HIGHWAY RESEARCH RECORD

Number 196

Aggregates and Concrete Durability

5 Reports

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Foreword

Question can arise when concrete displays less than expected durability as to whether knowledge of the materials engineer is sufficient to prescribe concrete ingredients actually capable of providing an enduring product or whether best construction practices have been followed. In any event, when shortcomings are discovered the maintenance engineer finds it necessary to improvise remedial measures.

This RECORD represents a multi-pronged attack on several aspects of the durability of highway concrete, i. e., resistance to deicing chemicals, identification of aggregates which contribute to internal or surface distress, necessary cover over reinforcing steel to prevent salt attack, and factors determining susceptibility to wear from high-speed traffic.

The concrete materials engineer will find a wealth of information on surface coatings, integral admixtures, and aggregate durability; whereas, effects of surface manipulation during placement and curing techniques will be of more interest to the construction engineer. Maintenance engineers are constantly seeking improved methods of rehabilitating scaled or polished concrete surfaces. The paper on epoxy surface treatments contains valuable information on measures to achieve thoroughly effective seals resistant to passage of water or deicing chemicals.

In several instances, research has progressed to the point that promising avenues for well-planned field trials are indicated.

Thin epoxy surface treatments have recently received considerable attention, especially for skidproofing or sealing concrete bridge decks. Minimum increase in dead load makes their use attractive. Kubitschek reports laboratory effort to solve a problem wherein it has been observed that although the initial epoxy coating positively provides complete coverage and effectively seals the surface, examination subsequent to dropping in or troweling on the cover fine aggregate reveals continuity of the film has been impaired. This peculiarity has been attributed to "wicking" of the aggregates, drawing the epoxy away from the surface and thus leaving voids for later entrance of aggressive salt solutions. Careful stepwise study employing an interesting electrical means for detection indicates minimum formation of such discontinuities is achieved using rounded aggregates smaller than the No. 20 sieve (841 microns). Use of such rounded aggregates is not necessarily compatible with maximum skid resistance usually thought to be better achieved by use of angular aggregates.

Gray and Renninger trace the history of the "brass pencil" soft particles test which was developed to detect injurious quantities of soft fragments in concrete coarse aggregates. Information is developed leading to the conclusion that across-the-board application of a soft particle limitation to insure desired aggregate quality is inappropriate—particularly for the carbonates. Extensive laboratory testing for strength and durability of concrete made with a serpentine rock is reported wherein no detectable ill effects were found despite substantial quantities of soft fragments as measured by the test. Field service records were similarly good. Lack of reproducibility of the so-called scratch test for certain aggregates was also noted; it is concluded that more definitive methods of measuring aggregate hardness are sorely needed. Specification limits based upon such measurements need to be thoroughly correlated with field performance of concrete.

Studies concerning laboratory freeze-thaw tests of concrete and specimens exposed outdoors in the Washington, D. C., area were undertaken by Gaynor and Meininger with the major variable being the differing quality of some 56 coarse aggregates and 36 fine aggregates. Variations in sand quality had little influence on freeze-thaw durability of air-entrained concrete but both quality and moisture content of the concrete and contained coarse aggregate importantly influenced durability. Extensive correlations were attempted between freeze-thaw durability and aggregate properties such as sulfate soundness, petrographic examination for deleterious particles, content of lightweight chert; freeze and thaw of the aggregates in brine, and absorption. The authors propose that an important distinction should be made between coarse aggregates containing small amounts of highly deleterious constituents causing only surface popouts and the more serious situation where deep-seated distress of the concrete results from overall inferior aggregate quality.

After development of a method for determining the surface abrasion resistance of concrete, Spellman and Ames studied abrasion losses as influenced by slump, curing, time of finishing, use of monomolecular films to inhibit evaporation, use of water reducing retarding admixtures, and linseed oil coatings. The abrasion test adopted for the work is a modification of a previously developed California method for measuring abrasion resistance of compacted bituminous mixtures and uses as the test area the top surface of a 4-in. diameter drilled core. The authors found that slump and finishing techniques influenced abrasion resistance to a lesser degree than curing procedures. Evaporation control with monomolecular films appeared beneficial. It is concluded that even good construction practices will not positively insure thoroughly durable surfaces and benefit will result from later application of linseed oil sealing treatments.

Brink, Grieb and Woolf have continued the previous extensive work of the Bureau of Public Roads in determining the resistance of concrete surfaces to scaling by deicing chemicals. Previous studies concentrated on behavior of outdoor slabs in contact with the ground similar to exposure of pavements. The present report is devoted to slabs mounted on columns off the ground to simulate bridge decks. Study has been made of an unusually large variety of curing treatments, surface coatings, admixtures, and include use of expansive cements and lightweight aggregates. For example, 17 different surface coatings and 10 admixtures were tried. Additionally, study was made of the necessary depth of embedment of reinforcing steel in order to afford full protection from effects of deicing chemicals. Prospective users of the various products and those responsible for the formulations will find the observed performance of great interest. Among the surface coatings the epoxy, chlorinated rubber, and tar-based sealers gave the best results. Linseed oil coatings after the first winter were not as effective as had been previously reported. Calcium chloride accelerator and polysiloxane durability aid were the most effective admixtures except for air-entraining admixtures. A minimum 2-in. cover over reinforcing steel appeared necessary to give protection from rusting. Confirmation of the benefits of prolonged curing periods prior to initial application of deicing chemicals was again made.

—F. E. Legg

Contents

The Role of Aggregates in Open-Void Formation in Epoxy Resin Sealcoats

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A laboratory investigation was initiated to establish what role aggregates play in producing open voids in epoxy resin sealcoats. The number of voids was obtained using a photographic technique employing a sparking device. Their sizes and configurations were identified utilizing a light-box. Void formation was found to be a function of both aggregate particle shape and size. In general, as particle shape changed from angular to round, and particle size decreased, the number and size of open voids decreased. Single fractions produced sealcoats with fewer open voids than composite blends containing two or more of those fractions. Sealcoats that were essentially free of open voids were constructed with rounded aggregates, particularly when sizes smaller than 0.033 in. (No. 20 sieve) were employed.

•EPOXY resins have been used for over ten years in a variety of applications. They have become increasingly popular in their use as sealcoats for portland cement concrete (PCC) bridge decks. In this application, the binder seals the surface against materials which cause deterioration of PCC. Aggregate is commonly included in the film to provide necessary skid resistance.

Examination of field installations reveals that some sealcoats contain open voids which vary in both size and number. Sealcoats, so constructed, fail to protect the concrete substrate from attack by materials such as deicing salts. Paving technology identifies both poor stability and inadequate wear resistance with high voids in mixes. High-void sealcoats, lacking inherent lateral stability, behave similarly. A loss of skid resistance occurs as the surface layer is gradually lost. Open voids can also be associated with sealcoat detachment.

Inasmuch as voids form after aggregates are added to a continuous binder film, a laboratory investigation was initiated to establish what role aggregates play in open-void formation.

EXPERIMENTAL APPROACH

A review of aggregates recommended for use as cover stone for epoxy resin sealcoats shows that these aggregates vary both in particle shape and in particle size. Particle shapes range from angular to round. The sizes most often recommended fall into the No. 10 to No. 40 range (U.S. Series Equivalent Sieves) with the most popular gradations in the 12 × 30 and the 20 × 40 sizes.

To observe the behavior of these particle shapes and sizes in binder films, three sources of aggregates were selected. Each type was split into narrow fractions within the 10 to 40 range. Angular fractions were derived from a crushed granite, subangular from a portland cement concrete top sand, and rounded from three sizes (8-14, 10-20, 20-40) of eastern round-grain type silica sand. These aggregates were oven-dried and the sieves cleaned to prevent contamination of particle shapes. Five individual fractions (10 × 12, 12 × 16, 16 × 20, 20 × 30, and 30 × 40) were obtained from

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each aggregate type. In addition, some of these fractions were composited into two blends, a 12 × 30 (50% 12 × 16, 30% 16 × 20, and 20% 20 × 30) and a 20 × 40 (50% 20 × 30, and 50% 30 × 40).

GRAVITY-FILLING BINDER FILMS WITH AGGREGATE FRACTIONS DIFFERING IN PARTICLE SHAPE AND SIZE

These aggregate units were used in the preparation of sealcoat castings according to the following procedure: A casting mold ($\frac{5}{8}$ × $6\frac{1}{2}$ × $10\frac{1}{4}$ in.), fitted with Mylar film on the bottom and with Teflon-coated walls, is placed on a leveling table. The binder was a coal tar-modified epoxy resin system, a type used successfully in sealcoat applications for over ten years. A quantity of thoroughly mixed epoxy resin binder, sufficient to provide a 60-mil thick film, is weighed into the mold. Time is allowed for the binder to level and for entrained air to surface. Before adding aggregate to the film, surfaced bubbles are removed by passing a hot air stream (heat gun) quickly across the surface. To provide a uniform rate of application, aggregate is applied by shaking it through a colander whose openings accommodate the size of the particular aggregate fraction. Excess aggregate is added to insure complete utilization of the binder. The casting is allowed to cure undisturbed before it is taken from the mold. Excess aggregate is removed by wire-brushing and blowing with high-pressure air. The casting is weighed and the weight of binder subtracted to obtain the aggregate weight by difference.

To hold the number of variables to a minimum, binder age at the time of aggregate addition was 27 to 40 min (10 to 15% of gel time) and operating temperature was maintained at 75 F (including that of components and environment). Aggregates were applied to the binder films at essentially the same rate and from the same height.

With this procedure, the filling efficiency of aggregates as related to both particle shape and particle size was measured. The formation of open voids was identified photographically, using the spark-generating apparatus described in ASTM D-1670-62T (Method for Determination of Failure Due to Cracking of Bituminous Materials Undergoing Accelerated or Outdoor Weathering). A photographic identification of the open voids in a sealcoat casting was obtained by passing the sparking wand (12,000 volts) over a casting laid on a grounded metal plate with a sheet of contact print paper (Kodak-AZO-F3) between them. A spark, generated where an open void permitted the current to pass to ground exposed the photosensitive paper in the area of the void. With this technique, the number of open voids was visually recorded. The size and configuration of open voids obtained with this technique require special interpretation. Some castings contained void cavities at the interface, the spark passing through a small

TABLE 1

EFFECT OF AGGREGATE PARTICLE SHAPE AND SIZE ON EFFICIENCY OF FILLING BINDER FILMS BY GRAVITY[a]

Aggregate Fraction			Aggregate in Binder Film (%w)		
Sieve Size (No.)		Avg. Particle Size (in.)	Angular Particles	Subangular Particles	Rounded Particles
Passing	Retained				
10	12	0.071	76.2	77.3	77.0
12	16	0.055	74.5	76.7	77.6
16	20	0.040	74.3	76.0	75.9
20	30	0.028	72.7	75.9	76.4
30	40	0.020	72.3	75.3	78.3
12	30[b]	0.045	74.9	76.8	77.4
20	40[c]	0.024	72.9	75.8	77.9

[a] Aggregate applied in moderate excess by sifting fractions through appropriate colanders.
[b] 12 × 30 blend: 50% 12 × 16, 30% 16 × 20, 20% 20 × 30.
[c] 20 × 40 blend: 50% 20 × 30, 50% 30 × 40.

opening at the top of such a casting exposed the contact print paper in the area confined by the perimeter of the cavity (void cluster). This identified a small opening as a large open void. Where open voids were single holes, the spark correctly identified both the size and shape of the hole. The size and configuration of the open voids, therefore, were observed visually, by means of a light-box. The information thus obtained from each casting includes: (a) the ability of the aggregates to fill the binder film (Table 1); (b) the effect of aggregate particle shape and size on the number of open voids formed (Figs. 1, 2, 3); and (c) the configuration of the voids and the maximum size of

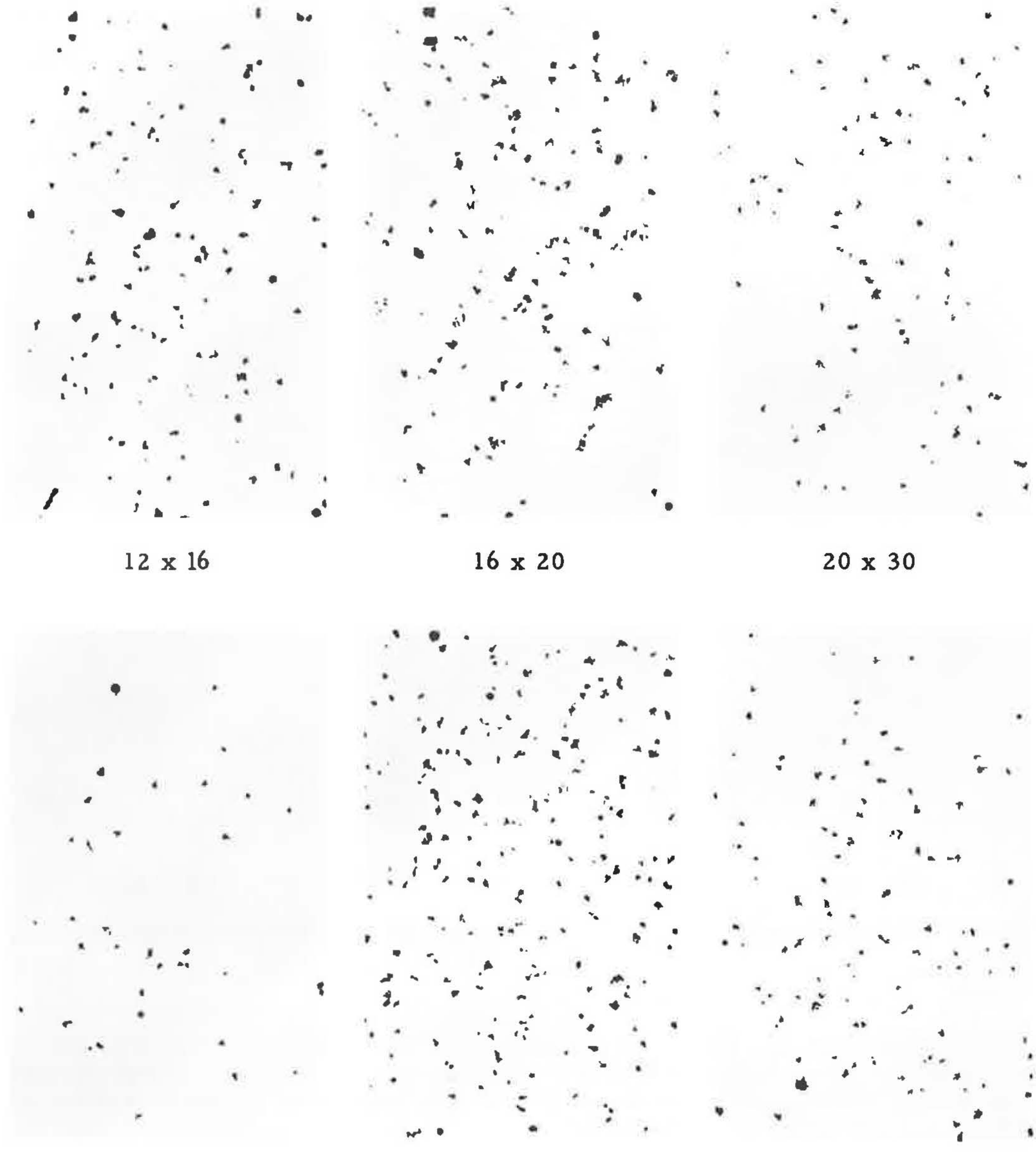


Figure 1. Effect of angular aggregate particle size on open-void formation when gravity-filling binder films.

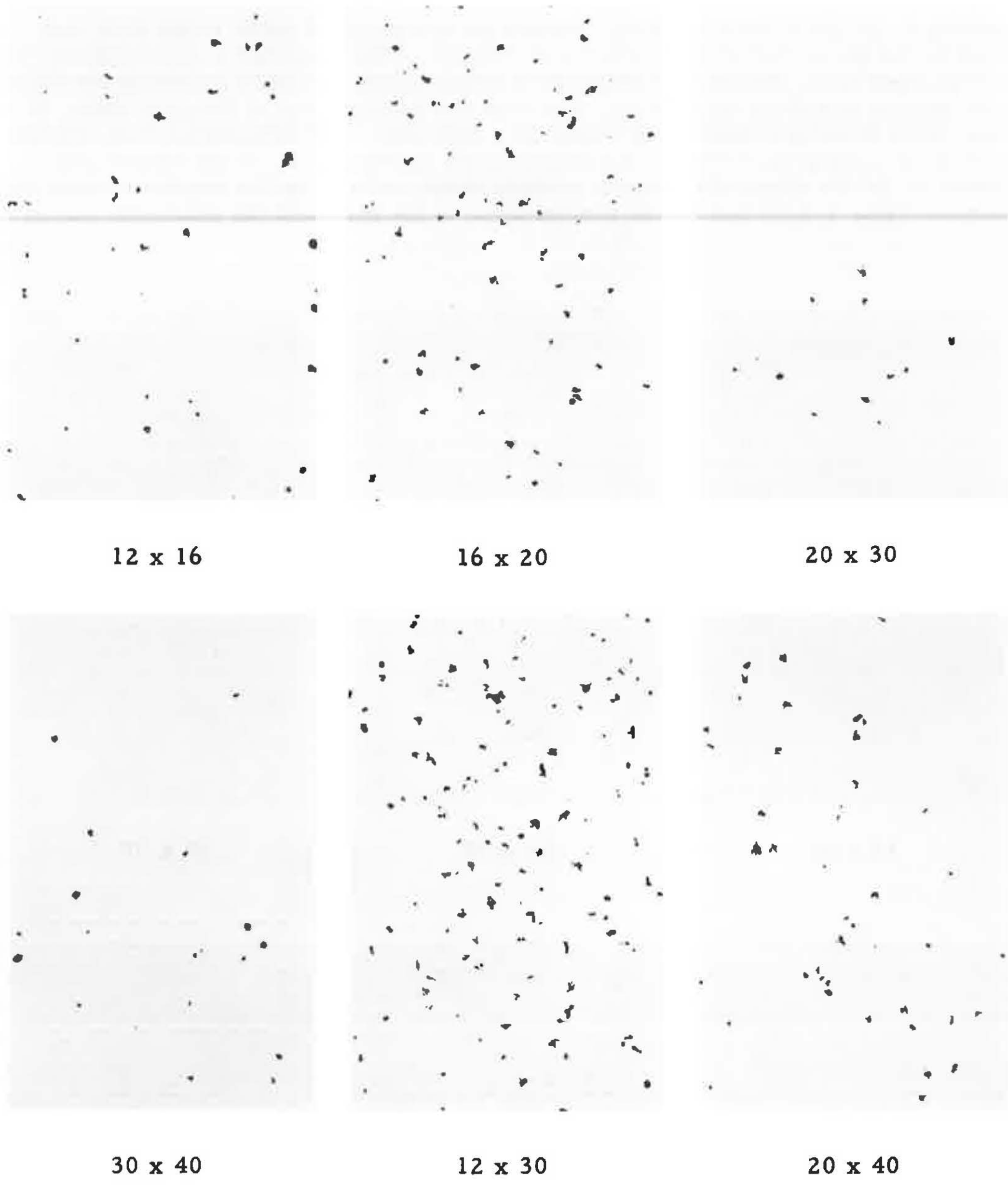


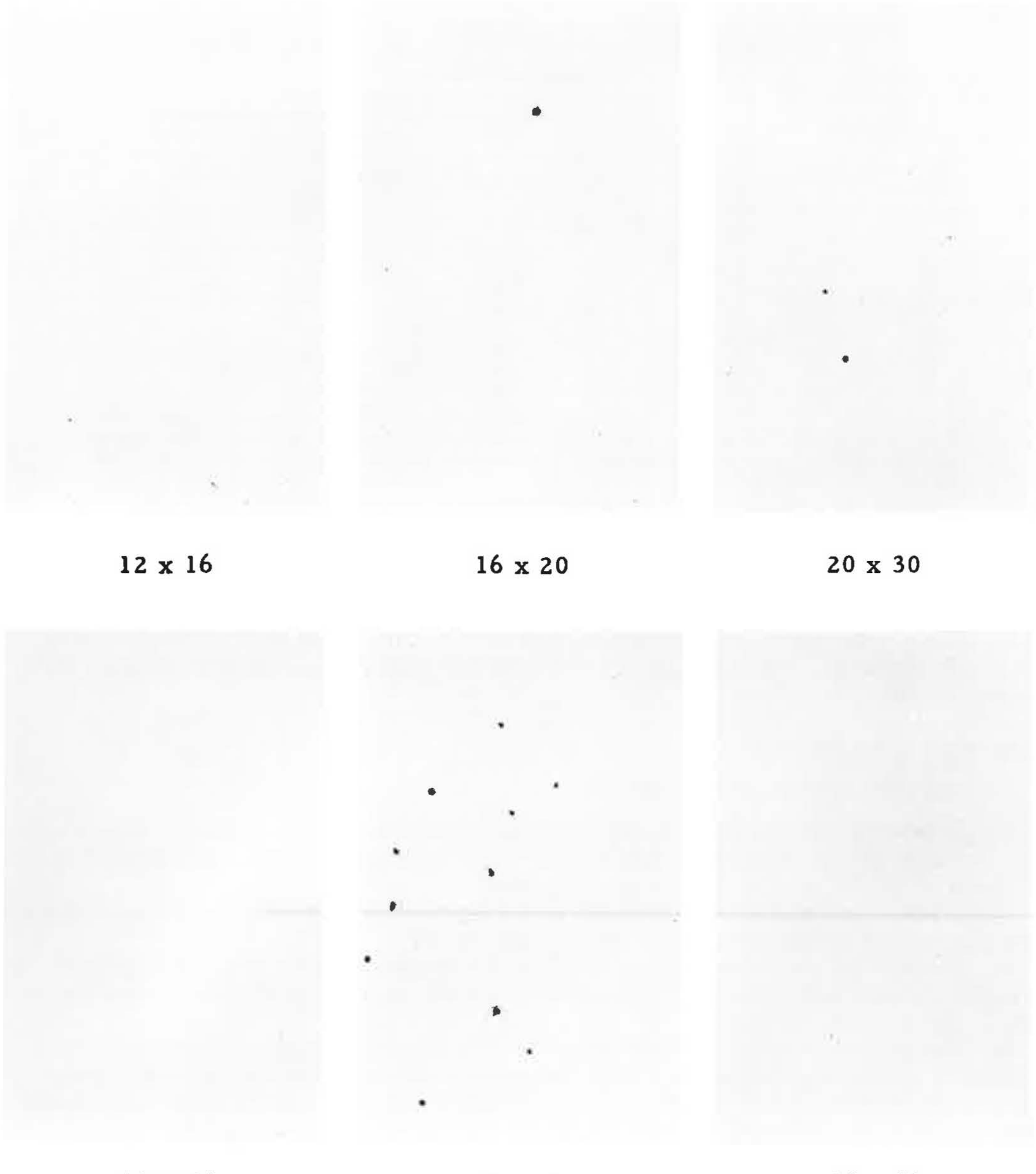
Figure 2. Effect of subangular aggregate particle size on open-void formation when gravity-filling binder films.

the open void (Table 2). The effect of an aggregate excess on these properties was also observed. Even the addition of 50 to 80 percent excess aggregate failed to produce a significant effect on either the amount of aggregate introduced into the film or on the number and size of the voids that formed.

Effect of Particle Shape and Size

As indicated in Table 1:

1. With each of the fractions, in general, the amount of aggregate included in the binder film increased as the particle shape became less angular.



12 x 16 16 x 20 20 x 30

30 x 40 12 x 30 20 x 40

Figure 3. Effect of rounded aggregate particle size on open-void formation when gravity-filling binder films.

2. With the angular and subangular fractions, the amount of aggregate included in the binder film decreased as particle size decreased. Rounded shapes showed a general reversal of this phenomenon.

3. Comparing the difference in the amount of filling between the largest single fraction (10 × 12) and the smallest single fraction (30 × 40), angular aggregate content decreased 3.9%w, subangular aggregate content decreased 2.0%w and round aggregate content increased 1.3%w.

4. With the graded fractions (12 × 30, 20 × 40), the level of filling was essentially the same as was accomplished with the largest single fraction in the blend. With the rounded 20 × 40 blend, however, filling was greater than with the larger 20 × 30 fraction.

TABLE 2

EFFECT OF AGGREGATE PARTICLE SHAPE AND SIZE ON SIZE AND CONFIGURATION[a] OF OPEN VOIDS FORMED IN GRAVITY-FILLED[b] BINDER FILMS

Aggregate Fraction			Open-Void Size (max) and Void Configuration					
Sieve Size (No.)		Avg. Particle Size (in.)	Angular Particles		Subangular Particles		Rounded Particles	
Passing	Retained		Size (in.)	Form	Size (in.)	Form	Size (in.)	Form
10	12	0.071	0.11	1	0.10	1	0.06	1
12	16	0.055	0.08	1-2	0.06	1	none	-
16	20	0.040	0.04	3	0.04	2	0.07	1
20	30	0.028	0.02	4	0.03	4	none	-
30	40	0.020	<0.01	4	0.01	4	none	-
12	30[c]	0.045	0.08	3	0.06	3	0.05	1
20	40[d]	0.024	0.01	4	0.01	4	none	-

[a]Voids appeared either as single holes or as clusters (clusters contained both open and closed voids): 1—voids are single holes, 2—void clusters (coarse-grain), 3—void clusters (medium-grain), and 4—void clusters (fine-grain).
[b]Aggregate applied in moderate excess by sifting fractions through appropriate colanders.
[c]12 × 30 blend: 50% 12 × 16, 30% 16 × 20, 20% 20 × 30.
[d]20 × 40 blend: 50% 20 × 30, 50% 30 × 40.

5. The graded aggregate fractions followed the same filling pattern displayed by the single fractions, i.e., the smaller the size the less the fill, and the rounder the shape the better the fill.

Open-Void Formation

As indicated in Figures 1, 2, and 3:

1. In general, the number of open voids decreased as the particle size decreased with the angular and subangular aggregates. The rounded fractions and rounded 20 × 40 blend produced relatively void-free castings.
2. There was a significant decrease in the number of open voids produced with each particle size as the particle shape became less angular.
3. There appeared to be a sharp break in the number of open voids that formed with the angular and subangular aggregates when the average particle size was smaller than 0.033 in. (No. 20 sieve).
4. The graded angular and subangular 12 × 30 blends showed an increase in the number of open voids that formed over the number observed for the individual fractions in the blends. Even with the 12 × 30 rounded aggregate blend, some open voids were in evidence. A possible explanation for this is the classification that occurs with graded aggregates either during storage or during application to a binder film. As larger particles settle at a faster rate, they get to the substrate-binder interface before the finer fractions. When this occurs, the greater capillary forces exerted by the smaller particles produce a strong "wicking action" (drawing of binder from the interstices between the larger particles) which often leads to open-void formation. This wicking action occurs when the coarser fraction separates and reaches the interface first (Fig. 4). A coarse fraction on top of a finer fraction produces no open voids. The angular and subangular 20 × 40 blends behaved similarly. The rounded 20 × 40 blend produced no voids.

Open-Void Size and Configuration

Size. As indicated in Table 2:

1. In general, the maximum size of open voids decreased as particles became less angular and as particle size decreased. There was a break in both the size and number

12 x 16 followed by 20 x 30

20 x 30 followed by 12 x 16

Figure 4. Effect of order of addition of rounded aggregates of different size on open-void formation.

TABLE 3

EFFECT OF AGGREGATE PARTICLE SHAPE AND SIZE ON EFFICIENCY OF FILLING BINDER FILMS, EMPLOYING COMPACTION[a]

Aggregate Fraction: Sieve Size (No.) Passing	Aggregate Fraction: Sieve Size (No.) Retained	Avg. Particle Size (in.)	Aggregate in Binder Film (%w): Angular Particles	Subangular Particles	Rounded Particles
10	12	0.071	82.4	83.2	83.9
12	16	0.055	80.2	82.7	84.3
16	20	0.040	79.0	81.1	83.1
20	30	0.028	78.1	79.8	82.8
30	40	0.020	77.2	79.1	82.7
12	30[b]	0.045	81.3	82.3	84.7
20	40[c]	0.024	77.3	79.8	83.0

[a]Compacted with a trowel over excess aggregate.
[b]12 x 30 blend: 50% 12 x 16, 30% 16 x 20, 20% 20 x 30.
[c]20 x 40 blend: 50% 20 x 30, 50% 30 x 40.

of large voids that occurred when angular and subangular particle sizes were smaller than 0.033 in. (No. 20 sieve).

2. There was a greater number of large voids in the angular and subangular 12 × 30 and 20 × 40 blends than in the individual fractions making up those blends. The sizes of these voids, however, were not so large as those formed with their fractions. With the rounded aggregate blends, only the relatively widely graded 12 × 30 composite (which classifies readily) produced open voids of essentially uniform size.

Configuration. Also indicated in Table 2:

1. With angular and subangular particles, voids appeared as single holes when sizes were greater than 0.046 in. (No. 16). Void clusters formed with smaller sizes, decreasing in texture as particles became finer. Clusters contained both open and closed voids.

2. When rounded fractions produced voids, they were of the single hole variety.

TABLE 4

EFFECT OF AGGREGATE PARTICLE SHAPE AND SIZE ON SIZE AND CONFIGURATION[a] OF OPEN VOIDS FORMED WHEN GRAVITY-FILLED BINDER FILMS ARE COMPACTED[b]

Aggregate Fraction: Sieve Size (No.) Passing	Aggregate Fraction: Sieve Size (No.) Retained	Avg. Particle Size (in.)	Open-Void Size (max) and Void Configuration: Angular Particles Size (in.)	Angular Particles Form	Subangular Particles Size (in.)	Subangular Particles Form	Rounded Particles Size (in.)	Rounded Particles Form
10	12	0.071	0.05	1	0.05	1	none	-
12	16	0.055	0.03	1	0.04	1	0.06	1
16	20	0.040	0.01	3	<0.01	1	none	-
20	30	0.028	<0.01	4	<0.01	1	none	-
30	40	0.020	<0.01	4	<0.01	4	none	-
12	30[c]	0.045	0.02	3	<0.01	1	none	-
20	40[d]	0.024	<0.01	4	<0.01	4	none	-

[a]Voids appeared either as single holes or as clusters (clusters contained both open and closed voids): 1—voids are single holes, 2—void clusters (coarse-grain), 3—void clusters (medium-grain), and 4-void clusters (fine-grain).
[b]Compacted with a trowel over excess aggregate.
[c]12 x 30 blend: 50% 12 x 16, 30% 16 x 20, 20% 20 x 30.
[d]20 x 40 blend: 50% 20 x 30, 50% 30 x 40.

