HIGHWAY RESEARCH RECORD

Number 226

Concrete Admixtures, Aggregates and Durability

6 Reports

Subject Area

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Foreword

Hydraulic-cement concrete can be made, by suitable selection of ingredients and their proportions, to possess at any given time in its life widely different levels of any of its properties. The builders and users of transportation systems desire that concrete in all portions of these systems possess at all times in its service life the necessary levels of relevant properties to insure adequate service of the structures in which it is used. The papers presented here contribute knowledge of a variety of sorts that should aid in the attainment of this goal.

The portland cement concrete used in constructing highway bridges and pavements should be made of properly selected materials, in appropriate proportions, and should be mixed, placed, and cured in accordance with proper construction practices so that the resulting product will have the necessary levels of its relevant properties to provide the desired service in the environment in which used.

The information given in these papers should be of value to those who prepare specifications for materials and construction, to those who test and select materials and proportions for concrete, to those who supervise and inspect construction work, to those who must investigate cases of less-than-adequate performance, and especially, to those who plan and carry out research and investigational work in these areas.

If concrete in service is subjected to freezing while saturated by water, it must be made using aggregates and cement paste of proper pore characteristics. Walker and Hsieh contribute to the understanding of the relations between aggregate pore characteristics and concrete durability. Torrans and Ivey further understanding of a number of factors for the achievement of the desired pore characteristics (air-void system) of the cement paste. Buth, Ivey, and Hirsch present results that suggest that the fine aggregate used in concrete can affect the relevant properties of the concrete as a function of the type and amount of clay in the fine aggregate.

Synthetic lightweight aggregates are being used in increasing amounts in concrete. Buth and Ledbetter report a number of characteristic differences in the behavior of these aggregates especially relating to the effects of their pore structure and pore volume on the rate and amount of water they absorb. It is shown that these variations affect frost resistance of concrete.

In recent years, concrete bridge decks have received increasing attention from the standpoint of concrete performance. Studies of such performance are reported by Larson, Cady, and Price and by Stewart and Neal. Larson and his coauthors present specific recommendations for improving the performance of bridge decks in Pennsylvania. Among other things, they call for changes in aggregate specifications, trial mixtures, frequent determination of moisture content of aggregates, determination of slump and air content on every load of concrete delivered, special requirements during hot weather, and elimination of overfinishing.

Stewart and Neal analyzed construction practices used on 28 decks in California. They found it difficult to execute a research program during actual construction. Great difficulty was experienced in learning, with confidence, the actual water content of a batch of concrete or in insuring that proper curing practices were followed. They concluded that the age of the concrete is a significant factor in the cracking pattern during the first months after placement.

—Bryant Mather

In Highway Research Record Number 226, there was an omission in the Table of Contents. Please insert this Corrected copy.

Contents

Air Void Systems Affected By Chemical Admixtures and Mixing Methods

PATRICK H. TORRANS and DON L. IVEY, Texas Transportation Institute, Texas A&M University

Variations in the entrained-air system in hydraulic cement mortars due to different chemical types of air-entraining agents and retarders and different mixing methods were investigated. Twenty-seven mortar batches were prepared at a fundamentally constant air content using different combinations of 3 mixing sequences, 3 air-entraining agents, and 3 retarders.

The Powers and Philleo spacing factors were determined on specimens from each batch, and were used as the criteria for comparison of the air void systems. Observed differences in the Powers spacing factor were found to be statistically significant for different mixing methods and different air-entraining agents. Comparatively large values of the Powers spacing factor were observed when the air-entraining agent and retarder were combined in the same water phase before being combined with the cement and sand. Relatively low values of Powers spacing factor were observed when the organic acid retarder was used, regardless of the air-entraining agent used.

•THE practice of intentionally entraining air bubbles in concrete was introduced in the 1930's and has since become one of the most important developments in concrete technology.

The primary purpose of air entrainment in concrete is to protect the paste from the potentially destructive hydraulic pressures developed during the freezing of moisture contained within the concrete matrix. Work done by T. C. Powers (1) in 1954 predicted the order of magnitude of this pressure and showed that not only was the total volume of air contained in the concrete of importance, but more importantly the size distribution and frequency of air bubble voids must be such as to provide protection to the paste. Powers states that ". . . a body of nearly saturated paste more than a few hundredths of an inch thick cannot possibly be frozen rapidly without incurring damage."

To indicate the thickness of the paste, Powers introduced a factor defined as the maximum average distance from a point in the paste to the nearest air void (Powers spacing factor, $\bar{L}$). This factor is indicative of the distance water would have to travel during the freezing process in order to reach an air bubble void. According to Powers, if these voids are spaced sufficiently close, the internal hydraulic pressure created as a result of the movement of moisture would be sufficiently low and rupture of the paste in tension would not occur.

More recent laboratory observations have supported Powers' findings that for a particular air-entrained concrete, the magnitude of the spacing factor serves as an indication of that concrete's ability to withstand freezing and thawing. That is, as the magnitude of $\bar{L}$ for a given concrete decreases, the durability of the concrete subjected to freezing and thawing increases.

The magnitude of $\bar{L}$ is dependent on the frequency distribution of the void sizes in a given concrete mixture. Therefore, either the spacing factor or the frequency

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distribution of void sizes can be used as an indication of a concrete's durability when subjected to freezing and thawing.

The spacing factor proposed by Powers is not the only indication of a concrete's ability to withstand freezing and thawing. In 1955, R. E. Philleo (2) suggested a factor based on what he termed the protected paste volume concept. Larson et al (3) reported evidence sufficient to justify further studies of this factor as an indicator of the frost resistance of concrete. To determine the Philleo factor, it is necessary to obtain a bubble size distribution from which the total number of bubbles per unit volume of paste may be calculated. As stated (3): "This number is used to calculate a factor indicating the protected paste volume, termed the Philleo spacing factor. This may be thought of as the thickness of spherical shells concentric with randomly distributed air voids such that the volume contained within all such spheres in a unit volume of paste constitutes a given percentage of paste."

As with Powers spacing factor, the magnitude of the Philleo factor is dependent on the frequency distribution of void sizes.

Because of the preceding considerations, it is believed that, regardless of the factor or factors chosen to indicate a particular concrete's ability to withstand freezing and thawing, the frequency distribution of the void sizes is of primary importance in frost resistance. Previous investigations (4, 5) have shown that numerous factors influence the frequency and void distribution of the entrained-air system in both concrete and mortar. The work reported in this paper was undertaken to study the effects of the following factors on the entrained-air system of mortars:

1. Effects of different mixing sequences;
2. Effects of different air-entraining agents;
3. Effects of different retarders; and
4. Interaction effects of mixing sequences, air-entraining agents, and retarders.

It was not expected that the numerical differences could be extrapolated directly to concrete. However, the significant differences in the air void system of mortars would seem to indicate that some differences would be encountered due to the variation of the corresponding factors in concrete.

The criteria used to compare the differences in the air void systems were the magnitudes of the Powers and Philleo spacing factors. The spacing factors were not themselves under a comparative investigation.

TABLE 1

WATER-CEMENT RATIOS

Batch Designation	Water-Cement (wt water/wt cement)
AV-M2	0.550
AD-M2	0.550
AH-M2	0.564
(a) Mixing Sequence M1	
AV-RO	0.536
AD-RO	0.543
AH-RO	0.543
AV-RL	0.550
AD-RL	0.550
AH-RL	0.550
AV-RP	0.543
AD-RP	0.550
AH-RP	0.557
(b) Mixing Sequence M2	
AV-RO	0.509
AD-RO	0.509
AH-RO	0.509
(b) Mixing Sequence M2 (cont'd)	
AV-RL	0.543
AD-RL	0.550
AH-RL	0.550
AV-RP	0.522
AD-RP	0.522
AH-RP	0.543
(c) Mixing Sequence M3	
AV-RO	0.536
AD-RO	0.530
AH-RO	0.530
AV-RL	0.522
AD-RL	0.543
AH-RL	0.550
AV-RP	0.536
AD-RP	0.536
AH-RP	0.550

Figure 1. Determining flow of the hydraulic cement mortar.

Figure 2. Determining air content of the hydraulic cement mortar.

TESTING PROGRAM

The mortars used in these tests were composed of Atlas Type I cement, Ottawa standard graded silica sand (as defined in ASTM C 185-59) and water. The cement/sand ratio of all mortars was 0.366 and sufficient water and air-entraining agent was used to produce a flow of 75 percent and air content of 11 ±1 percent. Tests were conducted in accordance with ASTM C 185-59 except that the dimensions of the cylindrical container were $2\frac{7}{16}$ in. in diameter by $3\frac{21}{32}$ in. in depth. Eleven percent air in mortar corresponds to 6 percent in a concrete with 57 percent mortar by volume.

Water/cement ratios for the mortars are given in Table 1. The procedure to determine the flow and air content of the hydraulic mortar is shown in Figures 1 and 2.

Three air-entraining agents (AV, AD, AH) and three retarders (RO, RL, RP) were used in this testing program. In the designation used, A indicates an air-entraining agent and R, a retarder: the letter following A indicates the type of air-entraining agent (V—vinsol resin, D—synthetic detergent, H—organic salt of sulfonated hydrocarbon);

TABLE 2

MIXING PROCEDURES

Designation	Description
M1	Sand and cement were placed in the mixer and allowed to dry mix 1 min at 150 rpm. Air-entraining agent and retarder were combined in the mixing water and introduced into the mixer over a 1-min time period. Mortar was then mixed an additional 2 min at 340 rpm.
M2	Sand and cement were placed in the mixer and allowed to dry mix 1 min at 150 rpm. Air-entraining agent was combined with one-half the mixing water and added over a 30-sec period. Retarder was combined with the remainder of the water and added over a 30-sec period. Mortar was then mixed an additional 2 min at 340 rpm.
M3	Sand was placed in the mixer and one-half the mixing water containing the air-entraining agent was added over a 30-sec time period at a mixing speed of 150 rpm. Cement was then added over a 1-min period at a mixing speed of 150 rpm. Remainder of the mixing water containing the retarder was added over a 30-sec period. Mortar was then allowed to mix for 2 min at 340 rpm.

Figure 3. Technicians operating linear traverse device.

Figure 4. Sawing mortar specimen to expose surface for microscopic examination.

that following R indicates the type of retarder (O—organic acid, L—lignosulfonate, P—hydroxylated polymer).

Each air-entraining agent was used in combination with each retarder and three mixing sequences (M1, M2, M3—Table 2) were employed to yield 27 batches of mortar. With the exception of retarder RL, the manufacturer's recommended quantity was used. Because of the air-entraining characteristic of retarder RL, it was necessary to reduce the quantity to maintain the proper air content while using a significant amount of air-entraining agent.

The method used to determine the parameters of the air void system was essentially in accordance with ASTM C 457-66T, except that the Rosiwal linear traverse technique was modified in order to record each individual chord length. The apparatus used for measurement of the parameters is shown in Figure 3.

Information necessary to determine the Philleo spacing factor was obtained using the mathematical methods outlined by Larson et al (3). To facilitate the tedious numerical analysis necessary to carry out this investigation, the data reduction was programmed for the IBM 7094 and IBM 1401.

From each mortar batch, a prismatic specimen was cast and allowed to moist cure for 14 days before being prepared for microscopic examination. Figure 4 shows the specimen being sawed to expose a surface approximately at right angles to the finished surface of the mortar. The exposed surface is then ground with silicon carbide abrasive (Fig. 5) until it is suitable for microscopic observation.

Figure 5. Grinding exposed surface with silicon carbide abrasive.

The air-void parameters of air content, specific surface area, and Powers spacing factor were determined in accordance with ASTM C 457-66T. Traverse lengths ranged from 50 to 60 in. on a surface of 9 sq in.

TEST RESULTS AND DISCUSSION

Using the method described by Lord and Willis (6), the number of voids per cubic centimeter in the 0 to 508 -μ range was determined as well as the total number in the 0 to 2,540 -μ range. The number of voids per cubic centimeter in the 0 to 508 -μ range was then determined using the mathematical approach described by Larson et al (3), thus enabling determination of the Philleo spacing factor. Table 3 summarizes the information of primary interest.

Effect of Mixing Sequences

The statistical technique used to determine the significance of the observed differences in the Powers and Philleo spacing factors was a three factor interaction analysis of variance. However, because there was no repetition of the batches, there is no guarantee that the estimate used for testing variation is not lower than the true variation.

The significance of the differences in the mean values was determined using Tukey's (7) h. s. d. procedure. The magnitudes of the Powers spacing factors are shown in Figure 6 for each mortar batch. Using a three factor interaction analysis of variance and the Powers factor as a criterion, a difference in mixing sequences significant at the 99 percent level was found to exist. The dashed lines shown represent the mean value of the spacing factors associated with each mixing sequence. It was found that between mixing sequences M3 and M1 and M2 and M1, the mean differences were significant at the 99 percent level with M1 giving larger values of Powers spacing factor. No significant difference was observed between M2 and M3. Therefore, a less desirable air-void system was evident in the mixing procedure where the admixtures were combined prior to their addition.

The same procedure was followed using the Philleo factor as a criterion for observing the effect of the mixing sequence. Although the magnitudes of this factor are not shown in graphical form, they are given in Table 3. Table 4 summarizes the mean

TABLE 3

PARAMETERS OF THE AIR VOID SYSTEM

Batch Designation	Microscopic Air Content (%)	Powers Spacing Factor (in.)	Philleo Spacing Factor (in.)	Specific Surface Area (in.$^{-1}$)	No. Bubbles/CC Mortar, 0-508-μ Range (3)	No. Bubbles/CC Mortar, 0-508-μ Range (6)
AV-M2	13.56	0.00565	0.00359	520	137,251	93,319
AD-M2	14.15	0.00771	0.00615	358	27,454	17,669
AH-M2	11.47	0.00611	0.00362	581	131,188	74,964
AV-RO-M1	15.75	0.00574	0.00567	420	31,355	45,629
AD-RO-M1	10.55	0.00815	0.00418	471	74,406	30,092
AH-RO-M1	8.67	0.00806	0.00356	557	94,044	41,772
AV-RL-M1	11.52	0.00888	0.00503	394	53,667	25,504
AD-RL-M1	11.84	0.01173	0.00514	289	47,407	8,618
AH-RL-M1	8.16	0.00954	0.00399	486	111,321	27,502
AV-RP-M1	13.63	0.00786	0.00519	365	52,431	39,697
AD-RP-M1	15.12	0.00837	0.00736	306	16,379	13,254
AH-RP-M1	8.80	0.01032	0.00579	434	42,357	41,420
AV-RO-M2	13.48	0.00645	0.00330	442	181,449	46,062
AD-RO-M2	13.50	0.00805	0.00550	353	40,560	25,124
AH-RO-M2	10.92	0.00600	0.00220	603	384,774	97,720
AV-RL-M2	12.52	0.00830	0.00521	382	39,278	24,674
AD-RL-M2	12.30	0.00931	0.00422	349	75,793	16,891
AH-RL-M2	10.21	0.00646	0.00344	620	154,309	78,478
AV-RP-M2	11.67	0.00688	0.00349	493	172,164	98,625
AD-RP-M2	11.16	0.00890	0.00505	400	57,123	38,025
AH-RP-M2	10.69	0.00755	0.00420	502	99,701	77,662
AV-RO-M3	11.16	0.00794	0.00481	452	64,799	49,605
AD-RO-M3	16.35	0.00708	0.00497	325	45,665	19,703
AH-RO-M3	11.00	0.00729	0.00347	498	145,312	51,414
AV-RL-M3	13.74	0.00721	0.00617	390	24,574	22,340
AD-RL-M3	12.51	0.00833	0.00547	381	40,414	21,061
AH-RL-M3	10.51	0.00569	0.00190	667	597,374	107,200
AV-RP-M3	11.38	0.00725	0.00344	484	154,537	38,667
AD-RP-M3	11.38	0.00779	0.00425	452	82,886	41,491
AH-RP-M3	10.57	0.00802	0.00444	480	87,397	63,365

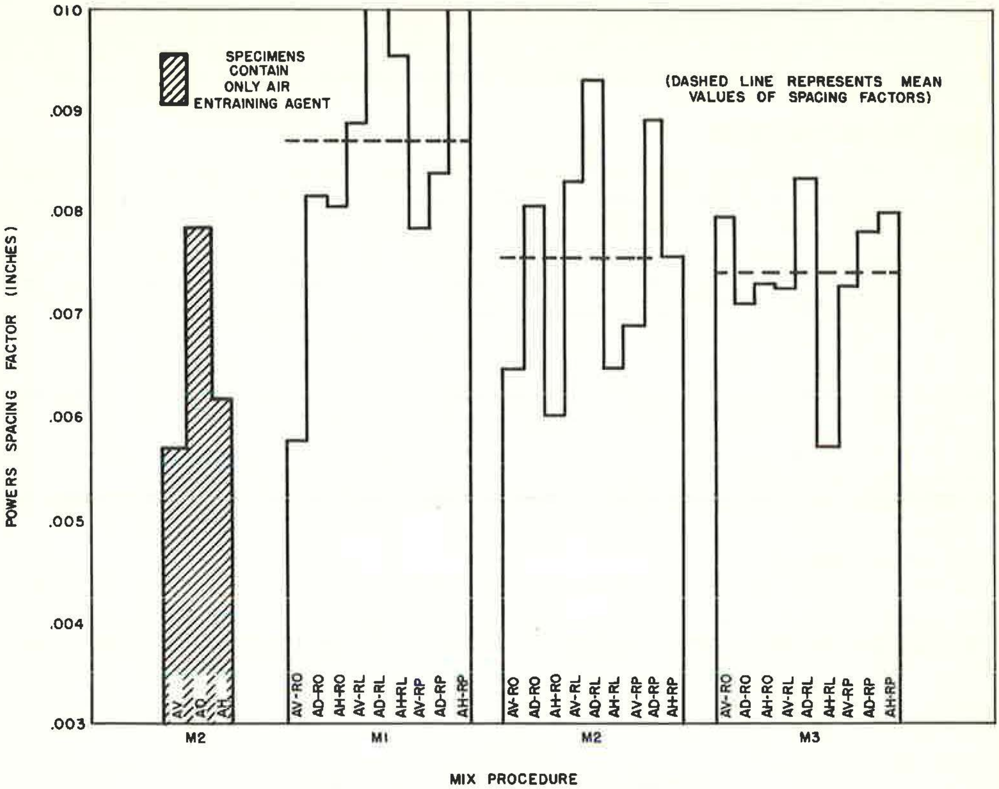

Figure 6. Comparison of Powers spacing factor between mixing procedures.

values. Using the Philleo factor as the criterion, no significant difference was indicated between mixing sequences.

Effect of Air-Entraining Agents

The effectiveness of the air-entraining agents was investigated using the same tests for significance. It was found that significant differences existed between the air-entraining agents. Air-entraining agent AD produced an air void system which yielded higher values of both the Philleo and Powers spacing factor. No significant difference was observed between agents AV and AH, whereas the differences in the average Powers spacing factors of AV and AD, and AH and AD were significant at the 99 percent and 95 percent levels, respectively. Mean values of Powers factor are shown in Figure 7.

Using the Philleo factor, a significant difference in the means of AD and AH was indicated at the 95 percent level.

