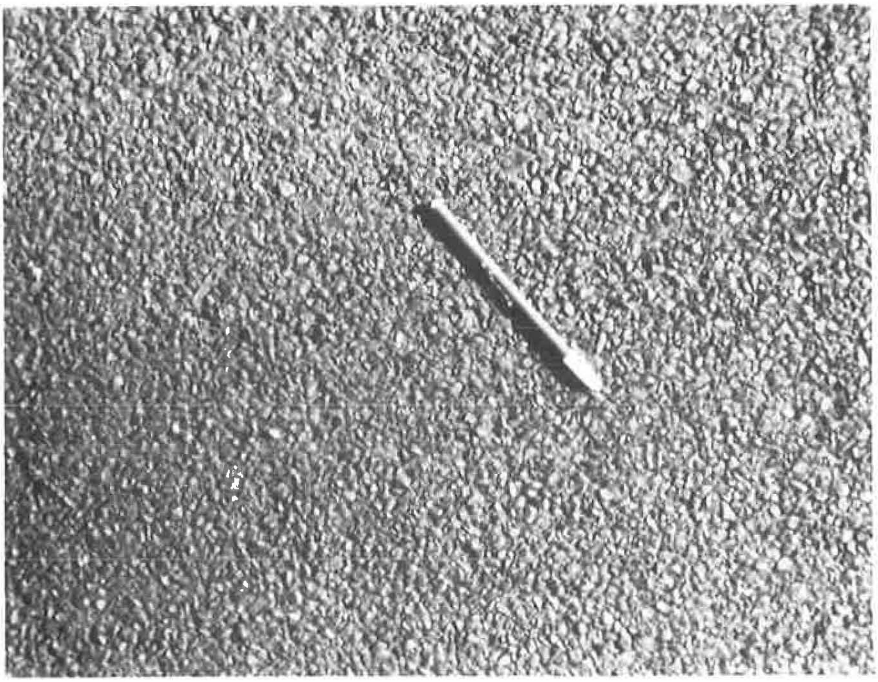

Figure 3. Close-up of plant-mixed seal coat.

In some cases, a sand or sand-gravel cover aggregate has been used, but the results have generally been less uniform and satisfactory than a regular chip seal, although the sand or sand-gravel seals have waterproofed the surface effectively.

A third type of seal frequently used when weather is not suitable for applying a chip or sand seal (as when a mat has been completed late in the construction season) is the so-called fog seal or black seal. A dilute emulsion, SS-1, or light RC or MC cutback is used without cover aggregate. The seal must be applied at a very light rate to prevent a slippery surface. The SS-1 has an advantage in this regard, because it may be diluted to any desired extent with water to provide a complete, yet very thin, cover of asphalt. Rate of application is about 0.02 to 0.05 gal of asphaltic residue per square yard.

A fourth type coming into increased use is the plant-mixed seal in which a semi-open-graded crushed cover aggregate of ⅜-in. maximum size is plant mixed with asphalt cement and spread with a regular paver at about 60 lb/sq yd (Fig. 3). The mix is quite rich (6 to 7 percent asphalt) and must be mixed at a relatively low temperature to permit retention of the required thick film of asphalt. As with the mat itself, an anti-stripping asphalt additive or hydrated lime must often be used to provide adequate resistance to stripping. As in uncoated cover aggregates used in seal coats or surface treatments, the need for an anti-stripping agent is determined by the static-immersion test (AASHO Designation T-182) with 95 percent retention required. Table 2 gives typical results in this test. Typical grading requirements for aggregate in plant-mixed seal are as follows:

Passing ⅜-in. sieve, 100 percent;
Passing No. 4 sieve, 30 to 50 percent;
Passing No. 8 sieve, 15 to 30 percent;
Passing No. 40 sieve, 0 to 10 percent; and
Passing No. 200 sieve, 0 to 3 percent.

This type of seal, if a large quantity is involved or where the contractor is already set up for hot plant-mix work, becomes less expensive than the chip seal and is far superior in appearance and durability.