COMPACTION OF GRAVITY-FILLED BINDER FILMS CONTAINING AGGREGATE FRACTIONS DIFFERING IN PARTICLE SHAPE AND SIZE

The investigation of the effects of aggregate particle shapes and sizes on the composition of sealcoats was expanded to include a study of the effects of compacting the same systems described previously. The sample preparation procedure used was the same, but a larger excess of aggregate was applied in each case (50 to 100% excess) to accommodate compaction. This was accomplished using a 4 by 5-in. trowel, applying vertical hand pressure with a slight sawing action. The amount of excess

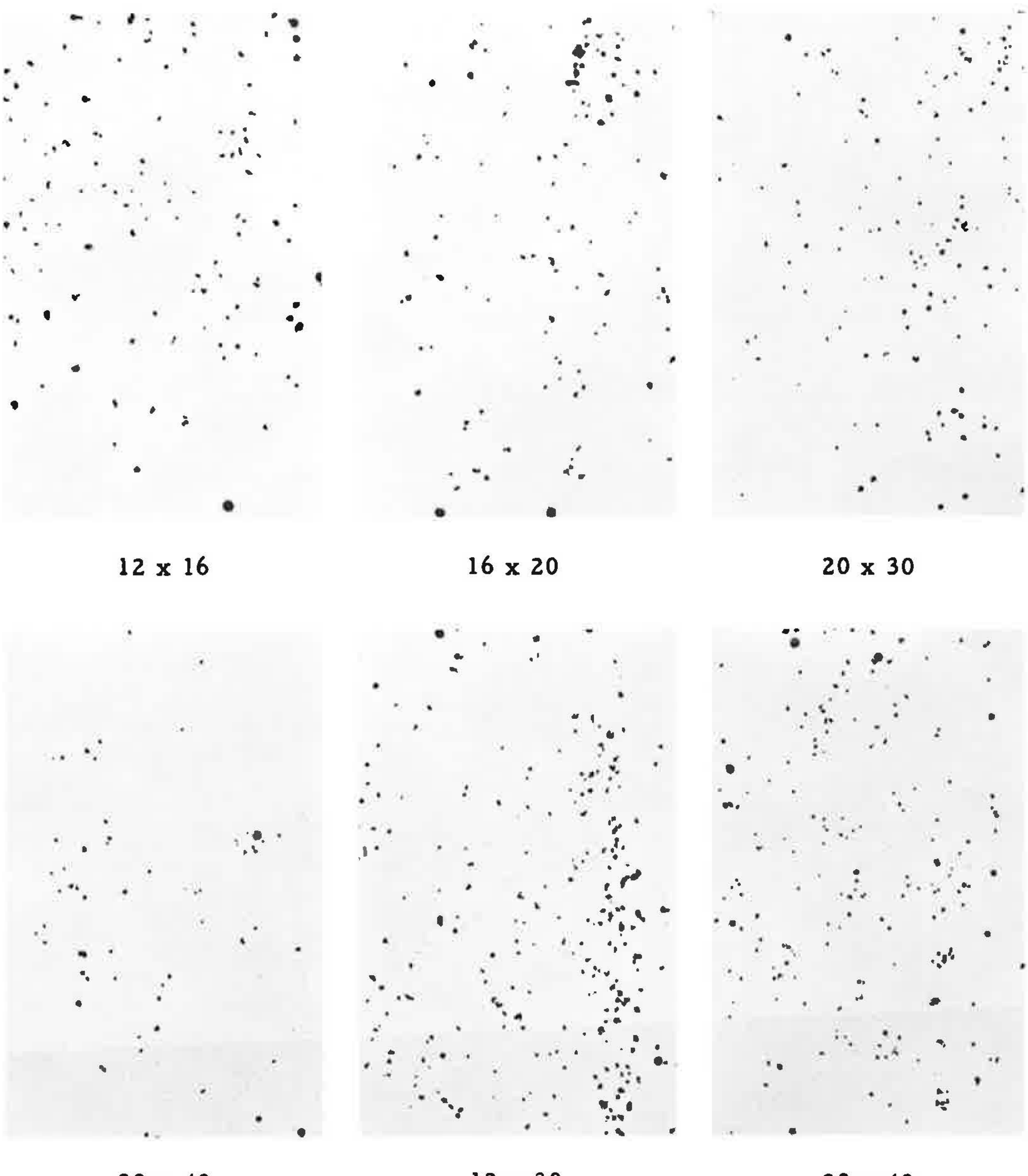


Figure 5. Effect of angular aggregate particle size on open-void formation when gravity-filled binder films are compacted.

aggregate required for this operation differed for the castings. Generally, the finer the particle size the more excess was needed to prevent binder from wicking through to the surface. Samples, thus prepared, were examined in the same manner as described where the binder films were gravity filled. Results are given in Tables 3 and 4 and Figures 5, 6 and 7. Figure 8 plots filling as a function of particle size for each aggregate type and illustrates the response of the systems to compactive effort.

The data may be evaluated simply by stating that the patterns formed as a function of aggregate particle shape and size are essentially the same as those for the gravity-

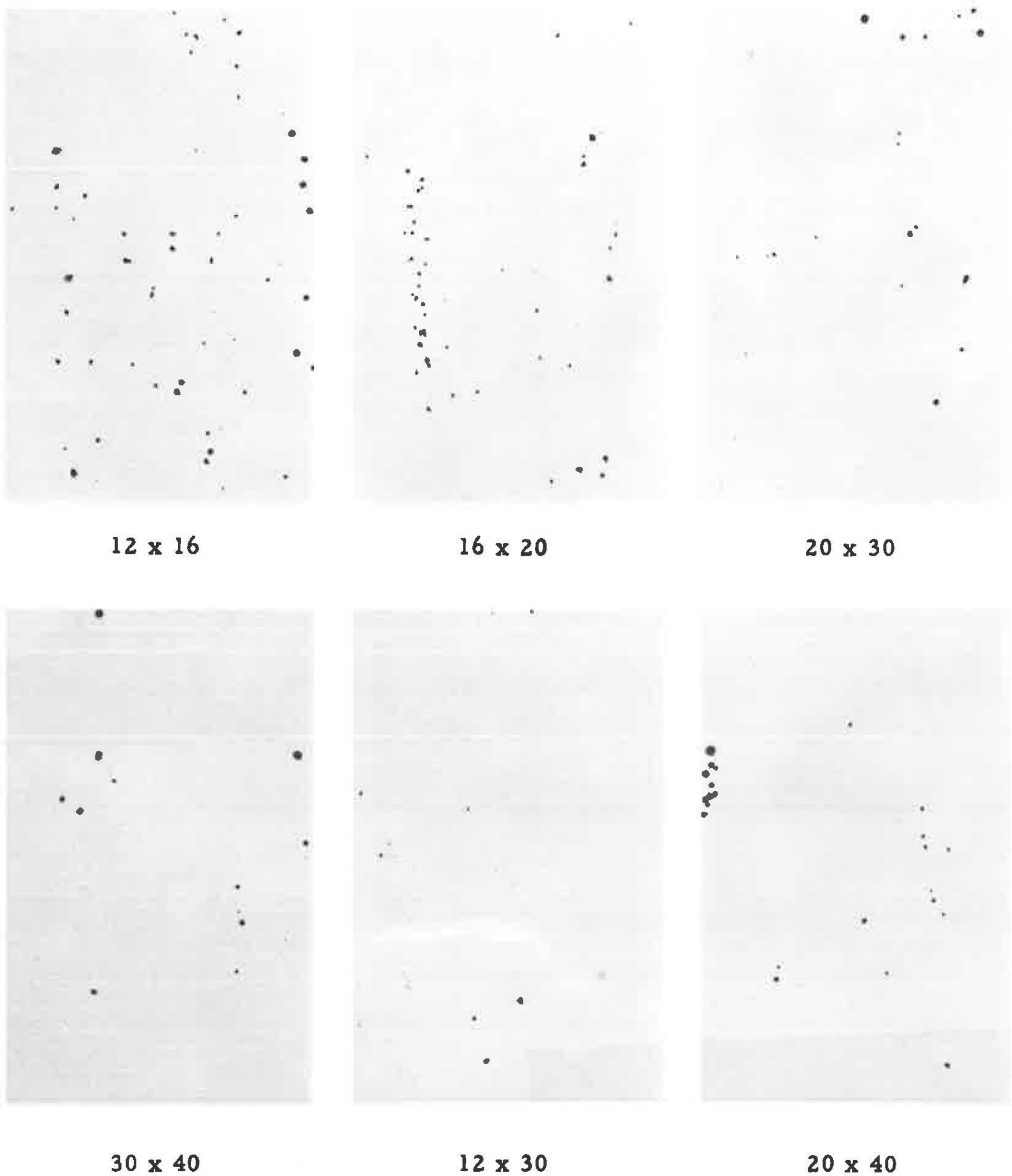


Figure 6. Effect of subangular aggregate particle size on open-void formation when gravity-filled binder films are compacted.

filled systems, only the magnitude of the values is different. The only reversal of order occurred with rounded particles, where, with compaction, the amount of aggregate included in the binder film decreased as the particle size decreased, following the same pattern as the angular and subangular materials.

Special comments for compacted samples are as follows:

1. The 12 × 30 and 20 × 40 aggregate blends are not sufficiently wide in gradation to benefit from compaction as far as more efficient filling of a binder film is

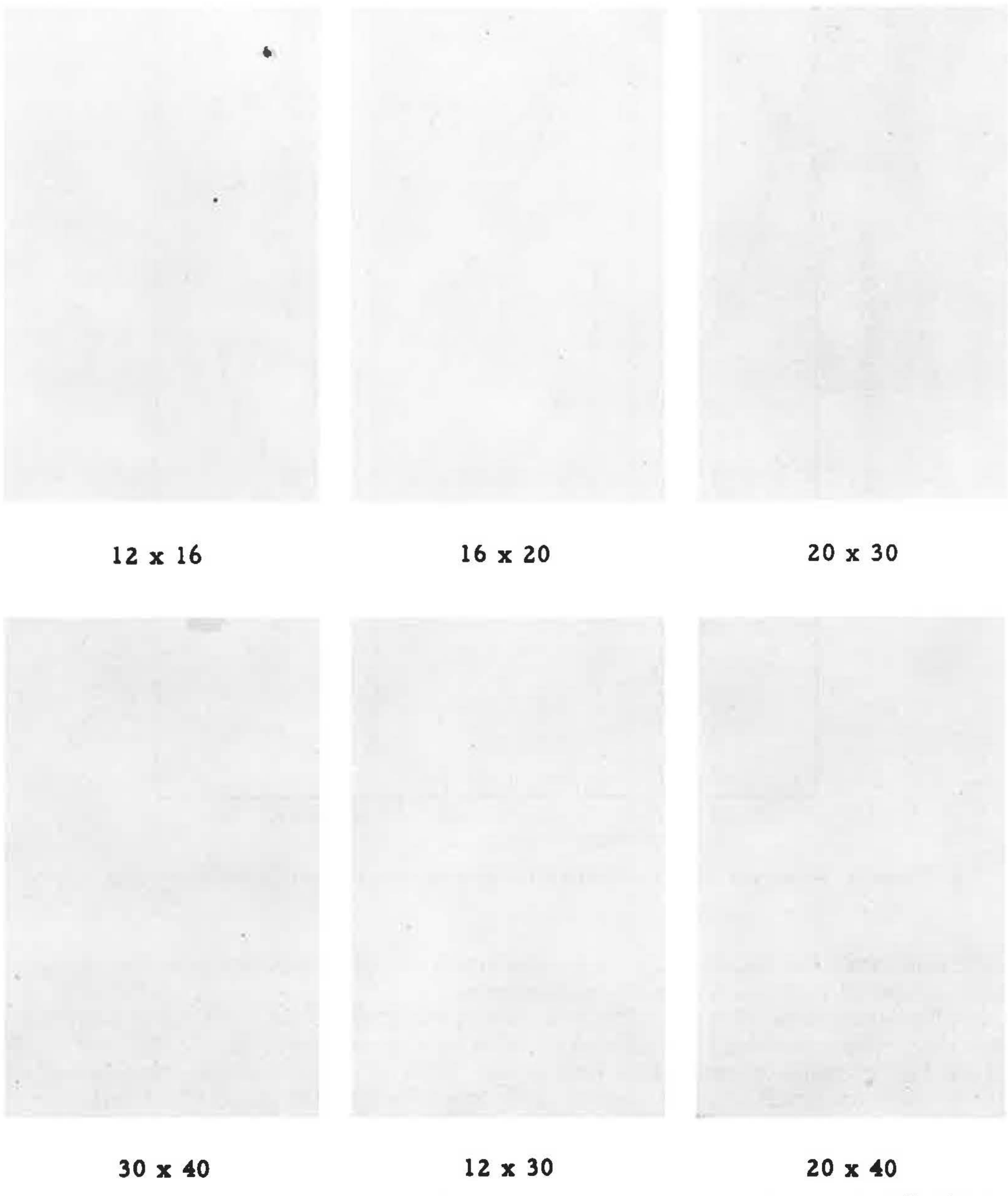


Figure 7. Effect of rounded aggregate particle size on open-void formation when gravity-filled binder films are compacted.

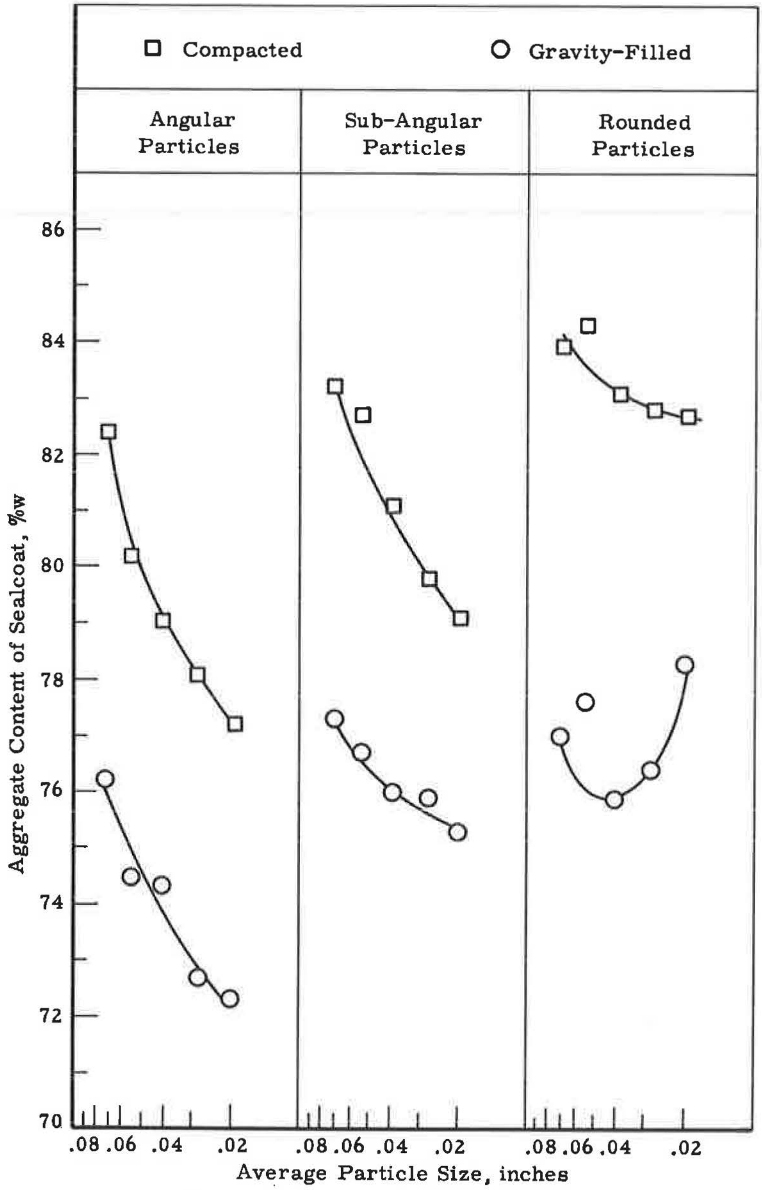


Figure 8. Aggregate filling of binder films as a function of particle shape and size.

concerned. Only the angular 12 × 30 blend showed a slight advantage over the filling quality of the 12 × 16 fraction in its composition.

2. The subangular 30 × 40 and 20 × 40 (which contains 50% 30 × 40) aggregates had voids that formed fine-textured clusters. All single sizes larger than 0.023 in. (No. 30) and the 12 × 30 blend formed single hole voids. With the 30 × 40 single fraction and the 20 × 40 blend the small number of open voids was very small in size (<0.01 in.).

3. With the rounded individual particles, no open voids formed when the particle size was smaller than 0.046 in. (No. 16). No voids formed with the blends.

QUALITIES OF SEALCOAT CASTINGS PREPARED BY GRAVITY-FILLING BINDER FILMS VS THOSE COMPACTED AFTER FILLING

1. Compaction increased the amount of aggregate included in a binder film, regardless of particle shape or size.
2. Compaction of systems containing angular and subangular aggregate fractions reduced the number and size of void clusters (containing both open and closed voids).
3. In general, compaction had the greatest effect on systems containing rounded particles as far as filling the film was concerned.
4. In general, for all systems, compaction reduced the size of the open voids.
5. More compaction was accomplished with larger particle sizes than with the smaller, regardless of particle shape.
6. With the aggregate blends, the 12 × 30 angular material showed essentially no decrease in the number of voids as a result of compaction. The subangular and rounded 12 × 30 blends showed striking improvement after compaction. All of the 20 × 40 blends were compacted to a significantly lower void content than was achieved by gravity filling (the rounded system was voidless whether compacted or not).

CONCLUSIONS

1. Aggregate particles derived from crushed aggregate sources (angular materials) do not efficiently fill a binder film even when compacted. Sealcoats constructed with these materials are high in void content (regardless of particle size selection) and produce sealcoats that are permeable to the substrate which they are intended to protect. These high-void systems lack stability because of their void structure and, as a result, probably wear faster. A high degree of skid-resistance is the only attribute that can be claimed for sealcoats constructed with angular aggregates.
2. Aggregate particles that are subangular in configuration produce sealcoats with varying degrees of permeability. The number of voids formed depends on particle size and whether or not the sealcoat is compacted.
3. Sealcoats made with rounded aggregates are essentially free from open voids (impermeable) whether or not compaction is employed. The graded, coarse blend, 12 × 30, can produce open voids if the binder film is gravity filled, but becomes void free with compaction. Rounded particles, because of their greater mobility, are able to find their place more readily in the binder film and, as a result, fill the film to a greater extent than either angular or subangular materials. This has the effect of producing sealcoats with greater stability. Wear resistance should be at a high level.
4. Aggregate particles larger than No. 20 are less effective as sealcoat cover stones because they tend to produce overlays with the greatest number of voids. Furthermore, they do not respond as well to compaction as the smaller sizes.
5. Void-free sealcoats can best be obtained by employing finely divided, rounded aggregates.

SUMMARY

The effect of aggregate particle shape and particle size on void formation in epoxy resin sealcoats was investigated. Results showed that void formation was a function both of particle shape and size. As particle shape changed from angular to round, and particle size decreased, the number and size of open voids decreased. Finer aggregate fractions produced void clusters (containing both open and closed voids), whereas sizes larger than 0.033 in. (No. 20 sieve) produced single hole voids. Because graded aggregates tend to classify, they produced higher void systems than any of the single fractions making up the composites. When classification occurred, larger particles migrated to the binder substrate interface first. The finer fractions, settling on top of the coarser fractions, drew binder from the interstices between the larger particles,

creating voids. Compaction reduced the number and size of voids, regardless of particle shape. Rounded aggregates, in general, produced sealcoats that were free from open voids.

Other factors affecting the performance of epoxy resin sealcoats are being investigated.

The Concept of Hardness As Applied to Mineral Aggregates

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A historical review is given of the soft particle requirements in specifications showing, with the development of the scratch hardness test, a broadening of the application of the test without sufficient supporting data to indicate that the test measures a property which actually affects performance. Originally softness was used to indicate the presence of unsound material which caused pop-outs in concrete. Softness as measured by the scratch hardness test is considered to be a measure of deleterious material in aggregate and is a specification requirement without association to any other property and without sufficient information to show that softness, per se, is in fact a deleterious property of aggregate. Data are presented that relate the scratch hardness test in terms of Mohs hardness scale which indicates that both procedures may classify minerals but not rocks of variable mineral composition. Finally, data are presented which demonstrate that aggregate which fails the scratch hardness test may make strong, sound, durable concrete. In light of the philosophy of materials testing, the historical review, and the test data submitted, it is suggested that the present concept of aggregate hardness be reconsidered and re-evaluated.

•A DICTIONARY might define the word hardness as "the state of being hard" or "the state of being firm or unyielding." That same dictionary, if it were scientifically oriented, might go even further and define hardness as "that quality of a mineral which resists scratching." Both definitions, that of being firm or unyielding and that of scratchability, relate to the concept of hardness as it is currently applied to mineral aggregates. When consideration is given to the hardness of an aggregate, one generally thinks in terms of (a) its resistance to abrasion as measured by the Los Angeles test (ASTM Designation C 131) or (b) the amount of soft or scratchable material present as determined by the scratch hardness test (ASTM Designation C 235). It is this second characteristic, that of aggregate softness or scratchability as a measure of quality, that is the basis for this report.

A soft particle limitation may be found in most concrete aggregate specifications, many of which are modeled after the ASTM C 33 specification which includes, as a quality requirement, the provision that not more than 5 percent of the coarse aggregate particles may be soft as determined following the procedure outlined in C 235, which is the determination of whether a piece of aggregate can or cannot be scratched by a brass rod. This discussion will be confined specifically to the ASTM scratch hardness test as a measure of quality and as an index of performance of aggregate in portland cement concrete.

Paper sponsored by Committee on Mineral Aggregates and presented at the 46th Annual Meeting.

HISTORY OF THE SOFT PARTICLE REQUIREMENT

The ASTM specification for concrete aggregates (Designation C 33) was first released in 1921 (1, 2). That portion relating to the coarse aggregate requirements was rather general, specifying little more than a suggested grading. With the expanding use of portland cement concrete and in recognition of the deleterious behavior observed in concretes containing certain constituents of natural aggregate deposits, it was soon realized that such a general, open type specification was of little real value. Soft, weathered chert particles, poorly bonded sandstone fragments, agglomerations of clay, shale, ochre, weathered schist, particles of coal, etc., had performed poorly as coarse aggregate particles in concrete. These fragments had one characteristic in common: they were all "soft" by comparison with the sound, durable particles comprising the majority of the deposit. By association, then, particle softness or hardness became connected with aggregate quality. The need to limit the tolerable amount of these deleteriously soft materials and to develop a test with which they might readily be recognized was felt desirable.

Douglass Test

In 1928, a provision limiting the soft particle content of coarse aggregate was placed in the ASTM concrete aggregate specification. At the same time a test with which to identify and measure these soft particles was proposed. A Method of Test for Determination of Soft Pieces in Gravel, commonly known as the Douglass test, was a simple compression test in which individual particles were subjected to a 75-lb load. Particles which yielded under this load were considered soft. The Douglass test, however, was not adopted by ASTM although the 5 percent limitation on soft pieces was placed in C 33.

The 1930 version of C 33 cited, as the method for soft particle determination, the Douglass test as adopted by AASHO. The 1936 revision cited this same test as then identified as AASHO Designation T-8. The Douglass test last appeared as an AASHO method in their 1938 printing. In 1940 action was taken to delete T-8 and the procedure was not published in the 1942 edition of the AASHO test methods. The 1949 version of C 33 contained the same five percent limitation on soft pieces, but referenced no method by which to make the determination. It was during this same period that research work relative to soft particle determination was conducted by the Bureau of Public Roads (then the Public Roads Administration). In 1941, Subcommittee IX of ASTM C-9 on Concrete and Concrete Aggregates had requested such an investigation. The results of BPR's investigations were reported by Woolf (3) in 1947. It was this work which ultimately led to the standardization of the current ASTM scratch hardness test (Designation C 235), first adopted by the society in 1949.

The 1952 revision of C 33 corrected the deficiency present in the 1949 version by again providing a method by which to determine the soft particle content. The correction approved was simply to reference the scratch hardness test; the 5 percent limitation on soft pieces was left unchanged. Since 1952 the status of C 33 with respect to the scratch test has remained unchanged, the test has gained wide usage, and softness, per se, as determined by the scratch test, has become firmly entrenched as being a deleterious quality. Even though the AASHO specification for concrete aggregate, M 80, does not explicitly refer to the scratch hardness test, it does have a recommended limit of 2 percent and a maximum limit of 5 percent on soft fragments and the only AASHO method of test for soft particles is T 189 (ASTM C 235).

From the above, it is apparent that since the inception of the soft particle limitation in 1928, the significance or interpretation of that limitation has undergone a gradual metamorphosis. The net result of these changes has been to apply the 1928 "consensus" limitations under a dramatically different and drastically enlarged set of circumstances—different with reference to the strict enforcement and literal compliance of specification provisions currently advocated and enlarged with the present interpretation of softness, per se, as a deleterious characteristic of aggregates in general. The changes offered and enacted through the years have also been made without sound research data supporting the validity of a general overall soft particle limitation, and the applicability of the various soft particle tests employed. The 5 percent limitation established in 1928

TABLE 1

ROSIWAL EQUIVALENT FOR MOHS SCALE MINERALS

Mineral	Mohs Value	Rosiwal Value[a]
Talc	1	0.04
Gypsum	2	2.0
Calcite	3	5.6
Fluorite	4	6.4
Apatite	5	8.0
Orthoclase	6	50.2
Quartz	7	175.0
Topaz	8	194.0
Corundum	9	1,000.0
Diamond	10	140,000.0

[a]Abrasive loss with corundum numbering 1,000.0.

was to be enforced by the Douglass test, designed and developed to detect softness in gravel aggregates. No mention of its applicability to other materials was cited. Woolf's data (3) relative to the development of the scratch hardness test were also limited to gravel aggregates and also gave no indication as to applicability to other materials. Woolf did point out, however, that the scratch test was more selective than the Douglass test, that is, the scratch test gave results greater by a factor of two or more. The 5 percent limitation originally based on the Douglass test remained unchanged, however, with the adoption of the scratch hardness test.

Scratch Hardness vs Mohs Hardness

The rather arbitrary Mohs scale, in which hardness values are assigned with respect to the ease or difficulty with which a given substance may be scratched by a set of certain scale minerals, has proven extremely satisfactory in the science of mineralogy. It performs admirably when uniform substances, pure crystals, materials composed of a single mineral, or in essence, materials having a uniform hardness characteristic, are subjected to test. Geologists are reluctant to apply the Mohs hardness scale to rocks due to their heterogeneous mineral composition, yet it has been felt by certain materials engineers that a modification of this same test (i.e., using as the scratching medium a certain size and type brass rod) is universally applicable to all aggregate materials.

Hardness, or conversely softness, has not been satisfactorily defined as there are a number of different physical concepts associated with the term. Compressive hardness, impact hardness, abrasive hardness, and indentation hardness are the major measures of hardness applicable to materials. Each measures hardness from a different point of view depending upon the nature of the stress applied and each might be expected to evaluate the "relative hardness" of a group of materials in a somewhat different order.

The indentation type hardness tests most generally find applicability in the field of metallurgy although values may be found for certain of the common rock forming minerals.

There have been many test methods and many sclerometers described over the years as being applicable to the measurement of "soft pieces" in aggregates. Several impact tests have been proposed. Woolf (3) described one that was AASHO Standard T-6 but never an ASTM method. The Douglass test measured compressive hardness. The present scratch hardness test may be considered a particularly arbitrary measure of abrasive hardness.

The Mohs scale of scratch hardness (the concept on which the scratch hardness test is based) was developed in 1812 as a simple guide to aid in the identification of minerals. The scale chosen was a qualitative measure of abrasive hardness intended solely to relatively evaluate the normal rock forming minerals. This simple scale of 1 to 10 takes on a very different aspect, however, when abrasive hardness is determined quantitatively by the method outlined by Rosiwal (data obtained through private correspondence with Dr. Ralf Villwock, Wiesbaden, Germany). Table 1 gives the quantitative Rosiwal equivalent for each of the Mohs scale minerals.

Since approximately 70 percent of the 780,000,000 tons of crushed stone produced in this country in 1965 is in the 3 to 4 Mohs hardness range; since the quantitative range in abrasive hardness between calcite (Mohs 3) and fluorite (Mohs 4) is the least of the entire hardness scale; and since the "relative hardness" of brass varies appreciably, the scratch hardness test would appear to be without any established reliability or precision. Consequently, its general acceptance can only lead to trouble for the industry as it is applied by way of a specification item. The main point of this discussion is that no difficulty is encountered with established methods of test in separating very good from very bad materials, but that it is always difficult to separate borderline materials. The above is true of the present scratch hardness method, but by the very nature of the test a tremendous "borderline" category, based on an arbitrary characteristic and including the majority of the crushed stone produced, is created. Strict enforcement of its provisions can only result in many ill-founded arbitrary decisions.

PHILOSOPHY OF MATERIALS TESTING

Indeed the entire philosophy behind the soft particle tests and the specification limitation is open to question in light of an article by Plum et al (4) published recently in the Rilem Bulletin. Plum and his co-workers in reviewing the basic philosophy underlying the science of materials testing state:

> The object of testing materials is to obtain a prediction of its serviceability. If an indication of the performance in service can be expressed quantitatively, comparison has been made possible and the information can be used as a basis for choice between alternatives. The testing of materials is, therefore, connected with the definition of serviceability, or the requirements to the performance on the one hand and with the criteria for making the choice, or the economic consequences on the other.