TABLE 4

SUMMARY OF MEAN VALUES OF POWERS AND PHILLEO SPACING FACTORS

Mean Spacing Factor	Powers Spacing Factor (in.)			Philleo Spacing Factor (in.)		
Mixing procedure	M1 0.00874	M2 0.00754	M3 0.00740	M1 0.00510	M2 0.00407	M3 0.00432
Air-entraining agent	AV 0.00739	AD 0.00863	AH 0.00766	AV 0.00470	AD 0.00513	AH 0.00367
Retarder	RO 0.00720	RL 0.00838	RP 0.00810	RO 0.00418	RL 0.00451	RP 0.00480

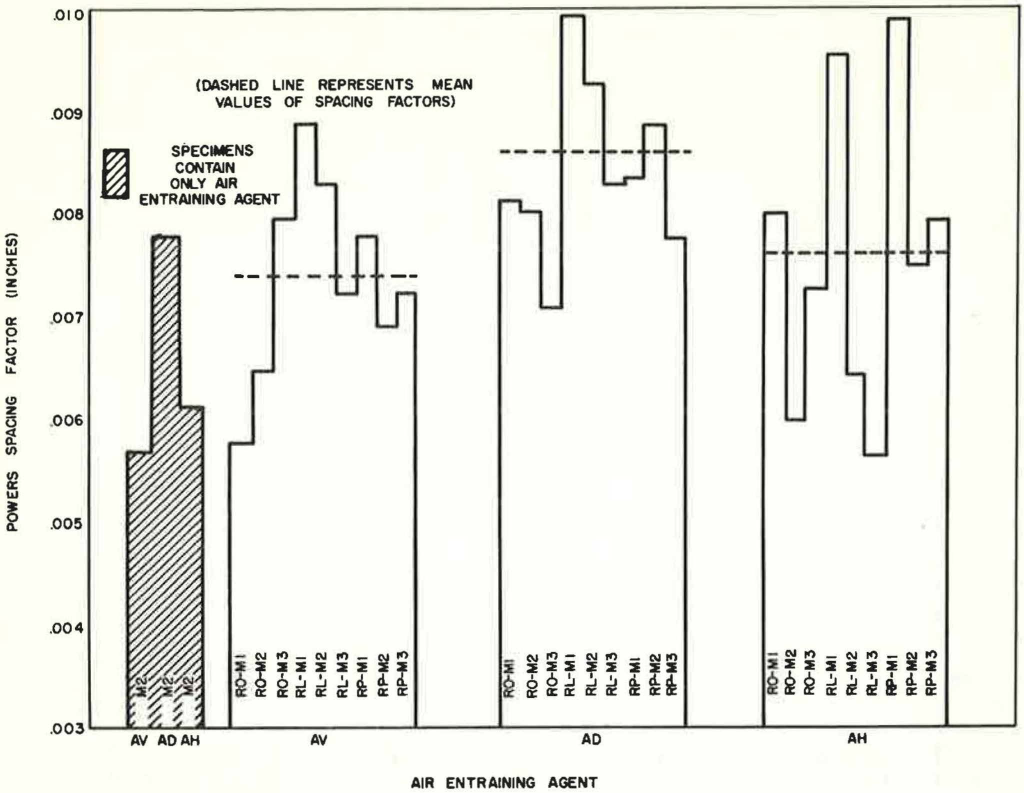

Figure 7. Comparison of Powers spacing factor between air-entraining agents.

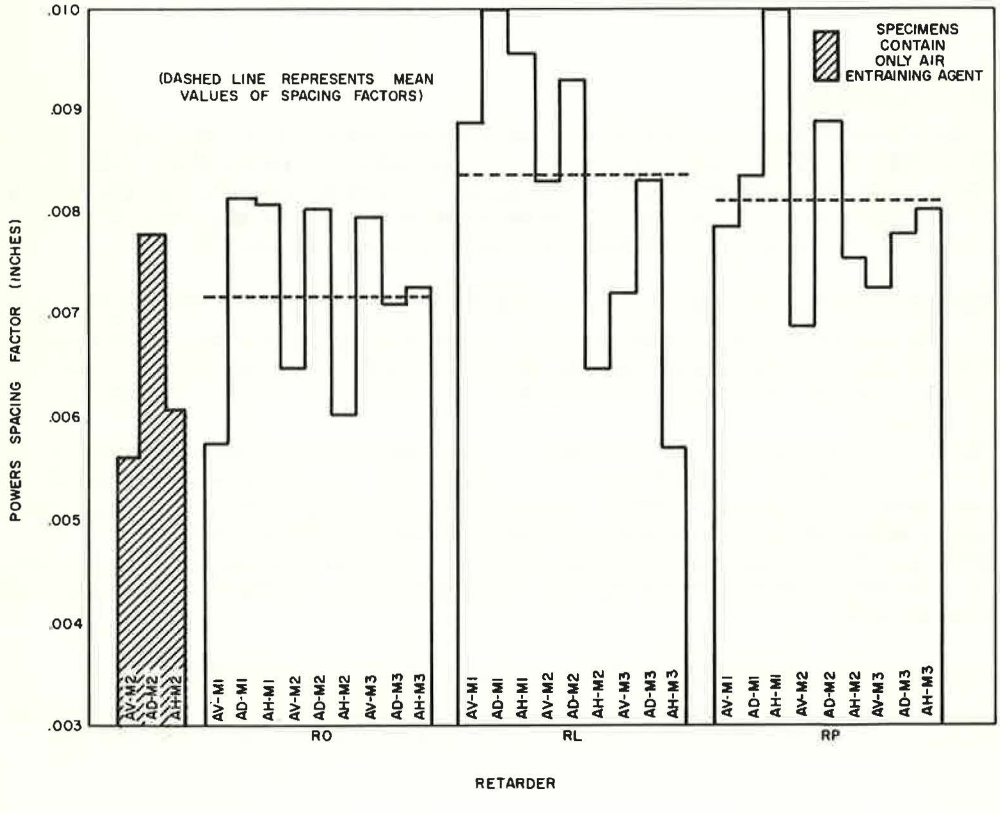

Figure 8. Comparison of Powers spacing factor between retarders.

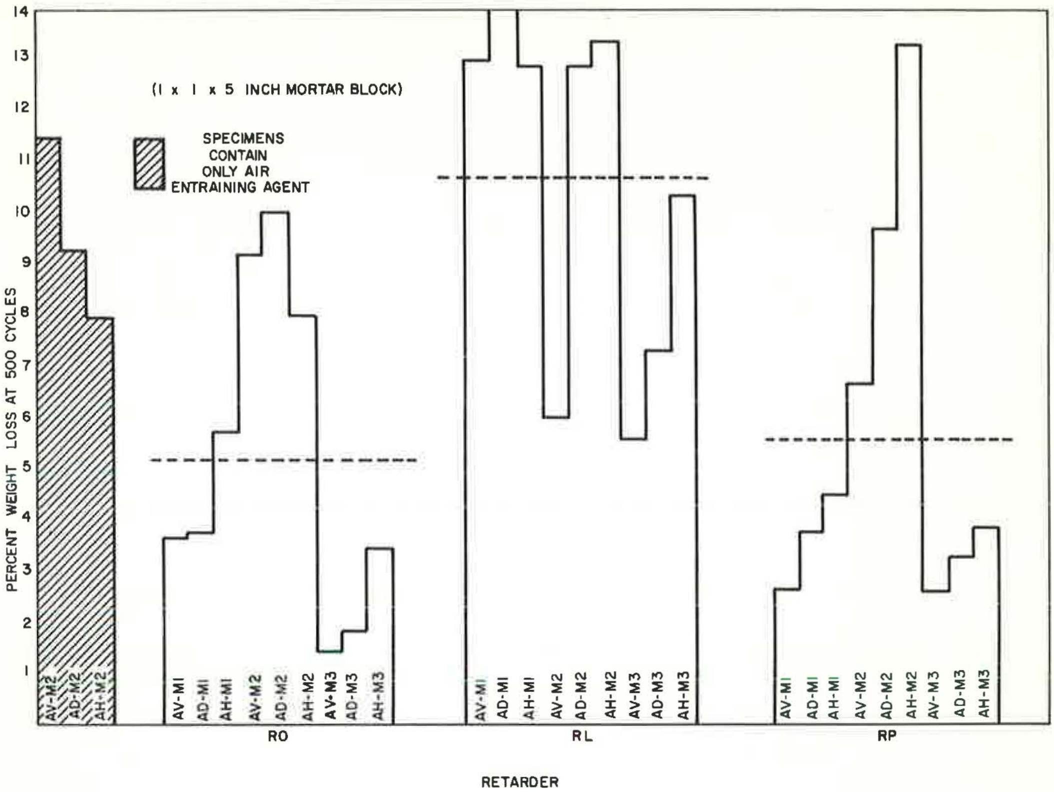

Figure 9. Comparison of freeze-thaw weight loss between admixtures.

Effect of Retarders

A significant difference at the 99 percent level existed between different combinations of retarders and air-entraining agents when compared on the basis of the Powers spacing factor (Fig. 8). The difference in the mean values of RO and RL was significant at the 99 percent level, whereas the difference between RO and RP showed a 95 percent significance. No significant difference was noted between RP and RL.

Interaction Effects of Mixing Sequences, Air-Entraining Agents and Retarders

The interaction of mixing sequences, air-entraining agents and retarders was found to produce values of the Powers spacing factor with differences significant at the 95 percent level; however, no significance was found to exist between different values of the Philleo factor.

When both retarders and air-entraining agents were used, introduction into the batch in accordance with the method described in mixing sequence M1 was least desirable.

With the three mixing procedures and the three retarders used, it was found that air-entraining agent AD performed less satisfactorily than did AV and AH.

Under the three conditions of mixing and in combination with the three air-entraining agents, retarder RO was found to be less detrimental to the development of a desirable air-void system.

Resistance of Mortar to Deterioration as a Result of Freezing and Thawing

Two 1 by 1 by 5-in. mortar specimens were cast from each mortar batch. These specimens were moist cured for 14 days in 100 percent relative humidity and then

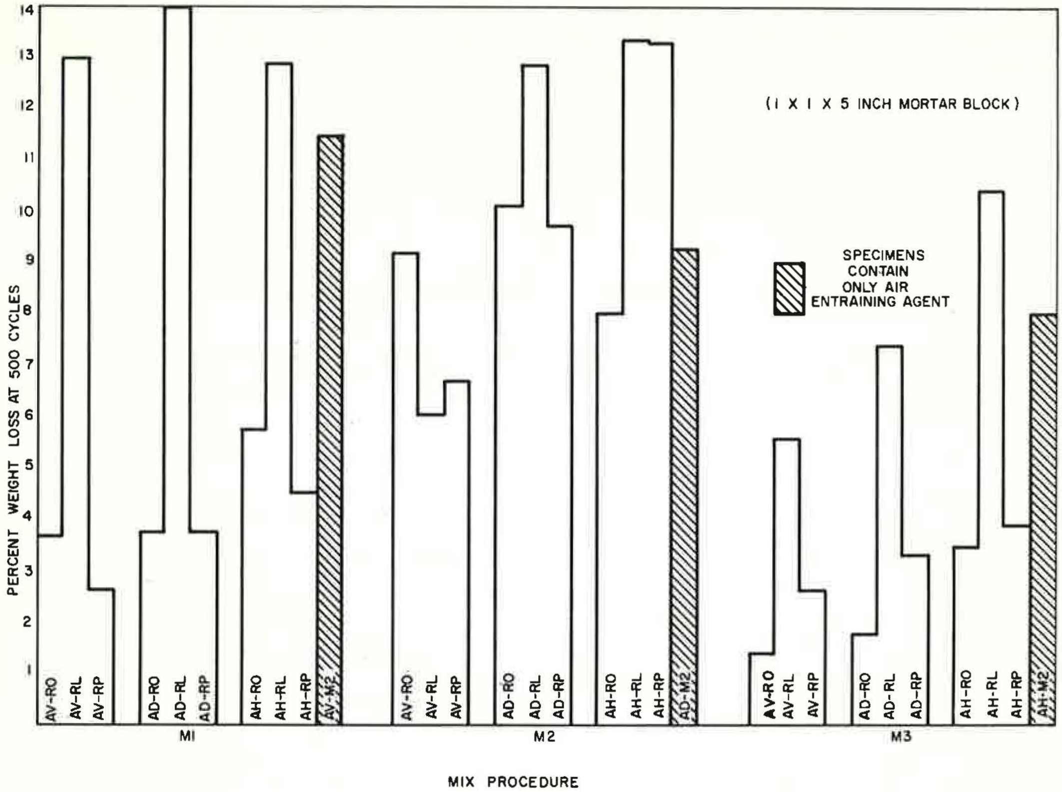

Figure 10. Comparison of freeze-thaw weight loss between air-entraining agents and mixing methods.

subjected to rapid freezing and thawing in water in accordance with ASTM C 290-63T. Weight loss of the specimens was progressively recorded until termination at 500 cycles.

It has not been shown that weight loss in mortar is a good criterion for predicting concrete freeze-thaw durability. However, significant discernible trends do seem apparent between the air-entraining agents and retarders (Fig. 9) and mixing procedures. One indication of correlation between this test and concrete freeze-thaw durability, as indicated by loss in resonant frequency, is shown by observing the similarity of these differences between retarders and the differences shown in Research Report 70-3 (8).

Data in Figure 10 indicate lower weight losses occur in those specimens mixed in accordance with procedure M3.

CONCLUSIONS

1. Of the three mixing procedures, significantly larger values of the spacing factor were found in mortars where the air-entraining agent and retarder were mixed in the same water phase and introduced into the sand and cement (M1, Table 2).

2. The air-entraining agents investigated differed in their abilities to produce a system of closely spaced air voids. The vinsol resin (AV) and the organic salt of sulfonated hydrocarbon (AH) produced an entrained-air system of closely spaced voids as indicated by the smaller values of the spacing factors.

3. The lignosulfonate (RL) and hydroxylated polymer (RP) retarders, used in combination with the three air-entraining agents, produced systems of voids with larger spacing factors than did the organic acid retarder (RO) when used with the same air-entraining agents.

4. Lower weight losses due to freezing and thawing were observed in mortar specimens which were mixed by first introducing the air-entraining agent and one-half the mixing water into the sand and blending before adding the other constituents (M3, Table 2).

5. Comparatively larger values of weight loss due to freeze-thaw were observed in mortar specimens containing the lignosulfonate retarder (RL).

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Review of a Three-Year Bridge Deck Study in Pennsylvania

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The results of the initial three years of continuing research project on the durability of concrete bridge decks in Pennsylvania are presented. The study involved field surveys of 38 bridge decks comprising 2,782 ten-ft long survey units. A total of 154 cores were taken from 34 of the decks. The cores were subjected to detailed laboratory analyses including determination of air void parameters by linear traverse techniques, w/c ratio determinations, and petrographic examination. Seven bridge decks were observed during construction to examine and evaluate the effect of construction practices on durability. Bridge decks were resurveyed annually to establish rates.

The major types of deterioration found were transverse cracking, fracture planes, potholes, and surface mortar deterioration. The primary causes of deterioration were indicated to be materials (aggregates) and workmanship (overfinishing, poor quality control with respect to entrained air and w/c, and improper placement of reinforcement). Recommendations to alleviate the causes of poor performance and suggested areas of needed research are included.

•A CONTINUING study of deterioration of concrete bridge decks in Pennsylvania was undertaken by Pennsylvania State University in 1964 under the auspices of the Pennsylvania Department of Highways and the Bureau of Public Roads. The results, conclusions, and recommendations based on the initial three years of the study are summarized.

The broad purpose was to investigate the causes and remedies of concrete bridge deck deterioration in Pennsylvania. To accomplish this goal, 38 bridge decks were field surveyed and 154 core samples were taken from 34 of them. A total of 2,782 ten-foot long by one traffic lane wide sections were surveyed. The core samples were subjected to detailed laboratory analyses which include determination of air-void system parameters, water-cement ratio determinations, and petrographic examinations. Annual resurveys were conducted to define the rate of advances in deterioration. To observe and evaluate the role of construction practices on bridge deck durability, 7 decks were observed under construction. Several special studies were also undertaken to investigate particular areas of interest that would contribute to the knowledge of bridge deck durability.

The observed bridges were selected on the basis of their particular interest to the Highway Department. They comprise a small but representative sample of deteriorating structures in 10 of the 11 Pennsylvania highway districts. The approximate location of these bridges is shown in Figures 1, 2, and 3.

MAJOR TYPES OF DETERIORATION

The major types of deterioration found on Pennsylvania bridges were transverse cracking, fracture planes, potholes, and surface mortar deterioration. Less common

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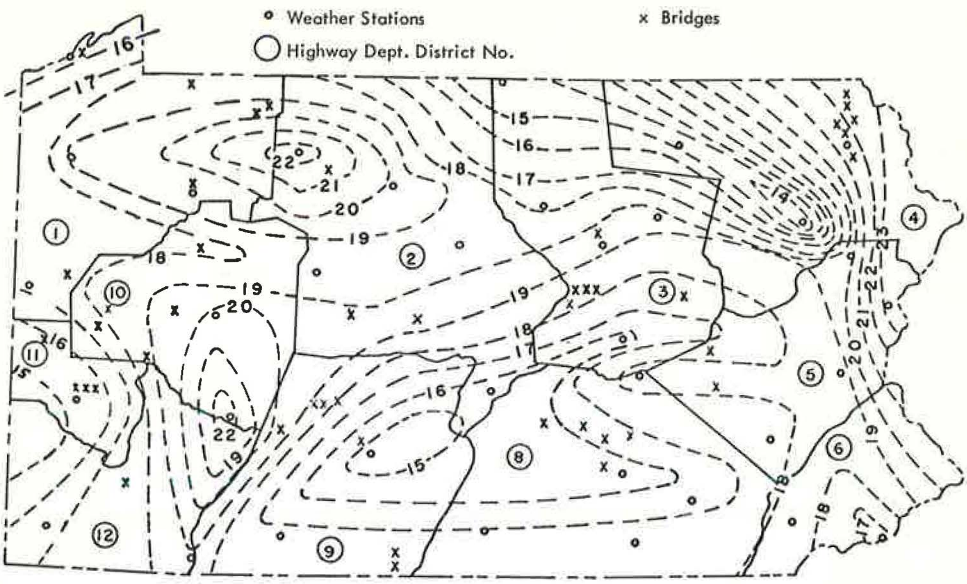

Figure 1. Average annual rainfall during the winter season (in.), 1950-1966.

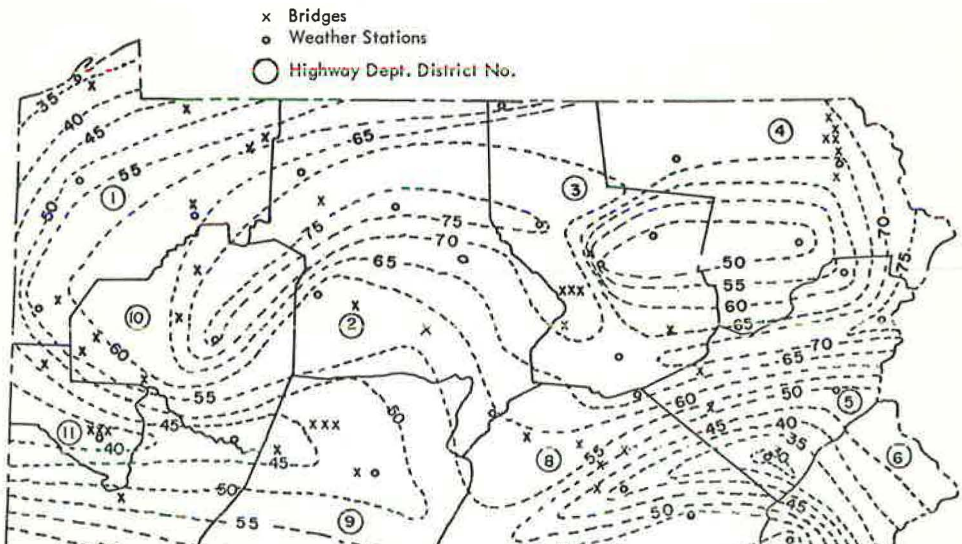

Figure 2. Average yearly freeze-thaw cycles, 1950-1966.

types of distress encountered were longitudinal, diagonal and pattern cracking, hair checking, pitting, and popouts. Examples of the four major types of deterioration are shown in Figures 4 through 7.