TABLE 2

STATIC-IMMERSION TEST RESULTS

Utah Project	Type Asphalt	Additive	Stripping (%)
1	SC-3	-	35-90
		1% A	45-50
	RC-4[a]	-	85-90
		1% A	5-10
	120-150	-	70-75
		1% A	0
	RC-4[b]	-	85-90
		1% A	0
2	SC-3	-	75-80
		1% A	25-30
	RC-4[a]	-	60-65
		1% A	5-10
	RC-4[b]	-	15-20
		1% A	0
3	RC-4[a]	-	70-75
		1% A	2-5
	RC-4[b]	-	0
		1% A	0
6	SC-3	-	65-70
		1% A	5-10
	RC-4[a]	-	70-75
		1% A	2
	RC-4[b]	-	2-5
		1% A	0
	120-150[b]	-	5-10
		1% A	0
182-SA-61	MC-3[b]	-	90-95
		1% C	20-25
	RC-4[a]	-	30-35
		1% B	0-2
	120-150[a]	-	5-10
		1% B	0-2
	RS-1[c]	-	10-15
	RS-2[d]		0-2
	RS-2[c]		10-15

[a]Asphalt source. [b]Asphalt source B.
[c]Anionic. [d]Cationic.

Another type of seal receiving increasing acceptance is the so-called slurry seal using a sand, mineral filler and dilute SS-1 emulsion. This seal is inexpensive, but the results have not been as consistently good as the plant-mixed seal described.

When mention is made of seal coating hot plant-mix or asphaltic concrete pavements, surprise is often expressed by engineers from other parts of the country—particularly from farther east—experience has led them to expect a properly constructed pavement to 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 unnecessarily, there still remains a significant difference between the water resistance of pavements in the Rocky Mountain area and those in some other areas. The difference may arise from difference in materials, in climate or something lacking in construction procedures. There are many hydrophilic aggregates, but then probably this is true elsewhere as well.

As an example of what States in the Rocky Mountain area are doing about the problem, Wyoming adopted new design criteria and specifications for plant-mixed surface for the 1962 season. The new specifications require the aggregate to be 100 percent crushed (including fines), provide a grading straddling a maximum density curve (0.45 exponential chart, 7), and require a compacted density of at least 95 percent of standard Marshall (ASTM D-1559). The design criteria requires a minimum wet-dry stability ratio of 70 percent for high-type plant mix, 75 to 85 percent of voids filled with asphalt, and a net void content of 3 to 5 percent in the compacted pavement. Commonly, hydrated lime must be added to obtain the required 70 percent immersion-compression ratio.

Probably because of the high mechanical stability of the 100 percent crushed aggregate, some difficulty has been encountered in obtaining the specified 95 percent plus of Marshall density a very high percentage of the pit material in gravel deposits has had to be wasted. This is not only expensive, but in some areas there is not enough aggregate available to permit such extravagant use. Consequently, the specifications have now been revised to require only 50 percent crushed particles in the plus No. 4 size, and most of the pit fines will now be used. Admittedly the standards have been thereby lowered. In many cases, the pit fines contain undesirable portions of the deposit, but any undesirable characteristics will probably be corrected by the necessity of complying with the 70 percent minimum immersion-compression ratio.

EFFECT OF PI

Tests run on four typical Wyoming aggregates by the Bureau of Public Roads showed that aggregates having PI values of 5 and 6 only gave wet-dry stability ratios of 55 and

60 percent (using Marshall specimens), whereas when these same aggregates had their plastic fines removed by washing and replaced by limestone fines, the ratios jumped to 77 and 86 percent. If the proper minimum wet dry stability ratio is 70 percent using standard 4- by 4-in. double-plunger compacted specimens tested for unconfined compression, the corresponding minimum for Marshall specimens is 75 percent. Washing of aggregates for bituminous pavements has seldom been practiced in this area, but from these tests, it appears to offer one method of improving pavement quality.

The normally accepted limit of 6 PI is proving too high in a great many cases and the general opinion is that the specifications for plant-mix aggregate should require the fines to be nonplastic. Even where specifications do not require nonplastic fines, the 70 percent minimum wet-dry stability ratio requirement makes it necessary to add hydrated lime in most cases where the pit fines have plasticity, thereby resulting in nonplastic fines.