The development and adoption of the present soft particle limitation and its associated scratch hardness test without the benefit of supporting data appears to be in conflict with this basic philosophy. The literature search for making the historical review did not uncover a single attempt to correlate the scratch hardness test with performance of aggregate.

RESEARCH STUDIES OF SCRATCH HARDNESS TEST

In the past the soft particle test and limitation have taken on the guise of a "sleeper clause" in specifications, seldom enforced. However, the present trend toward rigorous specification enforcement and contract compliance from the legal standpoint is heralding the day of strict enforcement of both this test and its compliance requirements. As an example of just such an instance, a commercially quarried serpentine aggregate has recently been rejected for noncompliance with the scratch hardness requirements despite the fact that the stone meets all other physical and chemical requirements and has a long-standing satisfactory service record.

A rather comprehensive laboratory investigation was made on the physical characteristics and the concrete making properties of this rock. The results of this study were most revealing.

Description of Testing Program

The quarry was examined, ledge samples chosen, and separated in the field into "hard" and "soft" fractions using a brass rod mounted as a pencil meeting the requirements of the scratch hardness test. On being returned to the laboratory, the two fractions (hard and soft) of ledge rock were re-examined, using the standard scratch hardness device, and re-classified. Any doubtful pieces which could not readily be termed hard or soft on the basis of the scratch test were not used in further analysis. This laboratory selection procedure was followed by two individuals independently, its purpose being simply to insure that the separation into hard and soft was the best that could be expected. Each sample was crushed to 2-in. maximum size aggregate in a laboratory jaw crusher and tested again, this time in strict accordance with the scratch hardness test. The test was performed on the same samples by different individuals. The results, shown in Tables 2 and 3, demonstrate the variability of the test. That portion classified as soft in the field had 64 percent soft when tested in the laboratory. This 64 percent when crushed and re-tested had 65 percent soft. Likewise, the portion classified as hard in the field had 70 percent hard when tested in the laboratory. This 70 percent, when crushed and re-tested, had 92 percent hard.

TABLE 2

SUMMARY OF SCRATCH HARDNESS TEST RESULTS[a]
(No. 6823—Soft Sample)

Part A. Selection, by Geologist, of "Soft" Rocks in the Field
Total weight—1503 lb

Part B. Examination and Classification of Field Sample in the Laboratory
Soft—956 lb (64 percent of original)
Hard—547 lb (36 percent of original)

Part C. Results of Scratch Hardness Test Performed on Crushed Aggregate Prepared From That Identified as Soft in Part B Above

Operator	Percent by Weight Soft Pieces				
	2-1½ In.	1½-1 In.	1-¾ In.	¾-½ In.	½-⅜ In.
A	63	52	47	61	55
B	60	78	77	82	83
C	60	65	66	80	75
Average	61	65	63	74	71

[a]Method of Test—ASTM Designation C 235.

The same samples subjected to the scratch hardness procedure were also subjected to the Mohs test. The last separations into hard and soft fractions made on the laboratory prepared samples were left intact and each fraction of each size of each sample was tested to determine its range in Mohs hardness. The results obtained are given in Tables 4 and 5. Representative results are shown in Figure 1.

The $\frac{1}{16}$-in. diameter brass rod specified for use in the ASTM scratch hardness test is described as being capable of scratching a Lincoln penny but unable to scratch a Jefferson nickel. Translated into the Mohs scale of index minerals, it should then be capable of scratching the mineral calcite but not the mineral fluorite. In effect, then, the brass rod might be described as having a Mohs hardness between 3 and 4. According to the present test method, those materials which are scratched or have fragments dislodged by the brass rod are considered soft and hence unsatisfactory for use as concrete aggregates. As has been indicated, the rock tested was a serpentine, that is, a rock composed essentially of the serpentine minerals. It was, therefore, somewhat easier to apply the Mohs and scratch hardness tests to these samples than it would be to a granite or some other rock with a more heterogeneous mineral composition.

The serpentine minerals are described as having a Mohs hardness varying from 2 to 5 depending on type. The massive serpentines, generally, fall above 3 on this hardness scale. Tables 4 and 5 and Figures 1 through 3 indicate that the samples all

TABLE 3

SUMMARY OF SCRATCH HARDNESS TEST RESULTS[a]
(No. 6824—Hard Samples)

Part A. Selection, by Geologist, of "Hard" Rocks in the Field
Total weight—1385 lb

Part B. Examination and Classification of Field Sample in the Laboratory
Soft—423 lb (30 percent of original)
Hard—962 lb (70 percent of original)

Part C. Results of Scratch Hardness Test Performed on Crushed Aggregate Prepared From That Identified as Hard in Part B Above

Operator	Percent by Weight of Soft Pieces				
	2-1½ In.	1½-1 In.	1-¾ In.	¾-½ In.	½-⅜ In.
A	2	5	6	4	7
B	15	10	5	10	17
Average	8	8	6	7	12

[a]Method of Test—ASTM Designation C 235.

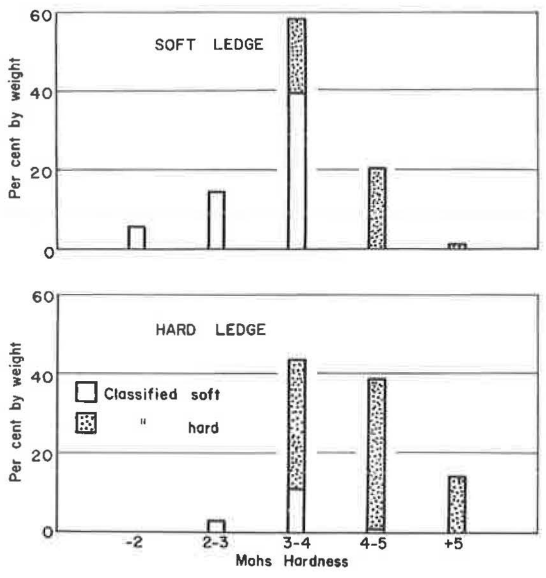

Figure 1. Mohs hardness vs scratch hardness (+1½ inch sieve size).

fell within this 2 to 5 range and are concentrated in the 3 to 4 hardness group. Taken as a whole, the only difference between the hard and soft samples was the amount of material having a hardness of 4 or more or 3 or less, respectively. In other words, the tails of the skewed distribution histograms were simply reversed from one sample to another (Figs. 2 and 3).

In each instance the brass rod classified a portion of that material in the 3 to 4 Mohs hardness range as hard and a portion as soft. Apparently the separation of this hardness group is largely arbitrary and extremely difficult to make. This would account for the rather significant differences noted between operators conducting the scratch test on the same samples (Tables 2 and 3). There is, apparently, little difficulty in classifying, with the brass rod, that material having a Mohs

TABLE 4

MOHS HARDNESS CLASSIFICATION
(No. 6823—Soft Sample)

Mohs Hardness	-2	2-3	3-4	4-5	+5
	Percent by Weight				
Sieve Size					
+1½ in.	5.5	14.7	58.4	20.4	1.0
1½-1 in.	1.3	8.3	64.4	17.8	8.2
1-¾ in.	3.5	10.9	73.9	6.5	5.2
¾-½ in.	1.7	30.2	53.6	7.6	6.9
½-⅜ in.	2.2	42.9	44.0	8.4	2.5
Arithmetic Avg.	2.8	21.4	58.9	12.1	4.8

NOTE: The brass rod in the NCSA scratch hardness apparatus has a Mohs hardness between 3 and 4. This range coincides with that of a majority of the sample, making separation extremely difficult and largely arbitrary. This is evidenced by the appreciable overlap of the brass rod classified hard and soft fractions in the Mohs hardness range of 3 to 4 (see Fig. 1).

TABLE 5

MOHS HARDNESS CLASSIFICATION
(No. 6824—Hard Sample)

Mohs Hardness	-2	2-3	3-4	4-5	+5
	Percent by Weight				
Sieve Size					
+1½ in.	0.0	3.0	43.9	39.0	14.1
1½-1 in.	0.0	2.5	53.3	38.4	5.8
1-¾ in.	0.0	0.9	55.4	41.6	2.1
¾-½ in.	0.0	0.0	69.9	18.7	11.4
½-⅜ in.	0.0	4.2	60.2	20.3	15.3
Arithmetic Avg.	0.0	2.1	56.5	31.6	9.8

NOTE: The brass rod in the NCSA scratch hardness apparatus has a Mohs hardness between 3 and 4. This range coincides with that of a majority of the sample, making separation extremely difficult and largely arbitrary. This is evidenced by the appreciable overlap of the brass rod classified hard and soft fractions in the Mohs hardness range of 3 to 4 (see Fig. 1).

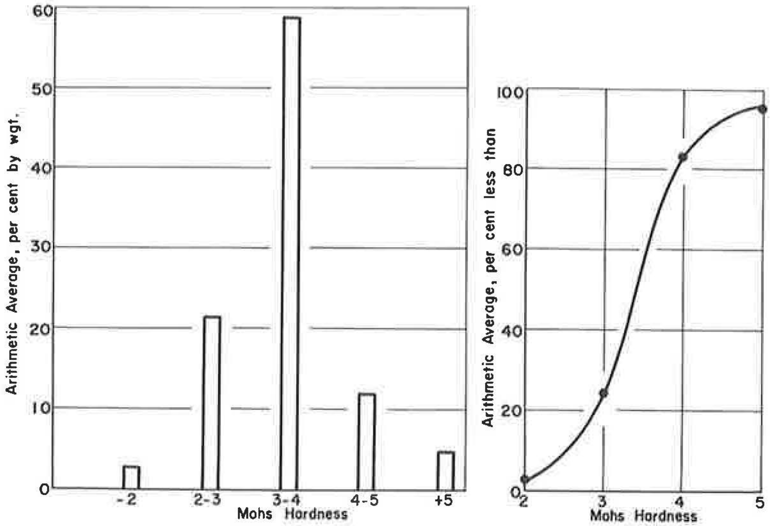

Figure 2. Mohs hardness distribution (No. 6823—soft ledge).

hardness of less than 3 or greater than 4, that is, in the separation of obviously soft from obviously hard. This statement gains support when Woolf's (3) data are re-examined. He synthesized samples by mixing a quartz gravel (Mohs hardness of 7) with obviously soft material and experienced no difficulty in making a rather precise separation with the scratch hardness test. The separation difficulty arises with the 3 to 4 hardness group, a range quite common to the majority of crushed stone aggregates, and the range which comprised roughly 60 percent of the materials involved in this investigation. Pursuing this line of reasoning, an individual working with the brass rod and the soft sample might classify anywhere from 24.2 to 83.1 percent of the sample as soft. Likewise, with the hard sample it might be reasonable to expect a classification of soft particles to fall within a range from 2.1 to 58.6 percent (Tables 4 and 5). If, therefore, a sample were chosen at random from the commercially produced aggregate, one might expect a soft particle classification anywhere from 2.1 to 83.1 percent. Although it is true that some of the variation might, in fact, reflect a true difference, the greatest

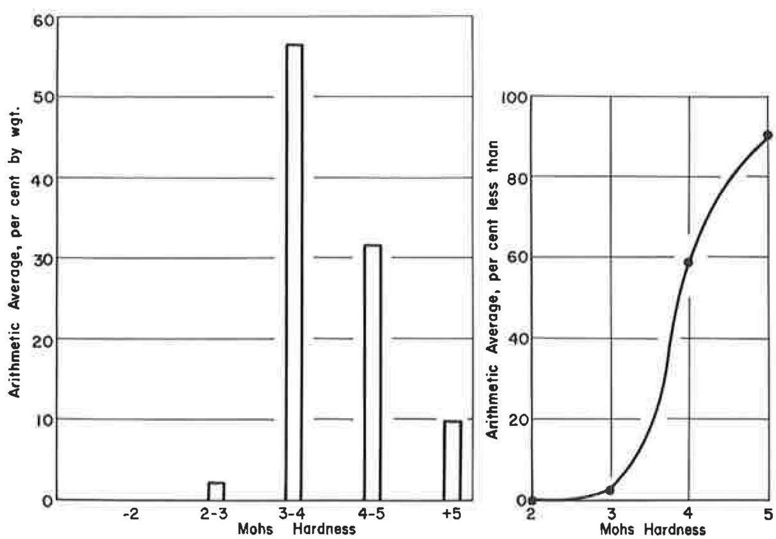

Figure 3. Mohs hardness distribution (No. 6824—hard ledge).

TABLE 6

RESULTS OF PHYSICAL TESTS PERFORMED ON CONCRETE AGGREGATES

Laboratory No. Identification	6824 Hard	6793 & 6794 Commercial	6823 Soft
Gradation (ASTM C 136)			
Total percent passing 2½ in.	100	100	100
2 in.	96	96	96
1½ in.	72	72	72
1 in.	50	50	50
¾ in.	38	38	38
½ in.	20	20	20
⅜ in.	10	10	10
No. 4	—	1	—
Specific gravity (ASTM C 127) bulk dry	2.71	2.70	2.66
Absorption, percent (ASTM C 127)	0.6	1.1	1.6
Dry rodded unit weight, pcf (ASTM C 29)	96.7	103.7	96.4
b/b_o	0.57	0.61	0.58
Soft particles (ASTM C 235) weighted average, %	8	17	65
Los Angeles abrasion, grading "A" (ASTM C 131) % loss in 500 revolutions	17	17	24
Sodium sulfate soundness (ASTM C 88) weighted % loss in 5 cycles	0.6	0.3	9.9
California durability index (Calif. 229-C)[a] durability factor, D_c	66	—	53
Water-alcohol test (HRB Bull. 201)[b] % loss in 16 cycles	0.1	1.5	11.2

[a]See text for description and interpretation.
[b]See text and Ref. (6) for interpretation.

majority of it would be the result of the difficulty in adequately classifying the 3 to 4 Mohs hardness group with the brass rod.

Performance in Concrete

The second phase of the laboratory investigation was the determination of the physical properties of the hard and soft fractions and a study of the performance of these materials as coarse aggregate in portland cement concrete. In addition to the prepared samples, a sample of the commercially produced concrete aggregate was secured and included in these tests. These three samples of aggregate had, respectively, 8 percent soft particles in the laboratory hard stone, 17 percent in the commercial stone, and 65 percent in the laboratory soft stone. The laboratory crushed samples were graded the same as the commercial aggregate and the physical properties of each determined (Table 6).

The air-entrained concretes were designed to have a cement factor of 6.25 sacks/cu yd, a slump of 1 to 2 in., and an air content of 4.0 ± 0.5 percent, in accordance with NCSA Bulletin No. 11 (5). Each mix was repeated at least once and specimens prepared and tested for compressive and flexural strength, as well as for the effect of freezing in air and thawing in water. The use of the small specimens (3 by 3 by 12 in.) for the determination of freezing and thawing resistance necessitated wet screening the fresh concrete over a 1-in. sieve prior to casting the prisms. As a check against the effect of the abnormally high cement factors thus produced, 3 by 3-in. beams were also cut from the 6 by 6-in. beams used in the flexural strength test and these, too, subjected to the freezing and thawing procedure. The results of the tests performed on both the plastic and hardened concretes are summarized in Table 7.

The performance of the three aggregates in the various physical tests is summarized in Table 6. Even the hard ledge (No. 6824) had 8 percent soft particles, and thus would not be acceptable under the C 33 provision. The water absorption increased in direct

TABLE 7

RESULTS OF TESTS ON PLASTIC AND HARDENED CONCRETE

Mix No.	A	B	C
Coarse aggregate laboratory No.	6824	6793 & 6794	6823
Coarse aggregate identification	Hard	Commercial	Soft
Soft particle content, %	8	17	65
Actual batch proportions, per cu yd of concrete			
Cement, lb	595	600	596
Water, lb	240	238	241
Fine aggregate, lb[a]	1040	830	1000
Coarse aggregate, lb	2150	2330	2160
Vinsol resin solution, oz	5.8	5.9	5.9
Actual cement factor, bags/cu yd	6.3	6.4	6.3
Actual water content, gal/cu yd[b]	28.8	28.6	28.9
Actual water-cement ratio, gal/bag[b]	4.57	4.47	4.59
Slump, in.	1.0	1.6	1.1
Air content, %	3.8	4.3	3.6
Unit weight, pcf	149.8	149.2	149.2
Compressive strength, psi (28 day)	5230	4740	4510
Flexural strength, psi (28 day)	790	735	730
Freezing and thawing resistance (ASTM C 291)	Durability Factor at 300 Cycles		
Wet screened over 1 in.	101	102	101
Cut beams[c]	98	97	96

[a] Laboratory stock—natural sand; bulk dry specific gravity = 2.62; absorption = 0.6 percent.
[b] Net water content; does not include water for absorption of aggregates.
[c] 6 by 6-in. flexural strength beams cut to 3 by 3 by 12-in. prisms for F & T Tests.

proportion with the soft particle content but even with a soft particle content as high as 65 percent, did not exceed 1.6 percent, certainly not a level for concern. The Los Angeles abrasion test results indicate that even that sample containing 65 percent soft particles would be well within any specification set forth for concrete aggregate. The sulfate soundness results, although a measurable difference was recorded between that sample having 8 percent soft fragments and that containing 65 percent, indicate that each of the materials was sound and should perform satisfactorily in concretes subjected to a freezing and thawing environment.

Two non-standard physical testing procedures were also employed in an effort to develop as much information relative to the three aggregate materials as was possible. The California durability index test (California Division of Highways Method 229-C) is a test designed to detect those base course aggregates which, when subjected to traffic, might be expected to degrade excessively and produce plastic fines. In a sense it might be considered a form of "hardness" test more akin, possibly, to the abrasion tests. The results are evaluated on an empirical scale ranging from 0 to 100, the harder, more resistant materials yielding higher numerical results. Certainly the difference recorded for the hard and soft ledges would not be considered critical in light of the California specification which permits materials having an index as low as 35. The final physical test performed, the water-alcohol test as described by Brink (6), is the one test whose results might suggest that all is not as it should be. The loss (11.2 percent) recorded for the soft ledge would, under the interpretation of the test as advanced by Brink, be suggestive of a material which might not yield frost resistant concrete. The results of the freezing and thawing tests on the concrete specimens (Table 7) contradict such a conclusion in this instance.

Analysis of Test Data

The performance of the three aggregates in portland cement concrete is given in Table 7, as well as the proportions and properties of the fresh concrete. Both the compressive and flexural strength results tend to decrease somewhat as the soft particle content increases. The increase from 8 to 17 percent soft particles resulted in approximately a 500-psi decrease in the recorded compressive strength and a 50-psi decrease in the measured flexural strength. A further increase in soft particle content to 65 percent resulted in an additional change of roughly 250 psi in compressive strength but little if any additional change in flexural strength. The effect on strength, therefore, is not uniform and in light of these results and the fact that even with a soft particle content as high as 65 percent, both the compressive and flexural strengths were well in excess of those normal to 6.25 bag concrete; the 5 percent soft particle limitation present in many concrete aggregate specifications is without practical significance.

The durability factors recorded following the procedure set forth in ASTM Designation C 291 likewise lend no support to the validity of limiting soft particles, per se, to a maximum of 5 percent. All the concretes, regardless of soft particle content or initial treatment, performed satisfactorily when subjected to a freezing and thawing environment.

SUMMARY

To summarize the results of all the laboratory tests performed, the following statements seem to be in order:

1. Difficulty was encountered when subjecting the serpentine samples to the scratch hardness test. Results had poor reproducibility and when compared with those of the Mohs hardness test indicated that the scratch hardness results were largely arbitrary and of little significance in the 3 to 4 Mohs hardness range.
2. The coarse aggregate samples performed well in all the physical tests except the ASTM scratch hardness test. None of the aggregate samples prepared would meet the 5 percent soft particle limitation set forth in ASTM C 33.
3. Despite the nonconformance to C 33, each of the materials produced strong, durable concretes, thus casting doubt on the strict interpretation of the soft particle limitation, and on the significance of the scratch test. When the difference between 8 and 65 percent soft fragments fails to make a critical difference in performance, the 5 percent limitation is without validity.

More comprehensive research is needed to re-evaluate the present concept of aggregate hardness.

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Investigation of Aggregate Durability in Concrete

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The effects of aggregate on the freezing and thawing durability of concrete and methods of specifying aggregate quality have and continue to be of concern to both aggregate producers and highway engineers. The reported data highlight certain difficulties in the interpretation of conventional empirical soundness, freezing and thawing, and other tests.

•WHEN certain kinds of aggregate particles are used in concrete and become highly saturated and are then frozen, they expand and produce disintegration. The disintegration may be in the form of discrete popouts or pits or as a general overall deterioration of the concrete member. Investigators have searched for methods of identifying and limiting permissible quantities of these materials in specifications. Numerous test methods and procedures have been used. The conventional soundness tests have been in use for many years and continue to be popular. In the soundness tests, unconfined aggregate particles are subjected to alternate immersion in sodium or magnesium sulfate salt solutions and drying in an oven. A number of similar alternate freezing and thawing tests of unconfined aggregate in water, brine or alcohol-water solutions have been tried (1).

Many specifications place limits on certain classes of rock particles such as chert, shale, limonite, soft particles, and absorbent particles. Laboratory freezing and thawing tests of the aggregate after incorporation in concrete offer much promise, but for various reasons only a few agencies have used them for specifying aggregate quality.

Each of these three types of tests—soundness, limitations on rock types, and laboratory freezing and thawing tests—present certain difficulties and have certain limitations. The sodium sulfate soundness test was first standardized by ASTM almost 35 years ago. Both this early version and current standard methods recognize that the test should be considered only as a guide to the selection of aggregate and that consideration should be given to the service record of the aggregate.

As noted, laboratory freezing and thawing tests have not received wide acceptance. A number of different procedures have been suggested, and at this time, three have been standardized by ASTM (2). All three methods warn that "the method is not intended to provide a quantitative measure of the length of service that may be expected from a specific type of concrete."

Over the years, attempts to relate results of the soundness, laboratory freezing and thawing, and service records of aggregates have met with only moderate to poor success. In a general sense, most aggregates with high soundness losses tend to have poor durability, but there are numerous exceptions. Perhaps it is too much to expect a single simple laboratory test to predict the performance of aggregate in service when the field exposure is so complex. The basic climatic exposure differs greatly in different parts of the country and even in a given location the temperature and moisture exposure of different types of structural elements will be greatly different.

The Joint Research Laboratory has been actively studying the effects of aggregate on the durability of concrete for many years. This paper discusses three series of tests.

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Of principal interest will be data relating to the correlations between results of laboratory freezing and thawing and routine aggregate tests; the effects of drying on resistance to laboratory freezing and thawing durability; and the correlation between laboratory freezing and thawing and exposure to natural weathering.

SCOPE OF TESTS

Data from Series 161, 156, and 171 will be discussed. The detailed testing procedures are described in the Appendix.

In Series 161, 56 coarse aggregates and 36 sands were incorporated in air-entrained concrete. Each of the coarse aggregates was combined with a single high-quality siliceous sand and each of the sands was mixed with a high-quality natural quartz gravel. The test coarse or fine aggregate was saturated under vacuum to produce a high degree of saturation of the aggregate particles prior to incorporation in concrete. All concretes were cured continuously moist before exposure to freezing in air at 0 F and thawing in water at 40 F in accordance with ASTM Method C 291.

In Series 156, 16 coarse aggregates were incorporated in 5, 6, and 7 sack air-entrained concrete. The coarse aggregates were soaked 24 hours prior to incorporation in concrete rather than vacuum saturated.

In Series 171, coarse aggregates from five of the sources tested in Series 161 were incorporated in air-entrained concretes. One 14 by 14 by 6-in. slab and one 3 by 4 by 16-in. beam were made from each batch for exposure to natural weathering in Maryland. Two similar beams were made from each batch for exposure to the standard ASTM C 291 laboratory freezing and thawing test. One beam was cured continuously moist and the other moist with a short drying period. At this time, the concretes have been in the outdoor exposure plot for 6 years.

DISCUSSION OF TEST RESULTS

Series 161. Tests of Highly Saturated Aggregate and Concrete

The average results of tests of coarse aggregates are given in Tables 1, 2, and 3. The fine aggregate results are summarized in Table 4.

Measures of Concrete Durability. The three measures of concrete durability given in Table 1 are durability factor at 300 cycles, cycles required to reduce the relative dynamic modulus of elasticity 50 percent and the number of cycles required to reduce dynamic modulus 20 percent. Each has its particular advantages and, in general, they are closely but not precisely related to each other. The durability factor provides no information on specimen behavior past 300 cycles of exposure, but it has the advantage that it provides a number for all specimens regardless of whether the specimen fails or not. If a specimen is unaffected by freezing and thawing, the number of cycles to reduce E either 50 or 20 percent is indeterminate. Cycles to reduce E 20 percent have the advantage that more specimens reach this value than reach 50 percent.

The logarithm of the number of cycles to reduce relative E appears to be a more significant index of relative durability than the number of cycles per se. Small differences in number of cycles are significant when the number of cycles is small, but of little significance when the number of cycles is large. For instance, the difference between 100 and 10 cycles is probably as significant as the difference between 100 and 1,000 cycles or 1,000 and 10,000 cycles.

Half the specimens tested in Series 161 were wrapped in aluminum foil in an attempt to prevent specimens from drying out and thereby provide a more severe exposure. The weight change data suggest that drying was retarded, but there was no consistent difference in resistance to freezing and thawing for the two groups of specimens. The values for plain and wrapped beams have been averaged for all subsequent analyses.

The distribution of durability of concretes made with different coarse aggregates is shown in Figure 1. The average number of cycles required to reduce E 20 percent ranged from 1 to 1,079. Only about 15 percent of the aggregates withstood more than 300 cycles. Half the concrete withstood less than 35 cycles and 20 percent withstood less than about 9 cycles of freezing and thawing.

TABLE 1

ROUTINE COARSE AGGREGATE TEST RESULTS, SERIES 161[a]

Ref. No.	Sp. Gr. Bulk Dry	Absorption		Soundness Loss (%)			L. A. Abrasion Loss (%)		Dry Rod Voids (%)
		24 Hr Soak	Vac. Sat.[b]	Sodium Sulfate (5 cyc.)[c]	Mag. Sulfate (5 cyc.)[c]	Brine F & T (10 cyc.)	100 Rev.	500 Rev.	
2	2.54	2.0	2.5	22.5	16.8	10.1	6.5	32.4	37.2
4	2.52	1.6	2.3	5.6	10.1	12.2	5.9	29.1	34.5
6	2.52	1.9	2.6	9.5	12.2	3.4	8.6	36.0	36.6
8	2.45	2.0	2.9	5.6	5.4	1.6	5.3	23.6	36.3
11	2.61	0.7	0.9	2.0	3.9	0.8	7.5	33.5	36.0
12	2.60	0.6	0.9	2.6	3.4	1.1	7.7	34.1	36.9
13	2.61	0.5	0.7	1.4	2.0	1.2	19.1	61.7	35.7
14	2.61	1.6	1.7	18.4	19.6	12.9	6.6	33.0	39.1
17	2.66	1.5	1.9	6.4	6.0	1.9	6.0	28.6	35.8
18	2.24	5.0	6.8	10.0	3.6	2.9	3.6	19.8	36.9
20	2.53	1.8	2.6	7.7	10.1	3.3	7.9	32.5	35.7
22	2.63	1.2	1.5	7.2	7.1	4.1	5.4	25.1	37.5
24	2.59	1.4	1.6	7.0	8.7	3.4	5.9	28.4	37.2
26	2.60	0.8	1.0	3.9	6.0	2.8	7.8	32.4	37.9
28	2.49	2.0	3.5	6.5	9.9	3.4	5.4	24.9	41.6
30	2.58	0.8	1.0	3.0	7.7	3.6	24.6	67.2	37.1
31	2.30	3.8	5.6	7.4	4.0	2.4	2.6	13.2	38.6
32	2.52	1.7	2.2	15.4	7.8	4.0	7.0	29.6	38.9
34	2.53	1.5	1.9	12.4	14.4	6.4	14.9	47.8	38.7
36	2.53	2.0	3.0	11.6	23.5	6.8	10.1	37.9	36.4
37	2.68	0.7	0.5	0.8	1.4	1.0	2.6	13.2	35.5
39	2.60	0.5	0.6	2.4	4.0	2.2	15.9	56.5	36.8
42	2.61	0.6	0.7	2.6	4.4	2.3	18.1	57.8	36.7
46	2.50	2.4	2.7	17.6	25.6	10.4	6.2	28.8	39.8
47	2.47	2.5	2.9	17.2	22.0	13.0	6.8	29.7	37.5
48	2.60	0.9	1.0	5.8	12.2	2.4	8.4	36.4	35.7
49	2.67	1.0	1.3	2.7	2.6	1.2	3.5	19.4	33.5
50	2.65	1.2	1.6	5.0	5.1	2.3	4.2	21.0	34.0
51	2.52	2.7	3.6	20.0	20.4	12.5	9.4	33.7	36.6
52	2.58	1.7	1.6	17.8	16.6	15.8	6.0	27.5	40.8
53	2.51	4.1	4.6	24.1	30.9	19.8	8.0	34.6	36.4
54	2.56	2.6	3.1	13.2	16.2	9.5	6.8	30.4	35.6
55	2.63	1.4	1.7	4.6	6.9	2.3	4.6	26.3	35.5
57	2.63	1.1	1.3	3.9	6.5	1.4	7.3	33.3	39.4
58	2.70	0.9	1.1	2.6	1.6	1.5	3.4	17.8	34.9
59	2.66	1.3	1.7	4.8	4.6	2.8	4.5	21.3	35.9
60	2.59	2.1	2.7	8.4	11.2	6.1	6.1	27.1	36.7
62	2.62	1.8	2.8	7.8	9.8	2.8	5.4	25.5	36.3
64	2.64	0.9	1.3	5.0	5.8	2.0	5.6	24.1	39.3
66	2.66	0.8	1.1	2.4	4.2	2.3	5.0	24.4	38.3
68	2.62	1.0	1.3	2.2	3.9	1.9	5.0	24.6	38.2
69	2.82	0.2	0.3	1.3	3.7	1.1	11.1	45.1	38.8
71	2.62	1.7	2.3	6.3	5.8	2.3	6.6	28.8	36.3
73	2.62	1.1	1.6	3.2	3.9	1.4	4.7	22.9	37.3
75	2.62	1.3	1.4	4.1	8.6	1.6	8.3	33.8	35.1
77	2.64	1.3	1.4	10.1	18.1	4.7	13.3	46.3	37.7
79	2.44	3.2	3.5	17.1	24.5	16.1	6.3	28.3	35.1
81	2.40	3.1	5.0	20.7	1.4	3.5	10.3	34.7	39.4
85	2.62	1.7	2.0	8.8	8.3	3.0	6.4	26.0	37.3
86	2.56	2.8	3.6	16.7	16.6	6.5	7.7	32.7	42.5
87	2.65	1.4	1.6	6.3	5.0	2.4	4.2	19.8	35.2
88	2.67	1.1	1.3	2.8	3.0	1.7	3.3	17.7	35.8
89	2.63	1.4	1.9	5.2	4.2	2.6	3.8	18.7	35.0
90	2.63	1.4	1.9	4.7	3.7	2.4	4.2	19.8	35.1
91	2.66	1.2	1.5	3.0	2.3	1.4	2.9	16.6	34.6
92	2.66	1.1	1.6	4.2	3.4	2.2	3.6	18.6	35.2

[a]Specific gravity and absorption values are weighted averages of tests of 4 sizes used in concrete. Soundness tests are averages of tests of 3 samples of each size fraction. Abrasion tests are results of single tests of "B" gradings by ASTM Method C 131-55. Void content of dry rodded aggregates are averages of 3 tests.