The terminology used in classifying deterioration and the rating procedure employed were adopted from Axon, et al (2).

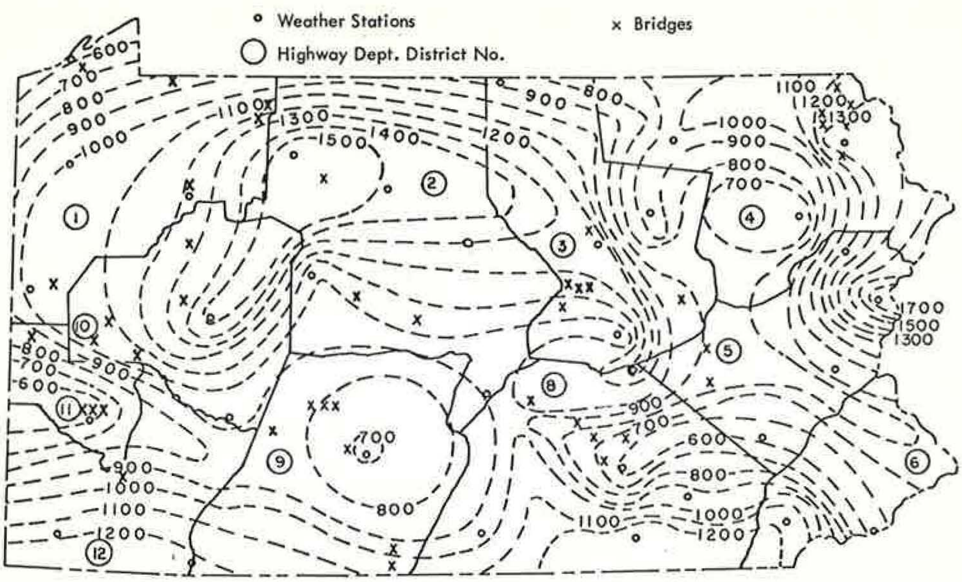

Figure 3. ASTM weathering index, 1950–1966.

Figure 4. Example of a major transverse crack.

Figure 5. Core showing a fracture plane at the top reinforcing steel.

Figure 6. Example of a pothole.

Figure 7. Severe surface mortar deterioration; as much as 2 in. of the deck has eroded away.

Table 1 shows the extent of the major types of deterioration found during each study year and an average for the 3-yr study; resurvey data are not included. Surface mortar deterioration occurred on 53.5 percent of the 10-ft sections. The next most common type of distress, transverse cracking, occurred on 35.9 percent of the sections, followed by potholes and fracture planes at 14.6 and 13.9 percent, respectively. An average of 14.1 percent of the sections showed no defects.

CAUSES OF DETERIORATION

Several factors influence the deterioration of concrete bridge decks. The 3-yr study indicated that the most important factors are (a) cracking, (b) entrained air, (c) aggregate, (d) construction practices, (e) degree of quality control maintained, and (f) the severity of the service environment.

Cracking

Although cracking is itself a manifestation of deterioration, it is also the most important forerunner of other modes of destruction. This type of distress is mainly attributable to traffic loads, resistance by reinforcement to subsidence of the concrete, drying and plastic shrinkage phenomena, and design.

High early loads and the repetition of large live loads with time have the most obvious influence on the development of transverse cracks. Cracking due to resistance to subsidence is caused by the settling of the plastic concrete under the rigidly supported, top reinforcing steel. Concrete above the steel does not have freedom for vertical movement as does the concrete between bars. When settlement occurs the result is a pulling apart of the concrete over the bars and a vertical crack is created. Plastic shrinkage is caused by an excessively rapid evaporation of water from the concrete surface. It is influenced by the temperature, relative humidity, and wind velocity of the surrounding air. When the evaporation rate exceeds the bleeding rate of the concrete, hydrostatic tension is likely to cause plastic shrinkage cracks.

TABLE 1

FREQUENCY OF MAJOR TYPES OF DETERIORATION

Study Year	No. of 10-Ft Sections	Type of Deterioration: Transverse Cracking, No. of Sections	Transverse Cracking, Percent	Fracture Planes, No. of Sections	Fracture Planes, Percent	Potholes, No. of Sections	Potholes, Percent	Surface Mortar Deterioration, No. of Sections	Surface Mortar Deterioration, Percent	No Defects, No. of Sections	No Defects, Percent
1964-65	256	132	51.5	91	35.5	98	38.3	177	69.1	10	3.9
	10	0	0.0	0	0.0	0	0.0	4	40.0	0	0.0
	62	33	53.2	16	25.8	16	25.8	62	100.0	0	0.0
	236	82	34.7	1	4.2	0	0.0	67	28.4	13	5.5
	22	7	32.7	10	45.4	9	40.9	22	100.0	0	0.0
	62	11	17.7	3	4.8	3	4.8	26	41.9	11	17.7
Subtotal	648	265	40.9	121	18.7	126	19.4	358	55.2	34	5.2
1965-66	12	0	0.0	0	0.0	3	25.0	12	100.0	0	0.0
	172	2	1.2	0	0.0	3	1.7	40	23.3	78	39.5
	34	30	88.2	0	0.0	3	8.8	21	61.8	0	0.0
	170	121	71.2	11	6.5	23	13.5	126	74.1	6	3.5
	518	146	28.2	2	0.4	19	3.7	372	71.8	12	2.3
	10	8	80.0	4	40.0	6	60.0	10	100.0	0	0.0
	40	38	95.0	6	15.0	2	5.0	35	87.5	0	0.0
Subtotal	956	345	36.1	23	2.4	59	6.2	616	64.4	96	10.0
1966-67	154	112	72.7	47	30.5	42	27.3	8	5.2	23	14.9
	184	8	4.3	5	2.7	7	3.8	48	26.1	123	66.8
	172	97	56.4	19	11.0	23	13.4	78	45.3	29	16.9
	296	76	25.7	40	13.5	23	7.8	250	84.4	16	5.4
	144	52	36.1	39	27.1	38	26.4	92	63.9	9	6.2
	300	38	10.0	07	43.5	84	42.0	12	6.0	62	31.0
	28	7	25.0	5	17.8	5	17.8	27	96.4	0	0.0
Subtotal	1,178	390	33.1	242	20.5	222	18.8	515	43.7	262	22.2
TOTAL	2,782	1,000	35.9	386	13.9	407	14.6	1,489	53.5	392	14.1

TABLE 2

CRACKS PER SECTION FOR SIMPLE AND CONTINUOUS CONCRETE AND STEEL-SUPPORTED BRIDGES

Type of Support	Type of Span							
	Simple				Continuous			
	No. of Bridges	No. of Spans	No. of Sections	Cracks per Section	No. of Bridges	No. of Spans	No. of Sections	Cracks per Section
Concrete	9	18	190	1.3	—	—	—	—
Steel	5	13	140	1.1	—	—	—	—
Steel	—	—	—	—	10	48	750	1.2
Steel	5	9	112	0.05	5	19	640	0.9

With the occurrence of cracking, other types of distress are likely to follow. Cracks facilitate the ingress of water and deicing chemicals which attack the reinforcing steel. Corrosion of the steel produces expansive pressures in the surrounding paste which may produce a fracture plane. Also, the ingress of water produces a saturated zone in the plane of the reinforcing steel which is vulnerable to the destructive action of frost. When the concrete above the fracture plane is lost, a pothole is created. These processes are usually accelerated if the steel has a shallow concrete cover. Cracking is thus a progressive type of distress leading to more serious types of deterioration.

Cracking and Bridge Design

The frequency of occurrence of transverse cracks has been studied with respect to span design type. Spans were classed as concrete (simple spans) and steel (simple or continuous spans).

There were 9 bridges in the first category, T, I, and box beam bridges, and they had an average of 1.3 cracks per 10-ft section. Two of these bridges had no cracks and one had as many as 2.7 per section. The 9 bridges represent 18 spans and 190 sections.

The simple spans of steel-supported bridges had an average of 0.6 cracks per section with a range of 0 to 2.4. There were 10 bridges in this category totaling 22 spans and 252 sections. The 67 continuous spans had 1,390 sections on 15 bridges. They showed an average of 1.1 crack per section and the range was from 0 to 3.3. Five of the steel-supported bridges had both simple and continuous spans. Both types of spans have nearly the same frequency of cracking except for the simple spans of bridges that have both simple and continuous spans. These spans show very little cracking, averaging less than one-tenth of a crack per section. Four of the 9 spans in this group showed no cracking. The reason for the low frequency of transverse cracking on these spans could be attributable to the fact that they are relatively short approach spans. These observations are summarized in Table 2.

Cracking has also been related to the skew of the bridge and length of spans for various types of bridges. Linear correlations were made between these two variables (Table 3). Only the simple spans of steel-supported bridges showed a correlation at the 95 percent significance level between skew and number cracks per section. When cracking and length of span were analyzed, again only the simple spans of steel supported bridges correlated at the 95 percent level. Hence, it appears that with increasing span lengths and skews on simple steel spans the frequency of cracking increases.

TABLE 3

CORRELATION BETWEEN CRACKS PER FOOT AND DEGREE OF SKEW AND SPAN LENGTH

Parameters	Type of Bridge	No. in Group	Significant at 95% Level
Cracking vs skew	Concrete—simple spans	9	No
	Steel—simple spans	5	Yes
	Steel—continuous spans	10	No
Cracking vs span length	Concrete—simple spans	18	No
	Steel—simple spans	13	Yes
	Steel—continuous spans	48	No

Entrained Air in Hardened Concrete

The primary function of air voids is to protect the hardened concrete from frost damage. They provide chambers into which water can enter when forced by hydraulic pressure during freezing and thus reduce disruptive pressures that might otherwise result. If there are no voids to accommodate the migrating water, frost damage in the form of surface mortar deterioration is likely to occur.

For the voids to be effective in reducing hydraulic pressures, they must be of adequate number and size, well distributed throughout the concrete, and close enough to allow water to migrate to them with minimum resistance. The larger size voids, greater than 1,000 μ, are not effective in the protection of the concrete from frost action, and they result in a decrease in strength. This type of void is called entrapped air; voids less than 1,000 μ are called entrained-air voids and provide the necessary protection for the concrete.

There are, however, deteriorating bridge decks which supposedly were made with concretes having an approved air content. Surface scaling, in particular, seems to be a common type of deterioration in these cases. Several reasons may be advanced for this: (a) the air content of the affected concrete is less than the reported average test values, (b) deterioration mechanisms other than freezing and thawing are involved, (c) the general quality of the affected concrete is so low that entrained air cannot protect it, and (d) air volume is adequate, but the size, distribution, and spacing of the air bubbles are not locally in harmony with other compositional features of the concrete.

Air Content and Deterioration

The locations from which core samples were taken on the bridge decks were recorded and the air content was determined for each core by linear traverse methods. Table 4 indicates that, of the the 154 core samples taken during the three year study, 91 (59%) had less than 4 percent total air content and 79 (87%) of the 91 cores came from distressed areas. Comparable information is given for entrained air content for only the last year of the study because it was not collected during the other two years' work. These data emphasize that if a concrete has a low air content, surface mortar deterioration can be expected.

TABLE 4

SUMMARY OF LABORATORY FINDINGS FOR AIR CONTENTS

Highway District No.	County	No. of Bridges Cored	No. Cores Taken	No. Cores Less than 4% Total Air	No. Cores Less than 4% Entrained Air	No. Cores Less than 4% T.A. from Distressed Areas	No. Cores Less than 4% E.A. from Distressed Areas
	(a) 1964-1965 Study						
8-0	Cumberland	1	7	3	—	3	—
9-0	Blair	1	3	3	—	2	—
9-0	Cambria	3	12	2	—	2	—
8-0	Dauphin	1	6	5	—	4	—
2-0	Elk	1	9	9	—	8	—
9-0	Fulton	2	10	8	—	6	—
	Subtotal	9	47	30	—	25	—
	(b) 1965-1966 Study						
3-0	Lycoming	1	3	3	—	3	—
3-0	Union	1	3	1	—	1	—
4-0	Lackawanna	1	3	3	—	3	—
4-0	Susquehanna	4	13	12	—	11	—
5-0	Berks	2	12	2	—	1	—
8-0	Perry	1	4	4	—	4	—
8-0	York	1	4	4	—	4	—
	Subtotal	11	42	29	—	27	—
	(c) 1966-1967 Study						
1-0	Forest	1	5	2	2	2	2
1-0	Warren	3	12	4	5	4	4
10-0	Armstrong	1	7	4	4	4	4
10-0	Butler	4	18	7	9	5	7
11-0	Allegheny	3	12	10	11	7	8
11-0	Beaver	1	6	1	2	1	2
12-0	Westmoreland	1	5	4	4	4	4
	Subtotal	14	65	32	37	27	31
	TOTAL	34	154	91	—	79	—

Air-Void System Studies

During the 1965-1966 study year, an investigation was conducted to search for and evaluate a new air-void system parameter. The parameter which offered the most promise was suggested by R. E. Philleo and involved what he called protected paste volume concept. It required an accurate estimate of the number of air bubbles per unit volume of paste and to meet this demand, a new method of describing bubble size distribution was developed. This distribution was based on the chord distribution and approximated from a frequency distribution function. A laboratory test program was conducted to evaluate the two parameters called the Philleo spacing factor and the number of bubbles per cubic centimeter of paste by the Penn State method. Using dilation of test specimens as a criterion of durability, the new parameters and the other common parameters were correlated with durability by a least squares linear regression method of analysis. The two new parameters showed the best correlations, followed by voids per inch, the Powers spacing factor, total air content, and the number of bubbles per cubic centimeter of paste by the Lord and Willis method.

This study prompted another in which the effect of entrapped air on the air void system parameters and on the correlation of the parameters with durability were investigated. Entrapped air was eliminated in increments from the calculation of the air-void parameters and after each iteration a correlation was made between the parameters and durability of laboratory made specimens. New data collected in the laboratory, data from the previous study, and data obtained from field core samples taken from the bridge decks were used. The major findings were (a) that there are relatively few entrapped-air voids in laboratory made specimens (approximately 1% of the total number of voids) but they have a significant effect on the numeric value of some of the air-void parameters; (b) as the degree of surface mortar deterioration increases on successive bridge decks, so does the effect of entrapped air on the air-void system parameters; (c) on bridges that were in relatively sound condition, the effect of entrapped air on the parameters was comparable to that of laboratory specimens; (d) air content and number of voids per inch are not reliable indicators of durability when they are calculated using the total air content, but they are reliable indicators when they are calculated from the entrained-air content; (e) specific surface and average chord length alone are not good indicators of durability.

Distribution of Air in Deck Cores

Although a concrete may have, on the average, an adequate volume of entrained air, it is not necessarily protected from frost damage. One of the criteria for frost resistance, an adequate distribution of entrained-air bubbles, must be satisfied to insure protection; a nonuniform distribution renders portions of the concrete susceptible to freeze-thaw damage.

The distribution of air within the core samples taken during this project was analyzed in two ways: air contents were determined and compared for the top, middle and bottom thirds of the cores, and air contents on the deck surface were measured for several cores.

An analysis of the distribution of air within 62 core samples by a statistical sign test revealed that there is no significant difference (at the 95 percent significance level) between the air content in the top third of the cores compared with the air contents in the bottom third, middle third, or the average of the air in the bottom and middle thirds. In approximately one-half the cores examined, the air content was lower in the top third, and in the other half of the cores the air content was higher in the top third. In several cases, the highest air content was in the middle third of the core. There was no observed pattern in the air distribution from top to bottom. However, an analysis of the top wearing surface of 29 cores showed that the air contents in the top surfaces of 18 cores were less than the air contents in the body of the same cores (Table 5). The paste volume of the top surface of the cores was from 16 to 126 percent higher than the paste volumes in the body of the cores, which means that the air volume at the top surface is distributed throughout a much larger paste volume and, therefore, would be less effective. To demonstrate the reduction in effectiveness, the air volumes at the top of the

TABLE 5

COMPARISON OF AIR CONTENT AT THE CORE SURFACE WITH AIR CONTENT FOR THE WHOLE CORE

Legislative Route No.	Sta. No.	Core No.	Top Surface		Whole Core		Equivalent Air at Top Surface[a] (%)
			Air (%)	Paste (%)	Air (%)	Paste (%)	
41049	1 + 66	1	1.1	50.7	1.2	28[b]	0.5
41049	1 + 66	2	2.6	54.1	2.9	27.9	1.6
1009	964 + 54	3	1.8	51.3	2.1	28[b]	1.1
1001	134 + 28	3	2.1	56.4	1.7	28	1.0
1001	303 + 83	1	1.8	46.5	2.0	26.3	1.1
1001	393 + 40	3	3.4	63.2	2.2	28[b]	1.5
286	23 + 52	4	4.0	40.9	3.9	28[b]	2.9
195	126 + 44	2	1.4	52.9	0.4	32[b]	0.8
333	1434 + 34	2	0.5	60.5	1.2	32.8	0.3
511	11 + 18	3	4.4	48.4	4.9	28.8	2.6
		5	4.4	46.5	3.8	28.0	2.5
61064	1 + 19	3	2.8	47.7	4.3	29.4	1.7
209-4	459 + 10	2	7.3	42.5	5.9	23.2	4.0
		3	7.2	47.2	5.2	26.4	4.0
209-5	507 + 06	4	9.6	51.5	7.3	25.1	4.7
71	136 + 00	1	4.9	45.9	6.8	29.6	3.2
		4	4.2	54.0	7.4	27.7	2.2
10027	43 + 26	3	0.3	42.7	1.2	31.7	0.2
		4	0.3	49.9	1.1	31.2	0.2
78	468 + 59	1	5.2	38.8	6.1	30.9	4.1
1021	516 + 00	1	3.6	45.9	3.8	29.9	2.3
		3	2.4	50.0	4.4	30.6	1.5
1021	1446 + 81	2	4.5	33.8	6.8	25.7	3.4
		4	6.4	45.3	6.1	28.4	4.0
70	62nd St.	5	3.7	41.1	3.8	31.7	2.8
802	229 + 84	1	1.9	39.7	1.9	29.2	1.4
		3	2.0	32.7	1.7	28.1	1.7
1057	197 + 80	4	0.2	37.6	0.4	24.7	0.1
51	104 + 55	4	1.5	45.2	1.9	27.9	0.9

[a]Calculated by proportioning the air content at the top surface to an equivalent air content for the paste content of the whole core.
[b]Assumed paste content, based on mix proportioning data.

cores were recalculated based on a paste volume equivalent to the paste volume in the main body. On this basis the volumes determined at the top of the cores were less than those for the body in 28 of the 29 cases.

In summary, it has been found that there is no significant difference in the air content in the top third compared with the air in the rest of the core. However, the air content at the top wearing surface is typically lower than in the main body of the core. A deficiency of entrained air at the surface leaves the most vulnerable portion of the deck unprotected and surface mortar deterioration is probable in such situations.

Performance of Aggregates in Concrete

The paste phase in concrete can be made durable by proper proportioning and by rigid quality control during construction. However, such deterioration mechanisms as freezing and thawing, carbonaceous and silicious aggregate reactions, and possibly, the growth of ice bodies can also operate in the aggregate phase. The need for sound mineral aggregates cannot be overemphasized inasmuch as aggregate comprises about 70 percent by volume of all concrete.

The selection of aggregates for bridge decks deserves particular attention because (a) the top surface exposure is more severe than on the roadway, both with regard to temperature and deicer application, (b) dynamic loading effects are more pronounced, and (c) the concrete is often mixed, placed, and cured in ways inferior to those used on the roadway itself.

Petrographic examination of cores from 22 of the 34 sampled bridges showed that the coarse aggregates were performing poorly. The gravel coarse aggregates in 10 bridge decks were fracturing along bedding planes. The stone aggregates in another 10 decks were fracturing also. Some slag coarse aggregates in 2 bridges were corroded. A typical example of freeze-thaw fracturing of an aggregate particle in a bridge deck core is shown in Figure 8; the crack between the particle and the paste illustrates the disruptive volume change that had occurred.