FACTORS INFLUENCING IMMERSION-COMPRESSION RESULTS

The tests generally show better results in the immersion-compression test when heavier asphalts and more asphalt are used. Hydrated lime permits the use of more asphalt and increases dry and wet stabilities. Wet stabilities with lime are often higher than corresponding dry stabilities, possibly because of continued reaction between lime and aggregate fines during the wet soaking period. With some materials, chemical additives are as effective as hydrated lime at increasing wet-dry stability ratios, but tend to reduce the dry stabilities somewhat.

SUMMARY

Bituminous pavements in the Rocky Mountain area have always lacked resistance to the disintegrating effect of water. When road mixes were replaced with plant mixes the problem was reduced, but still existed. Additives—both chemical and hydrated lime—have helped the situation as have closer quality control of the materials and construction procedures. 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. So far there is a way to go before reaching that goal and it is necessary to know just what further changes can be made in design, materials, or in construction procedures to achieve it. Results are improving, but there does not yet seem to be any positive, completely effective solution to the problem. There are indications, however, that considerable improvement in reducing the stripping and raveling problems can be made by close attention to the following details:

1. Determining in advance by laboratory tests, the probable action of the compacted paving mixture in the presence of water. The immersion-compression test is recommended for this purpose.
2. On basis of these tests, either eliminating unsatisfactory aggregates, improving them by screening or washing, or compensating for them by suitable admixtures (chemical anti-stripping additives, hydrated lime, filler or other aggregate sizes to improve gradation).
3. Following good design procedures for the paving mixtures and maintaining close construction control (particularly mixing times, temperatures, and compaction procedures, including more concentrated rolling when the mixture is still warm enough for the rolling to be effective).

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Discussion

ROBERT E. OLSEN, Materials Research Division Bureau of Public Roads.—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 the relative quality characteristics of aggregates from four sources in Wyoming. Some of the data collected pertaining to the physical characteristics of these aggregates are given in Table 3; these include aggregate gradation, sand equivalent test results, liquid limit and PI results on the minus No. 40 and minus No. 200 sieve aggregate fractions and hydrometer analysis of the minus No. 200 sieve material.

One phase of the study included the determination of the effect of water on bituminous mixtures prepared with these aggregates. Test specimens were prepared at previously determined optimum asphalt contents and tested by the immersion-compression test for percent retained strength following 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 1 day at 140 F. The same asphalt, an 85 to 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 4. A comparison of the data in Tables 3 and 4 shows increasing percentages of retained strength and decreasing percentages of volumetric swell with decreasing values of PI 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 one half that which the respective aggregates originally contained. The asphalt contents of these mixtures were reduced by 0.5 percent to insure 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 as to pretect the aggregate particles from the effect of water.

The results of tests of mixtures using the washed aggregates are also shown in Table 4. It will be noted that the level of dry stabilities 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 percent retained strength

TABLE 3

PHYSICAL CHARACTERISTICS OF AGGREGATES FROM WYOMING

Type	Passing Sieve (%)									Hydrometer Analysis (%)						Clay[a] (%)	Sand Equiv.	PI	
	3/4 in.	1/2 in.	3/8 in.	No. 4	No. 10	No. 20	No. 40	No. 80	No. 200	Passing No. 200	Finer Than							-No. 40 Fraction	-No. 200 Fraction
											0.50 mm	0.20 mm	0.0005 mm	0.002 mm	0.001 mm				
P	100	90	84	70	55	39	28	17	7.6	100	83	55	36	29	24	2.7	28	5	21
PR	100	88	71	40	25	20	17	12	8.0	100	87	55	31	23	19	2.5	24	6	13
R	100	94	85	63	48	34	25	14	9.4	100	84	45	21	14	11	2.0	31	-	8
GC	100	90	78	53	37	30	21	10	5.9	100	88	56	28	18	14	1.6	46	-	8

[a]In total aggregate calculated from percent finer than 0.005 mm.