[b]Dried, saturated under vacuum and soaked 24 hours.

[c]By ASTM Method C 88-56T.

TABLE 2

COARSE AGGREGATE LITHOLOGICAL CHARACTERISTICS AND TESTS, SERIES 161[a]

Ref. No.	½ to ⅜-In. Size: Heavy Liquid Separation (% by wt) <2.35	<2.45	Chert <2.35	Chert <2.45	Total Chert	Total Deleterious	¾ to ½-In. Size: Chert	Deleterious	Soft ASTM C 235
2	0.4	6.6	0	0.6	10.0	32.5	5	5	14
4	6.7	14.8	2.1	5.9	12.5	12.5	11	13	1.2
6	4.4	14.3	2.3	3.4	6.6	73.0	7	13	2
8	7.8	17.2	7.7	17.0	68.5	68.5	76	76	0
11	1.3	4.5	0.5	0.5	5.4	6.6	2	2	0
12	1.1	3.6	0.4	1.1	3.7	0.4	2	2	0
13	0	0	0	0	0	0	0	0	0
14	0	0	0	0	0	0	0	0	0
17	0.8	4.3	0.2	0.3	11.1	11.8	12	12	1.7
18	28.8	68.1	28.8	68.1	100	100	100	100	0
20	5.2	15.2	2.5	3.6	7.4	8.2	11	22.5	0.7
22	0.3	3.3	0.3	0.7	0.7	4.2	2	2.6	1.2
24	0.3	0.9	0	0	0	2.9	2	10	1.6
26	0.9	3.8	0	0	0	0.9	0	0.3	1.5
28	4.6	21.4	0	0	0	0	0	0	0
30	0	0	0	0	0	0	0	0	0
31	29.6	61.0	29.6	61.0	100	100	97	97	0
32	4.6	15.6	4.7	15.6	59.0	59.0	42	42	0
34	10.0	13.5	7.2	9.1	13.2	18.0	8	8	1.0
36	2.1	21.2	1.2	2.0	4.0	13.5	6	35.2	3.1
37	0.6	1.1	0	0	0	0.8	0	2	0.8
39	1.0	1.5	0.4	0.4	0.4	0.5	0	0	0
42	0	0.6	0	0	0	0	0	0	0
46	12.5	15.6	7.9	9.5	10.9	17.4	10	13	0.8
47	8.1	13.3	5.5	6.9	9.0	13.5	13	18	0
48	0	0	0	0	0.7	0.7	0	17	0
49	1.6	4.1	0.4	1.1	9.0	10.7	7	7.6	0.4
50	11.7	16.8	0	0.9	10.8	11.7	10	13.8	0.9
51	0.6	2.1	0.5	1.0	4.4	20.6	4.7	29.2	15.4
52	0	2.5	0	0	0.4	0.4	3	7	5.5
53	4.6	17.2	0	0.3	0.3	50	0.4	29.0	22.4
54	4.8	9.4	0.9	2.2	8.6	21.9	3.2	25.9	16.3
55	1.0	5.2	0.8	2.2	5.9	14.1	4.7	11.0	1.2
57	1.1	2.5	0	0.3	0.3	8.8	0	11.9	6.0
58	0.2	1.8	0.2	1.2	8.4	10.8	9	11.4	0.1
59	1.4	4.8	0.1	0.6	7.8	13.6	10.7	15.7	2.1
60	3.3	9.7	1.1	2.7	5.2	10.7	4.2	20.6	5.8
62	2.1	3.4	1.1	2.0	5.8	7.5	3.8	5.0	0.7
64	1.4	4.1	0	0	0.2	5.4	1.1	3.2	2.1
66	0.2	1.0	0	0	0	0.2	0	4.0	1.0
68	1.6	3.6	0	0	0	0	0.6	7.1	1.0
69	0	0	0	0	0	0	0	0	0
71	3.4	8.4	3.2	8.1	25.6	27.4	14.1	14.1	0.2
73	1.5	4.6	0.4	1.2	1.4	2.7	1.9	5.1	1.2
75	0	0.7	0	0	0	4.0	0	2.7	0
77	0	0	0	0	0	0	0	3.3	0
79	18.1	25.1	15.7	19.4	21.0	23.3	15.7	15.7	1.9
81	1.0	27.2	0	0	0	0	0	0	0
85	2.6	8.0	1.1	4.3	11.4	17.6	10.3	25.4	7.4
86	0.9	3.6	0	0	2.3	2.3	3.2	13.8	10
87	3.2	6.3	0.9	2.5	9.0	12.0	7.9	11.7	3.2
88	2.4	4.5	0.9	2.0	11.6	13.6	5.9	7.9	1.2
89	3.2	7.3	1.1	3.5	15.8	18.4	9.7	14.7	4.1
90	2.6	7.7	1.0	4.7	13.7	16.0	9.4	12.8	2.7
91	0.9	3.6	0.9	2.8	11.3	12.3	7.9	7.9	0
92	2.0	3.8	0.6	0.8	4.7	7.7	2.4	4.8	0.9

[a]All tests made on samples of approximately 300 particles. Analyses of ¾ to ½-in. size made over a period of 2 years. Heavy liquid tests of ½ to ⅜-in. size made over a period of a week. Samples for heavy liquid tests were saturated under vacuum and soaked 24 hours before separation.

TABLE 3

CHARACTERISTICS OF FRESH CONCRETE AND RESULTS OF FREEZING AND THAWING TESTS, SERIES 161[a]

Ref. No.	Fresh Concrete			Results of Freezing and Thawing Tests								
				Durability Factor at 300 Cycles			Cycles to Reduce E					
							50 Percent			20 Percent		
	Slump (in.)	H_2O (gal/cu yd)	Air (%)	Plain	Al. Foil	Avg.	Plain	Al. Foil	Avg.	Plain	Al. Foil	Avg.
2	3.3	33.5	4.7	17	15	16	100	75.1	87.5	43.1	35.6	39.4
4	3.4	32.1	4.8	10	21	16	61	127	94	9.8	20.9	15.4
6	3.4	33.6	4.5	45	45	45	286	277	282	59	85.8	72.4
8	3.1	32.7	4.4	1	1	1	5.2	6.3	5.7	2.3	2.6	2.4
11	3.7	33.5	4.7	65	51	58	—[b]	—[b]	—	145	103	124
12	3.5	33.5	4.7	93	66	80	—[c]	—[c]	—	—[c]	153	—
13	3.7	33.5	4.6	97	99	98	—[c]	—[c]	—	—[c]	—[c]	—
14	3.7	35.0	4.7	29	27	28	174	161	168	85	78	81
17	3.4	32.7	4.5	30	28	29	180	169	174	36	32	34
18	3.6	35.4	4.3	0	0	0	1	1	1	1	1	1
20	3.4	33.4	4.2	45	50	48	298	298	298	73.2	76	74.6
22	3.3	33.8	5.0	80	60	70	—[b]	—[b]	—	322	215	268
24	3.7	33.6	4.8	36	19	28	213	150	182	57.5	41	49.2
26	3.5	33.5	4.5	74	66	70	673	426	550	269	174	222
28	3.9	34.7	4.8	56	37	46	358	225	292	90	57	74
30	3.2	32.9	4.9	98	100	99	—[c]	—[c]	—	—[c]	—[c]	—
31	2.9	33.6	4.9	1	1	1	3	3	3	1.3	1.3	1.3
32	3.0	33.5	4.0	1	1	1	7	7	7	2.5	2.6	2.6
34	4.1	35.2	4.3	5	7	6	28	40	34	6.5	10.6	8.6
36	3.6	32.9	4.1	21	12	16	124	87	106	21.9	18.4	20.2
37	3.7	32.3	4.7	51	52	52	331	309	320	125	123	124
39	3.2	33.2	4.3	55	54	54	417	357	387	66	67	66
42	3.5	33.1	4.6	96	98	97	—[c]	—[c]	—	—[c]	—[c]	—
46	3.3	34.4	4.6	44	38	41	271	229	250	53	40	46
47	3.4	33.0	4.4	9	12	10	56	72	64	10.6	10.3	10.4
48	4.0	32.8	4.5	73	76	74	1277	567	922	228	225	226
49	3.9	32.8	4.1	9	13	11	54	78	66	18.8	27.4	23.1
50	4.0	32.8	4.2	8	9	8	48	52	50	16.9	16.6	16.8
51	3.4	33.1	4.8	8	12	10	45	70	58	13.1	26.1	29.6
52	3.3	34.7	4.8	27	13	20	161	135	148	66	52.9	59.4
53	3.7	32.6	5.1	1	2	2	10	11	10	4.2	4.5	4.4
54	3.7	32.8	4.5	1	3	2	7	18	12	2.9	4.1	3.5
55	3.8	32.8	4.6	9	12	10	53	74	64	14.8	24.9	19.8
57	3.4	34.5	4.3	63	61	62	—[c]	—[c]	—	133	161	147
58	3.3	31.4	4.5	22	21	22	134	126	130	48	48	48
59	3.4	31.4	4.8	12	9	10	74	54	64	20.8	18.9	19.8
60	3.8	32.9	4.2	4	7	6	23	44	33.5	7.9	17.1	13.0
62	3.4	31.4	4.9	16	14	15	96	84	90	26.6	23.9	25.2
64	3.1	33.6	4.8	79	82	80	617	644	631	296	269	282
66	3.0	33.9	4.4	65	83	74	581	692	636	261	328	294
68	2.9	34.0	4.2	30	22	26	177	130	154	32.1	27.5	29.8
69	3.2	33.5	4.6	97	97	97	—[c]	—[c]	—	—[c]	—[c]	—
71	3.8	32.3	4.5	7	8	8	44	48	46	8.7	7.4	8.0
73	3.5	33.2	4.0	89	78	84	799	576	688	464	290	377
75	3.1	31.4	4.8	90	92	91	—[c]	1111	—	1416	742	1079
77	3.2	32.2	4.2	85	82	84	1409	944	1176	612	420	516
79	3.2	32.4	4.8	4	5	4	23	31	27	4.9	5.6	5.3
81	2.6	33.4	4.9	20	19	20	118	114	116	30.1	30.8	30.6
85	3.9	33.3	4.7	6	7	6	36	45	40.5	7.3	8.2	7.8
86	3.4	35.4	4.9	29	27	28	172	162	167	84.7	86.9	85.8
87	3.7	32.5	4.5	12	11	12	70	69	69.5	18.6	12.1	15.4
88	3.8	32.4	4.8	14	15	14	85	89	87	19.6	20.4	20.0
89	4.1	31.9	4.3	3	3	3	19	20	19.5	5.6	4.7	5.2
90	3.7	32.2	4.2	7	7	7	42	39	40.5	19.8	9.4	14.6
91	4.2	32.1	4.7	12	11	12	74	66	70	19.5	19.7	19.6
92	4.1	32.2	4.9	29	28	28	176	167	176	55.0	56	55.5

[a]All values are averages of tests of 3 batches or specimens. Concrete mixed by hand in 1/4-cu ft batches. Coarse aggregate saturated under vacuum and soaked 24 hr before incorporation in concrete. All coarse aggregates were graded 25 percent each of 1 to 3/4 in., 3/4 to 1/2 in., 1/2 to 3/8 in., and 3/8 in. to No. 4. Two 3 x 4 x 16-in. beams molded from each of 3 batches mixed on different days. All beams were cured continuously moist for 14 days before exposure to alternate freezing in air at 0 F and thawing in water at 40 F. One beam from each batch was wrapped in heavy gage aluminum foil during freezing and thawing.

[b]E 55 to 80 percent at 300 to 350 cycles.

[c]E 90 to 100 percent at 300 to 350 cycles.

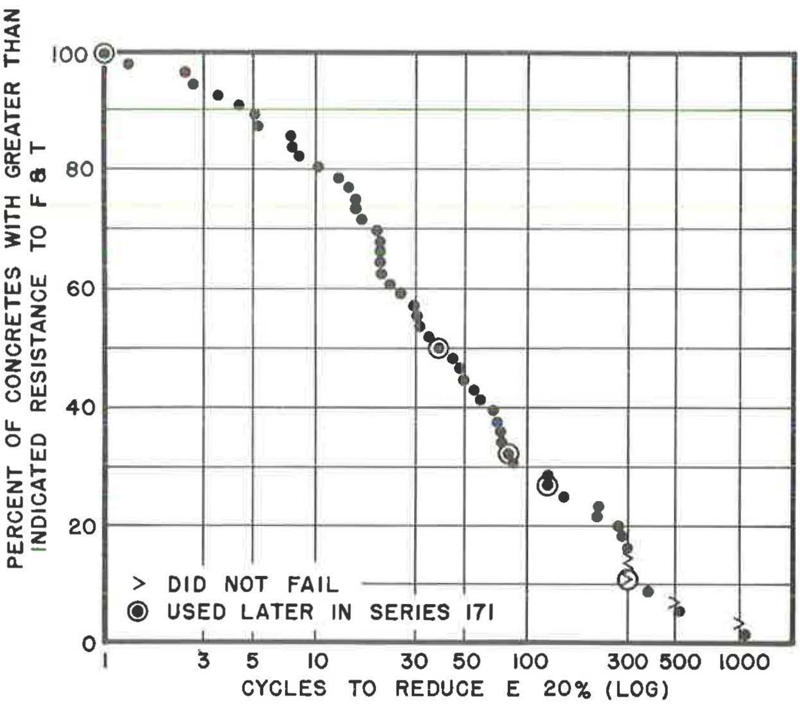

Figure 1. Distribution of coarse aggregate durability (Series 161).

Coarse Aggregate Tests and Resistance to Freezing and Thawing. ASTM Specifications for Concrete Aggregates, C 33, limit the loss in 5 cycles of sodium sulfate to 12 percent and that of magnesium sulfate to 18 percent. Of the 56 coarse aggregates tested, 12 had losses exceeding the limit for sodium and 7 exceeded the limit for magnesium sulfate. Figure 2 shows the relationships between the three soundness tests and the durability of concrete made with these coarse aggregates when tested in one particular laboratory freezing and thawing test.

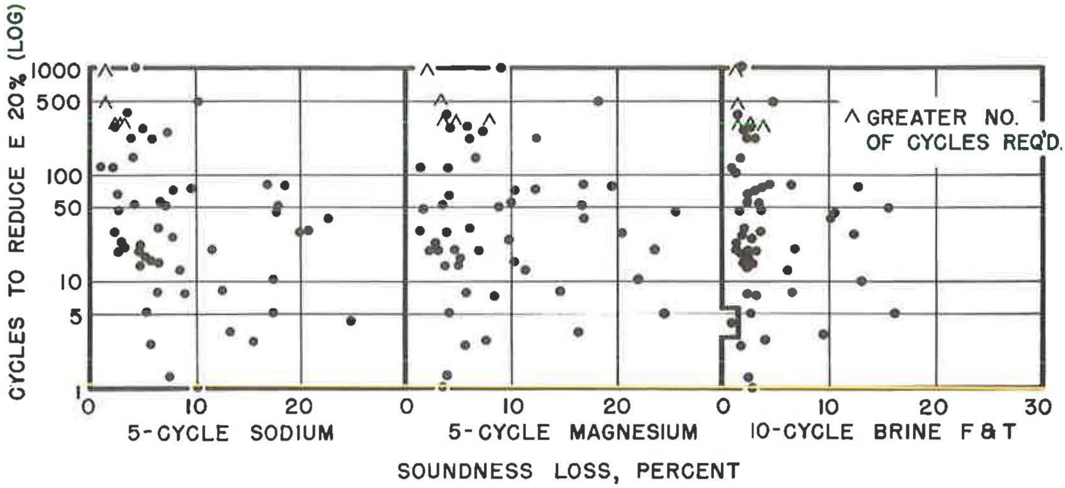

Figure 2. Relationship between soundness loss and resistance to freezing and thawing (Series 161).

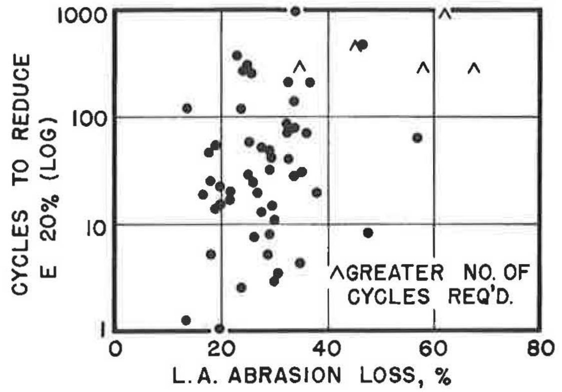

Figure 3. Abrasion loss vs resistance to freezing and thawing (Series 161).

There is no direct evidence that these soundness tests perform their intended purpose. It may be of some importance that few high soundness loss aggregates had excellent durability and that the soundness tests failed to detect several aggregates of low durability.

Figure 3 shows the relationship between Los Angeles abrasion loss and resistance to freezing and thawing. There is no evidence that high abrasion loss is indicative of low durability. In fact, in these tests correlation analyses show that high abrasion loss is associated with improved resistance to freezing and thawing. Although not shown here, the abrasion loss at 100 revolutions did not correlate with freezing and thawing resistance. The ratio of the loss at 100 revolutions to that at 500 revolutions has been suggested as a measure of the quantity of soft material in the aggregate. In these tests, that ratio was not correlated with freezing and thawing resistance.

Many investigations have found that the deleterious particles in a gravel are often concentrated in the lighter specific gravity fractions. Generally, these particles have higher absorption and are, therefore, more vulnerable to freezing when saturated. The heavy limonite particles are notable exceptions. The percent of the aggregate lighter

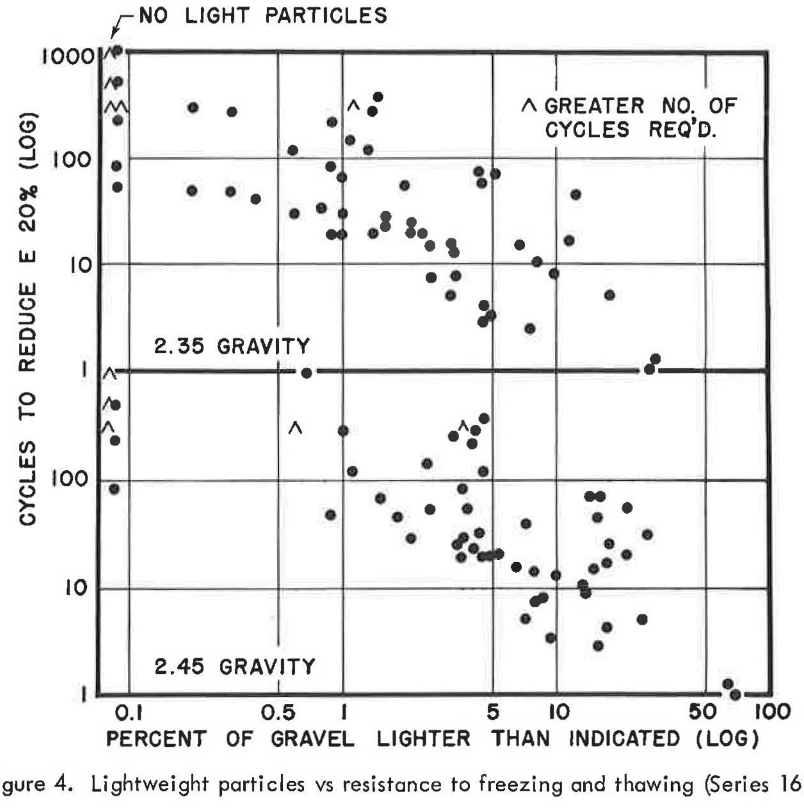

Figure 4. Lightweight particles vs resistance to freezing and thawing (Series 161).

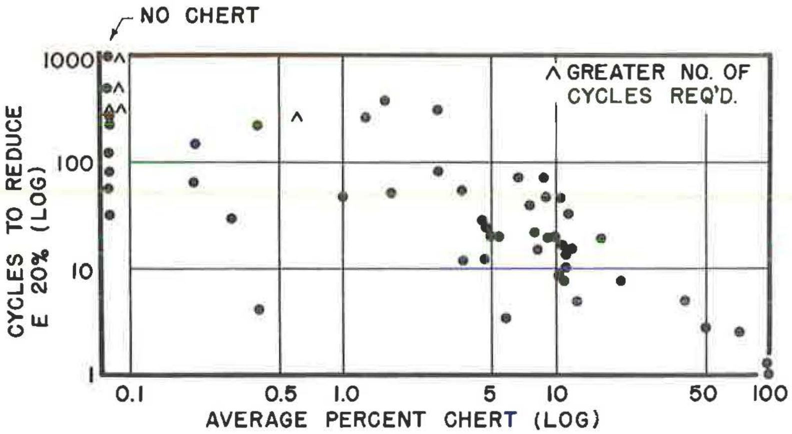

Figure 5. Chert vs resistance to freezing and thawing.

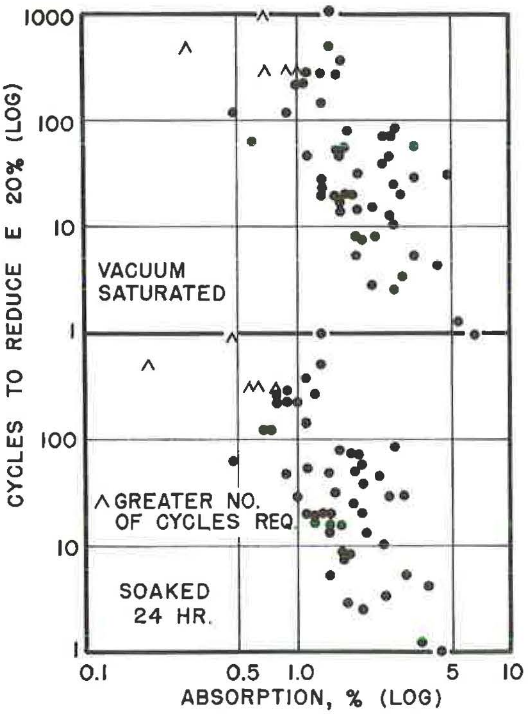

Figure 6. Absorption vs resistance to freezing and thawing (Series 161).

than either 2.35 or 2.45 is inversely related to resistance to freezing and thawing (Fig. 4). These data suggest that small percentages of lightweight particles have quite important effects on resistance to freezing and thawing.

The relationship between average chert content and durability is shown in Figure 5. In general, this relationship is as good or slightly better than the previous one. The relationships between freezing and thawing resistance and lightweight chert were similarly good.

The total percentage of deleterious particles was equally well related to durability. A principal difficulty with the use of "deleterious particles" as a specification criterion is that the determination of it depends on the experience of the particular petrographer. A similar difficulty exists with respect to chert, shale, absorbent particles, or any other lithological class of particles.

Figure 6 shows the relationships between freezing and thawing resistance and both 24-hour and vacuum-saturated absorption. Absorption is a better measure of freezing and thawing resistance than any of the sulfate soundness tests.

Tests of Sands. The results of tests of the 36 sands are summarized in Table 4. Absorption ranged from 0.3 to about 3.8 percent. The highest soundness losses were 9.8, 18.6, and 6.2 percent for sodium sulfate, magnesium sulfate and brine freezing and thawing, respectively. These can be compared to ASTM specification limits of 10 percent in 5-cycle sodium sulfate and 15 percent in magnesium sulfate, respectively. Figure 7 plots magnesium sulfate soundness loss and freezing and thawing durability of concretes made with the sands.

TABLE 4

SUMMARY OF AVERAGE TEST RESULTS FOR FINE AGGREGATES, SERIES 161[a]

Ref. No.	Sp. Gr. Bulk Dry	Abs. %		Soundness Loss %			Fineness Mod.	Mix. H_2O (gal)	Air (%)	Durability Factor at 300 Cycles (50% E)		
		24-Hr	Vac. Sat.	Sod. (5 cyc.)	Mag. (5 cyc.)	F & T (10 cyc.)				Plain	Al. Foil	Avg.
1	2.59	1.1	1.2	3.7	5.4	2.1	2.53	30.9	4.5	100	100	100
3	2.56	2.1	2.2	6.3	15.2	3.8	2.77	31.9	5.1	100	100	100
5	2.59	1.5	1.5	3.7	10.8	2.0	2.72	30.5	4.7	100	101	100
7	2.55	1.9	2.0	6.2	16.9	4.3	2.62	33.4	4.6	101	104	102
9	2.57	1.3	1.4	10.9	8.0	1.6	2.46	34.1	4.3	102	103	102
10	2.58	1.2	1.3	2.7	8.5	2.0	2.53	34.2	4.2	102	102	102
15	2.62	0.3	0.3	2.1	4.1	1.8	2.60	30.0	4.1	100	100	100
16	2.62	1.1	1.1	2.8	5.2	1.6	2.86	30.3	4.3	100	98	99
19	2.60	1.3	1.4	4.1	8.2	2.2	2.52	31.9	4.4	101	98	100
21	2.61	1.5	1.5	6.6	15.7	4.4	2.33	31.8	5.0	99	98	98
23	2.59	1.2	1.2	4.8	8.6	3.0	2.83	33.3	5.1	98	89	94
25	2.60	0.9	0.9	3.1	6.6	2.1	2.87	32.6	4.4	101	99	100
27	2.48	3.7	3.8	6.8	19.2	6.0	2.64	32.1	4.7	98	93	96
29	2.57	1.0	0.9	3.9	8.4	1.8	2.62	35.7	4.8	102	104	103
33	2.54	1.7	1.7	3.9	6.5	2.1	2.74	32.9	4.6	100	101	100
35	2.58	1.8	1.9	5.6	13.7	4.5	2.75	31.7	4.0	98	96	97
38	2.61	1.5	1.6	2.3	5.2	1.8	2.70	32.0	4.0	99	98	98
40	2.60	0.9	0.8	3.2	7.7	2.1	3.12	31.6	4.4	98	98	98
41	2.61	0.7	0.7	4.6	6.1	2.0	2.92	32.6	4.3	99	98	98
43	2.53	1.6	1.6	6.3	10.5	3.5	2.63	31.3	4.6	98	94	96
44	2.54	1.6	1.5	5.2	9.9	3.2	2.53	31.6	4.5	99	98	98
45	2.59	1.0	1.1	3.5	9.8	2.2	2.66	31.0	4.5	98	97	98
56	2.64	1.5	1.5	3.8	15.0	2.6	2.97	32.1	4.6	102	96	99
61	2.62	1.9	1.9	5.0	12.1	1.7	2.81	30.8	4.8	97	94	96
63	2.57	2.0	2.2	6.0	14.6	3.1	2.76	32.5	4.1	99	100	100
65	2.62	1.2	1.3	3.8	12.0	2.7	2.85	32.7	4.3	100	101	100
67	2.62	1.3	1.3	3.0	11.0	2.5	2.55	32.7	4.2	101	103	102
70	2.62	1.2	1.3	3.4	6.8	1.9	3.08	29.1	4.6	97	97	97
72	2.58	2.1	2.2	4.0	13.0	2.5	2.77	32.8	4.0	100	100	100
74	2.60	1.3	1.3	2.1	8.4	1.2	2.76	31.8	4.6	100	101	100
76	2.63	1.2	1.2	3.5	12.7	2.1	2.83	32.4	4.4	100	101	100
78	2.53	1.7	1.7	5.4	11.8	3.6	2.98	31.8	4.4	100	100	100
80	2.57	1.6	1.7	6.6	10.0	2.0	2.39	32.6	5.1	98	99	98
82	2.56	1.3	1.5	4.5	10.3	2.8	2.70	33.0	4.3	100	102	101
83	2.52	2.3	2.3	9.8	18.6	6.2	2.62	31.6	4.7	100	99	100
84	2.57	2.1	2.1	6.4	14.5	0.9	2.94	32.0	4.5	98	95	96

[a]All fine aggregates were saturated under vacuum and soaked 24 hours before incorporation in concrete. A single 1-in. maximum size quartz gravel was used with all sands. Sands were used in their as received grading. Beams were cured continuously moist for 14 days before exposure to freezing in air at 0 F and thawing in water at 40 F in accordance with ASTM Method C 291.

When these sands were vacuum saturated and incorporated in concrete with a high-quality quartz gravel, durability factors at 300 cycles all exceeded 94. Only 5 of the coarse aggregates performed this well. Generally the tests were discontinued after 300 to 400 cycles of freezing and thawing. However, whenever freezer space was available or when a specimen had developed some slight scaling the exposure was continued. A total of 13 sands was exposed to more than 450 cycles. With failure defined as a reduction in relative E of 20 percent, all six individual specimens from Sands 25 and 27 failed in averages of 840 and 920 cycles, respectively. The specimens wrapped in aluminum foil with Sands 43 and 83 failed in 500 and 830 cycles, respectively, and the unwrapped beams did not fail in 600 to 1,500 cycles. Two of the six specimens made with Sand 84 failed. None of the specimens with Sands 16, 19, 44, 63, 65, 72, 74, or 76 failed in 500 to 600 cycles.