Construction Practices

The durability of bridge deck concrete is most dependent on the quality of materials used in the concrete and the workmanship involved in the proportioning, mixing, placing, finishing, and curing. To evaluate some of the construction factors, 7 bridge decks were observed during construction. Construction practices which may have a significant effect on performance and factors which may have influenced these practices were noted.

The most commonly observed construction practices that are likely to affect the quality of the concrete adversely were sprinkling and reworking the deck surfaces and delays between the end of the finishing and beginning of curing procedures. These practices were noted on all 7 decks.

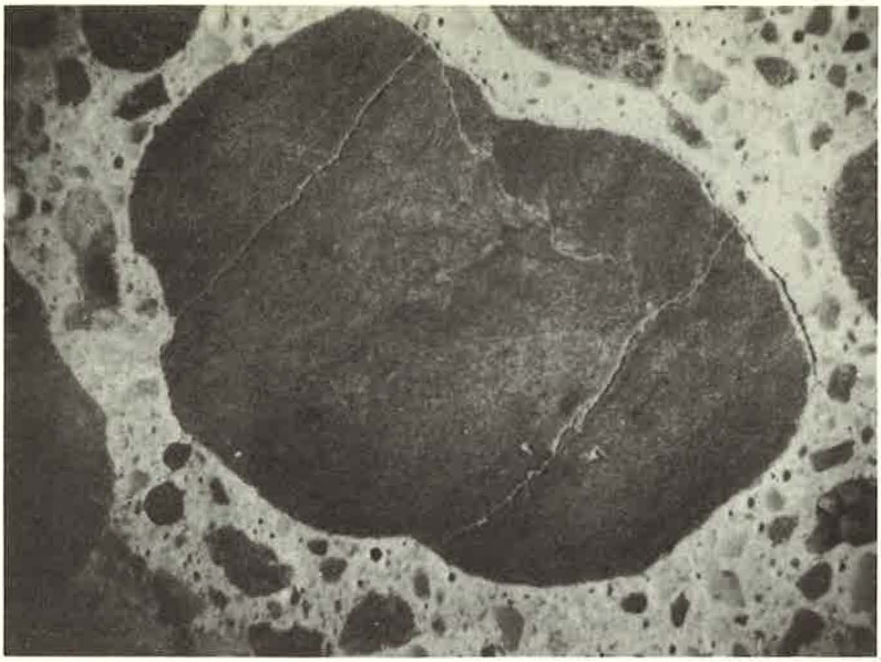

Figure 8. A typical aggregate fracture (mag. 6X).

The result of sprinkling and reworking a concrete surface is the production of a high w/c ratio surface layer, deficient in air voids. Immediately below this may be a zone of high porosity and even lower strength due to a high concentration of bleed water and air. The concrete below these zones is relatively unaffected. This layered system is subject to failure for several reasons: (a) a concentration of water or salt solution can exist in the porous layer, (b) the surface is deficient in air, and (c) differential thermal coefficients are created in these layers.

The procedure for correctly finishing a deck is a complicated one. Evaporation of water, caused by high temperatures, low humidity, and wind, requires additional water if the concrete is not finished immediately after placement. This was true on several of the bridge decks. When air temperatures were in the 80's and 90's and there were mild or periodic winds (the humidity was not recorded), there were serious difficulties in finishing the decks even though set retarders were used (the use of set retarders has not been entirely satisfactory, but they appear to offer some improvement). Delays caused by arrival of trucks and placement procedures resulted in delay of the start of finishing which in turn complicated the finishing process.

The reworking of bridge decks is often necessitated by the requirement for smooth riding surfaces which are true to grade, line, and cross section. Although this is a basic requirement, it leads to overfinishing.

Delays of up to three or more hours have been observed between the end of finishing procedures and the initiation of curing. On several of the decks, areas of drying were noted and on one bridge the workmen were able to walk on the deck to place the burlap for curing without indenting the surface.

Location of Reinforcing Steel

The location of the reinforcing steel in a bridge deck slab may have an effect on its durability. Steel placed near the surface is more vulnerable to chemical attack from deicer salts than steel well imbedded in the concrete. In core samples taken from

Figure 9. Shattered paste resulting from steel corrosion (mag. 12x).

Figure 10. Deteriorated surface showing exposed reinforcing bars; the steel is close to the top surface of the deck.

distressed bridge decks, one of the causes of deterioration has been the corrosion of the steel (Fig. 9).

Of the 7 bridges observed under construction, the minimum depth of steel in 4 was $1\frac{1}{2}$ in. The steel depth was not measured in 1 deck, and in the other 2, measurements were made from the bottom of corrugated metal forms to the top steel. This does not give the actual depth of steel from the top surface but a theoretical depth can be calculated. The theoretical depth in all cases was at least $2\frac{7}{8}$ in.; however, coverage of the top reinforcing steel was as little as $\frac{1}{2}$ in. on some bridges (Fig. 10).

The steel appeared to be adequately tied and supported on all but 1 deck, where the steel was poorly supported and the weight of the construction crew caused the reinforcing mat to sag. In some areas, it almost touched the lower reinforcing bars. In cases where the reinforcing steel was not positioned according to the design, modifications in the structural behavior of the slab result and cracking may occur.

Quality Control

Slump and air content determinations were made on essentially all truckloads of concrete placed on each bridge deck observed under construction. The average test values for each truckload have been grouped according to their relation to Pennsylvania Department of Highways specifications for Class AA concrete which is specified for bridge decks (Table 6).

For 2 of the decks, the average slump of the concrete in 50 percent of the truckloads was more than the specification limit of 3 in. On 1 bridge, as many as 68 percent of the slumps were above the specifications. There was one average slump less than 1 in. Of all the average slumps for the 7 bridges, 66 percent were within specifications (1 to 3 in.), 34 percent above, and an insignificant number below specifications.

The current highway department specification for air content is $6\frac{1}{2} \pm 1\frac{1}{2}$ percent or a range of 5 to 8 percent. An overall average of 57 percent of the average air contents were within the specifications, 34 percent below, and 9 percent above. The bridge which had 68 percent of the slumps above specifications had 84 percent of the air contents within specifications. Another bridge deck had as many as 65 percent of the average air contents below 5 percent.

These results indicate that significant amounts of the concrete placed on bridge decks are not within specifications. There is a lack of uniformity in the concrete on the decks among the bridges and in the concrete placed on the same bridge.

TABLE 6

NUMBER OF SLUMP AND AIR CONTENT DETERMINATIONS ABOVE, BELOW, AND WITHIN PDH SPECIFICATIONS

Bridge Identification (district, county, L.R. No., Sta.)	No. of Slumps			No. of Air Contents		
	<1 in.	1 to 3 in.	>3 in.	<5 Percent	5 to 8 Percent	>8 Percent
2, Clearfield: L.R. 1009-30, Sta. 225+46	0	18	3	10	11	0
3, Columbia: L.R. 609, Sta. 214+25	0	52	3	36	19	0
9, Cambria: L.R. 1022, Sta. 622+43	0	20	15	16	19	0
10, Clarion: L.R. 1009-8, Sta. 501+83	0	19	21	2	26	9
8, Perry: L.R. 1033-3, Sta. 712+57	1	14	8	9	10	4
1, Erie: 38th St.	0	17	11	0	23	5
2, Centre-Clearfield: L.R. 1009-35, Sta. 1166+19	0	6	13	2	16	1

TABLE 7

CALCULATED W/C RATIOS AND CEMENT FACTORS FOR CORE SAMPLES

Bridge Identification	Values Proportioned		Calculated Values		
	W/C (by wt.)	C.F. (sk/yd)	No. of Cores	Avg. W/C (by wt.)	Avg. C.F. (sk/yd)
L.R. 209, Sta. 459+10	0.43	6.25	4	0.48	5.85
L.R. 209, Sta. 507+06	0.43	6.25	4	0.45	6.40
L.R. 10027, Sta. 43+26	0.43	6.25	5	0.53	6.49
L.R. 71, Sta. 136+00	0.49	6.00	5	0.59	5.70
L.R. 1021, Sta. 516+00	0.45	6.00	5	0.67	5.32
L.R. 1021, Sta. 1446+81	0.44	6.25	4	0.59	5.33
L.R. 41049, Sta. 1+66	0.49	6.00	2	0.46	6.90
L.R. 1009, Sta. 964+54	0.45	6.00	2	0.38	7.50
L.R. 790, Sta. 331+41	0.46	6.00	2	0.49	6.77
L.R. 1001, Sta. 35+75	0.47	6.00	3	0.49	7.26
L.R. 1001, Sta. 134+28	0.47	6.00	2	0.46	6.22
L.R. 1001, Sta. 303+83	0.47	6.00	2	0.45	7.09
L.R. 1001, Sta. 393+40	0.46	6.00	2	0.49	7.05
L.R. 286, Sta. 23+52	0.47	6.00	4	0.53	6.39
L.R. 784, Sta. 37+50	0.50	6.10	5	0.48	5.95
L.R. 195, Sta. 126+44	0.49	6.00	4	0.77	4.58
L.R. 333, Sta. 1434+34	0.47	6.00	4	0.43	7.99

Compositional Analysis of Hardened Concrete

A laboratory investigation of a test method for estimating the original composition of hardened concrete was conducted. Analysis of laboratory-made specimens shows the average error in the determination of the water-cement ratio to be approximately 5 percent; and for cement content, approximately 6 percent.

The test method was applied to 59 core samples from 17 bridge decks. Selection of the cores was based on the availability of mix proportion data; the results are given in Table 7. In several cases, the calculated value for water-cement ratio and cement content were in close agreement with the specified values and within the error found in

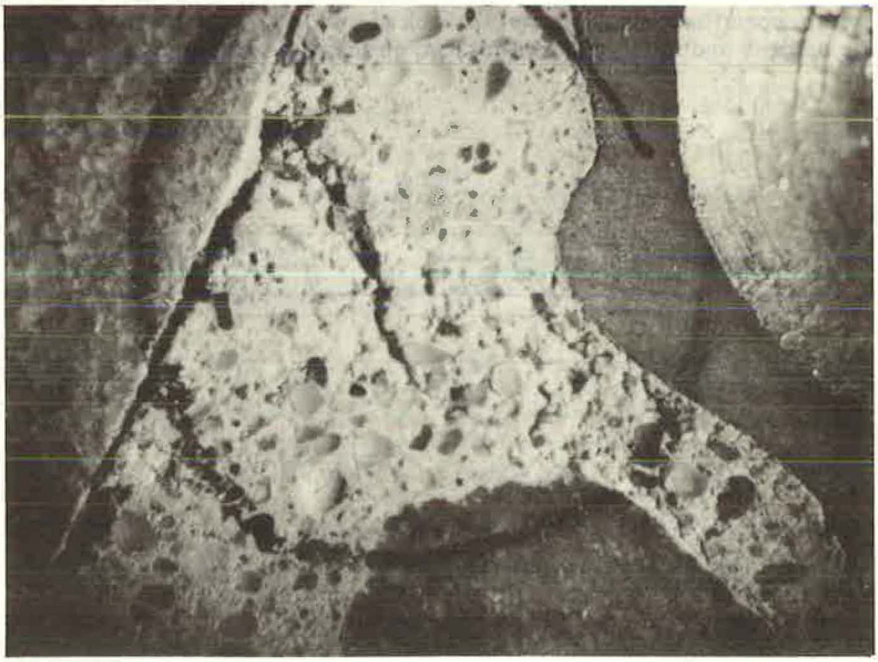

Figure 11. Water void space under an aggregate particle (mag. 12.5x).

laboratory-made specimens. In others, there was a significant difference between the proportioned and calculated values. Petrographic examinations invariably supported the results obtained by the test method. Evidence of excess mixwater, e.g., numerous bleed channels and water void space under aggregate particles, was encountered in those cores which had high water-cement ratios (Fig. 11).

The test provides a measure of water-cement ratio and cement content in hardened concrete that is consistent in its reliability with the effect of these factors on concrete performance.

Service Environment

Bridge deck deterioration is complicated by exposure conditions which are never mutually exclusive in their occurrence. Adverse climatic conditions coupled with the use of deicing chemicals and traffic loadings are deleterious forms of exposure to concrete bridge decks.

Pennsylvania has severe climatic conditions and the highway maintenance forces follow what is essentially a "bare pavement policy." Deicing chemicals (sodium and calcium chlorides) are applied at rates of 400 to 800 lb/mi, according to need.

The use of deicer salts and climatic conditions can be grouped together as forms of deleterious exposure; obviously, as more severe climatic conditions occur, the use of deicers increases. The most important climatic conditions which have a direct influence on the performance of field concretes are temperature and precipitation. Specifically, the durability of exposed concrete depends to a large degree on the freezing and thawing of absorbed water. Precipitation may be related to the relative saturation of the concrete, and freeze-thaw cycles indicate the number of cycles experienced by exposed concrete.

Weather data were gathered and plotted for Pennsylvania (see Fig. 1 for rainfall distribution during the winter months of October through April, and Fig. 2 for the yearly freeze-thaw cycles). A freeze-thaw cycle is a cycle of air temperature from above 34 F to below 26 F. The ASTM weathering index combines precipitation and the number of freeze-thaw cycles (see Fig. 3).

Because most of the bridges surveyed were in a moderately or severely deteriorated condition and because the total number was small, no generalized statement concerning the climatic conditions and the severity of bridge deck deterioration throughout the State can be made. However, it can be stated that there is no one type of deterioration that is unique to any part of the State. All types of distress have been found throughout Pennsylvania.

These exposure conditions undoubtedly have an effect on the performance of concrete. They may or may not be of paramount importance in individual cases of deterioration but are probably contributory in most cases.

RECOMMENDATIONS FOR IMPROVING PERFORMANCE OF BRIDGE DECKS

This investigation into bridge deck deterioration has pointed to numerous causal factors rather than a few major ones that might easily be identified and eliminated. Similar investigations in other states and by national organizations have reached the same conclusion. Design, materials, construction, environment and maintenance may be suspect in an individual way, or more likely, their effects may be superimposed one on another.

This research has indicated that materials and construction are principal contributors to the problem of poor bridge deck performance. This is supported in a general way by the fact that good and bad bridges exist side-by-side as twin structures on multilane highways throughout the country. The following recommendations, therefore, deal primarily with materials and construction.

Quality of Materials

The poor performance of aggregates in concrete bridge decks has shown that a more critical evaluation of aggregates is needed before approval of sources is granted.

Specifications should be changed so that the aggregates for concrete bridge decks are subjected to more detailed and critical testing before approval. Aggregates found acceptable for use in bridge decks should be identified as such.

Construction Procedures

1. Trial mixes, particularly when admixtures are involved, should be made in the same manner as the job concrete. If transit-mix trucks are to be used, the trial mixes should be made in transit-mix trucks.
2. The moisture content of concrete aggregates must be determined frequently. Moisture tests should be made early in the morning and throughout the day and the mix adjusted accordingly.
3. To insure uniformity of bridge deck concrete, a quality control program should be followed. A minimum of 1 slump and 1 air content determination per truckload of concrete should be required. A random cubic yard should be used as the sampling unit, augmented by selective sampling at the discretion of the inspector.
4. Particular concern must be shown for hot weather concreting. Reduction of evaporation rates can be attained by concrete temperature controls. The use of cold water or crushed ice as part of the mixing water requirements, protection of aggregates from heat, and concreting during cooler times of the day are a few items that may be considered.
5. Rigid support and accurate placement of the reinforcing steel must be maintained. Consideration should be made of using platforms in front of the placing operations and of suspending them above the deck so that construction crews can work from above the deck surface.
6. The rate of placement of concrete should be strictly controlled.
7. Specifications concerning the problem of overfinishing decks should be emphasized further. Any practices which contribute to overfinishing, such as sprinkling water on the surfaces, must be eliminated.
8. Methods for curing of concrete should be carefully specified. Initial curing should begin as soon as the concrete has been placed and hardened sufficiently. The concrete surface must not be allowed to dry before curing is begun.
9. The entire plan of operation, placing and finishing times, and equipment of the contractor must be evaluated to insure that the operation can be performed smoothly and efficiently.
10. Complete and detailed diaries of the construction procedures must be kept. Not only should all unusual conditions or practices be recorded, but also any minute items that may have an effect on the quality of the bridge deck.
11. Visual evaluation of the construction practices must be performed by qualified inspectors.
12. Specifications limiting the range of slump from 1 to 3 in. for vibrated concrete should be rigorously enforced.
13. Specifications for air content should be set at 6 ± 1½ percent. With high air contents the concrete is subject to abrasion by traffic and is considerably weakened by the volume of air. Low air contents may lead to premature frost damage.

Environmental Conditions

1. In a severe climatological area, bridge deck concrete should be allowed a period of drying after curing, before it is subjected to freeze-thaw cycles. A drying period after standard placement and curing is highly effective in producing more durable concrete; therefore, a construction season should be specified.
2. Coatings of linseed oil should be applied to clean, dry surfaces of new bridge decks.

Maintenance of Bridges

1. Bridge decks should be cleaned after the application of deicers.
2. Linseed oil coatings should be applied to new bridge decks annually for 3 to 4 consecutive years.

SUGGESTIONS FOR FURTHER INVESTIGATIONS

1. One of the causes of overfinishing bridge decks is the requirement for smooth-riding surfaces. The riding quality achieved by finishing as opposed to the price of deterioration should be studied. Perhaps the concrete should be placed with no finishing procedures and ground to the necessary elevations.
2. New aggregate test methods should be studied in light of the poor performance of some of Pennsylvania's aggregates.
3. A quality control program should be developed to insure uniformity of concrete in bridge decks.
4. The benefits of automation in batching and mixing in central mix plants over transit mixing should be investigated.
5. The present method of placing decks should be reevaluated. Conventional methods of paving instead of the "custom" operation should be investigated to determine the feasibility of building decks in other ways.
6. New repair methods for deteriorated bridge decks should be investigated.
7. New wearing surfaces for bridge decks should be investigated.

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Dirty Aggregate, What Difference Does It Make?

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The effects of clay in concrete fine aggregate on the properties of concrete are presented and discussed. The two most generally used tests, the loss by decantation and sand equivalent tests, are examined. The ability of these tests to measure the observed effects of the clay fraction of an aggregate on the properties of concrete is illustrated.

The concrete properties under study were water requirement, strength, shrinkage and freeze-thaw durability. Both the quantity and activity of the clay fraction were found to be influential. Increases in quantity and activity of clay cause increases in water requirement and shrinkage and decreases in strength and durability. Because of the influence of the activity of the clay fraction, the sand equivalent test is the better indicator of the effect on concrete of the clay fraction of an aggregate.

•SPECIFICATIONS for concrete aggregates represent a compromise between the desire for a perfect material and the necessity for using materials that are economically available. In many instances, the engineer is faced with the problem of writing a specification to limit a certain property and finds that sufficient information concerning that property, or how to measure it, is not available. These encounters have resulted in the use of phrases such as "harmful amounts," "excessive amounts," or in the assignment of some arbitrary quantitative measures which have been adjusted first in one direction then in another, resulting in a loss of confidence in some specifications. One of the examples of this type of specification is in the area of cleanliness of concrete aggregates.

Before the development of the sand equivalent test, the criteria to control the clay and silt size fractions of concrete aggregates were based strictly on the amount of these fractions, without regard to other properties. Even if these other properties were generally the same for different aggregates, sufficient information was not available to define quantitatively the effects of this fraction on the properties of concrete.

This study was undertaken to determine the factors of the minus No. 200 mesh fraction influencing concrete properties and to determine quantitatively the effect of these factors on concrete properties.