TABLE 4

TEST RESULTS OF BITUMINOUS MIXTURES WITH AGGREGATES FROM WYOMING

Type	Unwashed Aggregate										Washed Aggregate							
	A. C. (%)	Compressive Strength[a] (psi)		Retained Strength[a] (%)	Swell[a] (%)	Voids[a] (%)	Stability[b] (lb)		Ret. Stab.[b] (%)	Voids[b] (%)	A. C. (%)	Dust (%)	Limestone Dust Added (%)	Passing No. 200 Sieve (%)	Stability[b] (lb)		Ret. Stab.[b] (%)	Voids (%)
		Dry	Wet				Dry	Wet							Dry	Wet		
P	6.10	246	92	37	4.6	7.8	1,646	900	55	4.3	5.60	1.8	2.0	3.8	1,192	912	77	6.3
PR	5.25	246	105	43	3.4	6.7	1,525	910	60	4.3	4.75	1.0	3.0	4.0	1,252	1,074	86	4.8
R	5.75	289	192	66	1.6	6.5	2,044	1,570	77	3.9	5.25	1.3	3.4	4.7	1,560	1,508	97	4.3
GC	4.50	249	181	73	1.2	6.9	1,678	1,350	80	3.7	4.00	0.8	2.2	3.0	1,236	1,249	101	4.8

[a]Specimens 4 by 4 in. [b]Marshall specimens.

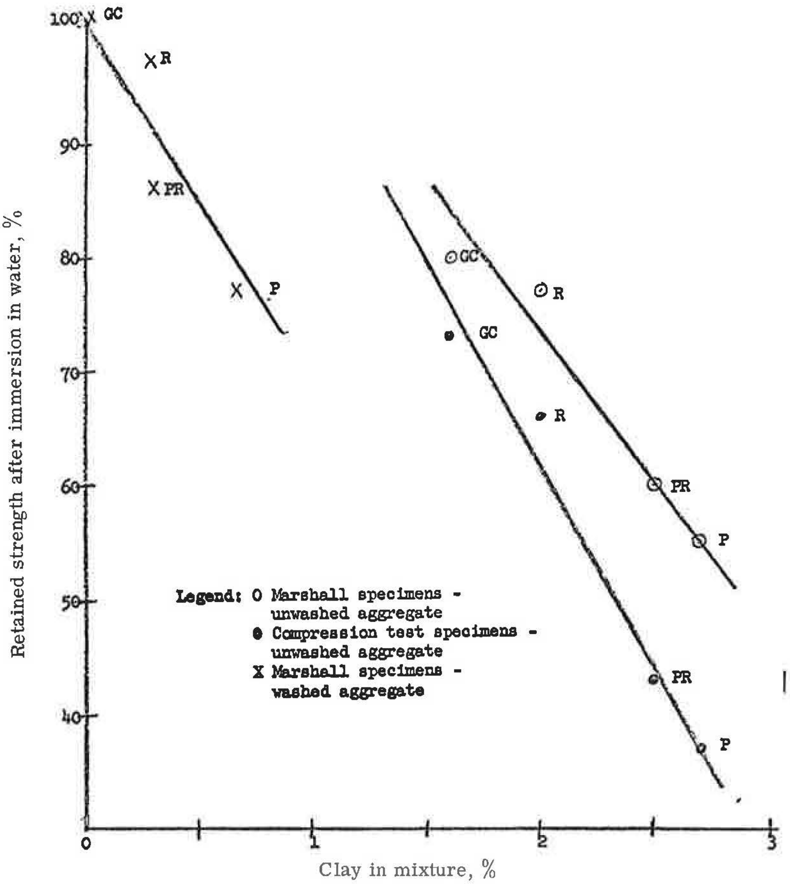

Figure 4. Relationship of percent clay in bituminous mixtures and percent retained strength.

of each mixture, however, is from 25 to 50 percent higher than the comparable mixtures using the unwashed aggregates. This supports the statement that the washing of aggregate may be a logical step toward the upgrading of local aggregates.

Figure 4 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 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 percent clay in the washed aggregates is, therefore, not absolutely correct.