As suggested by the "critical size" concept, an adequate air void system will protect the mortar phase of concrete from damage (3).

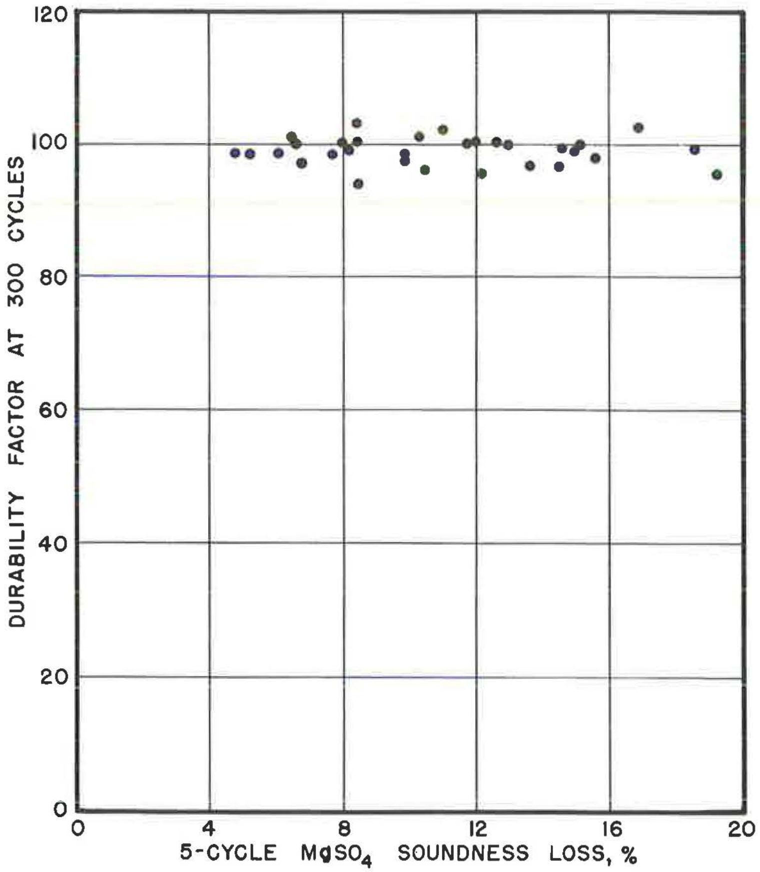

Figure 7. Relationship between soundness loss of sand and concrete durability (Series 161).

Series 156. Tests of Moist Cured Concretes Made With Less Highly Saturated Aggregates

Sixteen different coarse aggregates were incorporated in 5, 6, and 7 sack air-entrained concretes. These tests can be compared with those made in Series 161. There are two important differences, however. In these tests, the gravels were soaked 24 hours before incorporation in concrete rather than saturated under vacuum and the maximum size of the coarse aggregate was $1\frac{1}{2}$ in. instead of 1 in. The 24 hours of soaking should greatly reduce the severity of the exposure and the use of the larger maximum size might be expected to increase slightly severity of exposure for most gravels. The curing procedures and freezing and thawing exposures were comparable in both series.

Of the 16 aggregates only Aggregates 4 and 10 had durability factors at 300 cycles less than 93. For these two, relative E was reduced 20 percent in from 133 to 312 cycles. Two-thirds of the No. 15 coarse aggregate specimens failed in an average of 840 cycles and the remaining specimens were in excellent condition at 1,500 cycles. Single specimens from Aggregates 8, 12, and 13 had either failed or shown some evidence of failure when exposure was terminated at either 370 or 800 cycles. These specimens were markedly more durable than those tested in Series 161.

Table 5 compares results of the four aggregates which were tested in both Series 156 and 161. The aggregates were from samples taken two to three years apart. Only Aggregate 4 failed in Series 156. Based on the number of cycles to reduce relative E 20 percent, it was eighth from the poorest in Series 161. In Series 156, Aggregates

TABLE 5

COMPARISON OF FOUR AGGREGATES TESTED IN SERIES 156 AND 161

Agg. No.		Dur. Factor at 300 Cycles		Avg. Cycles To Reduce E 20%		Rank in Series 161
156	161	Series 156	Series 161	Series 156	Series 161	
2	11	98	58	300[a]	124	15
4	71	75	8	233	3.0	54
7	20	100	48	813[a]	74.6	18
9	37	100	52	350[a]	124	16

[a]Test discontinued at indicated number of cycles with no evidence of deterioration.

TABLE 6

COMPARISON OF SERIES 171 AND 161 LABORATORY TEST RESULTS

Series 171				Series 161		
Agg. No.	Cyc. To Reduce E 20%		Absorption, V. S. (%)	Agg. No.	Cyc. To Reduce E 20%	Rank
	Moist Cure	Air Dry			Moist Cure	
1	0.3	21	6.6	18	1	56
3	23	60	2.1	2	39.4	28
5	43	958	2.0	14	81	18
4	177	+2700[a]	0.8	11	124	15
2	1318	+2700[a]	1.3	30	+300	6

[a]Specimens unaffected by 2700 cycles and exposure terminated.

2, 7, and 9 showed no evidence of failure when the test was discontinued at number of cycles ranging from 300 to 813. In Series 161, these three aggregates were among the better one-third of those tested.

Eleven of the 16 aggregates contained chert and 6 of the 16 contained more than 10 percent chert. The results of the usual specification tests do not provide a means of identifying the two aggregates which failed in freezing and thawing in this series.

In Series 156, concretes were made with cement factors of 5, 6, and 7 sacks/cu yd, but there is no evidence that cement factor significantly affected resistance to freezing and thawing.

Series 171. Laboratory and Outdoor Exposure Tests

Five coarse aggregates were selected from those tested in Series 161 to provide a range in resistance to freezing and thawing. The Series 171 samples were obtained about 2 years later than those for Series 161. All aggregates were saturated under vacuum. The slab and beams for outdoor exposure tests and one of the beams for the laboratory C 291 tests were cured continuously moist for 14 days as in Series 161. The other beam for the laboratory tests was given 4 days of air-drying during its curing period.

Table 6 compares the results of laboratory tests made in Series 171 to those in Series 161. The 5 aggregates were rated in the same order in both series of tests. The agreement between the tests of moist cured concretes is considered excellent.

The Series 171 data provide an excellent comparison of the effects of 4 days of drying during the curing period. In all cases, freezing and thawing resistance was increased several fold by drying. Aggregate 3 seems to have benefited least from drying. As other investigations have demonstrated (4), different aggregates will not be benefited to the same extent by drying. The pore structure of the aggregate will have important effects on the amount of water taken from the aggregate pores on drying and the rate at which these pores become critically saturated during subsequent soaking (5).

Figure 8 contains the main results of the outdoor exposure tests. Data on the 3 by 4 by 16-in. beams are given in the upper two portions of the figure and that from the slabs in the lower portion. The specimens were placed in the exposure plot in December 1959 and received their first freezing cycle within a few days. Since that time there have been a total of 441 cycles of freezing and thawing. A cycle is defined as a temperature of 30 F or less followed by one of 34 F or more. The numbers of natural freezing cycles in excess of some minimum may not be particularly significant. In any event, they should not be compared with the laboratory exposure on a per cycle basis.

Only the concretes made with Aggregate 1 have deteriorated in the field exposure. All of these specimens developed popouts and cracks during the first winter. The sur-

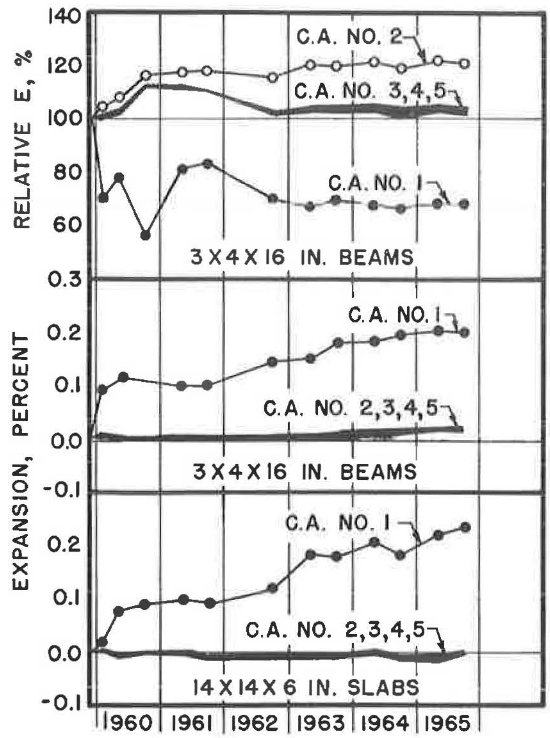

Figure 8. Field exposure of beams and slabs (Series 171).

faces which are in contact with the moist subgrade are most severely affected. The relative dynamic moduli of beams made with this aggregate were reduced to 70 percent the first winter.

The dynamic moduli of all other beams increased. The significance of the 20 percent increase in dynamic modulus which has been maintained throughout the exposure is not clear, but in any event does not indicate a proportionate increase in quality.

Both beams and slabs made with coarse Aggregate 1 expanded the first winter and have continued to expand since that time. At the end of six winters the expansion is 0.2 percent for beams and 0.24 percent for slabs. These values can be compared to those developed in the laboratory exposure ranging from 0.05 to 0.15 percent at the time dynamic modulus had been reduced 50 percent. The expansion of all other outdoor specimens was negligible.

Miscellaneous Data

Tables 7, 8, and 9 give additional data concerning Series 156. Table 10 gives the average results of laboratory freezing and thawing tests for Series 171.

TABLE 7

MISCELLANEOUS PHYSICAL TESTS OF COARSE AGGREGATES, SERIES 156

Ref. No.	Lot No.	Specific Gravity[a]			Absorption (%)[a]	Weight (pcf)[b]	Voids (%)[c]	Soundness Loss (%)[d]	L. A. Abrasion Loss (%)[e]
		Bulk Dry	Bulk SSD	Apparent					
1	2863	2.62	2.63	2.65	0.4	110.1	32.7	0.3	42.7
2	2836	2.60	2.62	2.64	0.6	107.3	33.8	1.8	31.6
3	2838	2.56	2.61	2.68	1.7	109.3	31.6	5.9	27.2
4	2839	2.62	2.67	2.75	1.7	111.7	31.7	8.1	31.7
5	2840	2.64	2.64	2.65	0.2	112.4	31.6	0.3	41.8
6	2845	2.58	2.60	2.65	1.1	109.6	31.9	4.3	27.2
7	2866	2.56	2.60	2.66	1.6	106.8	33.1	4.5	32.6
8	2867	2.46	2.52	2.62	2.5	103.2	32.8	10.9	39.6
9	2855	2.67	2.68	2.70	0.4	112.9	32.2	1.0	14.0
10	2878	2.66	2.69	2.74	1.1	109.2	34.2	3.9	22.2
11	2879	3.00	3.01	3.03	0.3	111.8	40.2	0.4	20.4
12	2881	2.70	2.73	2.78	1.0	112.1	33.5	3.9	27.6
13	2882	2.56	2.57	2.60	0.6	105.7	33.8	0.8	26.6
14	2892	2.60	2.62	2.65	0.6	111.1	31.5	0.8	17.2
15	2903	2.62	2.64	2.65	0.4	109.0	33.2	2.8	40.0
16	2910	2.60	2.62	2.65	0.7	107.8	33.5	2.6	25.7

[a] ASTM Method C 127; average of at least 4 tests; aggregate soaked 24 hours.
[b] ASTM Method C 29, for grading used in concrete.
[c] ASTM Method C 30, for grading used in concrete.
[d] 5-cycles sodium sulfate, ASTM Method C 88-46T; weighted average based on grading used in concrete.
[e] ASTM Method C 131-51, "A" grading, average of 2 tests.

TABLE 8

LITHOLOGICAL CHARACTERISTICS OF COARSE AGGREGATES, SERIES 156[a]

CA Ref. No.	Soft Particles ASTM C 235	% Crushed Particles, Fractured Faces		Rock Types (%)						
		1 or more	2 or more	Quartz and Quartzite	Hard Sandstone	Friable Sandstone	Misc. Igneous & Metamorphic	Calcareous	Chert and Flint	Misc.
1	0	31	19	100	—	—	—	—	—	—
2	0	64	53	82	8	—	1	—	3	6
3	1	20	15	4	44	10	8	23	11	—
4	0	64	56	4	3	2	7	67	17	—
5	0	67	56	96	—	4	—	—	—	—
6	3	47	38	82	—	—	2	1	10	5
7	0	38	35	18	51	5	10	6	7	3
8	3.2	32	28	17	53	6	6	5	11	2
9	0.5	29	25	40	—	—	16	44	—	—
10	3	39	36	11	—	5	23	57	4	—
11	0	100	100	—	—	—	100	—	—	—
12	3.5	48	44	6	2	—	64	27	1	—
13	0	36	24	18	2	—	12	—	68	—
14	0	5	3	54	—	—	9	20	17	—
15	0	53	49	100	—	—[b]	—[b]	—	—	—
16	0.8	48	44	48	—	—	50	1	1	—

[a]Analyses made on individual size fractions; values are weighted averages for grading used in concrete and involve analyses of at least 900 particles for each aggregate.
[b]Trace, less than 0.5 percent.

TABLE 9

RESULTS OF FREEZING AND THAWING TESTS, SERIES 156[a]

CA No.	Durability Factor at 300 Cycles (50% E)			Last Cycle (exposure stopped)			Relative E at Last Cycle			Cycles To Reduce Relative E 20%		
	5 sx	6 sx	7 sx	5 sx	6 sx	7 sx	5 sx	6 sx	7 sx	5 sx	6 sx	7 sx
1	102	101	101	298	295	295	102	101	101	—	—	—
2	99	96	98	298	298	297	99	96	98	—	—	—
3	102	101	101	381	352	361	103	103	101	—	—	—
4	81	62	75	737	395	515	48	49	48	312	133	253
5	102	102	100	418	390	418	103	102	101	—	—	—
6	101	100	100	428	400	400	101	100	100	—	—	—
7	100	100	100	813	813	813	102	102	100	—	—	—
8	99	99	98[b]	813	813	826[b]	95	97	88[b]	—[c]	—	—[c]
9	100	100	100	373	326	354	100	99	99	—	—	—
10	78	52	62	513	319	339[b]	51	49	50[b]	277	173	189
11	99	100	99	465	465	470	100	100	99	—	—	—
12	97	98	85	457	455	370	96	99	76	—	—	—[d]
13	95	93	93	375	375	375	95	93	92	—	—[d]	—
14	96	96	96	417	417	423	97	96	96	—	—	—
15	94	95	96	1270	1470	1370[e]	62	72	71[e]	710+[f]	970+[f]	830+[f]
16	96	96	96	1400	1400	1400	100	99	100	—	—	—

[a]1½-inch maximum size coarse aggregate immersed 24 hours before incorporation in concrete. Concretes designed to have a slump of 2 to 3 in. and an air content of 4 to 5 percent. Concretes mixed by machine in 1.1 cu ft batches. One 3 x 4 x 16-in. beam was molded from each batch and cured continuously moist for 14 days before exposure to alternate freezing in air at 0 F and thawing in water at 40 F in accordance with ASTM Method C 291. Specimens were subjected to a minimum of 300 cycles of exposure. In a few instances where relative moduli of elasticity were reduced, tests were continued to as many as 1500 cycles of freezing and thawing.
[b]One specimen failed suddenly at 225 cycles due to one large soft particle in the center of cross section. Value not included in average.
[c]One of 3 specimens showed evidence of impending failure when exposure discontinued at approximately 800 cycles.
[d]One of 3 specimens failed in 250 to 380 cycles. All other specimens unaffected by 370 cycles when exposure was discontinued.
[e]Test stopped at 1500 cycles. In each case at least 1 of the 3 specimens failed at about 1000 cycles, and at least one of the remaining two specimens was substantially unaffected by freezing and thawing.
[f]Average of 2 specimens which failed. In each instance one specimen had not failed at 1500 cycles. Actual averages would exceed 1000 cycles in all instances.

TABLE 10

AVERAGE RESULTS OF LABORATORY FREEZING AND THAWING TESTS, SERIES 171[a]

Coarse Agg. No.	Durability Factor at 300 Cyc. (50% E)	Cycles To Reduce E		Expansion at 50% E (%)[b]
		50 Percent	20 Percent	
(a) 14-Day Moist Curing—Vacuum Saturated Aggregate				
1	0.1	0.7	0.3	0.119
3	8	48	23	0.073
5	15	90	43	0.090
4	50	340	177	0.072
2	93	1995	1318	0.146
(b) 7-Day Moist, 4-Day Dry, 3-Day Moist—Vacuum Saturated Aggregate				
1	6	33	21	0.046
3	96	941	60	0.072
5	102	1253	958	0.118
4	102	+2700[c]	+2700[c]	—[c]
2	105	+2700[c]	+2700[c]	—[c]

[a] Coarse aggregate graded 1 in. to No. 4 and saturated under vacuum before incorporation in concrete. Concretes designed to have a slump of 3 to 4 in., an air content of 4 to 5 percent and a cement factor of 5.5 sacks/cu yd. Two 3 x 4 x 16-in. beams for laboratory tests and one beam and one slab for field exposure were made from each batch. Each value is an average of specimen from five batches mixed on different days. Both laboratory beams were cured 14 days before exposure—one continuously moist and the other moist for 7 days, dried in 100 F air for 4 days and moist for the remaining 3 days. Specimens were frozen in air at 0 F and thawed in water at 40 F in accordance with ASTM Method C 291.

[b] Expansion of specimens during freezing and thawing measured to the nearest 0.0001 in. over two 10-in. gage lengths on each specimen.

[c] Specimens substantially unaffected by 2700 cycles of freezing and thawing when tests were terminated.

SUMMARY

If coarse aggregates are vulnerable to freezing and thawing in a highly saturated condition, the durability of the concrete will be improved greatly by providing an opportunity for the aggregate particles to dry out. This drying can be accomplished either before the aggregate is incorporated in concrete or by insuring that the concrete will be afforded an opportunity to dry in the field exposure before freezing occurs.

Almost any conventionally used sand will be protected from laboratory freezing and thawing damage by an adequate air void system even under conditions conducive to maintenance of a high degree of saturation.

When a wide variety of different coarse aggregates was incorporated in concrete in a highly saturated condition and the concretes were not allowed to dry out before exposure to ASTM C 291 laboratory freezing and thawing, they failed quite rapidly. Under these conditions, the resistance to freezing and thawing: (a) did not correlate with results of the usual soundness or abrasion tests; and (b) was moderately well correlated with the absorption of the aggregate, the percentage of lighweight particles, the percentage of chert, the percentage of lightweight chert, and percentage of deleterious particles. Based on the results of the outdoor exposure tests where only one aggregate has failed in 6 winters, the ASTM C 291 test as made in Series 161 is an extremely severe test.

In the laboratory freezing and thawing test, relative performance appears to be directly proportional to the logarithm of the number of cycles of exposure rather than the number of cycles per se.

Small percentates of chert, deleterious particles, and lightweight particles had an inordinately large effect on the performance of the 3 by 4 by 16-in. beams exposed to laboratory freezing and thawing. The laboratory test may greatly overemphasize the importance of these small percentages of violently expansive particles that are of the type that produce popouts in larger structural elements. The principal difficulty may be in the large ratio of surface area to volume of the 3 by 4 by 16-in. beams and the relatively high rates of freezing in the accelerated test of small specimens. Deleterious aggregate constituents can cause either popouts or overall disintegration of a major portion of the structure. Overall disintegration is far more serious. Moderately large numbers of popouts do not lead to overall disintegration and in most instances, they are detrimental only to appearance.

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Appendix

TEST PROCEDURES

Series 161. Tests of 56 Coarse Aggregates and 36 Fine Aggregates

All concretes were designed to contain $5\frac{1}{2}$ sacks of cement per cubic yard, 4 to 5 percent entrained air and to have a slump of 3 to 4 in. The 56 coarse aggregates were separated into individual sizes and recombined for incorporation in concrete to consist of 25 percent each of 1 to $\frac{3}{4}$ in., $\frac{3}{4}$ to $\frac{1}{2}$ in., $\frac{1}{2}$ to $\frac{3}{8}$ in., and $\frac{3}{8}$ in. to No. 4. All coarse aggregates were used with a single high-quality siliceous sand. All 36 sands were used graded as received with a single high-quality quartz coarse aggregate. The test aggregates were saturated under vacuum to vacuum to produce a high degree of saturation before incorporation in concrete. Concretes were mixed by hand in $\frac{1}{4}$-cu ft batches. Two 3 by 4 by 16-in. beams were molded from each batch and cured continuously moist for 14 days before exposure. A minimum of three batches was mixed on different days for each aggregate combination. Freezing was in air at 0 F and thawing in water at 40 F in accordance with ASTM Method C 291. One specimen from each batch was exposed in the normal, unwrapped condition and one was wrapped in aluminum foil throughout the exposure. If the specimen developed no evidence of deterioration after 300 cycles, exposure was terminated. The relative dynamic modulus of elasticity was the principal index of specimen condition, and exposure was terminated when relative modulus had been reduced 50 percent. Because of limitations on available freezing and thawing equipment, the concretes were mixed over a period of about $1\frac{1}{2}$ years.

In Series 161, the coarse aggregates were subjected to a number of the usual physical tests including: specific gravity, absorption, dry-rodded unit weight, void content, abrasion, soundness, soft particles, lithological, and heavy liquid separation. The sands were tested for specific gravity, absorption, grading, organic impurities, lithological composition, and soundness. Five-cycle sodium sulfate soundness, 5-cycle magnesium sulfate soundness and 10-cycle unconfined freezing and thawing tests of unconfined aggregate in a 10 percent brine solution were made on all aggregates.

Series 156. Tests of 16 Coarse Aggregates

Sixteen $1\frac{1}{2}$-in. maximum size coarse aggregates were incorporated in 5, 6, and 7-sack concretes designed to have an air content from 4 to 5 percent and a slump from 2 to 3 in. The same high-quality natural siliceous sand was used in all concrete. The coarse aggregate was soaked 24 hours before incorporation in concrete, producing a markedly less severe condition than that in Series 161 where the aggregate was saturated under vacuum. In these tests, concretes were machine mixed in 1.1-cu ft batches. One 3 by 4 by 16-in. beam was molded from each batch and cured continuously moist for 14 days prior to exposure to freezing and thawing in accordance with ASTM Method C 291. Three specimens were tested, each from a different batch.

Series 171. Laboratory and Outdoor Exposure Tests

Coarse aggregates from five of the sources tested in Series 161 were used. The coarse aggregates were graded to 1-in. maximum size and incorporated in concrete in a vacuum saturated condition as before. The concretes were designed to contain 5½ sacks of cement per cubic yard, 4 to 5 percent air and 3 to 4 in. of slump. Three 3 by 4 by 16-in. beams and one 14 by 14 by 6-in. slab were molded from each batch of machine mixed concrete. Five batches were mixed for each condition. The slab and one beam were cured continuously moist for 14 days and placed in the outdoor exposure plot. The beam was buried to one-half of its 3-in. depth and the slab to one-half of its 6-in. depth.

The two remaining beams were for laboratory freezing and thawing tests. One of these was cured continuously moist for 14 days before exposure to laboratory freezing and thawing in accordance with Method C 291. The other beam was cured 7 days moist, dried 4 days in 100 F air at 20 to 30 percent RH and resoaked in water for 3 days before exposure.

Length change was measured to the nearest 0.0001 in. on two 10-in. gage lines on each beam and four gage lines on each slab. Relative dynamic modulus of elasticity was the principal measure of the condition of specimens exposed to laboratory freezing and thawing. Laboratory exposure was continued until the relative E was reduced 50 percent or the specimen had been subjected to 2,700 cycles of freezing and thawing. Beams in the outdoor exposure plot were returned to the laboratory twice yearly and immersed in 40 F water overnight before determination of length and dynamic modulus. Dynamic modulus of slabs was not determined. Length measurements of slabs were corrected to a constant temperature based on thermal coefficients of contraction determined at an age of 14 days.

Factors Affecting Durability of Concrete Surfaces

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A laboratory research program on abrasion resistance of concrete surfaces, including the development of an impact-type abrasion test for testing of cored specimens, is described. Variables incorporated in this study were slump, finishing, curing, surface treatments (linseed oil and monomolecular film), and the use of admixtures. Sand with a low mortar strength was substituted for the laboratory stock sand in two tests. In addition to the abrasion tests, hardened concrete specimens were examined for air content and void distribution by the linear traverse method. A brief summary of reports by others on the causes and prevention of concrete wearing surface deterioration is also presented.

The abrasion samples were cored from laboratory slabs constructed to simulate some of the field conditions and other factors encountered in placing, finishing, and curing of concrete bridge decks and pavement. Although the data presented are limited in scope, relative values of "abrasion losses" are established and are indicative of what might be expected in the field.

Test results show that slump, curing, and time of finishing are the most important factors affecting the abrasion resistance of concrete surfaces. Concrete receiving an application of a monomolecular film to retard evaporation during the finishing period showed an increase in abrasion resistance. A two-coat surface treatment of linseed oil, applied when the concrete had partially dried, was found to increase abrasion resistance appreciably regardless of other variables.

•IN recent years, the deterioration of concrete wearing surfaces has become a major problem. This deterioration takes many forms, among which are scaling, raveling, abrasion, spalling, pitting and cracking. Highway Research Board Bulletin 323, 1962 (1), provides a comprehensive analysis of the problem and suggests preventive measures that may be taken both during and after construction. This Bulletin deals predominantly with the effects of deicing chemicals on concrete structures. The use of these chemicals is considered to have contributed to the observed increase in scaling and possibly other concrete defects as well.

Although air entrainment has been shown (1, 2) to provide protection against damage from deicing agents as well as from freezing and thawing, poor construction practices can negate much or all of this protection. Oleson (1, pp. 16-18) has cited an instance where the air content of concrete cores varied considerably between the surface and other height levels of the core. In this case, two cores were taken from adjacent lanes of a concrete pavement; one from a lightly scaled area, the other from an undamaged area. Both cores had a total air content of over 5 percent as determined by the linear traverse method. However, a determination of top surface air content revealed 3.7 percent air in the core from the scaled area, whereas the core from the unscaled area

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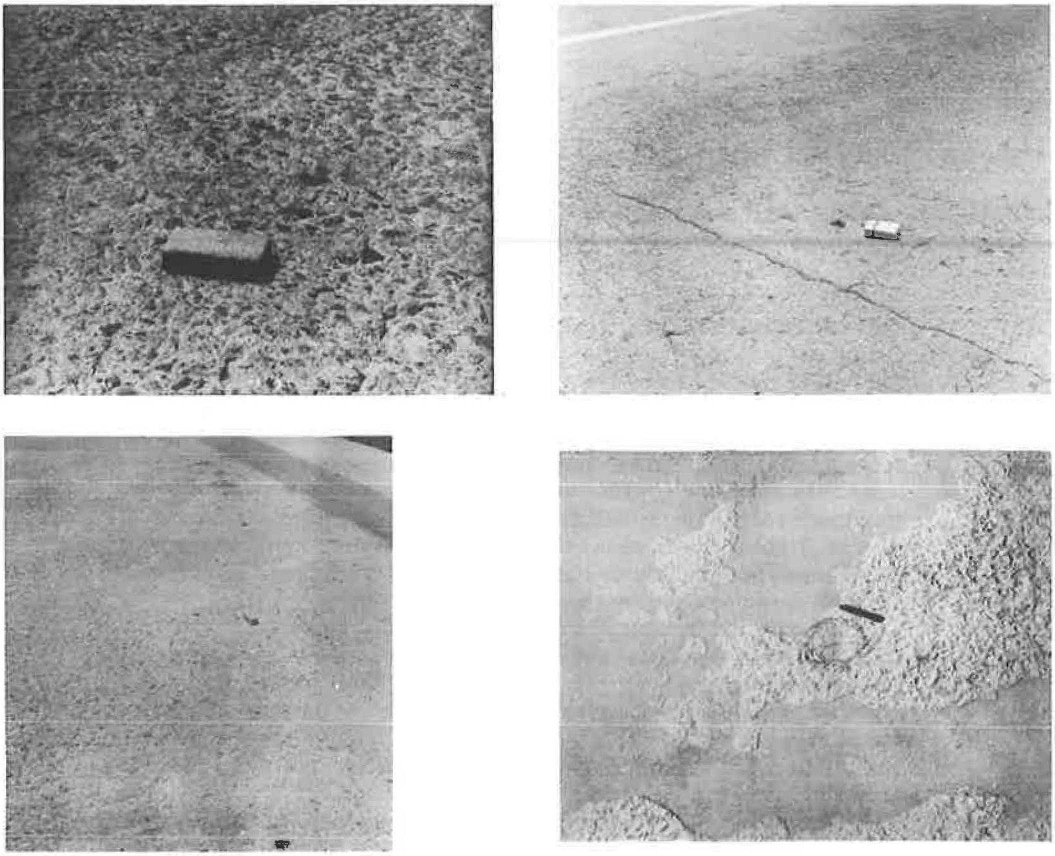

Figure 1. Bridge deck deterioration.

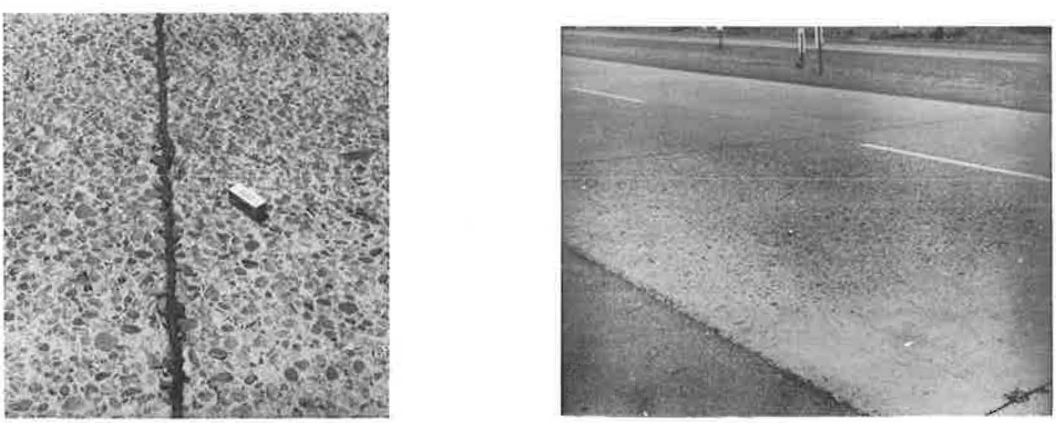

Figure 2. Concrete pavement surface deterioration.

had 7.6 percent air at the surface. Based on observations and photographs made during construction of the pavement, the low surface air content of the scaled area was attributed to the excessive use of water sprinkled on the surface as an aid in finishing.