EXPERIMENTAL PROGRAM

Two types of coarse aggregate were used in the concretes, a high-quality "clean" siliceous aggregate and a high-quality "clean" crushed limestone. A single high-quality siliceous fine aggregate was used in all concrete mixes. Both coarse and fine aggregates were washed with a detergent to remove all minus No. 200 mesh material. The desired amount and type of clay was then added to the fine aggregate at the time of mixing. The volumes of coarse aggregate and cement were held constant, and "clean" sand was adjusted so that the volume of sand plus clay fraction varied only with variation in water. The specified amounts of dry materials were used in each batch and the water was regulated to control the slump. Specimens were molded for the determination of strength, shrinkage and freeze-thaw resistance. In total, 64 batches of concrete were tested.

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Initially, two groups of concrete batches (Series A and B) were tested to determine the significance of the influence of the amount and activity of the minus No. 200 mesh fraction. (The activity of the clay is defined as its ability to attract an absorbed film of water; in this work, the liquid limit value (AASHO T 80-60) was chosen as an indicator of this activity.) The aggregates contained from 0 to 2¼ percent minus No. 200 mesh material with liquid limits ranging from 0 to 640 percent. Significant variations in the concrete properties were indicated. Two additional groups of batches (Series C and E) were then tested to substantiate the rough trends developed by Series A and B.

Physical properties of the aggregates used in Series A, B, C, and E tests are given in Table 1.

MIX DATA AND PROPERTIES OF CONCRETE

The concrete mix data and concrete properties for A and B Series are given in Tables 2 and 3. Similar information for the C Series is given in Tables 4 and 5. Table 6 gives the concrete mix data and compressive strengths for the E Series.

RESULTS AND DISCUSSION

Aggregate Tests

The fact that minus No. 200 mesh size particles in concrete aggregates are considered deleterious is reflected in almost all specifications. Usually, limits have been placed on the amount of this size material. Before the development of the sand equivalent test, little effort had been devoted to distinguishing between active and inert minus No. 200 mesh particles.

The sand equivalent test procedure (AASHO T 176-56) uses a graduated plastic cylinder and a solution of glycerin and calcium chloride to separate the clay and sand fractions. The sample and solution are placed inside the cylinder and a vigorous shaking action is imparted to the apparatus. An agitator tube is used to flush the clay size particles up in the solution. After a prescribed settling period, the height of the sand and the total height of sand plus clay are read. The ratio of the sand height to total height, expressed as a percentage, is the sand equivalent value (Table 7). The activity of the clay results in a magnification in volume of the clay fraction which decreases the sand equivalent value (1).

The loss by decantation procedure (Texas Method Tex-406-A) uses a pycnometer and water to separate the clay and sand fraction. A mild washing action is used and the clay size fraction is decanted over a No. 200 mesh sieve. Material retained on the sieve is returned to the sample. The loss is expressed on a weight basis and reflects only the amount of material decanted.

Variations in the liquid limit of the minus No. 200 mesh fraction of commercial concrete sands proved to be large enough to be of significance. The extremes in sand equivalent value for a 1 percent decantation loss were 77 and 91 (2).

TABLE 1
PHYSICAL PROPERTIES OF AGGREGATES

Item	A and B Series			C and E Series	
	Siliceous Coarse	Siliceous Fine	Crushed Limestone Coarse	Siliceous Coarse	Siliceous Fine
Unit weight, pcf (dry loose)	93.0	98.5	88.0	98.0	100.0
Specific gravity (SSD)	2.61	2.62	2.64	2.64	2.63
Absorption (% of dry wt.)	1.2	0.8	1.4	1.2	0.8
Sieve analysis, cumulative percent retained on					
¾ in.	0.0	–	0.0	0.0	–
½ in.	35.0	–	35.0	35.0	–
⅜ in.	60.0	–	60.0	60.0	–
No. 4	100.0	0.24	100.0	100.0	0.76
No. 8	–	10.10	–	–	15.20
No. 16	–	26.21	–	–	33.22
No. 30	–	41.21	–	–	54.28
No. 50	–	83.29	–	–	89.60
No. 100	–	98.62	–	–	98.42
No. 200	–	100.00	–	–	100.00

Plastic Concrete

The primary influence of clay on the plastic concrete was manifest in the amount of water required to maintain a given slump. The water-cement ratio for a 5-sack mixture with a 3-in. slump varied from an average of 0.56 to 0.78 by weight when the sand equivalent value changed from 100 to 30 (Fig. 1). However, the mixtures containing large amounts of highly plastic clay exhibited better workability and placeability than did the cleaner mixtures with the same slump.

TABLE 2

CONCRETE MIX DATA—A AND B SERIES

(Quantities per Cubic Yard of Concrete)

Batch	Aggregate		Type I Cement		Water (lb)	Minus 200 Mesh Fraction			Air (%)	Slump (in.)	Wet Unit Wt. (pcf)
	Coarse (lb)	Fine (lb)	Sacks	Pounds		Type*	Liquid Limit	% of Total Agg. Wt.			
A11	1840	1300	5.02	472	247	—	—	0.00	6.1	3½	143.0
A12	1810	1290	5.07	477	287	NC	35	0.74	5.0	3¼	144.3
A13	1960	1160	5.11	480	287	NC	35	1.48	4.5	3	146.0
A14	1960	1080	5.26	495	300	NC	35	2.36	4.1	3	145.0
A15	1820	1360	5.07	477	282	S	0	1.42	3.0	2¾	147.5
A16	1810	1280	5.05	475	273	S-M	35	1.48	4.9	2¾	144.0
A17	1780	1220	4.97	467	352	S-M	200	1.50	2.9	3¼	142.9
A18	1840	1100	5.11	480	386	S-M	400	1.57	3.0	3	142.9
A19	1700	1110	4.95	465	406	M	640	1.60	3.3	3	138.8
B11	1670	1490	4.97	467	287	—	—	0.00	4.1	3	145.0
B12	1680	1380	5.00	470	271	NC	35	0.74	6.0	3½	141.0
B13	1720	1380	5.11	480	289	NC	35	1.49	3.0	3	145.0
B14	1670	1330	4.97	467	334	NC	35	2.25	4.2	2¾	143.0
B15	1700	1400	5.05	475	296	L	0	1.48	3.1	3	145.5

*NC, natural clay; S, silica flour; S-M, silica-montmorillonite mixture; M, montmorillonite; L, limestone fines.

TABLE 3

CONCRETE PROPERTIES—A AND B SERIES

Batch	Dynamic Modulus of Elasticity (10^{-6} psi—ASTM C215)		Modulus of Rupture* (psi)		Compressive Strength (psi—ASTM C116)		Shrinkage** (μ-in./in.)	
	7 Day	28 Day	7 Day	28 Day	7 Day	28 Day	28 Day	120 Day
A11	5.86	6.25	810	780	3300	3670	235	435
A12	5.61	6.31	660	720	2690	3370	383	565
A13	5.79	5.99	640	580	2850	3220	353	490
A14	5.26	6.64	580	650	2390	3000	347	518
A15	6.40	6.46	880	770	2890	2920	265	420
A16	5.48	6.00	650	790	2750	3530	312	450
A17	4.81	5.16	510	560	2160	2520	373	630
A18	4.58	4.72	500	520	2370	2430	433	730
A19	3.96	4.33	410	450	1840	2290	465	768
B11	5.76	6.22	700	830	2900	3210	432	628
B12	5.44	5.64	580	760	2790	2640	370	560
B13	5.35	5.95	770	790	3570	3810	312	430
B14	5.14	5.38	600	730	2450	2750	440	665
B15	5.52	5.84	830	810	3120	3890	285	455

*Center point 3 by 4 by 16-in. prisms.

**ASTM C157 except specimens had 4 by 4-in. cross section and were internally vibrated; specimens were moist cured for 3 days then dried at 50 ±5 percent relative humidity and 72 ±2 F.

TABLE 4

CONCRETE MIX DATA—C SERIES

(Quantities per Cubic Yard of Concrete)

Batch	Aggregate		Type I Cement		Water (lb)	Minus 200 Mesh Fraction		Air (%)	Slump (in.)	Wet Unit Wt. (pcf)
	Coarse (lb)	Fine (lb)	Sacks	Pounds		Liquid Limit	% of Total Agg. Wt.			
C10	1770	1400	5.04	474	278	—	0.00	4.7	3	145.2
C11	1760	1350	4.99	469	297	0	1.59	5.3	3	145.6
C12	1800	1350	5.04	474	283	35	1.59	4.2	3	146.5
C13	1770	1290	4.98	468	224	70	1.61	4.4	3½	144.5
C14	1790	1270	4.99	469	321	0	3.24	5.2	3¼	144.8
C15	1790	1230	4.99	460	304	35	3.39	4.0	3½	144.0
C16	1790	1150	5.01	471	338	70	3.38	4.5	3	142.4
C17	1810	1080	5.05	475	359	0	5.55	5.2	3½	144.0
C18	1780	1140	4.97	467	319	35	5.43	4.8	3	142.8
C19	1800	1060	5.02	472	357	70	5.59	4.0	3	142.4

TABLE 5

CONCRETE PROPERTIES—C SERIES

Batch	Dynamic Modulus of Elasticity (10^{-6} psi—ASTM C215)			Modulus of Rupture (psi—ASTM C78)			Compressive Strength (psi—ASTM C39)			Shrinkage* (μ-in./in.)	
	7 Day	14 Day	28 Day	7 Day	14 Day	28 Day	7 Day	14 Day	28 Day	28 Day	120 Day
C10	6.21	6.45	6.54	620	630	660	4500	4730	4990	355	520
C11	5.60	6.49	6.78	680	695	685	4800	5230	5310	325	510
C12	5.82	6.46	6.73	635	675	670	4620	4900	5240	360	550
C13	5.89	5.99	6.26	550	625	650	4070	4310	4580	375	580
C14	6.02	5.49	6.31	585	585	540	3680	4210	4700	390	570
C15	6.02	6.08	6.26	590	615	635	3910	4150	4480	340	475
C16	5.38	5.39	5.63	535	460	465	3400	3810	3860	470	675
C17	6.03	6.27	6.35	590	585	620	3930	4340	4700	350	510
C18	5.46	5.46	5.61	525	490	520	3590	3820	4220	470	720
C19	5.36	5.55	5.77	540	515	530	3560	3950	4160	460	645

* ASTM C157 with specimens being moist cured for 7 days, then dried at 50 ±5 percent relative humidity and 72 ±2 F.

TABLE 6

CONCRETE MIX DATA AND COMPRESSIVE STRENGTHS—E SERIES

(Quantities per Cubic Yard of Concrete)

Batch	Aggregate		Cement		Water (lb)	Minus 200 Mesh Fraction*		Air (%)	Slump (in.)	Wet Unit Wt. (pcf)	Compressive Strength	
	Coarse (lb)	Fine (lb)	Pounds	Sacks		(lb)	% of Total Agg. Wt.				7 Day (psi)	28 Day (psi)
E10	1730	1370	455	4.84	244	0	0	8.5	4	140	3050	3570
E20	1800	1440	475	5.05	251	0	0	5.4	2	146	3590	4280
E30	1740	1430	458	4.87	255	0	0	5.2	2¾	144	3260	3920
E11	1770	1360	466	4.96	304	12.6	0.4	5.5	2¾	144	3440	4270
E21	1770	1360	466	4.96	253	12.6	0.4	6.2	3¼	143	3150	4030
E31	1770	1360	466	4.96	268	12.6	0.4	6.0	3½	144	3520	4010
E12	1760	1330	463	4.93	282	36.3	1.2	5.4	3¼	144	3240	3650
E22	1760	1330	463	4.93	277	36.3	1.2	5.9	3½	143	3190	3670
E32	1780	1340	469	4.99	272	36.8	1.2	5.9	3¼	144	3480	3940
E13	1770	1310	466	4.96	278	60.4	1.9	5.1	3	144	3170	3700
E23	1770	1310	466	4.96	287	60.4	1.9	4.5	3	144	3040	3760
E33	1760	1300	463	4.93	273	60.0	1.9	6.1	3	143	3130	3320
E14	1770	1290	466	4.96	292	84.4	2.8	4.5	3½	144	3040	3720
E24	1770	1290	466	4.96	280	84.4	2.8	5.2	3¼	144	3150	3360
E34	1760	1280	463	4.93	273	83.9	2.8	5.5	3¼	143	3110	3400
E15	1760	1230	463	4.93	303	131.7	4.4	5.0	3	144	2750	3120
E25	1770	1200	466	4.96	315	132.3	4.4	4.5	3½	144	2540	3100
E35	1770	1200	466	4.96	299	132.3	4.4	5.0	3¼	144	3020	3280
E16	1790	1080	472	5.02	321	194.9	6.8	4.5	3½	143	2550	2840
E26	1780	1070	469	4.99	326	193.7	6.8	5.5	3¼	142	2320	2610
E36	1760	1060	463	4.93	322	191.4	6.8	5.2	3¼	141	2280	2510
E17	1760	1010	463	4.93	350	250.6	9.0	4.4	3¼	142	2100	2460
E27	1750	1010	460	4.89	348	248.9	9.0	5.1	3¼	141	1900	2190
E37	1780	960	469	4.99	354	253.2	9.0	4.5	3½	142	1990	2220

*Liquid limit of minus 200 mesh fraction is 35 percent.

TABLE 7

CALCULATED SAND EQUIVALENT VALUES FOR AGGREGATES USED IN CONCRETE MIXES*

Batch	Sand Equiv. Value	Batch	Sand Equiv. Value
A11	100	C12	82
A12	90	C13	70
A13	80	C14	86
A14	70	C15	66
A15	94	C16	51
A16	81	C17	79
A17	49	C18	53
A18	30	C19	38
A19	22		
B11	100	E10, 20, 30	100
B12	91	E11, 21, 31	95
B13	82	E12, 22, 32	85
B14	75	E13, 23, 33	78
B15	94	E14, 24, 34	71
C10	100	E15, 25, 35	59
C11	94	E16, 26, 36	46
		E17, 27, 37	39

*Calculated by $SE = \frac{100}{1 + P\,(0.1318\,LL + 1.79)}$

This observation lends support to Tremper and Haskell's statement (3): "Instances are on record of the use of a soil as an admixture to concrete with evident improvement in workability and without serious effect on the compressive strength, particularly if the mix was lean and harsh."

It was also observed that the required quantity of air-entraining admixture for a given air content varied some 1,000 percent for the full range of sand equivalent values: from 5½ oz/bag for a sand equivalent value of 39, down to ½ oz/bag for a sand equivalent value of 100.

Strength of Concrete

The amount and type of minus No. 200 mesh material used in each batch strongly influenced the amount of water required to maintain a given slump. This created a significant variation in the water-cement ratio. With this in mind, the first and most obvious relationship is concrete strength and water-cement ratio (Fig. 2). Values of compressive strength for the mixes having water-cement ratios of 0.6 were used as the 100 percent value and the strengths are shown relative to this value. The correlation coefficient of 0.78 indicates the existence of a significant correlation. If variations in compressive strength are compared with the amount of minus No. 200 mesh material (liquid limit being held constant at 35 percent, the correlation coefficient is significantly improved (Fig. 3)). However, if the liquid limit is allowed to vary, this correlation breaks down due to strong influence of this parameter on concrete strength.

A very slight improvement on the correlation of strength with water-cement ratio was obtained by comparing the modulus of rupture and compressive strengths to the sand equivalent value. A correlation coefficient of 0.83 is obtained for each of these relationships (Figs. 4 and 5). The difference between the two correlation coefficients (0.78 for compressive strength vs water-cement ratio, 0.83 for compressive strength vs sand equivalent value) was tested and found not to be statistically significant. The possibility remains, however, that sand equivalent value reflects influential parameters

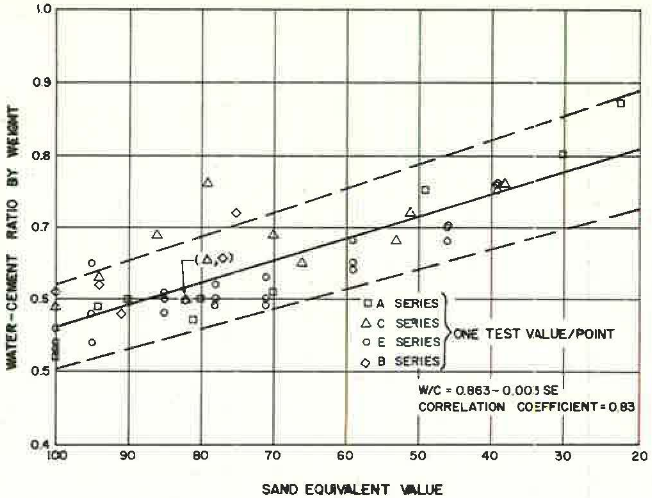


Figure 1. Effect of sand equivalent value on water-cement ratio for 5-sack mix with 3-in. slump.

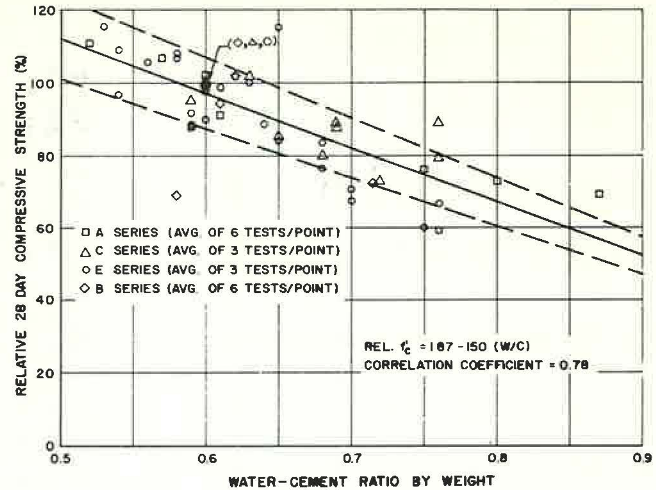


Figure 2. Relationship between 28-day compressive strength and water-cement ratio.

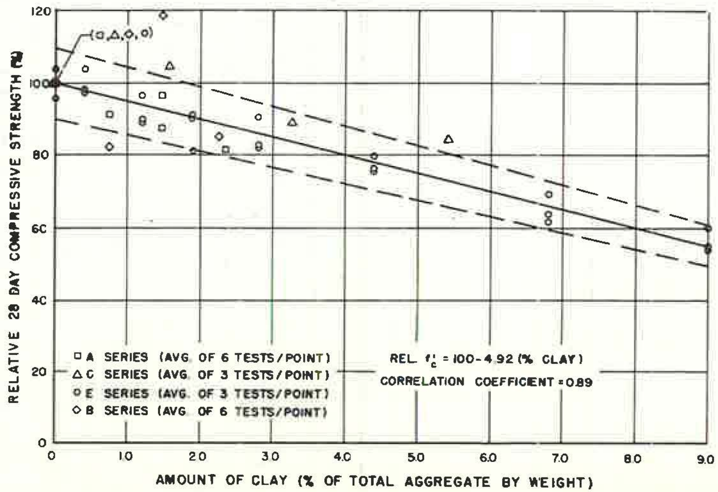


Figure 3. Influence of amount of clay fraction (LL = 35%) on 28-day compressive strength.

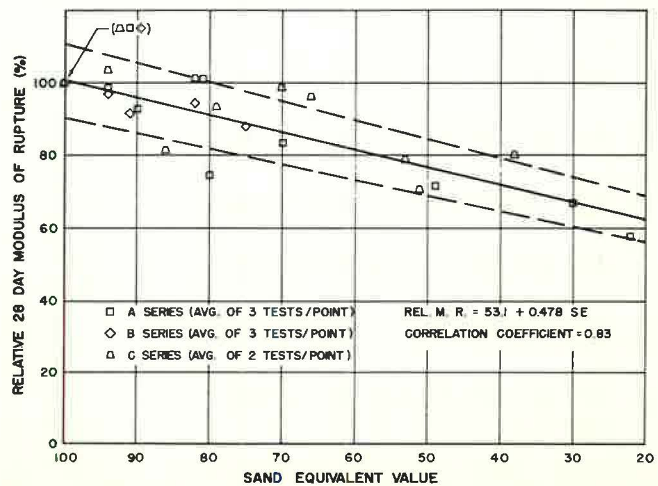


Figure 4. Relationship between 28-day modulus of rupture and sand equivalent value.

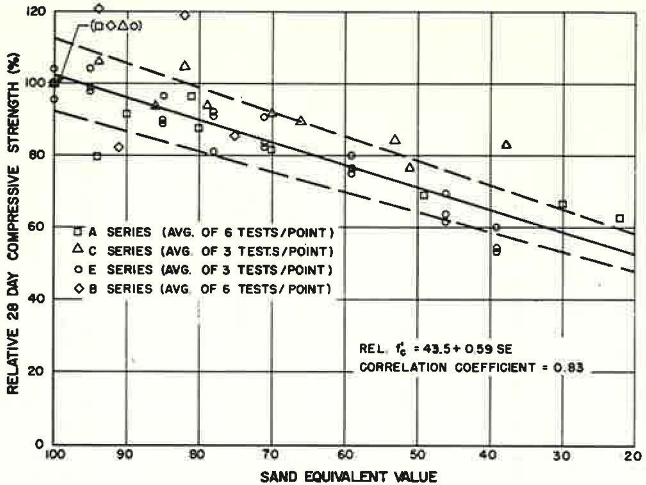


Figure 5. Variation in 28-day compressive strength with sand equivalent value.

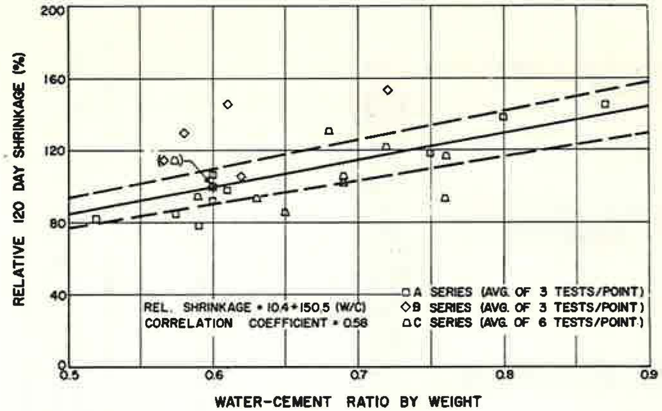


Figure 6. Relationship between 120-day shrinkage and water-cement ratio.

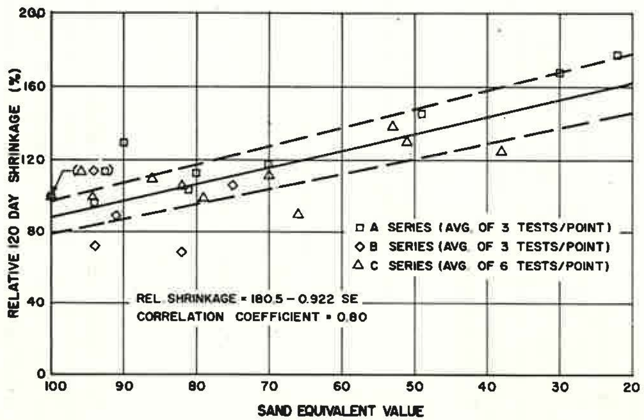


Figure 7. Relationship between 120-day shrinkage and sand equivalent value.

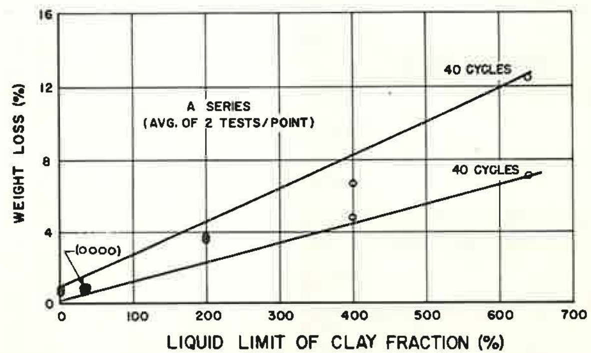
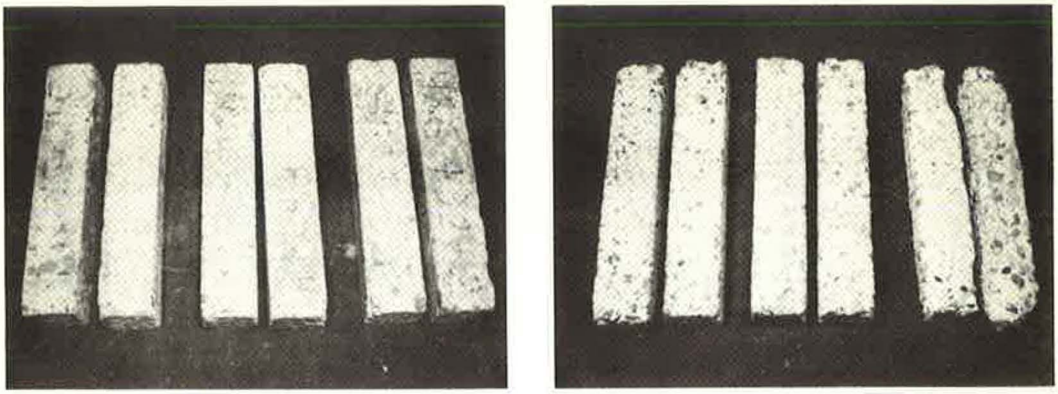


Figure 8. Relationship between weight loss of freeze-thaw specimens and LL of clay fraction after 300 cycles of ASTM C290.



A15	A16	A13	A17	A18	A19
1.42% Clay	1.48% Clay	1.48% Clay	1.50% Clay	1.57% Clay	1.60% Clay
0% LL	35% LL	35% LL	200% LL	400% LL	640% LL
SE = 94	SE = 81	SE = 80	SE = 49	SE = 30	SE = 22

Figure 9. Specimens from A Series batches after completion of freeze-thaw testing by ASTM C290.

other than the water requirement of the sand. One possible parameter is the difference in specific surface as it affects concrete strength. Another consideration might be differences in the load-resisting properties of active particles having adsorbed films of water as opposed to inactive particles that have comparatively little surface attraction for water molecules.

Shrinkage of Concrete

Shrinkage of the concretes studied correlates to some degree with water-cement ratio but to a higher degree with sand equivalent value (Figs. 6 and 7). In each case a decrease in sand equivalent value or an increase in water-cement ratio causes an increase in shrinkage.

Hveem and Tremper (4) reported a correlation coefficient of 0.66 between drying shrinkage of mortar (after 14 days of drying) and sand equivalent value of commercially produced concrete sands. However, when the absorption of the sand was included the correlation was significantly improved (correlation coefficient 0.83). Chamberlin (5, p. 32-33) reports:

> Interestingly, sand equivalents of the experimental aggregates also correlate with drying shrinkage and to a rather high degree.
>
> * * *
>
> The relative contribution of aggregate elasticity and clay content (as measured by sand equivalent) to the observed shrinkage cannot be distinguished by statistical methods alone. This is because the two factors correlate significantly with one another (coefficient of 0.80), that is, sands with low elastic moduli tend also to have low sand equivalents and both, therefore, would be expected to influence shrinkage in the same direction and in unison.

The lower dynamic modulus of elasticity for the concretes with low sand equivalent values tends to show an effect of the active minus No. 200 mesh particles in the hardened concrete. These particles may offer less restraint to the shrinking paste thereby accounting for some of the increased shrinkage not directly attributable to higher water-cement ratios.

Durability of Air-Entrained Concretes

Specimens from batches A13 and A15 through A19 were subject to 400 cycles of slow freezing and thawing in a chest-type freezer. Only very slight surface deterioration was observed and the results were inconclusive. The specimens were stored until a later date when they were subjected to freeze-thaw testing according to ASTM C290. Deterioration of most of these specimens was not indicated by fundamental frequency determinations but did show itself in changes in weight due to surface deterioration. The exceptions were the specimens containing clay with a 640 LL (batch A19). These specimens completely disintegrated after 40 cycles and were removed from testing. Figure 8 shows the weight loss after 300 cycles (except from batch A19) of ASTM C290 testing, and Figure 9 shows these specimens after completion of testing.

The tests indicate an insignificant loss in durability of specimens containing fine aggregates with sand equivalent values of 80 and 81 when the proper amount of air is entrained in the concrete.

SUMMARY AND CONCLUSIONS

The conclusions developed from this study are based on a limited number of aggregates and concrete batches. Care should be exercised in extending these conclusions to materials other than those studied.

It has been found that the activity as well as the amount of the minus No. 200 mesh fraction of concrete aggregates affects the properties of concrete. Both activity and amount are reflected in the sand equivalent value but not in the loss by decantation (2). Clay fractions in concrete aggregate affect concrete properties primarily through their effect on water demand. Concrete strength and shrinkage correlate to a high degree with sand equivalent value and to a slightly lesser degree with water-cement ratio indicating the possibility that the sand equivalent test indicates properties of the aggregate that are not accounted for solely by the aggregate's water demand in concrete.

The freeze-thaw durability of the concretes studied is related to the sand equivalent value. Decreases in the sand equivalent value bring about decreases in the freeze-thaw durability of the concretes. With the exception of batch A19, attrition of the surface reflected by loss in weight was the only apparent indicator of deterioration.

Research reported here and elsewhere (2, 6) has shown that the quality of aggregates can be increased considerably during processing by more thorough cleaning.

The need for sufficient processing to produce a relatively clean aggregate is emphasized by considering the quantitative effects on the properties of the concretes tested. The data developed indicate that for a given fine aggregate, as the sand equivalent value changes from 60 to 80, the concrete properties will exhibit the following changes:

1. Gain in 7-day compressive strength of 15 percent;
2. Gain in 28-day compressive strength of 16 percent;
3. Gain in 7-day modulus of rupture of 13 percent;
4. Gain in 28-day modulus of rupture of 12 percent;
5. Insignificant change in durability of air-entrained concrete (air content approximately 5 percent);
6. Decrease in relative 28-day shrinkage of 17 percent;
7. Decrease in relative 120-day shrinkage of 15 percent; and
8. Decrease in concrete mixing water demand of 9 percent.

ACKNOWLEDGMENT

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The Importance of Moisture Absorption Characteristics of Lightweight Coarse Aggregate

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Lightweight coarse aggregates from five sources, commercially available in Texas, were selected for investigation. All aggregates selected were structural lightweight aggregates produced from shale by the rotary kiln process.

The absorption characteristics of the aggregates, including total absorption and absorption rate, were studied. Concretes incorporating the five aggregates were subjected to repeated freezing and thawing (ASTM C290) and the results were compared to the aggregate absorption characteristics. The results demonstrate that the concretes which incorporated aggregates that have high absorption capacities and/or absorb water at a rapid rate are susceptible to freeze-thaw damage.

•IT is generally accepted that the freeze-thaw resistance of concrete is dependent on the properties of the cement paste and the properties of the aggregates (1). The influence of the properties of these two constituents are often thought of as being separate; however, conditions exist where one begins to affect the other and some interaction occurs. This report concerns the role of lightweight aggregate void characteristics in the freeze-thaw resistance of lightweight concrete.

For an aggregate to be potentially nondurable in terms of freeze-thaw resistance, it must be saturated with water beyond some critical value (2, 3). Aggregates which contain amounts of absorbed water below this critical value will generally not cause distress in the concrete when subjected to repeated cycles of freezing and thawing. But if the amount of absorbed water is greater than this critical value, freeze-thaw distress may be experienced.

The response that a critically saturated aggregate will exhibit upon freezing depends on its pore characteristics, the rate of freezing, and the size of the particle (4). An aggregate with a very low absorption may not cause disruption even if frozen instantaneously since the pressures built up can be accommodated by elastic dilation (5). A highly absorptive aggregate with low permeability may cause failure if the particles are so large that the freezing water cannot escape fast enough to relieve the internal pressures. If an aggregate has both high absorption and high permeability, the ability of the concrete to resist freeze-thaw damage will depend on the ability of the surrounding paste to accommodate the water being expelled from the aggregate. The high permeability will allow the relief of high internal pressures, but to insure that disruption will not occur, the paste must be capable of accommodating the expelled water. In actual service performance, the properties of the coarse aggregate and mortar, the service environment, and the interaction between the various parameters become more complicated.

Whether or not an aggregate becomes critically saturated will depend on how the aggregate is handled and mixed and the environment to which the concrete is subjected

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after being placed. It is possible for an aggregate to become critically saturated during handling and mixing operations; in that case, the occurrence of freezing temperatures before the concrete has time to dry sufficiently could be extremely detrimental. After concrete has been placed, some factors influencing the ease with which an aggregate particle will become saturated include the permeability of the aggregate, the thickness and permeability of the cement paste cover, the relative pore size of the cement paste and aggregate, and the availability of water at the surface of the concrete (6). Here, a low permeability would be advantageous in that it would take the aggregate longer to become saturated. However, once the aggregate particles become critically saturated, a low permeability would no longer be desirable.

EXPERIMENTAL PROGRAM

Five lightweight coarse aggregates commercially available in Texas were used in the experimental program (¾-in. maximum nominal size). These aggregates were all expanded shales produced in a rotary kiln. Values of absorption and absorption-time relationships for these aggregates were determined by the method proposed by Saxer (7) and later modified by Bryant (8). Using this method, absorption characteristics were determined without the difficulty of obtaining a saturated-surface-dry condition. The test is performed by placing a known weight of oven-dry material in a known volume container (pycnometer). The container is maintained with a specific water level above the aggregate and the entire apparatus is weighed at specific time intervals.

Absorption, absorption-time relationships, and other aggregate properties were obtained for all the aggregates used (Table 1). One new aggregate property which was determined was the aggregate absorption factor (AAF). The AAF is defined as the percent absorption at 1,000 min of immersion minus the percent absorption at 100 min, based on dry weight (9). The change in absorption between 3 and 14 days' immersion and the variation in aggregate properties will be discussed subsequently.

The concrete durability testing program included 15 batches (3 for each coarse aggregate) of concrete in which mixture proportions, slump, and air content were held as constant as possible, the only variable being the source of lightweight coarse aggregate. To obtain a relatively high degree of aggregate saturation, the coarse aggregates were immersed in water for 14 days and allowed to drain under cover for 24 more hours before mixing. This procedure produced the desired degree of saturation while providing for the removal of a large portion of the free water. A high quality, regular-weight, natural fine aggregate and Type I cement were used throughout the program. The air-entraining agent was a neutralized vinsol resin. Batch proportions and properties of the plastic concrete are given in Table 2.

TABLE 1
AGGREGATE PROPERTIES

Coarse Agg. Desig. and Lot No.	Absorption[a] 3 Days (%)	Absorption[a] 14 Days (%)	Bulk Specific Gravity[a] (SSD) 3 Days	Bulk Specific Gravity[a] (SSD) 14 Days	Aggregate Absorption Factor[b] (%)	Dry Unit Weight (pcf)[c]	Fineness Modulus[d]
R1	4.7	8.0	1.47	1.50	1.8	46.8	7.0
R2	6.0	8.2	1.52	1.55	1.8	48.1	6.5
C1	8.0	12.1	1.37	1.41	2.3	38.1	6.7
C2	7.8	12.0	1.39	1.50	1.6	38.5	6.5
E4	7.8	9.8	1.59	1.63	2.5	45.3	6.7
E6	6.4	8.9	1.50	1.51	2.3	44.9	6.8
S2	13.2	19.6	1.63	1.73	3.1	44.9	6.5
S3	12.4	18.8	1.58	1.68	3.5	42.0	6.7
D1	22.5	28.0	1.35	1.42	6.3	34.4	6.7
D2	20.2	26.9	1.42	1.50	5.5	35.8	6.6
H(fine)[e]	0.8	—	2.61	—	—	99.0	2.57

[a]Determined by Bryant method (8).
[b]Change in percent absorption of the aggregate between 100 and 1,000 min elapsed time, based on dry weight (9).
[c]Determined in accordance with ASTM Designation: C330 64T.
[d]Determined in accordance with ASTM Designation: C125 58; maximum size of coarse aggregate was ¾ in.
[e]This regular weight fine aggregate was used in all mixes.

TABLE 2

CONCRETE MIX DESIGN DATA

Coarse Agg. Desig. and Lot No.	Concrete Batch Code No.	Cement Factor (sk/cu yd)	Percent Absolute Volume					Slump (in.)	Initial Unit Weight (pcf)
			Cement	Water	FA	CA	Air		
R1	3FTR	4.6	8.2	20.9	34.9	31.7	4.3	4	114.4
R1	4FTR	4.9	8.7	19.0	33.1	33.8	5.4	3	113.2
R2	5FTR	4.9	8.7	18.7	33.0	33.6	6.0	$3\frac{3}{4}$	115.1
C1	3FTC	4.7	8.3	21.0	34.3	32.1	4.3	$3\frac{1}{2}$	117.6
C1	4FTC	4.7	8.4	20.2	34.4	32.2	4.8	4	115.2
C2	5FTC	4.6	8.1	21.3	33.3	31.8	5.5	3	115.6
E4	3FTE	4.7	8.3	17.6	36.0	32.2	5.9	$4\frac{1}{2}$	120.2
E4	4FTE	4.7	8.3	17.1	37.4	32.2	5.0	3	120.8
E6	5FTE	4.6	8.2	16.6	37.1	32.1	6.0	$3\frac{1}{2}$	117.1
S2	3FTS	4.9	8.7	20.7	31.7	33.6	5.3	3	116.8
S2	4FTS	4.9	8.7	21.0	32.4	33.6	4.3	$3\frac{1}{2}$	118.0
S3	5FTS	5.0	8.8	20.3	32.6	33.8	4.5	$3\frac{3}{4}$	118.2
D1	3FTD	5.0	9.0	18.9	35.4	31.9	4.8	$3\frac{1}{4}$	113.6
D1	4FTD	4.9	8.7	19.2	36.2	31.1	4.8	$3\frac{3}{4}$	115.2
D2	5FTD	5.0	8.9	17.8	36.7	31.6	5.0	$3\frac{1}{4}$	116.1

Compressive strength determinations for each batch of concrete were made on three 6 by 12-in. cylinders moist cured for 28 days. Also from each batch, three 3 by 3 by 16-in. prisms for freeze-thaw testing were cast and continuously moist cured. Freeze-thaw cycling of these specimens was initiated at 14 days, in accordance with ASTM designation: C290- 63T (Fig. 1). The concrete properties and freeze-thaw results are given in Table 3. In each case, structural quality lightweight concrete was produced. Twenty-eight-day compressive strengths ranged from 3, 240 to 4, 330 psi.

RESULTS AND DISCUSSION

Figure 1. Freeze-thaw equipment.

In comparison with natural aggregates, lightweight aggregates continue to absorb significantly larger amounts of water for a much longer time; in fact, absorption can continue for several weeks or months. The absorption-time curve for a typical aggregate (Fig. 2) does not approach the horizontal until after approximately 40 days' immersion in water. This should not be taken to mean that all voids in the aggregate are saturated even after 40 days since some voids are isolated and not connected to the surface. This long-time absorption property of structural lightweight aggregates is by no means unique with one or two aggregates. To the authors' knowledge, all lightweight aggregates exhibit these long-time absorption phenomena and these phenomena should be considered in mix design.