Excessive floating and overvibration can create a layer of surface mortar which will be weak and subject to early scaling and abrasion. These practices may also drive out entrained air making the surface susceptible to damage from deicing salts and freezing and thawing. Rigid adherence to the specifications through proper inspection and control testing can eliminate much of the danger of concrete deterioration due to these and other poor construction practices.

There remains some doubt that improvements in concrete technology and construction methods will ever completely solve the problem of concrete deterioration. Mitchell (3) is of the opinion that water is the common denominator of all generally recognized types of concrete deterioration, and a high degree of imperviousness is necessary to achieve permanent durability. Since concrete is essentially a porous product, he believes that surface treatments are necessary to insure adequate imperviousness.

Numerous materials have been investigated for use in sealing concrete surfaces. Among these are bituminous products, silicones, latexes, epoxy resins, vegetable oils, and special curing compounds. The primary objective in most of the investigations of surface treatments was to determine the effectiveness of the product in the prevention of surface scaling caused by deicing chemicals.

Linzell (1, pp. 95-96) has reported that the Ohio Department of Highways experimented with surface treatments in 1941. A mixture of linseed oil and kerosene was applied to five 1-mile sections of non-air-entrained concrete pavement. Remarkably improved resistance to scaling was claimed after repeated observations during subsequent years. Details concerning the mixture and the rate of application are not reported.

In a study by Smith of the Ontario Department of Highways (1, pp. 72-94) various surface treatments were evaluated to determine their effect on freeze-thaw durability and scaling resistance of concrete. The study consisted of the examination of existing treated structures and pavements, controlled field trials, and laboratory tests. Some phases of the investigation were not complete at the time this report was made and only limited conclusions could be drawn. Both laboratory and field tests indicated that silicones did not give lasting protection to concrete. Laboratory and field results with a 50:50 mixture of linseed oil and kerosene were contradictory. Although laboratory tests indicated that ultimate durability was probably not improved, field tests showed that two coats of the linseed oil-kerosene mixture offered effective protection, at least during the first winter. Since 1959, all new concrete pavements in Ontario have been treated with two coats of the linseed oil mixture and the general conclusion is that this is an effective and economical method of protecting concrete against freeze-thaw damage and scaling.

Wisconsin, as reported by Aten (1, pp. 15-16), has used silicones as a preservative treatment during construction, but field observations indicate that such treatments are not particularly effective. They are now experimenting with other types of surface treatments such as linseed oil, rubberized asphalt seal coats, and epoxies.

Faul and McElherne (1, pp. 19-22) wrote that in Iowa pavements and bridge decks placed after October 15, 1961, and which were to be subjected to deicers, were treated with two coats of a linseed oil emulsion. Michigan, according to Finney (1, pp. 26-42), has experimented with various water repellants and surface penetrants. Laboratory tests indicate two coats of linseed oil to be as good or better in preventing surface scale than any of the proprietary products studied.

In controlled laboratory tests performed by the Bureau of Public Roads (4), the application of two coats of linseed oil was found to be beneficial in preventing or delaying scaling caused by the use of deicing chemicals. The improvement in scaling resistance was minor for air-entrained concrete, but very pronounced for non-air-entrained concrete. The use of boiled linseed oil resulted in equal or slightly better resistance to scaling than raw linseed oil. The BPR Office of Research and Development now recommends the use of linseed oil applications where deicing chemicals are used.

The Texas Transportation Institute (5) has also conducted a study to determine the effectiveness of various waterproofing compounds. Eighteen products were evaluated by wetting and drying tests and freeze-thaw tests on non-air-entrained concrete. Results indicate that a two-coat treatment with linseed oil was the most effective of the products tested in reducing movement of water through concrete and in protecting against freeze-thaw damage.

California has also experienced problems with concrete surface deterioration in some mountainous areas. Some loss of surface has occurred in both pavement and bridge decks. Air entrainment was specified for all concrete in these freeze-thaw areas, but such defects as scaling and raveling have appeared, suggesting the possibility that either not enough air was entrained, or that surface finishing removed air needed at the surface for frost protection. Deterioration of bridge decks was so extensive that in 1964, a contract was let to resurface 34 decks with an epoxy-type overlay to retard further deterioration. The average age of these bridges was about five years.

The problem of concrete distress is not limited to mountainous regions where it is thought that deicing chemicals have contributed to the rapid deterioration of concrete surfaces. A highway near Los Angeles shows scaling and abrasion damage, although it is not subjected to freezing temperatures. Whether or not common factors exist which contribute to surface deterioration in these completely different environments has not yet been determined.

Kennedy and Prior (6) summarize much of the work done on abrasion resistance with an emphasis on the methods of testing. They point out that cement factor, air content, and curing have all been shown to be important factors. They concluded that since these factors are related to compressive strength, it is reasonable to accept strength as a criterion of wear resistance. Witte and Backstrom (7) also state that compressive strength is the most important factor controlling the abrasion resistance of concrete, abrasion resistance increasing as compressive strength increases. If compressive strength could be accepted as the sole criterion for abrasion resistance, the solution to the problem would be relatively simple: merely specify high strength concrete wherever surfaces are subject to abrasion. However, abrasion has also been noted on surfaces of concrete with relatively high compressive strength (above 4,000 psi). It appears, therefore, that other factors must be considered as affecting abrasion resistance, and compressive strength alone cannot be used as a criterion for durable concrete.

Concrete surfaces with poor durability have been a problem on some highways in California. In an effort to find means of improving surface durability, particularly of wearing surfaces, a research project was initiated by the Materials and Research Department. It was hoped that a test could be developed to measure the surface quality of concrete and to correlate this test with concrete performance in the field. Since this goal has not been fully realized, further work is planned to correlate the findings reported here with field performance.

TEST METHOD DEVELOPMENT

From literature research, it was found that various test methods have been used by investigators to determine resistance of concrete to surface wear. Among these are the dressing-wheel, revolving disk, shot-blast, rolling steel balls under pressure, and a modified Los Angeles rattler. These methods have met with varying degrees of success.

Since it was felt that an impact-type test might be more satisfactory for evaluating abrasion resistance, a test method of this type was devised. This was done by modifying an existing test which had been developed for determining the surface loss of a compacted bituminous mixture (8) when subjected to impact abrasion. The complete procedure, as adopted and used in this project, is described and illustrated at the end of this report. Briefly, the test consists of placing a 4-in. diameter by 2-in. high specimen in a 5-in. high container, adding steel balls and water, and shaking for three minutes on a mechanical shaker.

In preliminary tests, specimens were fabricated individually in gang molds. However, with this method, it is difficult to duplicate fabrication and finishing procedures. Another method involved casting concrete slabs 2 in. thick from which a number of cores could be taken. This practice did produce more consistent results, but coarse

TABLE 1

AGGREGATE GRADING[a]

Sieve Size	Percent Passing
3/4-In.	100
3/8-In.	65
No. 4	44
No. 8	35
No. 16	26
No. 30	17
No. 50	8

[a]Sand was reduced 2 percent for air-entrained mixes.

aggregate near the surface could still cause a nonuniform finish. To improve on the uniformity of the concrete surfaces and to simulate field conditions, the block thickness was increased to 5 in. Although this necessitated sawing each core to the 2-in. test height, it is believed that more realistic results were obtained. The appearance of the concrete surfaces after the impact abrasion test is very similar to the abraded concrete surfaces observed in the field.

TEST PROGRAM

A test program was designed to provide a comparison of the effects of various simulated field conditions, the use of certain admixtures, and the application of surface treatments on abrasion resistance. The test program included the following variables: slump, finishing, curing, admixtures consisting of one air-entraining agent and two water-reducing retarders, a fine aggregate which had failed the mortar strength test, and surface treatments with linseed oil. Blocks were also treated with a monomolecular film which is intended to retard water evaporation during the finishing period. Twenty-six 14 x 21 x 5-in. blocks were fabricated.

Figure 3. Vibration of test block.

Figure 4. Floating process.

Figure 5. Fog spray being applied to concrete during delayed finishing process.

Figure 6. Finished block showing broomed surface.

TABLE 2

FINISHING PROCEDURES

Time After Vibration (min)	Early Finish	Delayed Finish
15	Strike off 1/8 in. high (1/8-in. high wood strips placed on top of forms, then removed after strike-off leaving excess material to be removed during subsequent finishing) with 2 passes of float; alternate direction of each pass.	Same as early
30	Float in 4 passes, removing approximately one-half of excess mortar.	Spray with water, float in 4 passes removing approximately one-third of remaining excess mortar.
45	Float in 4 passes, removing excess mortar down to top of form; broom the surface.	Spray with water, float in 4 passes removing one-half of the remaining excess mortar.
180	No further treatment.	Spray with water, float in 4 passes removing excess mortar down to the top of form; broom the surface.

Since mixing had to be spread over an 8-day period, the variables to be mixed each day were selected by a statistically random method.

Fabrication

Test concrete was made using 3/4-in. maximum size aggregates from the American River near Sacramento (Table 1). An exception was made in two of the mixes where a lower quality sand was substituted for the American River sand. Cement was Type II modified at 6 sacks/cu yd. Mixing was done in an open tub-type mixer following a standard laboratory procedure. Slump, unit weight, and air content was determined and the concrete was then placed in 14 x 21 x 5-in. wooden molds. A 1 1/4-in. stinger-type vibrator was used for compaction by quickly inserting the tip of the vibrator into the concrete and removing it just slowly enough not to leave a void (Fig. 3). The same pattern of 20 vibrator strokes was used on each test slab.

Finishing

In an effort to simulate bridge deck finishing, the procedures in Table 2 were followed for the two conditions indicated (also see Figs. 4, 5 and 6).

Curing

For those blocks receiving a standard cure, wet mats were applied as soon as the surface sheen had left the concrete. Forms were removed the following day and the concrete blocks placed in the moist curing room. When the concrete was 7 days old, the test blocks were removed from the moist curing room and stored in laboratory air which is maintained at approximately 73 F and 50 percent relative humidity (Fig. 7).

Three of the test blocks received a freezing treatment intended to simulate unexpected freezing conditions which might occur during construction (Fig. 8). After approximately 24 hours of moist curing, the block was placed in a freezer and exposed to air at 15 F for 8 hours a day on 3 consecutive days. Blocks were covered with wet mats when not in the freezer. Thermocouples placed in the concrete indicated minimum

Figure 7. Storage in laboratory air.

Figure 8. Blocks being frozen during early curing period.

Figure 9. Test blocks in 100 F oven.

temperatures of 24 F at a 1-in. depth and 27 F at 2 in. At 4 days of age, the concrete blocks were stripped from the molds and placed in the moist curing room. They remained there for three days before being removed and stored in laboratory air.

To simulate a lack of proper curing during the first three days following placement, 8 test blocks were left uncovered for the first 24 hours, then placed in a 100 F oven for eight hours a day on three consecutive days (Fig. 9). Blocks were stored in laboratory air when not in the oven. After the "drying" period, blocks were stripped from the molds and placed in the moist curing room where they remained for three days before being removed and stored in laboratory air.

Monomolecular Film Surface Treatment

A series of blocks were sprayed immediately after strike-off with a solution to produce a monomolecular film which retards evaporation of the water from the fresh concrete surface. The material was supplied in a concentrated form, but was diluted with water according to the manufacturer's directions to an 80:1 solution before applying. Rate of application was 1-gal solution to 250 sq ft of concrete surface.

Linseed Oil Surface Treatment

A number of blocks received a two-coat treatment of linseed oil when the concrete was 14 days old. For the first coat, the linseed oil was diluted 1:1 with turpentine and the solution applied at the rate of 1 gal per 360 sq ft. The second coat was applied undiluted at 1 gal per 600 sq ft.

Admixtures

The air-entraining agent used was a Vinsol resin product. Two water-reducing retarders were used; a hydroxylated carboxylic acid type at 3-fl oz per sack of cement, and lignosulfonate type at 0. 25-lb per sack. No adjustments in aggregate proportions were made when the retarders were used, but sand was reduced 2 percent for the air-entrained mixes.

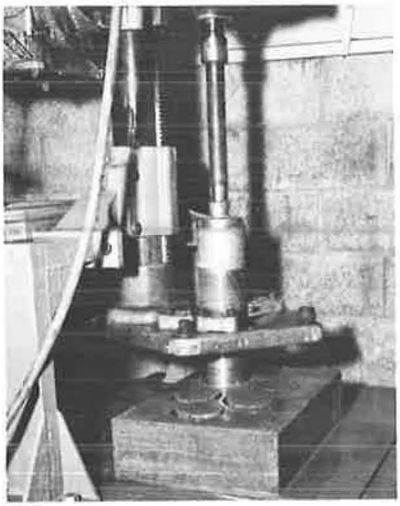

Figure 10. Coring operation.

Low Quality Sand

In making two of the blocks, stock sand was replaced by sand of lower quality. The lower quality material had a relative mortar strength of about 80 percent as determined by Test Method No. Calif. 515 (a modification of ASTM C 109), compared to 95 percent for the stock sand.

Test Specimen Preparation

Test blocks were cored when the concrete was 17 to 19 days old (Fig. 10). Five 4-in. diameter cores were taken from each block for the abrasion test. An additional core was taken from selected blocks for determining void characteristics by the linear traverse method.

Since the abrasion specimens were to be 2 in. high, the bottom 3 in. of each core was cut off and discarded (Fig. 11). The cores for void determination were sliced horizontally so that sections at the top surface, ½ in. below the top surface, and in two cases, the center of the core, could be examined individually.

Abrasion test specimens were placed in water to soak 24 hours before testing, in accordance with the adopted test procedure. At 21 days of age, specimens were tested (Fig. 12) according to the method described in the Appendix.

DISCUSSION OF FINDINGS

The results of losses on all five specimens from one slab were averaged to produce one test result. These results were compared to a control slab constructed to simulate our standard field practice on bridge decks. Individual variables were then isolated by averaging abrasion losses for all combinations having common factors for the two conditions of the variable selected for study. Thus, the effect of slump, for example, was compared in several different mixes involving different cures, finishing techniques, admixtures, and surface treatment.

Since the isolated effect of each variable was generally the same regardless of other variables involved, we believe the conclusions drawn relative to the effect of the factors selected for study are valid. Further study is needed before we can use abrasion loss as determined by this test method, as a quantitative measure of concrete surface durability.

Table 3 gives fresh concrete properties, test variables, abrasion losses on the hardened concrete at 21 days of age, and the relative abrasion losses. The effects of each variable on abrasion loss are shown in Table 4.

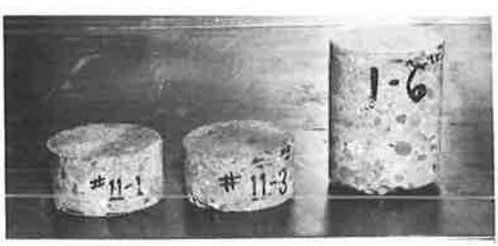

Figure 11. Specimens before and after sawing to 2-in. test height.

Effects of Slump

Four of the five comparisons between slumps of 3 in. and 6 in. show that this increase in slump causes an increase in abrasion loss, the average increase being about 15 percent (Table 4a).

Effects of Finishing

Results of tests on specimens which had received a delayed finish show that the average abrasion loss was decreased about

TABLE 3

ABRASION LOSS, TEST VARIABLES AND FRESH CONCRETE PROPERTIES

Slab No.	Abrasion Test Results								Fresh Concrete Properties			
	Nominal Slump (in.)	Finish[a]	Cure[b]	Admixture[c]	Surface Treatment[d]	Wt. Loss Range (gm)	Avg. Wt. Loss (gm)	% of Std. (No. 1)	Slump (in.)	Air (%)	Unit Wt. (pcf)	W/C (lb/sk)
1	3	E	Std.	None	None	25-27	26.2	100	3.0	1.4	154.2	47.5
2	3	D	Std.	None	None	16-24	20.4	78	2.5	1.4	154.0	47.2
3	6	D	Std.	None	None	20-27	23.0	88	5.25	0.8	153.8	51.5
4	3	D	Std.	AE	None	18-24	20.2	77	2.75	4.9	145.8	43.8
5	6	D	Std.	AE	None	22-26	24.0	92	5.5	6.0	144.1	47.8
6	3	D	Adv.	None	None	22-27	24.0	92	2.5	1.4	153.8	46.8
7	6	D	Adv.	None	None	25-36	30.0	115	5.5	1.3	152.2	52.2
8	6	E	Std.	None	None	23-27	25.4	97	6.0	1.2	152.7	52.2
9	3	E	Adv.	None	None	30-42	37.6	144	2.75	1.2	153.6	47.2
10	3	D	FT	None	None	20-25	22.4	85	2.5	1.4	153.8	46.8
11	3	E	Std.	None	LO	17-22	18.6	71	2.5	1.4	154.2	46.1
12	3	D	Std.	None	LO	12-17	14.4	55	2.5	1.4	154.0	47.2
13	6	D	Std.	None	LO	14-20	18.6	71	5.25	0.8	153.8	51.5
14	6	D	Adv.	None	LO	15-22	17.6	67	5.5	1.3	152.2	52.2
15	3	E	FT	None	None	23-30	26.2	100	3.0	1.3	152.2	48.1
16	3	E	FT	AE	None	25-30	27.8	106	2.5	5.2	147.9	44.1
17	3	E	Std.	None	MM	18-22	20.2	77	2.5	1.2	153.3	47.2
18	3	E	Adv.	None	MM	26-36	31.2	119	3.0	1.2	153.8	47.5
19[e]	3	E	Std.	None	None	24-29	26.2	100	2.75	1.2	150.4	56.7
20[e]	3	E	Std.	None	LO	13-18	16.4	63	2.75	1.2	150.4	56.7
21	3	E	Std.	HC	None	22-25	23.8	91	3.0	1.2	154.0	46.9
22	3	E	Std.	LS	None	23-29	25.6	98	2.5	2.4	152.3	44.5
23	3	D	Adv.	HC	None	26-31	28.8	110	3.25	1.2	152.5	47.7
24	3	D	Adv.	LS	None	20-27	25.0	95	3.0	2.3	154.2	44.7
25	3	D	Std.	None	MM	20-25	22.8	87	3.0	1.3	152.2	48.1
26	3	E	Adv.	None	LO	23-34	28.8	110	2.75	1.2	153.6	47.2

[a] E = early; D = delayed.
[b] Std. = standard; Adv. = adverse; FT = freeze-thaw.
[c] AE = air entrainment; HC = hydroxylated carboxylic acid type; LS = lignosulfonate type.
[d] LO = linseed oil; MM = monomolecular film.
[e] Low quality sand.

TABLE 4

EFFECTS OF VARIABLES ON ABRASION LOSS[a]

Nominal Slump (in.)	Finish	Cure	Admix.	Surface Treatment	Slab No.	Abrasion Loss	Slab No.	Abrasion Loss
				(a) Slump				
					3-In. Slump		6-In. Slump	
—	D	Std.	None	None	2	20.4	3	23.0
—	D	Std.	AE	None	4	20.2	5	24.0
—	D	Adv.	None	None	6	24.0	7	30.0
—	E	Std.	None	None	1	26.2	8	25.4
—	D	Std.	None	LO	12	14.4	13	18.6
Avg. abrasion loss						21.0		24.2
				(b) Finish				
					Early		Delayed	
3	—	Std.	None	None	1	26.2	2	20.4
6	—	Std.	None	None	8	25.4	3	23.0
3	—	Adv.	None	None	9	37.6	6	24.0
3	—	Std.	None	LO	11	18.6	12	14.4
3	—	FT	None	None	15	26.2	10	22.4
3	—	Std.	None	MM	17	20.2	25	22.8
Avg. abrasion loss						25.7		21.2
				(c) Standard vs Adverse Cure				
					Standard		Adverse	
3	E	—	None	None	1	26.2	9	37.6
3	D	—	None	None	2	20.4	6	24.0
6	D	—	None	None	3	23.0	7	30.0
3	E	—	None	LO	11	18.6	26	28.8
6	D	—	None	LO	13	18.6	14	17.6
Avg. abrasion loss						21.2		28.2
				(d) Freezing During Curing Period				
					Standard		Freeze-Thaw	
3	D	—	None	None	2	20.4	10	22.4
3	E	—	None	None	1	26.2	15	26.2
3	E	—	AE	None	—	—	16	27.8[b]
Avg. abrasion loss						23.3		24.3
				(e) Admixtures				
					No Admixture		Admixture	
3	D	Std.	AE[c]	None	2	20.4	4	20.2
6	D	Std.	AE[c]	None	3	23.0	5	24.0
3	E	FT	AE[c]	None	15	26.2	16	27.8
Avg. abrasion loss						23.2		24.0
3	E	Std.	HC[c]	None	1	26.2	21	23.8
3	D	Adv.	HC[c]	None	6	24.0	23	28.8
Avg. abrasion loss						25.1		26.3
3	E	Std.	LS[c]	None	1	26.2	22	25.6
3	D	Adv.	LS[c]	None	6	24.0	24	25.0
Avg. abrasion loss						25.1		25.3

TABLE 4 (Continued)

EFFECTS OF VARIABLES ON ABRASION LOSS[a]

Nominal Slump (in.)	Finish	Cure	Admix.	Surface Treatment	Slab No.	Abrasion Loss	Slab No.	Abrasion Loss
				(f) Linseed Oil Treatment				
					No Treatment		Linseed Oil	
3	E	Std.	None	—	1	26.2	11	18.6
3	E	Adv.	None	—	9	37.6	26	28.8
3	D	Std.	None	—	2	20.4	12	14.4
6	D	Std.	None	—	3	23.0	13	18.6
6	D	Adv.	None	—	7	30.0	14	17.6
3	E	Std.	None	—	19[d]	26.2	20[d]	16.4
Avg. abrasion loss						27.2		19.1
				(g) Monomolecular Film Treatment				
					No Treatment		MM Film	
3	E	Std.	None	—	1	26.2	17	20.2
3	E	Adv.	None	—	9	37.6	18	31.2
3	D	Std.	None	—	2	20.4	25	22.8
Avg. abrasion loss						28.1		24.7
				(h) Low Quality Sand				
					Stock Sand		Low Quality	
3	E	Std.	None	None	1	26.2	19	26.2
3	E	Std.	None	LO	11	18.6	20	16.4
Avg. abrasion loss						22.4		21.3

[a]See footnotes to Table 3 for definitions of abbreviations.
[b]Not included in average.
[c]Admixture type used in comparison.
[d]Concrete contained low quality sand.

20 percent over similar blocks which received an early finish. This was contrary to expectations as it was thought that the addition of water to the surface before each floating would weaken the concrete and that the delayed floating would prevent the forming of a dense surface. It is possible that the added water was entirely removed, together with some or all of the surface mortar during the finishing process, and thus had no detrimental effect (Table 4b).

This finding agrees with a report by Klieger (9) that to improve scaling resistance (and surface durability), it appears desirable to delay final finish as long as possible. Michigan has adopted a procedure of this type for certain portions of bridge construction

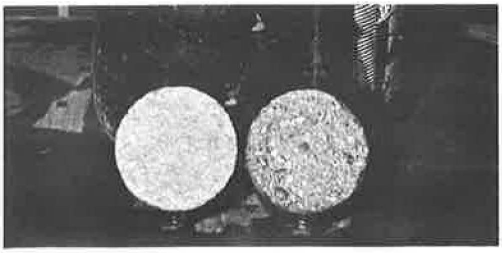

Figure 12. Untested (left) and tested (right) specimens.

according to Finney (1, pp. 26-42). Deep structural members are built up slightly higher than the finished dimension and struck off to proper elevation when bleeding has stopped. This is intended to eliminate a possible weak top layer of mortar formed with excessive water.

Effects of Curing

The lack of proper curing resulted in a significant increase in abrasion loss. This was also noted during preliminary testing when different curing procedures were followed. Although the adverse cure might have been more severe than would occur in actual practice, the results clearly indicate the need for proper curing (Table 4c).

Effects of Freezing and Thawing During Curing Period

The interruption of the curing cycle by intermittent freezing caused only a slight increase in abrasion loss (Table 4d).

Effects of Admixtures

Abrasion test results on concrete containing air-entraining agents and water-reducing retarders are given in Table 4e. The effect of these agents on abrasion loss is inconclusive, but does not appear to be significant.

Effects of Linseed Oil Treatment

The application of two coats of linseed oil increased the abrasion resistance in every case, the average increase being about 30 percent. This supports the findings of others that linseed oil improves the durability of concrete surfaces. It is evident that linseed oil strengthens the surface and deters the entry of moisture. The beneficial effects of the linseed oil treatment would be expected to be more pronounced for poorly cured or porous concrete surfaces than for high-density, properly cured concrete (Table 4f).

Effects of Monomolecular Film

The results of tests on concrete which had been treated with a monomolecular film are given in Table 4g. Specimens that received the treatment and an early finish showed an increase in abrasion resistance of approximately 20 percent when compared to untreated specimens. For some undetermined reason, the treated specimens with delayed finish did not show the increase. Possibly the additional manipulation destroyed the continuity of the film.

Effects of Low Quality Sand

The substitution of sand of low relative mortar strength (80%) for stock sand (95%) did not appear to have any significant effect on abrasion loss. However, it is probable that insufficient tests were made to provide conclusive data (Table 4h).

Cumulative Effects of Variables

Figure 13 shows cumulative effects of four variables selected from this study as a hypothetical case. The magnitude of the ordinates are plotted on a relative scale based on our findings. We began with a 6-in. slump, adverse curing, and standard finishing without any surface treatment. The first drop in abrasion loss of about 15 percent was obtained by decreasing the slump from 6 in. to 3 in. The most important factor of all under the test conditions encountered was curing which reduced abrasion losses by 33 percent. This does not necessarily mean that curing is more important than water-cement ratio or slump, as the test conditions for curing may have represented a broader range of field conditions than those for slump. Delaying the final finish decreased the loss about 18 percent. Finally, Figure 13 shows that the application of linseed oil further decreases abrasion losses. The relative effect of linseed oil would be expected to be greater when applied to poor quality mortar than when applied to a high-quality surface.

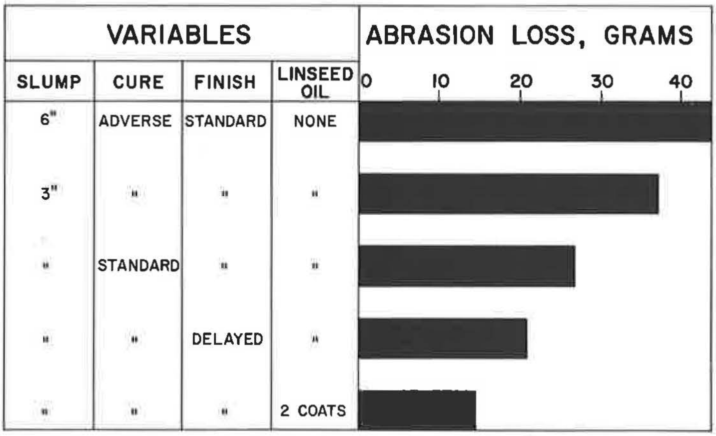

Figure 13. Cumulative effects of variables (hypothetical).

Linear Traverse Data

Data obtained by linear traverse examination of core sections taken from different depths below the finished surface are given in Table 5. The top surface was ground and polished just enough to produce a plane surface. For each of the variables so examined, the air content at the surface was calculated to be considerably lower than that determined on the fresh concrete. Even at ½ in. below the surface, the air content was significantly lower. Air content determined at the center of cores from air-entrained concrete checked closely with that measured in the fresh concrete. It appears that vibrating the concrete drove off part of the air, especially from the surface. It is possible that this action contributes more to decreasing the durability of concrete surfaces (exposed to freezing and thawing) than has previously been realized. In certain instances, the top surface of air-entrained concrete placed in the field has probably been left with insufficient entrained air for protection against freezing.

The spacing factor of the air void system has been considered by some (10, 11) to provide a criterion for determining satisfactory frost protection. Powers (10) has

TABLE 5

VOID DISTRIBUTION OF HARDENED CONCRETE CORES BY LINEAR TRAVERSE

Test Slab No.	Location of Bubble Count	Air Content (%)		Voids per Linear In.	Avg. Chord Intercept	Spacing Factor
		Fresh	Linear Traverse			
1	Surface	1.4	0.3	0.88	0.0039	0.0148[a]
1	½ In. below surf.		0.8	0.69	0.0112	0.0300
2	Surface	1.4	0.2	0.74	0.0025	0.0122[a]
2	½ In. below surf.		0.8	0.74	0.0109	0.0293
4	Surface	4.9	2.1	13.14	0.0016	0.0028[a]
4	½ In. below surf.		3.5	9.97	0.0035	0.0049
4	2½ In. below surf.		4.8	11.47	0.0042	0.0050
16	Surface	5.2	3.6	17.1	0.0021	0.0029[a]
16	½ In. below surf.		4.0	12.37	0.0032	0.0043
16	2½ In. below surf.		5.3	14.09	0.0038	0.0044
21	Surface	1.2	0.2	0.66	0.0033	0.0152[a]
21	½ In. below surf.		0.7	0.73	0.0102	0.0283
22	Surface	2.4	0.6	2.08	0.0028	0.0085[a]
22	½ In. below surf.		0.8	1.30	0.0064	0.0170

[a]Actual paste content at surface not determined, therefore calculated spacing factors are low.

stated that the spacing factor should not exceed 0.01 in. As indicated in Table 5, the air-entrained concrete had spacing factors well below this value at all three levels of the core. The surface sections contained the least amount of air but indicated the lowest spacing factors. However, since the percentage of paste content used in the calculations was that of the total mix, there is an inherent error in the values given for the surface spacing factors. The paste content at the surface is higher than that for the mass; therefore, the actual spacing factors would also be higher if arbitrary corrections were made.

Since the mortar concentration is greatest at the surface, it follows that the air content at that point should be considerably greater than that in the concrete to provide equivalent frost protection. Mortar is generally considered to need 9 to 10 percent air by volume for frost protection, this being about equivalent to 4½ to 5 percent air by volume in concrete. Whether the surfaces of the air-entrained concrete in this test program had sufficient air to provide protection against freezing damage is questionable.

GENERAL COMMENTS

The results of the test program do not fully explain why concrete surfaces deteriorate under traffic. However, since the effects of the variables which decrease abrasion resistance are shown to be cumulative, it follows that one poor construction practice may only slightly reduce the durability of the concrete surface, whereas a combination of two or more poor practices may seriously lower the surface's resistance to weathering and abrasion. Such combinations probably occur randomly in the field, which would explain why only certain portions of a concrete surface show abrasion loss and others appear to be satisfactory. Further research is needed to determine the degree of correlation that exists between our laboratory findings and concrete surface deterioration in the field. Variables that have been found to affect abrasion resistance could be introduced into concrete bridge decks under rigid control to provide some of the answers. The dissipation of entrained air from the surface is a problem that should be studied both in the laboratory and in the field.

Throughout this report, references have been made to various types of concrete surface deterioration. The descriptive terms used are indicative of the lack of concrete surface durability. If the quality of a concrete surface is improved and made more resistant to impact abrasion, surface durability as affected by most factors, would also be improved. The described test method provides a means of measuring this resistance and establishes a basis for evaluating the effects of various contributing factors.

SUMMARY

The data show that each of the factors of slump, finishing techniques, and curing procedures has an appreciable effect on the abrasion resistance of concrete surfaces as measured by an impact-type test. Of these three factors, the broadest range of abrasion losses encountered were those associated with curing procedures. Concrete surfaces cured with adequate moisture to provide for proper hydration of the cement will have a relatively higher resistance to abrasion.

The use of a monomolecular water retention agent during the finishing period had a measurable effect on improving the abrasion resistance of the surface. The use of admixtures in the concrete had no appreciable effect on abrasion resistance.

The findings of this laboratory test program corroborate the experience reported by others with respect to the impressive benefits, both preventive and remedial, that may be realized by the application of linseed oil treatments. Adhering to good construction practices will not, under all circumstances, provide a durable abrasion-resistant surface. Since it has been shown that the factors adversely affecting abrasion resistance are cumulative, it is conceivable that the problem may manifest itself in virtually any environment. Therefore, general use of linseed oil application as a preventive maintenance measure appears justified. This procedure would be especially valuable in areas where deicing agents are used. The treatment serves to seal the concrete surface, thereby deterring the entry of water, as well as creating a toughened wearing surface which greatly improves surface durability of concrete. Recently some researchers have reported that there is a screening action that deters the entry of deicing salts.

The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the Bureau of Public Roads (Report No. M&R 250908-1, BPR D-3-7, November 1965).

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Appendix

Test Method No. Calif. ______
(Proposed)

METHOD OF TEST FOR DETERMINING THE SURFACE ABRASION RESISTANCE OF CONCRETE SPECIMENS

Scope

The surface abrasion test measures the ability of a concrete specimen to resist surface abrasion by impact in the presence of water.

Procedure

A. Apparatus

1. A mechanical shaker capable of agitating a mold assembly containing the test specimen, water, and steel balls, in a vertical direction of 1200 cycles per minute with a 1-in. stroke. (Drawings are available from the Materials and Research Department.)

2. One steel test mold, 4 in. ID by 5 in. high, fitted with a watertight base and cover [Fig. 14]. Three set screws are tapped through and positioned evenly around the perimeter of the mold $1\frac{1}{2}$ in. from the bottom.

Figure 14. Container for abrasion test.

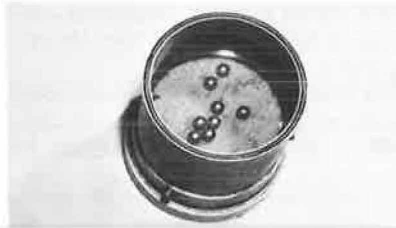

Figure 15. Specimen in place for abrasion test.

3. Eight steel ball bearings, $^{13}/_{32}$-in. diameter, weighing 4.5 grams each [Fig. 15].
4. One 200-ml graduated cylinder.
5. Balance sensitive to 1 gram.

B. Specimens

The test specimens shall be cylindrical in shape, 4 in. in diameter and 2 in. high, and may be either cores cut from hardened concrete or specimens molded from concrete. They shall be soaked in water for a minimum of 24 hours prior to testing.

C. Test Procedure

1. Surface dry the specimen, weigh, and record weight to the nearest gram.
2. Place specimen in the test mold with the surface to be tested facing up and secure the specimen in a level position by means of the set screws. Place the mold with specimen on the base and add 8 steel ball bearings and 200 ml of water. Attach the cover making sure rubber gaskets are in place and clamp the assembly to the mechanical shaker.
3. Agitate the assembly at 1200 ± 10 cpm for three minutes and remove from the mechanical shaker [Fig. 16].
4. Remove the specimen from the test mold. Flush off the abraded material, wash the specimen, surface dry, weigh, and record to the nearest gram.

Figure 16. Abrasion test in progress.

D. Calculations

The abrasion loss in grams is calculated by subtracting the weight of the surface-dry specimen after the test from the weight of the surface-dry specimen before test.

Reporting of Results

Report the amount of abrasion loss in grams. This amount shall be the average of at least three test specimens. Age of the concrete shall be included in the report.

Resistance of Concrete Slabs Exposed as Bridge Decks to Scaling Caused by Deicing Agents

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•THE use of deicing chemicals for the removal of snow and ice from concrete pavements frequently causes scaling of the concrete surface. This deterioration occurs on both road slabs placed on a subgrade and on bridge slabs. The latter is recognized as being the far more serious problem. The Bureau of Public Roads has previously published several reports (1, 2, 3) on the resistance of concrete surfaces to scaling. The results reported therein were based on the performance of test slabs placed on the ground so that freezing would occur in a manner similar to that occurring on a road slab; that is, from the top down, with the ground insulating the bottom surface.

In recognition of the more extensive deterioration of bridge deck slabs, it was considered advisable also to study the resistance to scaling of concrete surfaces in a simulated bridge deck environment. The findings reported here are based on the performance of slabs mounted on columns so that both the top and bottom surfaces were exposed to the air as in the case of bridge decks.

SUMMARY OF RESULTS

The following is a summary of the performance of concrete slabs exposed outdoors for periods up to three years. The slabs were subjected to an average of 47 cycles of freezing of water on the surface followed by thawing with deicing agents during each winter. It is doubtful that an equivalent number of deicer applications would be made each winter on bridge decks in most areas, so that performance over a 2 or 3-yr period as discussed in this report is probably more severe than in actual practice.

Effect of Surface Coatings

Sixteen of the 17 surface coatings tested were beneficial to different degrees in increasing the resistance of concrete to surface scaling. The better results were obtained with the epoxy resins, chlorinated rubber compounds, and a tar-based sealing compound. One of the coatings, an acrylic resin, did not give beneficial results.

The epoxy coatings provided excellent resistance to scaling regardless of whether they were the penetrating type or the surface covering type which is applied with a grit filler. However, some loss of the epoxy resin coating occurred when the grit was spread over the epoxy resin before it hardened. No such loss of coating occurred when an epoxy resin-grit mortar was applied on a hardened epoxy resin coating.

Chlorinated rubber compounds applied either as a curing agent or as a surface protective coating gave very good protection to scaling. The best results were obtained when this material was applied as a curing agent.

Slabs coated with fish oil or tall oil compounds had good resistance to scaling in exposures limited to about 1½ winters.

Linseed oil coatings as applied in these tests gave excellent resistance to scaling for the first winter (45 cycles of freezing and thawing), but little protection was found at later periods. This result suggests that linseed oil treatments should be repeated after one or two years of exposure.

Paper sponsored by Committee on Effect of Ice Control.

Effect of Admixtures

The effects of 10 admixtures were evaluated. Two admixtures were classified as retarders, two as durability aids, five as waterproofers, and one as an accelerator. Only two of these, a durability aid described as a polysiloxane (which also had a retarding effect) and calcium chloride, significantly increased the resistance to scaling. The other admixtures, including two waterproofers which entrained over 8 percent air, had little or no effect on the resistance to scaling.

Effect of Slump and Air Content

These tests verified the fact that resistance to scaling is a function of both the air content and water-cement ratio of the concrete, with air content having the greater effect. Non-air-entrained concrete was not scale resistant even with 1-in. slump (w/c = 5.9 gal/bag). Concrete with 5 percent air had good resistance to scaling except when 8-in. slump concrete was used (w/c = 7.0 gal/bag). Concrete with 8 percent air had good resistance to scaling even with 8-in. slump (w/c = 6.5 gal/bag), but such concrete had about 25 percent less strength than concrete with 5 percent air and 3-in. slump.

A maximum water content of 6 gal/bag of cement and a minimum air content of 5 percent were indicated by these tests to be desirable limits for concrete subjected to deicing agents. Many states now specify concrete of this quality for exposure to freezing and thawing.

Effect of Miscellaneous Variables

A 2-in. bituminous concrete wearing surface has afforded good protection from scaling to both air-entrained and non-air-entrained concrete under the particular environmental condition of these tests.

A cover of 1½ in. of air-entrained concrete with maximum slump of 3 in. appears sufficient to prevent corrosion of reinforcing bars. When 8-in. slump air-entrained concrete was used similar protection was provided by 2 in. of concrete over the steel.

The desirability of permitting concrete to age before applying deicers was demonstrated by these tests. Concrete slabs which were 30 days old before the first application of deicing salts were much more resistant to scaling than similar slabs which were only 8 days old when deicers were first applied.

Lightweight concrete prepared with manufactured lightweight fine and coarse aggregates or with normal weight sand and lightweight coarse aggregate had good resistance to scaling. Both had equal or better resistance than similar concrete prepared with normal weight fine and coarse aggregates.

Concrete prepared with expansive cement was more susceptible to scaling at an early age than similar concrete prepared with type I portland cement. The greater susceptibility of this type of concrete to scaling was decreased somewhat, but not entirely eliminated, by permitting the concrete to age longer before applying deicers.

VARIABLES INCLUDED IN STUDY

The following four series of tests were included in this study:

Series I and IA—Surface Coatings: Seventeen different surface coatings.
Series II—Admixtures: Ten different admixtures.
Series III—Variable slump and air content.
Series IV—Miscellaneous: Bituminous concrete wearing surface on portland cement concrete, depth of reinforcing steel in concrete, lightweight aggregate concrete, and expansive cement concrete.

A description of the variables for each series is given in Tables 1 through 4. For the purpose of identification, each slab was given a number, the first digit of which indicates the test series and the remaining digits indicate the variable for that series. Replicate slabs for the same variable were given the same number.

TABLE 1

SURFACE COATINGS USED IN SERIES I AND IA

Variable No.	Description	No. of Coats	Remarks
		Series I	
1-1	Boiled linseed oil	2	Mixed with 50% mineral spirits
1-2	Chlorinated rubber sol. A	1	Applied 3 hr after casting
1-3	Chlorinated rubber sol. A	1	Applied after 21 days
1-4	Chlorinated rubber sol. B	1	Applied 3 hr after casting
1-5	Chlorinated rubber sol. B	1	Applied after 21 days
1-6	Potassium silicate sol.	2	–
1-7	Bituminous-rosin mixture	2	–
1-8	Tar-based sealer	2	–
1-9	Silicone solution	1	–
1-10	Acrylic resin solution	1	–
1-11	Epoxy resin A penetrating type	1	Top surface not treated
1-12	Epoxy resin A penetrating type	1	Top surface acid washed and wirebrushed
1-13	Epoxy resin B	1	Top surface acid washed and wirebrushed
1-14	Epoxy resin C	1	Top surface acid washed and wirebrushed
		Series IA	
1-15	Fish oil	2	Mixed with 50% mineral spirits
1-16	Distilled tall oil	2	Mixed with 50% mineral spirits
1-17	Tall oil fatty acid	2	Mixed with 50% mineral spirits

NOTE: Coatings applied at a rate of 1 gal/200 sq. ft. For variables 1-2 and 1-4, material applied as a curing agent only. All epoxy resins were the two-component type.

MATERIALS

The same cement, sand, and coarse aggregate were used in preparing all slabs, except in the case of some slabs of series IV where the type of cement or aggregates was a variable of the test program. The materials used were a type I portland cement having an alkali content of 0.8 percent, a siliceous sand having a fineness modulus of 2.68, and a uniformly graded crushed limestone, all passing the $1\frac{1}{4}$-in. sieve. A commercially available aqueous solution of neutralized Vinsol resin was used to obtain the desired air content. A general classification of the various surface coatings and admixtures is given in Tables 1 and 2.

MIXING PROCEDURE AND MIX DATA

The concrete for all slabs except those for series IA was mixed in a 5-cu ft capacity tilting drum mixer. Two batches were needed for each slab. A laboratory mixer of

TABLE 2

ADMIXTURES USED IN SERIES II

Variable No.	Type of Admixture[a]	Description	Amount Added per Bag of Cement
2-1	Retarder	Lignosulfonate	3 oz
2-3	Retarder	Lignosulfonate	0.25 lb
2-2	Durability aid	Polysiloxane	0.3 lb
2-4	Durability aid	Silicone	4.4 oz
2-5	Waterproofer	Asphalt emulsion	$1\frac{1}{2}$ gal
2-6	Waterproofer	Asphalt emulsion	$1\frac{1}{2}$ gal
2-7	Waterproofer	Latex	0.1 gal
2-8	Waterproofer	Calcium stearate	14 oz
2-10	Waterproofer	Resin derived from tall oil	0.08 lb
2-9	Accelerator	Calcium chloride	2 lb

[a]Type of admixture is the classification given by the manufacturer.

TABLE 3

VARIATIONS IN SLUMP AND AIR CONTENT USED IN SERIES III

Variable No.	Nominal Slump (in.)	Nominal Air (%)
3-1	1	1
3-2	3	1
3-3	8	1
3-4	1	5
3-5	3	5
3-6	8	5
3-7	1	8
3-8	3	8
3-9	8	8

2-cu ft capacity was used for preparing the slabs for series IA, so that five batches were required for each slab. Except when a property of the concrete was a study variable, a standard mix meeting the following nominal requirements was used: slump, 3 in.; water content, 6 gal/bag of cement; cement content, 5.7 bags/cu yd; and air content, 5 percent. Concrete of this quality is generally considered to be durable in a freezing and thawing exposure and is specified by many states. However, such concrete may not be entirely resistant to scaling in an accelerated freezing and thawing program as in this study, and the effects of the various treatments used may be shown more definitely. The data for the various mixes for all four series are given in Table 5. Three concrete cylinders for compressive strength tests were usually cast to represent the concrete made on each day for each mix design.

DESCRIPTION OF TEST SPECIMENS

The slabs used in these tests were 4 ft wide, 5 ft long and 6 in. thick. The top surface was given a broomed finish similar to that usually used on bridge decks. A raised edge was cast around the perimeter of the top surface to retain a shallow pool of water on the surface. When placed in the outdoor exposure area, the slabs were mounted on columns 3½ ft above the ground. A view of the exposure plot and some of the test slabs is shown in Figure 1.

PREPARATION OF TEST SLABS

The 115 slabs included in this study represent approximately 50 variations of slab preparation and treatment. Each variation is usually represented by duplicate slabs made on different days. However, a control slab was always made in conjunction with the slabs prepared each day to represent test variables.

The slabs were cast in the laboratory in watertight molds. All were molded, vibrated, and finished in the same manner. After the concrete had been placed and vibrated, the slabs were finished by three passes with a steel-edged float and then given

TABLE 4

MISCELLANEOUS VARIABLES USED IN SERIES IV

Variable No.	Nominal Slump (in.)	Nominal Air (%)	Description of Variables
4-1	3	1	2-in. asphalt wearing surface—no seal coat
4-2	3	1	2-in. asphalt wearing surface—seal coat
4-3	3	5	2-in. asphalt wearing surface—no seal coat
4-4	3	5	2-in. asphalt wearing surface—seal coat
4-5	8	5	Reinforcing bars ¾ in. from surface
4-6	8	5	Reinforcing bars 1¼ in. from surface
4-7	8	5	Reinforcing bars 2 in. from surface
4-8	3	5	Reinforcing bars ¾ in. from surface
4-9	3	5	Reinforcing bars 1¼ in. from surface
4-10	3	8	Lightweight F.A. and C.A. (½ in. max.)
4-11	3	8	Natural F.A. and lightweight C.A. (½ in. max.)
4-12	3	5	Expansive cement (C.F. = 7½ bags/cu yd)
4-13	3	5	Expansive cement (C.F. = 8½ bags/cu yd)

TABLE 5

MIX DATA AND PHYSICAL PROPERTIES OF THE CONCRETE[a]

Variable No.	Proportions by Weight (lb)	A. E. Admix. (oz/bag cement)	Other Admix.[b] (per bag cement)	Cement Content (bags/cu yd)	Water Content (gal/bag)	Slump (in.)	Air (%)	Comp. Str. 28 Days (psi)
				Series I				
All	94-215-335	0.6	None	5.6	6.0	3.1	5.2	4650
				Series IA				
All	94-215-335	None	None	5.8	6.0	2.9	2.2	5360
				Series II				
Control	94-215-335	0.6	None	5.6	6.1	2.8	5.1	4480
2-1	94-215-335	0.5	3 oz	5.7	5.7	3.0	5.4	5260
2-2	94-215-335	None	0.3 lb	5.7	5.6	2.6	4.9	4340
2-3	94-215-335	0.3	0.25 lb	5.7	5.4	2.8	5.2	5450
2-4	94-215-335	0.3	4.4 oz	5.7	5.8	2.6	5.0	4590
2-5	94-215-335	0.3	1½ gal	5.5	5.1[c]	3.5	5.3	3530
2-6	94-215-335	None	1½ gal	5.3	5.2[c]	3.0	8.+	1830
2-7	94-215-335	None	0.1 gal	5.8	5.8	2.6	4.6	4380
2-8	94-215-335	0.9	14 oz	5.7	6.1	2.7	4.8	4780
2-9	94-215-335	0.5	2 lb	5.6	6.1	3.5	5.0	5070
2-10	94-215-335	None	0.08 lb	5.4	6.0	3.4	8.+	2730
				Series III				
3-1	94-231-335	None	None	5.6	5.9	1.0	1.3	4980
3-2	94-231-335	None	None	5.6	6.6	3.2	1.3	4750
3-3	94-231-335	None	None	5.5	7.6	7.7	1.2	4040
3-4	94-215-335	0.8	None	5.8	5.4	1.0	5.0	4820
3-5	94-215-335	0.6	None	5.7	6.0	3.2	5.2	4430
3-6	94-215-335	0.4	None	5.6	7.0	8.0	5.8	3860
3-7	94-200-335	1.8	None	5.7	5.2	1.2	6.9	4630
3-8	94-200-335	0.9	None	5.6	5.8	3.3	7.0	4140
3-9	94-200-335	0.7	None	5.5	6.5	7.9	7.2	3420
				Series IV				
4-(1&2)	94-231-335	None	None	5.8	6.3	2.7	1.2	4950
4-(3&4)	94-215-335	0.6	None	5.7	5.9	2.9	5.2	4430
4-(5, 6, 7)	94-215-335	0.5	None	5.5	7.0	7.5	5.1	3850
4-(8&9)	94-215-335	0.6	None	5.7	6.0	3.0	4.5	–
4-10[d]	94-130-130	1.0	None	6.0	8.1[g]	3.1	9.5	4500
4-11[e]	94-190-130	1.0	None	6.1	6.9[g]	4.0	9.0	–
4-12[f]	94-155-250	0.6	None	7.4	4.6	3.4	4.4	4680
4-13[f]	94-120-216	0.7	None	8.6	4.1	4.0	5.5	–

[a]Data for series II, III, and IV are average of tests on concrete for two slabs for each variable.
[b]See Table 2.
[c]Water content does not include water in the admixture.
[d]Lightweight fine and coarse aggregates used.
[e]Natural sand and lightweight coarse aggregate used.
[f]Expansive cement used.
[g]Total water added including that absorbed by lightweight aggregates.

Figure 1. Exposure area.

a final surface finish with a hair broom. Except when a surface coating was being evaluated as a curing material as well as a protective coating, the slabs were covered with wet burlap while stored in the laboratory at a temperature of 70 to 75 F for 4 days. The slabs were then transported to the exposure plot and received no further curing except for that of natural weathering.

The slabs for series I, IA, II, and III were at least 30 days old before they were exposed to freezing. This length of time should permit sufficient curing so that differences caused by the age of the slabs could be considered immaterial. Some of the slabs for series IV, however, were only 8 days old when the first freeze occurred and this is believed to have had a bearing on the amount of scaling of these slabs.

Since the slabs for series IA were prepared about one year later than the slabs in the other series, they were made with non-air-entrained concrete to expedite scaling and thus enable the results to be reported at the same time as those for the other series.

TESTING PROCEDURE

The procedure followed for testing these slabs was similar to that in the previously reported tests for scaling (1). Each night when freezing was expected, the top surface of each slab was covered with approximately ¼ in. of water. The next morning, a mixture of equal parts of sodium chloride and calcium chloride was spread uniformly over the ice-encrusted surface at a rate of about 2.4 lb/sq yd. About 4 to 5 hours later, after the ice had melted, the salt water on the surface was broomed off and the slabs flushed with fresh water to remove all of the chloride solution. Fresh water was left on the surface of the slabs for the next freezing.

The slabs in series I, II, III and IV were exposed to outdoor weathering for 3 years. During this time, they were subjected to 140 cycles of the freezing and thawing procedure previously described. Forty-five cycles were obtained during the first winter, 42 during the second, and 53 during the third. The slabs for series IA have had only 75 cycles of freezing and thawing. All specimens were examined periodically and rated by visual observations of the amount and depth of scaling. The criteria for the ratings of the various degrees of scaling are the same as given in previous reports by the Bureau of Public Roads (1, 2, 3). A general description of the numerical ratings is as follows:

0 = no scale.
1 = scattered spots of very light scale.
2 = scattered spots of light scale with mortar surface above coarse aggregate removed.

3 = light scale over about one-half of the surface.
4 = light scale over most of the surface.
5 = light scale over most of the surface, with a few moderately deep spots, where the mortar surface was below the upper surface of the coarse aggregate.
6 = scattered spots of moderately deep scale.
7 = moderately deep scale over one-half of the surface.
8 = moderately deep scale over entire surface.
9 = scattered spots of deep scale with the mortar surface well below the upper surface of the coarse aggregate; otherwise, moderately deep scale.
10 = deep scale over entire surface.

SURFACE COATINGS—SERIES I

In series I the effect of 14 different surface coatings on scaling resistance to scaling was determined. Descriptions of the coatings as given by the manufacturer, and the methods of application are given in Table 1.

The standard mixture design previously described was used for the concrete for all slabs in this series. The mix data and compressive strength data given in Table 5 are the averages of the tests made on the concrete for the 30 slabs in this series. The actual ranges of these data were as follows:

Cement content, 0.1 bag/cu yd (5.6 to 5.7);
Water content, 0.3 gal/bag of cement (5.9 to 6.2);
Slump, 0.7 in. (2.7 to 3.4);
Air content, 1.1 percent (4.7 to 5.8); and
Compressive strength, 560 psi (4430 to 4990).

Coating Procedure

All of the surface coatings were applied by brushing. The rate of each application was 1 gal of solution per 200 sq ft. For some coatings, two applications were used. Except when the surface coatings were applied as a curing material, they were applied after the slabs had been moist-cured for 4 days and stored in the exposure area for 17 days of outdoor drying. When a second coat was required, it was applied the next day.

The surface of each slab was brushed to remove loose material just prior to the application of the surface coating, except for three of the four groups of slabs on which epoxy was applied. These received an acid wash followed by wire brushing.

TEST RESULTS FOR SERIES I

The effect of the various surface coatings on the resistance of concrete to scaling is given in Table 6 wherein the scaling ratings after 20, 45, 65, 87, 114, and 140 cycles of freezing and thawing are given for each slab on which the surface coatings were applied as well as the three control slabs. In this table other surface defects which may later develop into scaling are also noted. A comparison between the ratings of the slabs with the various surface coatings and the control slabs for series I is shown in Figure 2. The performance of each of the surface coatings is discussed in the following sections.

Boiled Linseed Oil (Variable No. 1-1)

Figure 2 shows that at the end of the first winter (45 cycles) neither of the linseed oil-coated slabs showed any scaling (rating of 0), whereas the uncoated control slabs had light scaling. Continued exposure to freeze-thaw cycles, however, caused the slabs treated with linseed oil to scale, with the result that after three winters (140 cycles) there was nearly the same degree of scaling on the linseed oil-treated and control slabs. These tests suggest that linseed oil provides good protection for the first year or so, but that subsequent treatments may be necessary to maintain a scale-free surface.

TABLE 6

RATINGS OF SLABS FOR SERIES I AND IA SURFACE COATINGS

Variable No.	Description	Rating After Fr. and Th. for Cycles Indicated						Other Defects
		20	45	65	87	114	140	
				Series I				
Control	No surface coating	2	3	4	5	5	6	Map cracking
Control	No surface coating	1	1	2	4	4	4	Lt. map cracking
Control	No surface coating	0	1	2	4	4	5	Map cracking
1-1	Linseed oil	0	0	2	2	3	4	None
1-1	Linseed oil	0	0	2	3	4	5	Lt. map cracking
1-2	Chlor. rubber (A)[a]	0	0	1	1	1	1	Lt. map cracking
1-2	Chlor. rubber (A)[a]	0	0	0	0	0	0	Lt. map cracking
1-3	Chlor. rubber (A)	0	1	1	3	3	4	Lt. map cracking
1-3	Chlor. rubber (A)	0	0	0	0	0	0	None
1-4	Chlor. rubber (B)[a]	0	0	0	0	0	0	None
1-4	Chlor. rubber (B)[a]	0	0	0	0	0	0	None
1-5	Chlor. rubber (B)	0	0	1	2	2	2	None
1-5	Chlor. rubber (B)	0	1	1	2	2	3	Lt. map cracking
1-6	Potassium silicate	0	0	1	1	1	1	None
1-6	Potassium silicate	0	0	1	2	2	2	None
1-7	Bituminous rosin	0	0	0	1	1	1	Lt. map cracking
1-7	Bituminous rosin	0	0	1	3	4	6	Lt. map cracking
1-8	Tar based sealer	0	0	0	0	0	0	Map cracking
1-8	Tar based sealer	0	0	0	0	1	1	Map cracking
1-9	Silicone	0	1	1	2	2	3	None
1-9	Silicone	0	1	2	4	5	5	Lt. map cracking
1-10	Acrylic resin	0	2	3	5	5	6	Map cracking
1-11	Epoxy A[b]	0	0	0	0	0	0	None
1-11	Epoxy A[b]	0	0	0	0	0	0	None
1-12	Epoxy A[b]	0	0	0	0	0	0	None
1-12	Epoxy A[b]	0	0	0	0	0	0	None
1-13	Epoxy B	0	0	0	0	0	0	None
1-13	Epoxy B	0	0	0	0	0	0	None
1-14	Epoxy C	0	0	0	0	0	0	None
1-14	Epoxy C	0	0	0	0	0	0	None
	Series IA[c]	For cycles:	22	49	75			
Control	No surface coating		7	8	10			—
Control	No surface coating		7	9	10			—
1-15	Fish oil		0	0	1			None
1-15	Fish oil		0	0	1			None
1-15	Fish oil		2	5	6			None
1-16	Distilled tall oil		0	0	1			None
1-17	Tall oil fatty acid		0	1	1			None

[a]Chlorinate rubber solution applied as curing material.
[b]Epoxy A applied without mineral filler.
[c]Non-air-entrained concrete used in series IA.

Previous studies conducted by the Bureau of Public Roads (3) had indicated that the beneficial effects of linseed oil were persistant at least to the period required for 105 cycles of freezing and thawing. Although it is possible that the poorer performance of linseed oil in the tests reported here may be related to the greater severity of the test conditions as a result of the test slabs being supported above ground, it is considered probable that other factors may have had equal or greater influence. It is conceivable, for example, that the conditions under which the linseed oil was applied in the later testing program were less favorable for proper penetration than in the earlier series of tests. This possibility is suggested in view of the observation that much of the linseed oil coating flaked off the simulated bridge deck slabs during the first summer along with some of the mortar, which is indicative that penetration was incomplete.

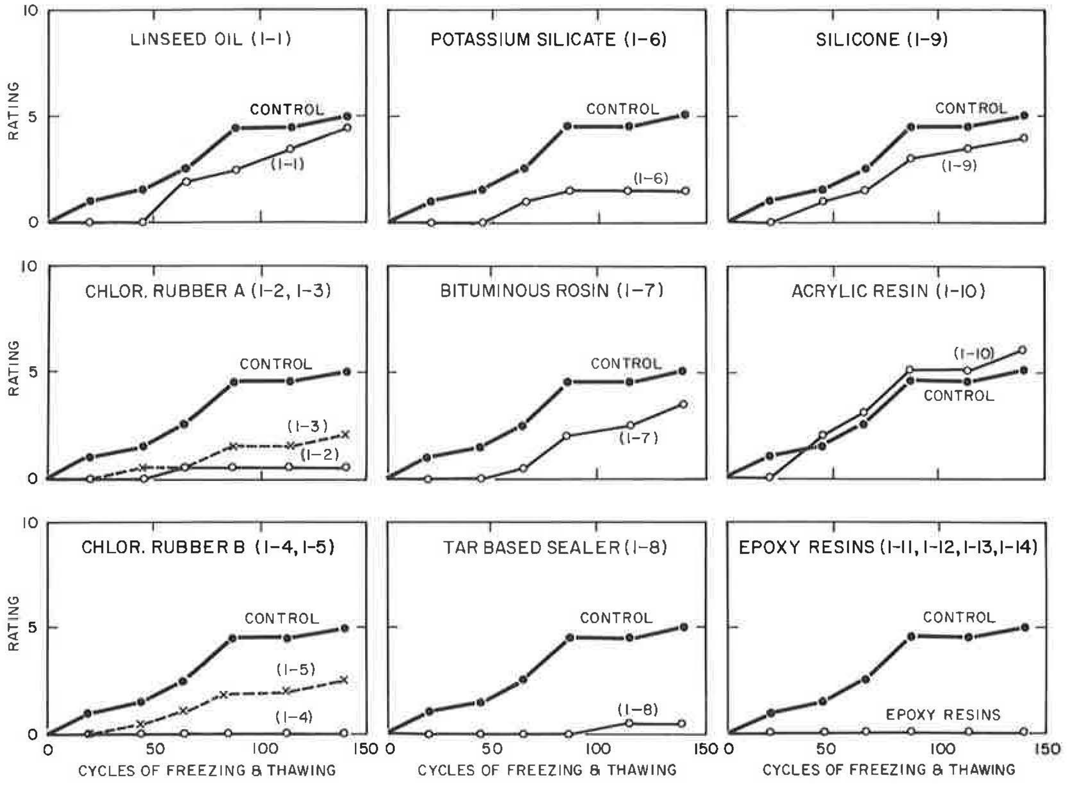

Figure 2. Effect of surface coating on resistance to scaling (series I).