As expected, the variations in aggregate properties are sizable between aggregates from different sources, even though they are all rotary kiln produced. These properties also vary between lots (or shipments) of aggregate from the same producer. Even within the same lot of aggregate the scatter in absorption-time data is apparent in

TABLE 3

CONCRETE PROPERTIES AND FREEZE-THAW RESULTS

Coarse Agg. Desig. and Lot No.	Cement Factor (sk/cu yd)	Air Entrainment (%)[a]	28-Day Compressive Strength (psi)[b]	No. of Freeze-Thaw Cycles to Failure (N_f)[c]	Concrete Batch Code No.
R1	4.6	4.3	3650	Rel E = 89%[d]	3FTR
R1	4.9	5.4	4330	Rel E = 92%[d]	4FTR
R2	4.9	6.0	4190	Rel E = 86%[d]	5FTR
C1	4.7	4.3	3780	95	3FTC
C1	4.7	4.8	3540	65	4FTC
C2	4.6	5.5	3240	46	5FTC
E4	4.7	5.9	3770	44	3FTE
E4	4.7	5.0	3930	Rel E = 90%[d]	4FTE
E6	4.6	6.0	3570	Rel E = 75%[d]	5FTE
S2	4.9	5.3	3740	15	3FTS
S2	4.9	4.3	3900	37	4FTS
S3	5.0	4.5	3930	11	5FTS
D1	5.0	4.8	3810	25	3FTD
D1	4.9	4.8	3980	30	4FTD
D2	5.0	5.0	4260	27	5FTD

[a]Determined in accordance with ASTM Designation: C173 58.
[b]Determined in accordance with ASTM Designation: C39 64.
[c]Determined in accordance with ASTM Designation: C290 63T; values given are averages for three specimens from each batch.
[d]Test was terminated after 100 cycles of freezing and thawing.

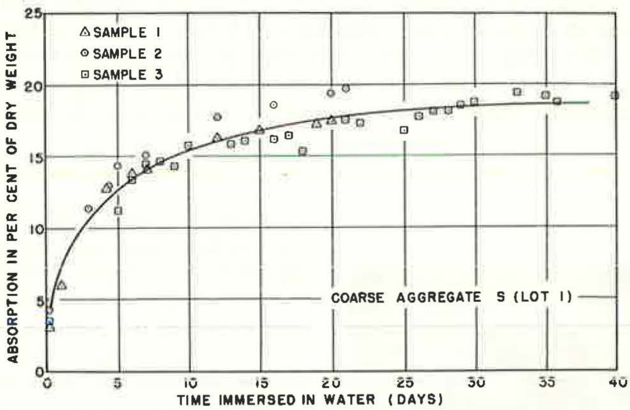

Figure 2. Typical absorption-time curve for Aggregate S.

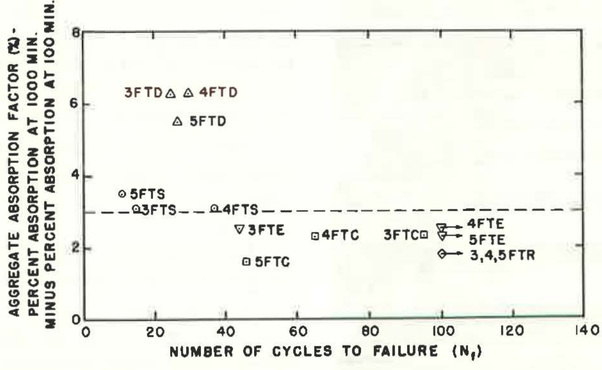

Figure 3. Relationship between aggregate absorption factor and number of cycles to failure by ASTM C290.

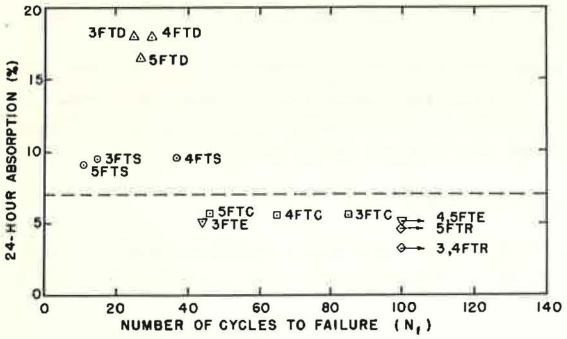

Figure 4. Relationship between 24-hr absorption based on dry weight and number of cycles to failure by ASTM C290.

Figure 2. (Analysis of some 30 determinations of 24-hr absorption on 1 sample of aggregate E indicated a coefficient of variation in results of 15 percent.) Such scatter contributes to the difficulty of relating given aggregate properties to end product behavior as well as the difficulty of obtaining consistent mix designs.

Permeability of the aggregate has been thought of as a measure of the rate at which an aggregate will become saturated. Hypothesizing that some measure of aggregate permeability may be an indicator of potential durability of resulting concretes subjected to freezing and thawing, the aggregate absorption factor (AAF) was taken as one measure of this permeability. The relationship between number of cycles to failure (N_f) of the concrete and the AAF is shown in Figure 3. Those aggregates with a high AAF exhibited poor durability, whereas those aggregates with a low AAF (say, below approximately 3) exhibited generally better durability against freezing and thawing. However, a low AAF will not insure durable concrete. Its simplicity precludes any such "magic answer." The AAF is a quick and easy means of quantifying at least one of the parameters that influences the freeze-thaw durability of lightweight concrete.

In regard to aggregate absorption values at selected time intervals, Figures 4 and 5 show the relationships between number of cycles to failure and 24-hr and 14-day coarse aggregate absorption values, respectively. These plots are very similar to the AAF-N_f relationship (Fig. 3) and classify the concretes in the same order. Limiting values of either AAF or absorption were found, above which nondurable concretes were produced. However, this concept is not as clear cut when aggregates C, R, and E are compared; for the concretes investigated, one aggregate produces a durable concrete, whereas another does not, and the difference cannot be explained on the basis of AAF or absorption alone.

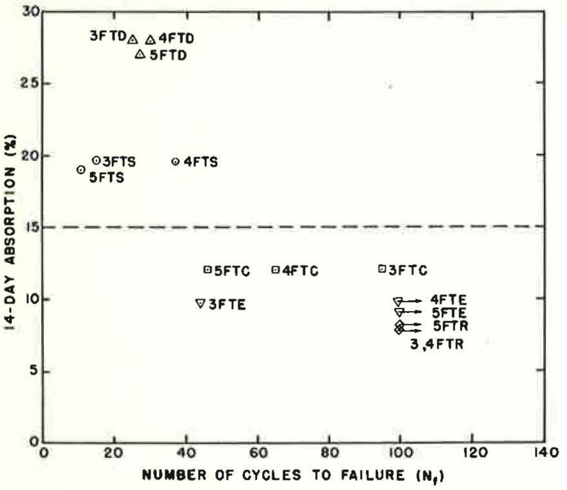

Figure 5. Relationship between 14-day absorption and number of cycles to failure by ASTM C290.

These results do not indicate that in actual service one of these concretes will be more durable than another. They do indicate that under prescribed conditions of prewetting, the concretes had widely varying durabilities. Some of these concretes, incorporating aggregates with high absorption capacities and absorption rates, were less durable under these conditions of prewetting. It is the authors' opinion that, in this case, the high absorption and absorption rate are the cause of low durability.

CONCLUSIONS

1. The absorption of water by lightweight coarse aggregates is a lengthy process often requiring several weeks for saturation to occur.

2. The absorption characteristics of the five aggregates studied

varied widely. Total absorption after 3 days of immersion ranged from 4.7 to 22.5 percent.

3. When tested under the prescribed conditions, wide differences exist between the freeze-thaw resistance of concretes made with the lightweight aggregates studied.

4. Under the conditions of prewetting imposed by the tests, those concretes whose aggregates had high absorption values and/or absorption rates were less durable.

ACKNOWLEDGMENT

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Relationship Between Aggregate Pore Characteristics and Durability of Concrete Exposed to Freezing and Thawing

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Various coarse aggregates ranging from crushed limestones and traprock to gravels from glacial and nonglacial sources were obtained from different parts of the United States and Canada. Concrete specimens made from these aggregates were exposed to laboratory freezing-and-thawing cycles. Specimens for a mercury intrusion porosity test were hand-picked to represent the aggregate being tested and broken into small particles.

For the eight aggregates tested, freeze-thaw durability was compared to aggregate pore characteristics as determined by the mercury-intrusion porosity test. Several characteristics were found to relate quite well with freeze-thaw durability, especially the amount of pores found in the + 8-μ range.

•THE pore characteristics of aggregate have a profound effect on the performance of portland cement concrete exposed to freezing and thawing. The character of the pores not only determines how much water the aggregate within the concrete will contain, but perhaps of more importance, how that water is distributed within the aggregate. The size of the pores also determines how easily the aggregate shall be filled with water and, if such pores become filled with water, how easily a particular degree of saturation can be maintained.

The importance of pore characteristics on concrete durability is well documented in the literature. Verbeck (1) stated, "Aggregate constitutes about 75 percent of the volume of concrete. Consequently, the properties of the concrete are significantly influenced by the characteristics of the aggregates." As early as 1946, Rhoades and Mielenz (2) asserted that, "Pore characteristics of aggregates are important controls of chemical and physical stability and they strongly influence the bond with cement. They significantly affect the strength of any material, and also determine absorption and permeability. As a result, they control durability under freezing and thawing conditions and the rate of chemical alteration."

Lewis, Dolch, and Woods (3) stated, "It would be difficult to prove that any other physical property is of greater importance than the porosity characteristics (amount, size, and continuity of the pores) in either natural or artificial aggregates."

As investigated by Schuster and McLaughlin (4), cherts with high porosity are more susceptible to freeze-thaw deterioration than those with low porosity. However, other characteristics such as pore size and continuity of aggregate pores are recognized to be more important than total porosity. Lewis and Dolch (5) maintained that the harmful pore size is that which is sufficiently large to permit water to readily enter much

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of the pore space, but not large enough to permit easy drainage. Pore size determines the properties of permeability, absorption, and capillary potential. Dolch (6) explained, "A rock with larger pores will have a higher absorptivity than one with smaller pores. It will also have a higher permeability and a lower capillary potential. This means that rock with smaller pores will acquire water more slowly but will retain it longer and more tenaciously than will a rock with larger pores."

Study by Sweet (7) has indicated that critical pore size for freezing-and-thawing durability for limestone aggregates may be about 5 μ. Blanks (8) found that, under natural conditions of freezing and thawing, voids less than 5 μ in diameter, and particularly those less than 4 μ, will drain effectively only at hydrostatic pressures that exceed the tensile strengths of some rocks and concrete. Sweet (7) also noted that in Indiana limestone aggregates the volume of voids < 5 μ in diameter expressed as ratio of the total volume, was less than 0.057 for aggregates with good field performance records and greater than 0.091 for aggregates with poor service records.

Later work by Schuster and McLaughlin (4) showed that the significance of 5-μ pores was questioned: "The freeze-thaw durability of concrete containing chert apparently is not as dependent on pores in the chert less than five microns in diameter as has been postulated by Sweet (7)."

Because of these conflicting conclusions, it seemed desirable to make more thorough research of the relationships of pore characteristics as determined from total porosity and pore size distribution measurements. Attempts were also made to correlate these pore characteristics of aggregates with durability of concrete when exposed to freezing and thawing.

TESTING PROGRAM

Various coarse aggregates ranging from crushed limestones and traprock to gravels from glacial and nonglacial sources were obtained from different parts of the United States and Canada. Concrete specimens, 3 by 3 by 16 in., were fabricated and exposed to alternate cycles of freezing and thawing while in water, in accordance with ASTM test designation C 290 63T. Specimens for the porosity test were hand-picked to represent the aggregate being tested and broken into small particles having an approximate volume of $^1/_{64}$ cu in.

TABLE 1

PHYSICAL CHARACTERISTICS OF COARSE AGGREGATES

Aggregate	Specific Gravity		Absorption (%)	Estimated Field Performance	Brief Description
	B.S.S.D.	Bulk Dry			
A	2.64	2.63	0.40	Excellent	Quartzite gravel from eastern Piedmont
B	2.94	2.93	0.58	Excellent	Crushed trap rock from eastern U.S.
C	2.72	2.66	2.28	Unknown	Crushed L.S. from south-eastern Canada
D	2.32	2.16	7.21	–	Shale, handpicked from G.
E	2.53	2.45	3.09	Poor	Midwestern chert river gravel
F	2.39	2.23	7.41	Very poor	Ohio River chert gravel
G	2.51	2.40	4.42	Poor	Midwestern glacial gravel, floated at S.G. 2.55
H	2.48	2.23	11.14	Good	Blast furnace slag from eastern U.S.

Concrete Mix Design

All concrete mixes were designed using a low-alkali type I cement, a cement factor of 5.5 bags per cu yd, an air content of 5.5 percent, and sufficient water to obtain a 3-in. slump. ACI 613 method of selecting coarse aggregate content was used in the design of the mixes.

A single high-quality quartzite sand was used throughout the entire project and had a fineness modulus of 2.64, an absorption of 0.34 percent, and a bulk specific gravity of 2.59. The coarse aggregates were graded from 1 to ¼ in. The physical properties of the coarse aggregates are given in Table 1. Absorption and specific gravity values of the coarse aggregates were determined by means of vacuum-saturation techniques.

More than sufficient water for absorption was added to the fine aggregate 24 hr before mixing. The coarse aggregates were vacuum-saturated 24 hr before mixing, using a vacuum of approximately 2-cm mercury.

Mix design D, which used a hand-picked shale aggregate, utilized the shale only in the 1 to ½-in. size. Aggregate A was used in the ½ to ¼-in. size.

Detailed petrographic analyses of the aggregates as well as other data concerning them are included as part of NCHRP Report 12 (9). The aggregates may be identified in NCHRP Report 12 as the letters in parentheses as follows: A(A), B(B), C(C), D(Shale), E(F-1), F(G), G(H), and H(D).

Freeze-Thaw Testing

All specimens after curing in lime-water for 13 days after 1 day in the mold were exposed to approximately 7 cycles per day of rapid freezing and thawing in water in accordance with ASTM designation C 290 63T. The equipment was designed and developed at the Engineering Experiment Station, Utah State University (10) and was manufactured by the Logan Refrigeration Company of Logan, Utah. The apparatus is 82 in. long, 34 in. wide and 10 in. deep. A ½-hp commercial compressor cools a cooling plate upon which rest copper containers holding the concrete specimens. Thawing is accomplished with electric resistance heaters placed along the sides of the containers. The copper containers were ¼-in. larger than the actual dimensions of the specimens, thus leaving a ⅛-in. layer of water on all four sides of the specimen. Dynamic modulus was measured at regular intervals until the specimen had lost 50 percent of its original dynamic modulus or had experienced 100 cycles of freezing and thawing. Except for aggregate H, a minimum of 6 specimens was made in 3-beam batches. Aggregate H, being hand-picked, had 3 specimens made from it in 1-beam batches.

Pore Characteristic Testing

An Aminco-Winslow mercury intrusion porosimeter was used to test aggregate particles operated up to pressures of 15,000 psi. In using the porosimeter, the aggregate particle is first evacuated down to a pressure of 50 μ of mercury, and then mercury is forced into the pores by applying pressure. The pressure is applied in steps up to 15,000 psi. Since the diameter of pore that the mercury enters is inversely proportional to applied pressure, a pore-size distribution curve may be plotted. The principle of operation of the porosimeter is described by Washburn (11) and details of operation may be obtained from the American Instrument Company, Silver Spring, Md.

One problem was to select aggregate particles for porosimeter testing that would be representative of the aggregate source in question. Aggregates A, B, C, D, and H were quite uniform and the results given are the average of 6 different runs on 3 similar aggregate particles (2 particles broken from a larger one). The other aggregates, E, F, and G, were heterogeneous but were mostly chert (but multitudinous varieties of chert). A petrographer selected 12 particles from these aggregates, with emphasis on the particles having the greater population. The results given are a direct average of these 24 runs (12 particles, 2 runs each). It is recognized a better solution to the problem is required; however, it is felt that the procedure used is not without merit.

TABLE 2

SUMMARY OF 100-CYCLE DURABILITY FACTORS

Mix Design	No. of Specimens	DF_{100}		90% Conf. Limits
		Avg.	σ	
A	10	97.5	2.62	99.0, 96.0
B	10	98.5	1.10	99.1, 97.9
C	8	83.9	7.95	89.2, 78.6
D	5	40.4	15.7	55.4, 25.4
E	19	3.0	1.7	3.7, 2.3
F	11	1.0	0	1.0, 1.0
G	12	5.1	1.9	6.1, 4.1
H	8	58.6	11.6	66.4, 50.8

RESULTS AND DISCUSSION

Terminology

Results of this study are presented using several terms which are described as follows:

1. DF_{100} is the durability factor at 100 cycles as described by ASTM C 290 63T. These factors range from 0 to 100 percent.
2. n is porosity and is defined as the ratio volume of voids to the volume of solids and voids, expressed in percent. When a percent porosity is given in the results for a definite pore-size interval, such as between 8 and 4 μ, this is represented by the ratio of volume of voids within that interval to the volume of solids and total void volume. Thus, when the porosity values for all of the pore-size intervals are summed, it should agree with total porosity.
3. p is pore index and is determined by summing the porosities at points corresponding to a predetermined set of diameters and dividing the sum by 10. The diameters used in this study were: 8, 4, 2, 1, 0.5, 0.2, 0.1, 0.05, and 0.02 μ. Thus, the pore index is a cumulative value based on summing porosities have increasing values. Nine porosity values are summed, down to 8 μ, down to 4 μ and finally down to 0.02 μ. The final porosity summed is actually the total porosity.

Results

A summary of 100-cycle durability factors (DF_{100}) is given in Table 2. The relationships between DF_{100} and absorption, pore index and total porosity are shown in Figures 1, 2, and 3, respectively. Figures 4 through 13 show the relationships between DF_{100} and porosities for different pore-size intervals.

Discussion

As is normally the case, Figure 1 shows that the aggregates making very durable concretes had relatively low absorptions. The other aggregates had relatively high absorption values; aggregate G had the lowest, about 4.4 percent. Most researchers would agree that an aggregate having an absorption of 5 percent or more would have a high potential for producing a concrete of low durability. There are, of course, exceptions to this; aggregate H, a blast furnace slag, is one. This aggregate has a very high absorption, is very porous, yet performed reasonably well in the freezing-and-thawing test. Available information indicates that the field performance of pavement concrete made with this aggregate is very good.

Aggregate D also has a very high absorption and a durability not as low as compared to aggregates E, G, and F. However, inasmuch as aggregate D

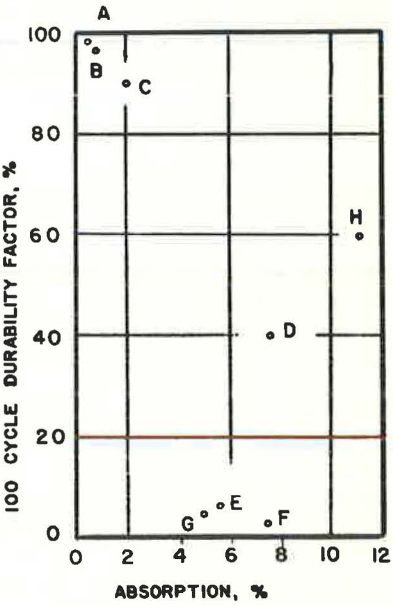

Figure 1. Relationship between DF_{100} and absorption.

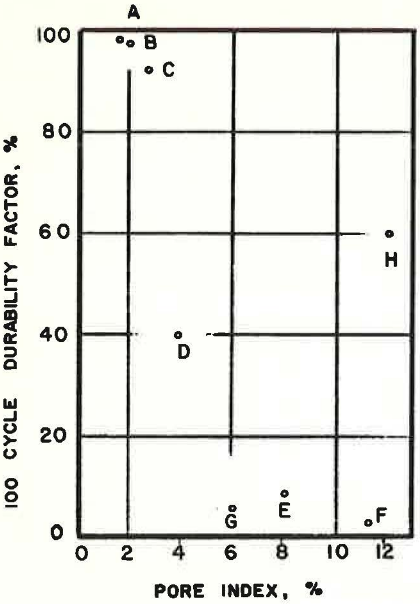

Figure 2. Relationship between DF_{100} and pore index.