Chlorinated Rubber Coatings (Variables 1-2, 1-3, 1-4, and 1-5)

Two commercially available membrane-forming chlorinated rubber curing materials (identified as A and B) were included in these tests. Water retention tests performed according to ASTM Method C 156 showed these materials to be curing compounds of good quality. Losses of 0.034 and 0.038 gram/sq cm were obtained for materials A and B, respectively, which compare favorably with the 0.055 gram/sq cm limit imposed by ASTM Specification C 309 for liquid membrane-forming curing compounds.

Each of the materials was applied separately as a curing agent and as a surface protective coating. When used as a curing agent, each material was applied approximately 3 hours after the slabs were cast and no further curing was given. When either material was used as a surface protective coating, it was applied after the slab had been given 4 days moist curing in the laboratory followed by drying for 17 days in the outdoor exposure area.

A comparison between the slabs on which chlorinated rubbers A and B were used and the control slabs is provided by two graphs in Figure 2. For the slabs identified as 1-3 and 1-5, the coating was applied as a surface protective coating and for slabs 1-2 and 1-4, it was applied as a curing material. The results obtained with the two chlorinated rubber compounds were essentially the same. In both cases, these materials gave much better resistance to scaling than the control slabs and were more beneficial when used as curing compounds.

Potassium Silicate (Variable 1-6)

The slabs on which this material was applied showed very good resistance to scaling. The average rating for these slabs after 140 cycles was $1\frac{1}{2}$ as compared to an average of 5 for the control slabs.

Bituminous-Rosin Mixture (Variable 1-7)

The two slabs treated with this material were not consistent in their resistance to scaling. Although both slabs showed light map cracking, one slab had a rating of 1 after 140 cycles of freezing and thawing, whereas the other slab had a rating of 6. Although there is no ready explanation for the inconsistent performance it is most probable that some variation in slab preparation, particularly the surface finish, may be responsible.

Tar-Based Sealer (Variable 1-8)

Slabs treated with the tar-based sealer were in excellent condition after 140 cycles of freezing and thawing. One slab showed a slight amount of scaling and it was given a rating of 1, whereas the other slab showed no scaling. Both slabs showed fine map cracking over their entire surfaces. The map cracking on these slabs as well as those coated with the bituminous-rosin mixture may have been brought about by higher surface temperatures resulting from the dark color of the coatings.

Silicone Solution (Variable 1-9)

The silicone solution used in these tests provided no significant improvement in resistance of concrete to scaling.

Acrylic Resin (Variable 1-10)

Only one slab was tested using this surface coating. Scaling on this slab had started by the end of the first winter, and at the end of the third winter it had a rating of 6 with considerable map cracking. The use of this material appeared to offer no protection to the concrete surface.

Epoxy Resin (Variables 1-11, 1-12, 1-13, and 1-14)

Three different two-component epoxy resin systems, identified as A, B, and C, were used in this study. The surfaces of the slabs identified as 1-11 were prepared for coating by brushing to remove loose material. The other slabs (identified as 1-12, 1-13, and 1-14) were given an acid wash, rinsed thoroughly, dried, and then wire brushed prior to coating. About 5 minutes after the two components were mixed, the mixture was brushed on the surface of the slabs at a rate of 1 gal/200 sq ft.

Epoxy A, which was a penetrating type, was applied to slabs 1-11 and 1-12 without the addition of a grit cover. Penetration into the surface of these slabs occurred quite rapidly. For the slabs coated with Epoxy B (1-13), a special sand-sized grit was broadcast uniformly on the plastic epoxy resin at a rate of approximately 0.25 lb/sq yd. For the slabs coated with Epoxy C (1-14), a mixture of one part epoxy resin and three parts grit was troweled on the surface after the initial epoxy resin coating had hardened. The rate of this application was not recorded, but the objective was to obtain as thin a mortar layer as possible with the size of grit used.

All of the slabs which had the epoxy resin surface coatings gave excellent resistance to scaling. After 140 cycles of freezing and thawing, they all had a rating of 0; that is, no indication of surface scaling. For the slabs on which Epoxy A was used (1-11 and 1-12), the epoxy appeared to have penetrated into the surface of the concrete and there is no difference between the slabs which had the acid wash and those which did not. Where Epoxy B was used (1-13), the epoxy did not adhere to the concrete as well as Epoxy C. By 140 cycles of freezing and thawing, approximately 20 percent of Epoxy B had flaked off, whereas the slabs on which Epoxy C was used are still completely coated. The method of applying the grit apparently was a factor in the better adherence of the Epoxy C coating.

SURFACE COATINGS—SERIES IA

Series IA includes tests on three surface coatings of the penetrating oil type which were received too late to be included in series I. The mixture proportions of the concrete from which the slabs for this series were made were the same as those for series I, except that no air-entraining agent was added. Since series IA was started 1 year later than series I, non-air-entrained concrete was used to reduce the number of cycles required to evaluate the coatings. It was hoped that the change in air content for series IA would cause an acceleration of the scaling so that the results for the two series could be reported at the same time.

The curing procedure for the slabs in series IA was similar to that for series I except that after the 4-day moist curing, the series IA slabs remained in laboratory air for a minimum of 30 days prior to storage in the exposure area, whereas series I slabs were taken directly to outdoor storage.

The scale ratings of each slab through 75 cycles of freezing and thawing are given in Table 6 and comparisons of the ratings of the treated slabs with the corresponding control slabs are shown graphically in Figure 3. The single slabs treated with distilled tall oil (variable 1-16) and tall oil fatty acid (variable 1-17), as well as two of the three slabs treated with fish oil (variable 1-15), showed no appreciable scaling after 75 cycles. One slab treated with fish oil had a rating after 75 cycles of 6, for which no ready explanation is availabe. This nevertheless represents less scaling than the rating of 10 shown by the control slabs. Considering that the slabs were composed of non-air-entrained concrete, this group of coating materials provided good resistance to scaling.

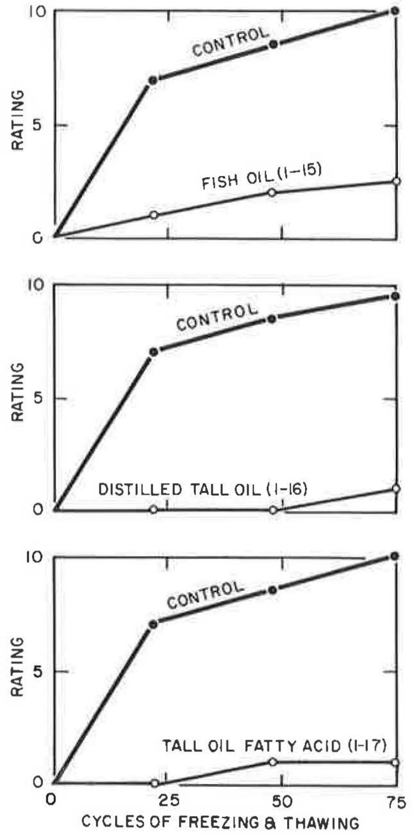

Figure 3. Effect of surface coatings on resistance to scaling (series IA).

ADMIXTURES—SERIES II

This series of tests included slabs prepared with various admixtures which have been recommended for use in bridge deck concrete. The manufacturer's classification of these admixtures, a description of each, and the amounts used are given in Table 2. This test series included two retarders, two durability aids, five waterproofers, and one accelerator. The admixtures classified as retarders and durability aids could also be classified as water-reducing admixtures, as the water required for the desired slump was from 0.3 to 0.7 gal/bag of cement less than that required for the control mix with the same slump.

All of the admixtures except one (variable 2-8) caused some air-entrainment in the concrete so that less air-entraining agent had to be added to the concretes containing them than to the control concrete in order to obtain the desired 5 percent of air. Because of difficulty in obtaining the desired air content with variable 2-8, only one slab was made with this admixture.

Two of the waterproofing admixtures (variables 2-6 and 2-10) produced over 8 percent air, causing reductions in compressive strength of 39 and 59 percent, respectively, below that of the control slabs.

The effect of the various admixtures on the resistance of concrete to scaling is given in Table 7. A comparison between the slabs prepared with the various ad-

TABLE 7

RATINGS OF SLABS FOR SERIES II ADMIXTURES

Variable No.	Type of Admixture	Rating After Fr. and Th. for Cycles Indicated						Other Defects
		20	45	65	87	114	140	
Control	None	1	1	2	4	4	5	Map cracking
Control	None	0	1	2	4	4	4	Map cracking
Control	None	1	1	2	2	2	3	None
2-1	Retarder	1	2	2	4	4	4	Lt. map cracking
2-1	Retarder	1	3	3	4	4	5	Lt. map cracking
2-2	Durability aid	1	1	1	1	1	1	Map cracking
2-2	Durability aid	1	2	2	2	2	2	None
2-3	Retarder	1	3	3	4	4	4	Map cracking
2-3	Retarder	0	1	2	3	3	3	Lt. map cracking
2-4	Durability aid	1	4	4	4	4	4	Map cracking
2-4	Durability aid	1	2	4	4	4	5	Lt. map cracking
2-5	Waterproofer	0	1	3	3	4	4	Map cracking
2-5	Waterproofer	0	1	2	2	3	3	Map cracking
2-6	Waterproofer	0	1	3	5	5	5	Heavy map cracking
2-6	Waterproofer	1	1	3	3	3	4	Map cracking
2-7	Waterproofer	0	1	1	2	2	3	None
2-7	Waterproofer	1	2	5	6	7	8	
2-8	Waterproofer	0	1	1	2	2	3	None
2-9	Accelerator	0	0	1	1	1	1	Lt. map cracking
2-9	Accelerator	0	0	1	1	1	1	Lt. map cracking
2-10	Waterproofer	0	1	3	3	4	5	Map cracking
2-10	Waterproofer	0	1	3	3	4	4	Map cracking

mixtures and the corresponding control slabs is shown in Figure 4. The performance of the slabs prepared with each of the admixtures is discussed in the following sections.

Retarders (Variables 2-1 and 2-3)

The slabs prepared with the two retarders had scale ratings of 3½ and 4½ after 140 cycles as compared to a rating of 4 for the control slabs. Thus, the retarders had no significant effect on the resistance of the slabs to scaling.

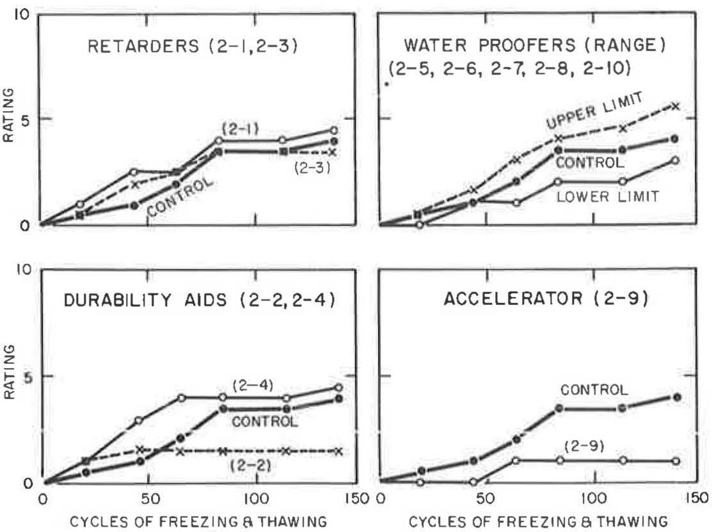

Figure 4. Effect of admixtures on resistance to scaling (series II).

Durability Aids (Variables 2-2 and 2-4)

The slabs prepared with the polysiloxane durability aid (2-2) gave very good resistance to scaling, having an average rating after 140 cycles of 1½. However this admixture retarded the setting time of the concrete more than can usually be tolerated. The other durability aid (2-4) showed no improvement in resistance to scaling.

Waterproofers (Variables 2-5, 2-6, 2-7, 2-8, and 2-10)

The range in the average ratings of the slabs prepared with the five waterproofers is shown in the upper right graph of Figure 4. This indicates that the waterproofing admixtures were generally ineffective in improving the resistance to scaling of air-entrained concrete. It should also be noted that even though admixtures 2-6 and 2-10 produced over 8 percent air in the concrete, there was no increase in scaling resistance over the control concrete with only 5 percent air. This result suggests that the air void system produced by these admixtures may not be as effective as that produced by the air-entraining agents normally used.

Accelerator (Variable 2-9)

The slabs prepared with calcium chloride showed excellent resistance to scaling, having an average rating of 1 after 140 cycles. This finding may have implications with respect to the mechanism by which deicing agents accelerate scaling during freezing and thawing of concrete surfaces. Other researchers (4) have postulated that $CaCl_2$ used as a deicer increases scaling partly as the result of the establishment of a concentration gradient near the surface of the concrete. The increased resistance to scaling obtained in this study supports this view since the integral incorporation of $CaCl_2$ in the concrete, as was done in this study, would tend to minimize the concentration gradient near the surface and thus should improve scale resistance.

VARYING SLUMP AND AIR—SERIES III

The object of this series of tests was to verify existing views on the effect of water content and air content on the resistance of concrete to scaling. For this purpose concrete was prepared with slumps of 1, 3, and 8 in. and air contents of 1, 5, and 8 percent.

TABLE 8

RATINGS OF SLABS FOR SERIES III
VARYING SLUMP AND AIR CONTENT

Variable No.	Slump (in.)	Air (%)	Rating After Fr. and Th. for Cycles Indicated						Other Defects
			20	45	65	87	114	140	
3-1	1	1	0	5	6	8	8	9	—
3-1			1	5	8	10	-	-	—
3-2	3	1	8	9	10	-	-	-	—
3-2			6	9	10	-	-	-	—
3-3	8	1	8	10	-	-	-	-	—
3-3			6	10	-	-	-	-	—
3-4	1	5	0	1	2	3	4	4	Map cracking
3-4			0	1	1	1	1	1	None
3-5	3	5	0	0	2	4	5	5	Map cracking
3-5			0	0	1	1	1	1	None
3-6	8	5	0	2	3	4	5	6	Map cracking
3-6			0	1	3	5	6	7	
3-7	1	8	0	0	1	1	2	2	Map cracking
3-7			0	0	0	0	0	1	None
3-8	3	8	0	0	0	0	1	1	None
3-8			0	0	1	1	1	1	None
3-9	8	8	0	1	2	3	4	5	Map cracking
3-9			0	1	1	1	1	1	None

The mix proportions used for all concretes in this series were the same except for a variation in the sand content to compensate for the varying air contents. It was also necessary to increase the water required for a given air content approximately 1.5 gal/bag of cement as the slump was increased from 1 to 8 in.

The ratings for each slab in this series are given in Table 8 and comparison of the ratings of the slabs with 1, 3, and 8-in. slump for each air content is shown graphically in Figure 5.

Concrete With 1 Percent Air (Variables 3-1, 3-2, and 3-3)

All of the slabs prepared with non-air-entrained concrete had very poor resistance to scaling. The slabs with 3 and 8-in. slump (variables 3-2 and 3-3) both had an average rating of 7 after 20 cycles of freezing and thawing, whereas the slabs with 1-in. slump (variable 3-1) had an average rating of ½. After 45 cycles, the slabs with 3 and 8-in. slump had average ratings of 9 and 10, respectively, while those with 1-in. slump had ratings of 5. It should be noted from the mix data given in Table 5 that the water content for the 1-in. slump mix was 5.9 gal/bag of cement, whereas it was 6.6 and 7.6 gal for mixes 3-2 and 3-3. This result suggests that a significant decrease in durability occurs for non-air-entrained concrete when 6 gallons of water per bag of cement is exceeded.

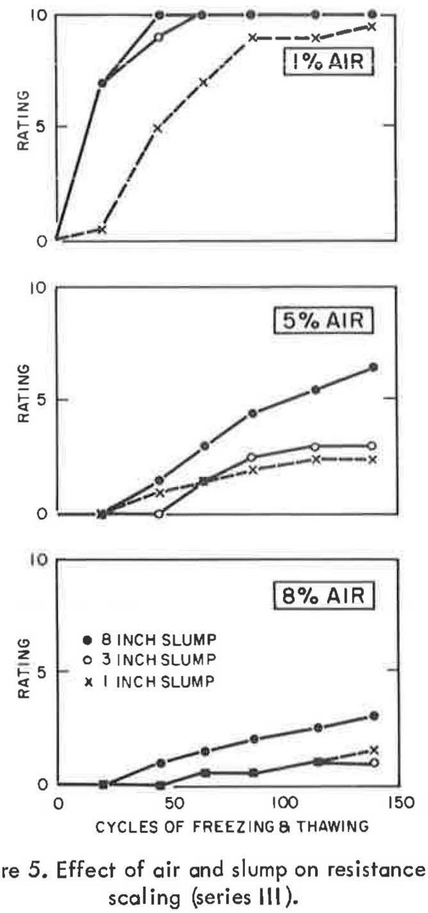

Figure 5. Effect of air and slump on resistance to scaling (series III).

Concrete With 5 Percent Air (Variables 3-4, 3-5, and 3-6)

The adverse effect of increased water-cement ratio was also evident for the concretes having an air content of 5 percent. After 140 cycles, severe scaling had occurred only on the 8-in. slump concrete which had a water content of 7 gal/bag of cement. The concretes prepared to have 1 and 3-in. slumps showed only scattered light scaling.

Concrete With 8 Percent Air (Variables 3-7, 3-8, and 3-9)

The slabs prepared with 8 percent air all showed good resistance to scaling; the highest average rating being found for the slabs cast from the 8-in. slump concrete. Apparently, the adverse effect of a high water content decreases as the air content is raised. However, it should be noted that the 8-in. slump concrete had approximately 25 percent less strength than concrete having 3-in. slump and 5 percent air.

MISCELLANEOUS TESTS—SERIES IV

This series of tests includes a number of miscellaneous independent studies for which variables are described in Table 4. The scale ratings given each of the slabs prepared for these variables are given in Table 9. The performance of the slabs for this series is discussed in the following sections.

TABLE 9

RATING OF SLABS FOR SERIES IV (Miscellaneous Variables)

Variable No.	Description	Rating After Fr. and Th. for Cycles Indicated						Other Defects
		20	45	65	87	114	140	
4-1	1% air PCC—2-in. asphalt surface						—a	
4-1	1% air PCC—2-in. asphalt surface						—a	
4-2	1% air PCC—2-in. asphalt surface[b]						—a	
4-2	1% air PCC—2-in. asphalt surface[b]						—a	
4-3	5% air PCC—2-in. asphalt surface						—a	
4-3	5% air PCC—2-in. asphalt surface						—a	
4-4	5% air PCC—2-in. asphalt surface[b]						—a	
4-4	5% air PCC—2-in. asphalt surface[b]						—a	
4-5	8-in. sl.—steel ¾ in. from surface	0	3	5	7	7	8	Cracks[c], rust[d]
4-5	8-in. sl.—steel ¾ in. from surface	1	5	7	9	9	10	Cracks[c], rust[d]
4-6	8-in. sl.—steel 1¼ in. from surface	1	4	6	8	8	9	Cracks[c], rust[d]
4-6	8-in. sl.—steel 1¼ in. from surface	1	4	6	9	9	9	Cracks[c], rust[d]
4-7	8-in. sl.—steel 2 in. from surface	1	3	5	7	7	8	None
4-7	8-in. sl.—steel 2 in. from surface	1	5	8	9	9	10	None
4-8	3-in. sl.—steel ¾ in. from surface	0	1	3	4	4	5	Cracks[c], rust[d]
4-8	3-in. sl.—steel ¾ in. from surface	0	1	3	4	5	6	Cracks[c], rust[d]
4-9	3-in. sl.—steel 1¼ in. from surface	0	1	3	3	4	4	None
4-9	3-in. sl.—steel 1¼ in. from surface	0	1	3	3	4	4	None
4-10	Lightweight F.A. and C.A.	0	0	1	2	2	2	None
4-10	Lightweight F.A. and C.A.	0	0	2	2	3	3	Map cracking
4-11	Normal F.A. and lightweight C.A.	0	0	0	0	0	0	Heavy map cracking
4-11	Normal F.A. and lightweight C.A.	0	0	1	1	1	1	Heavy map cracking
4-12	Expansive cem.—7½ bags/cu yd	7	7	7	7	7	7	—
4-12	Expansive cem.—7½ bags/cu yd	7	7	7	7	7	7	—
4-13	Expansive cem.—8½ bags/cu yd	7	7	7	7	7	7	—
4-13	Expansive cem.—8½ bags/cu yd	7	7	7	7	7	7	—

[a]No rating obtained—2-in. asphalt wearing surface not removed.
[b]Seal coat applied between concrete and 2-in. asphalt wearing surface.
[c]Cracks over steel reinforcing bars.
[d]Rust stains on surface of concrete.

Bituminous Concrete Surface Coating (Variables 4-1, 4-2, 4-3, and 4-4)

These tests were made to determine the effect of a 2-in. bituminous concrete cover on the resistance to scaling of the concrete surface on which it is applied. There is some opinion that salt solutions penetrate this type of overlay (usually through cracks in the overlay) and are trapped at the surface of the concrete with the result that deterioration of the concrete is accelerated. Both non-air-entrained and air-entrained concrete slabs were tested.

A coal tar pitch emulsion seal coat was applied to half of the slabs. After the sealer had dried, all of the slabs were lined up and 2 in. of bituminous concrete was applied in a manner similar to that which would be used on a bridge deck surface. The final compaction was obtained by 10 passes made with an 8-ton tandem steel-wheeled roller. The slabs were about 30 days old when the bituminous concrete was applied.

Because of difficulty in obtaining a dam which would hold water on the bituminous surface, these slabs were not salted regularly until the beginning of the second winter. They have now had approximately 100 cycles of natural freezing and thawing with deicing chemicals. Since it was desired to continue the exposure of these slabs beyond this period, examination of the concrete surface was made without removing the entire overlay. Eight-inch cores of the bituminous concrete were removed from the two slabs which were considered to be the most susceptible to scaling, that is, the non-air-entrained slabs which did not receive a seal coat prior to surfacing (variable 4-1).

The areas of concrete surface that could be inspected did not show any evidence of scaling. Although control slabs without a bituminous cover were not made specifically for comparison with the overlayed slabs, similar non-air-entrained slabs were used as controls in series IA and III. These uncovered slabs were all heavily scaled before 75 cycles.* Thus, the bituminous cover was highly effective in protecting the concrete under these particular conditions of test. One possible explanation for the apparent protective effect of the overlay is that the additional heat absorbed by the black surface layer, as well as its insulative effect, may have reduced the number and severity of the freezing cycles experienced by the concrete surface of these slabs. Another factor that may have a bearing on the performance of these slabs is the fact that they had over a year of curing before receiving regular applications of deicing chemicals.

Slabs Containing Reinforcing Steel (Variables 4-5, 4-6, 4-7, 4-8, and 4-9)

These tests were made to determine the minimum thickness of concrete cover for steel reinforcing bars needed to prevent cracking and spalling caused by corrosion of the steel. Slabs were made with air-entrained concrete with high (8-in.) and medium (3-in.) slumps. The steel reinforcement consisted of No. 5 (5/8-in. diameter) bars which were imbedded in the concrete with 3/4, 1 1/4, or 2-in. cover. The bars were spaced 8 in. from the edges and on 8-in. centers.

The slabs made from concrete with the 8-in. slump and with 3/4 and 1 1/4-in. cover (variables 4-5 and 4-6) showed cracks over each of the reinforcing bars, some rust stains on the surface, and several popouts. There were no cracks over the steel or rust stains on the surface of the concrete where the steel was imbedded in this concrete for a depth of 2 in. (variable 4-7). The slabs made with 3-in. slump concrete with the steel having 3/4-in. concrete cover (variable 4-8) showed cracks in the concrete over each of the bars. These tests indicate that a 2-in. concrete cover over reinforcing bars will insure protection from damage caused by corrosion of the steel even when a high slump air-entrained concrete is used. A 1 1/2-in. cover would apparently be satisfactory if a less permeable concrete is used, such as the 3-in. slump concrete of this series.

All of the slabs containing reinforcing steel showed considerably more scaling than would be expected for air-entrained concrete. This form of deterioration, however, would not be expected to be influenced by the depth of cover of the steel. In the case of these tests, it is believed that the excessive scaling is related to the fact that these slabs were only 8 days old when the first freeze and deicer application occurred. In support of this opinion, a comparison is shown in Figure 6 between the ratings of the slabs in this series and slabs from series III which were prepared with similar concrete but which were more than 30 days old when first subjected to freezing. For both the 8 and 3-in. slump concrete, the series IV slabs show considerably more scaling than similar slabs of series III. Thus, it is evident that during the early life of concrete there is a gradual improvement in resistance to scaling beyond the usual specified curing periods. This improvement in durability may be due to continued hydration and gain in strength or to the drying of the concrete, or both.

Lightweight Concrete (Variables 4-10 and 4-11)

These tests were conducted to determine the effect of using a manufactured lightweight aggregate on the resistance of concrete to scaling. Slabs were prepared with concrete in which the lightweight fine and coarse aggregates were used, and also where lightweight coarse aggregate was used in combination with a natural fine aggregate. These concretes were prepared with higher air contents (9.5 and 9.0 percent) than were used in the standard normal-weight concrete in view of the small maximum size (1/2 in.) of the lightweight coarse aggregate.

Three of the four slabs prepared with lightweight aggregates developed a form of surface map cracking with the more severe condition occurring when only the coarse aggregate fraction was the lightweight material. The map cracking, however, did not adversely affect the resistance to scaling. The slabs prepared with both fine and coarse lightweight aggregates had an average scale rating of 2 1/2 after 140 cycles; the slabs containing only lightweight coarse aggregate had an average rating of 1/2. On the basis of these ratings, the scale resistance of the lightweight aggregate concrete in series IV is slightly better than that of the normal-weight concrete used in the control slabs of series I, II, and III. Although the somewhat higher mortar air content of the light-

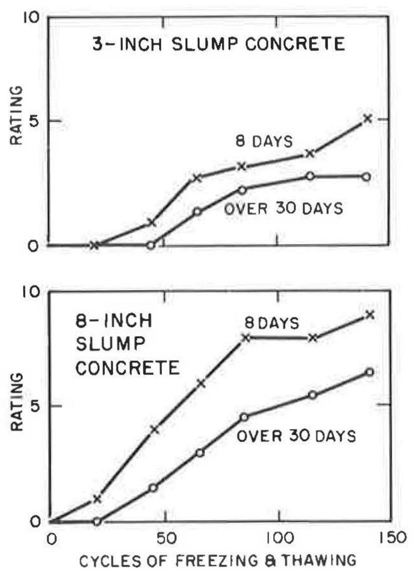

Figure 6. Effect of age of concrete when first salted on resistance to scaling.

weight aggregate concrete may have influenced the relative performance of the two types of concrete, these tests do indicate that there is no sacrifice of scale resistance by virtue of using lightweight aggregate.

Expansive Cement Concrete (Variables 4-12 and 4-13)

Scaling tests were made on slabs prepared with air-entrained concrete in which expansive cement was used. In keeping with the use of this type of cement in practice, cement contents of $7\frac{1}{2}$ and $8\frac{1}{2}$ bags/cu yd were used in the concrete and the slabs were heavily reinforced to provide the necessary restraint for the expanding concrete. These particular slabs received the standard 4 days of moist curing in the laboratory but were only 12 days old when they were first subjected to freezing and deicer agents.

All the expansive concrete slabs showed severe scaling prior to 10 cycles of freezing and thawing. At 10 cycles (the first rating of the slabs), they all had a rating of 7, which represents considerably more scaling at this age than was observed for any of the air-entrained concrete slabs prepared with the type I portland cement. Even the high slump concrete slabs prepared with type I cement, which were only 8 days old when the first freeze occurred (variables 4-5, 4-6, and 4-7), had an average rating of only 1 after 20 cycles of freezing and thawing. However, the amount of scaling on the slabs prepared with the expansive cement did not increase after this initial scaling.

It is thought that the poor resistance to scaling after only a few cycles may have been due at least in part to the early age of the slabs when the first freeze occurred. To investigate this possibility, duplicate slabs were prepared with concrete containing $7\frac{1}{2}$ bags/cu yd of concrete. They were moist-cured for 4 days and then stored in laboratory air for 60 days prior to storage in the exposure area. The first freeze occurred several days later.

These slabs showed some scaling after 10 cycles, although not as severe as the original slabs. The average rating at 10 cycles was $2\frac{1}{2}$, and at 75 cycles it was 4. This would indicate that the short curing period was a factor in the low resistance to scaling for the slabs prepared with this cement, but not the entire cause. Apparently, there was insufficient restraint of the surface mortar of the concrete slabs prepared with expansive cement to produce a scale-resistant surface.

CONTINUATION OF TESTS

The tests on these slabs will be continued for at least an additional winter. After the application of the deicing salts has been discontinued, cores will be drilled from many of the slabs to obtain additional information on the hardened concrete. As previously stated, in most cases duplicate slabs were made on different days for each variation. Of the 49 variables for which two or more slabs were prepared, only 8 showed a variation of more than 2 in the ratings given for the duplicate slabs. A study of the hardened concrete may explain some of the differences that occurred in these 8 cases.

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