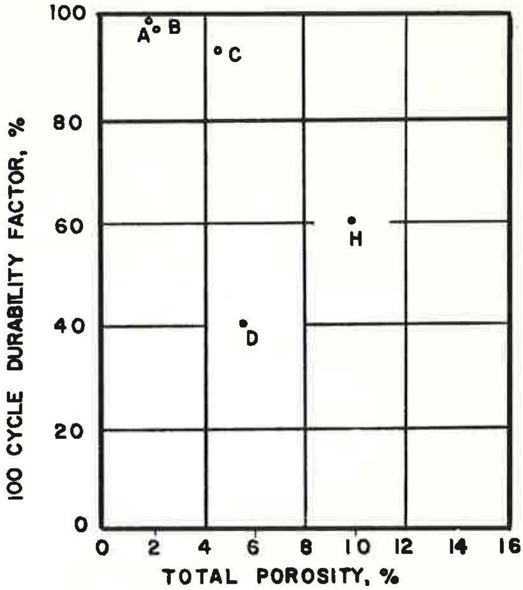

Figure 3. Relationship between DF_{100} and total porosity.

(hand-picked shale) was used in only the 1 to ½-in. size range (aggregate A was used for the ½ to ¼-in. size), it was a blend with an aggregate of known high durability; thus, one might expect lower durability if the shale were used throughout the coarse aggregate graduation. Aggregates E, G, and F all have poor service records and all have a fairly large quantity of porous chert. Their absorptions are reasonably high and their durability very low. Thus, if the durability of aggregate D were lowered and the highly absorptive slag (H) were eliminated, a fairly good relationship would exist between absorption and durability factor.

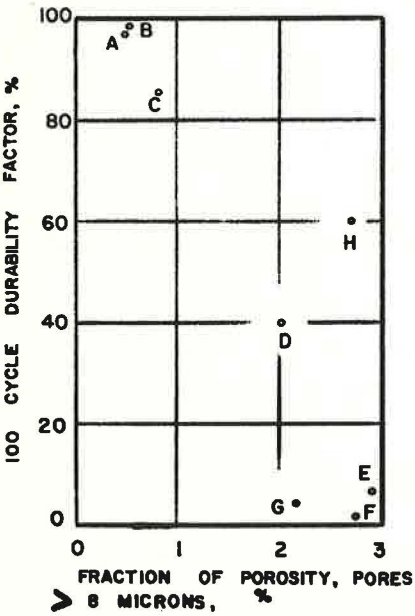

Figure 4. DF_{100} vs fraction of porosity of pores $> 8\ \mu$.

Pore index (Fig. 2) is apparently about as well related to DF_{100} as is percent absorption for the aggregates tested. Aggregates E, F, and G, the low-durability aggregates, all have higher pore indexes than the remaining aggregates (again excepting aggregate H). The high durability aggregates (A, B, and C) have the lowest pore indexes. However, it would not seem that the relationship shown is any better than the one given for absorption.

Total porosity (Fig. 3) shows a rather scattered or shotgun pattern with no real relationship discernible. Aggregates A, B, and C, although having lower porosities than the aggregates making low-durability concrete, have fairly high porosity values, in the case of aggregate C, more than 4 percent. Thus, here it would seem that total porosity is a relatively poor indicator of concrete durability, at least as far as the aggregates studied are concerned.

The breakdown of total porosity into fractions of porosity for different sizes of pores shows some interesting facets. For the data for >8-μ pores (Fig. 4),

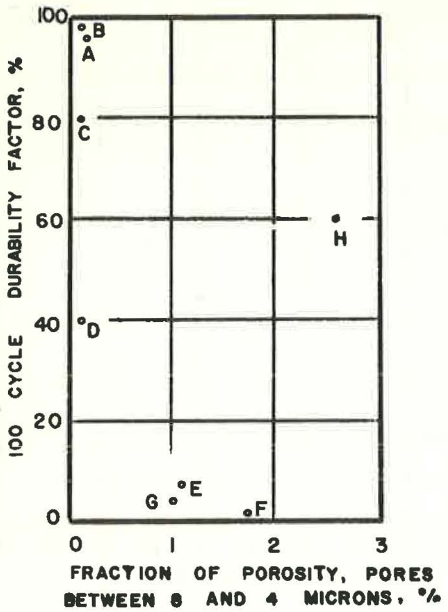

Figure 5. Relationship of DF_{100} vs fraction of porosity of pores between 8 and 4 μ.

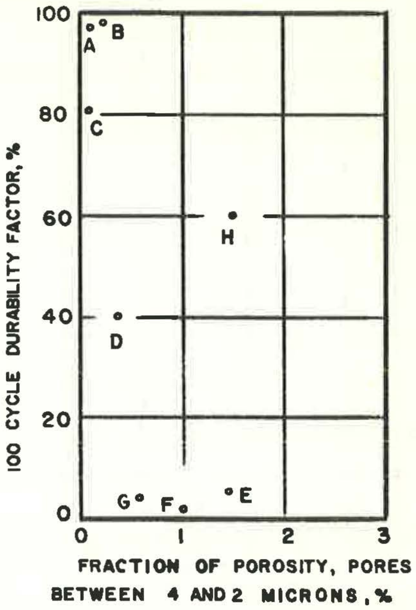

Figure 6. DF_{100} vs fraction of porosity of pores between 4 and 2 μ.

the porosity fraction seems to relate fairly well to concrete durability factor, again with the exception of aggregate H which has an abnormally high value, and for the value of aggregate D which is just about as high as the low-durability aggregate G. The high-durability aggregates, however, have low porosity values in this pore-size range as compared to the low-durability aggregates.

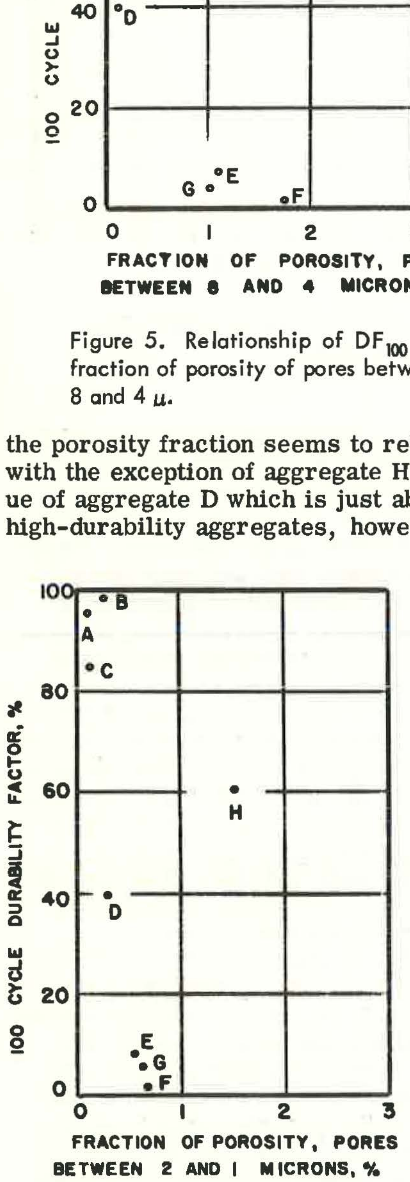

Figure 7. DF_{100} vs fraction of porosity of pores between 2 and 1 μ.

For the 8 to 4-μ range (Fig. 5), again aggregates A, B, and C have very low porosities as compared to aggregates E, F, G. Interestingly, shale aggregate, which is presumably of low durability and high absorption, also has a very low porosity in this size range. Compared with porosities for the other size ranges, the pore space for the shale is mainly in the + 8 μ and -0.2-μ range.

In the 4 to 2-μ range (Fig. 6), aggregates A, B, and C are well separated from aggregates E and F, but G is not so well separated. For the 2 to 1-μ range (Fig. 7), a fairly good separation would have existed between the high and low-durability aggregates except for aggregate B which shows relatively high porosity. In the 1 and 0.5-μ range (Fig. 8), aggregate B again confuses the relationship. In the 0.5 to 0.2-μ range (Fig. 9), aggregates A, B, C, and D have about the same fractional porosity, and these porosities are not much lower than that for the worst performing aggregate (F). In the much smaller size pores (0.2 to 0.1-μ range, Fig. 10), all the aggregates have fairly high values for this range except that aggregates A and B have low fractional porosities. In the still smaller range from 0.1 to 0.05 μ (Fig. 11), aggregates A and B have fractional porosities

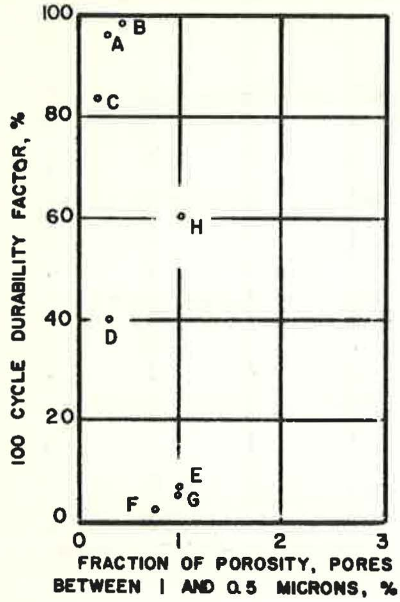

Figure 8. DF_{100} vs fraction of porosity of pores between 1 and 0.5 μ.

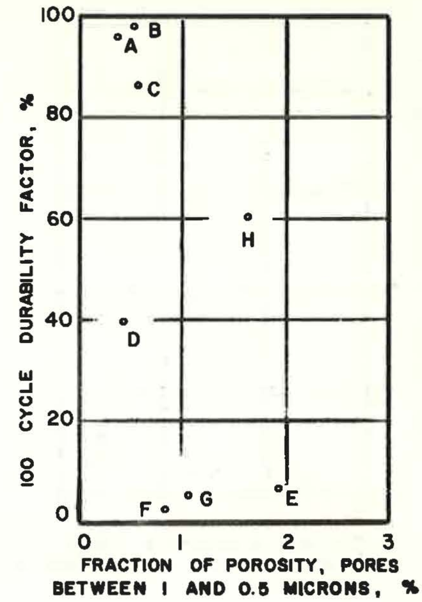

Figure 9. DF_{100} vs fraction of porosity of pores between 1 and 0.5 μ.

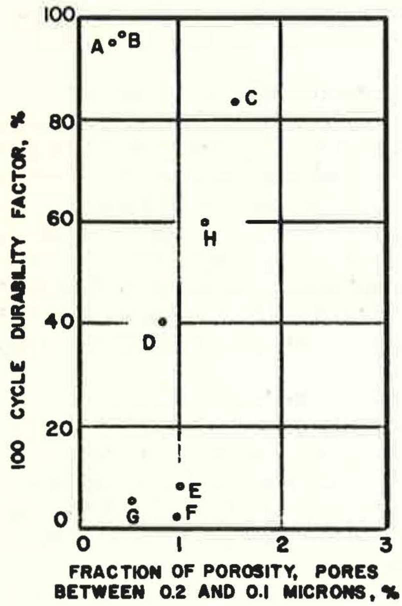

Figure 10. DF_{100} vs fraction of porosity of pores between 0.2 and 0.1 μ.

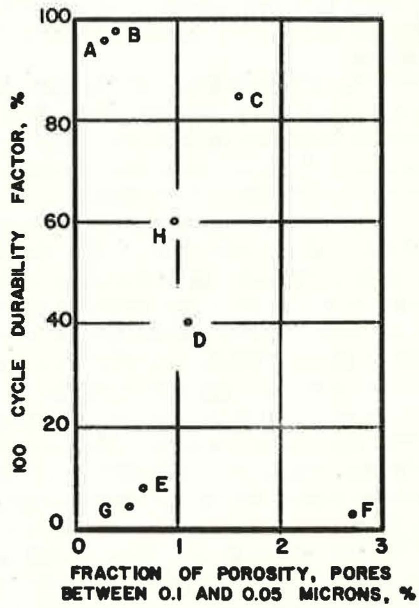

Figure 11. DF_{100} vs fraction of porosity of pores between 0.1 and 0.05 μ.

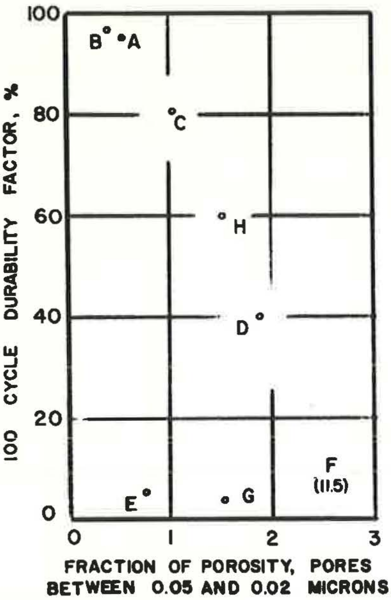

Figure 12. DF_{100} vs fraction of porosity of pores between 0.05 and 0.02 μ.

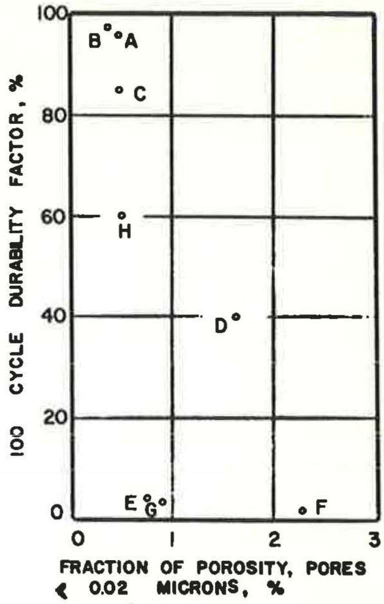

Figure 13. DF_{100} vs fraction of porosity of pores < 0.02 μ.

almost at the same level as the poorly performing aggregates E and G, so apparently no relationship exists. For the 0.05 and 0.02-μ range, poor performing aggregates E and G fall out of any possible relationship. In the less than 0.02-μ range, again aggregates E and G have about the same fractional porosity as the high-durability aggregates.

Of the data shown (Figs. 4-13), probably the best relationships were in the > 8-μ range and the 4 to 2-μ range, with the former having the best relationship of fractional porosity to concrete durability. The delineation mark of 4 μ has appeared in the literature as being the pore size that is most destructive in freezing and thawing. This is a pore-size range which is sufficiently small to permit the production of destructive hydraulic pressures, and yet is large enough to hold sufficient water to do some damage. Aggregate H is an exception in the relationship for the 4 to 2-μ range, but it has a peculiar pore structure as compared to most natural aggregates. Because of the vesicular nature of the pore structure of the slag, pores probably are less easily filled with water and thus have a lesser tendency to cause damage. The many large pores that do fill with water probably are able to drain by gravity.

Before summarizing our conclusions from this study, the shortcomings of the mercury-intrusion principle for determining porosity and pore-size determination should be discussed briefly. The calculation of the pore size is based on the assumption that the pores are cylindrical and interconnected. One possible important source of error is where there may be a large pore accessible only by a very narrow capillary. It will require a very strong pressure to force the mercury through the small capillary and to fill the large pore. Yet, as far as the reading from the porosimeter is concerned, the actual pore volume would be the sum of the large pore and the small capillary with the diameter approximating that of the small capillary. As serious as these assumptions may be, nevertheless, it would seem that if the values obtained are examined in the relative sense, information derived from such tests can be valuable.

SUMMARY AND CONCLUSIONS

It might be asked, "If, for example, it were shown that if concrete were made with an aggregate having more than "X" percent porosity in the + 8-μ range along with other specified characteristics, it would have a DF_{100} of less than 20 percent, than would the test described be of practical value?" A qualified answer to this would be "yes," if the aggregate were uniform and if it could be shown that a low DF_{100} of 20 percent means poor field performance. The value of freezing-and-thawing tests and durability factors in predicting field performance of concrete is another story and another argument. From the work presented here, it appears that there may be promise in relating specific pore characteristics with durability factor. Hopefully, this work may encourage further work on a much wider variety of aggregates.

In summary, only a few conclusions can be drawn, and even these must be qualified in the sense that they apply only to the group of aggregates studied.

1. With the exception of aggregate H (blast furnace slag), absorption, pore index, total porosity (by mercury intrusion), and porosities in the range of + 8 μ appear to separate the high from the low-durability aggregates.
2. The best separation between high and low-durability aggregates was with the + 8-μ porosity values.

ACKNOWLEDGMENT

The aggregates tested are described in some detail in NCHRP Report 12 (9). Although the freeze-thaw data for these aggregates are given in that report, the porosity data were obtained after the completion of the NCHRP project and have not been reported.

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Factors Affecting the Durability of Concrete Bridge Decks

Phase I: Construction Practices

C. F. STEWART and B. F. NEAL, California Division of Highways

Construction history was recorded on 28 concrete bridge deck placements incorporating planned variations in concrete slump, strike-off machine, finishing, texturing and curing.

The effect of these variations on deck durability is evaluated by comparing recorded construction data with cracking, surface defects, and abrasion and skid-resistance properties of the finished deck. A deck cracking index, a key factor in the evaluation, has been developed.

Comparison of initial and pre-traffic crack surveys show concrete age to have a significant effect on cracking. Also, the pre-traffic cracking pattern is significantly unlike that found on similar structures after normal traffic usage. Hence, conclusions on the study's objectives are deferred pending a post-traffic survey.

Normal construction problems hampered control of variations and data collection. These problems will probably reduce the study's overall effectiveness.

•THE effect of construction practices on concrete deck durability is the objective of a study on 7 grade separations, constructed by Peter Kiewit Company, on I-210 near Los Angeles. These structures were selected for the construction practices phase of an extensive study of deck durability due to the close proximity of the batching plant and the fact that each deck pour would be approximately the same size and shape. This would provide an opportunity to virtually eliminate the effects of common variables such as mixing time, aggregates, structure configuration, and duration of placement.

Figure 1. Typical structure.

Each structure is a single-span concrete box girder supported on abutment walls of 4-deg maximum skew. Thus, negative moment and skew variables are not present. Most significant, however, is that the absence of approach fills kept vehicular traffic off the decks for a considerable time after they were constructed,

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MACHINES

a. Bidwell (control)
b. Borges
c. Trueline
d. Clarey
e. Clarey overworked

FINISHING

a. Float once, approximately 45 min after strike-off, with a wooden 16-ft longitudinal plow handle float (control)
b. Float once as early as possible with wooden float
c. Float once as late as possible with wooden float
d. Float twice—early and late
e. Float once at standard time with two 6-in. diameter aluminum pipes placed parallel at 1-ft centers.

TEXTURING

a. Stiff bristle broom (control)
b. Burlap drag
c. Wooden finishing float

CURING

a. Fog as needed during finishing followed with wet rugs when set (control)
b. Delayed placement of wet rugs
c. Monomolecular evaporation retarder placed during strike-off and finishing operations followed with wet rugs when set
d. Membrane curing compound placed after texturing followed with wet rugs the next day.

SLUMP

a. 4-in. (control)
b. 2½-in.
c. 6-in.
d. pozzolith 8

Figure 2. Planned placement variables.

thus affording an excellent opportunity to separate shrinkage and traffic influence on deck cracking.

The structures vary in length from 60 to 91 ft (Fig. 1) and in width from 146 to 170 ft. The excessive width necessitated each deck being placed in 4 separate units. This resulted in 28 placements available for study.

VARIABLES

Seven of the placements, one on each structure, were selected as controls to which the others could be compared. Variations were made in one or more of the construction techniques in the remaining 21 placements. These included concrete slump, type of strike-off machine, type and application timing of finishing floats, method of texturing, and method and time of applying the cure. The control and variable techniques are shown in Figure 2.

A 4-in. slump was the control with 2½ and 6-in. slumps as variables. (California currently equates 1 in. of Kelly ball penetration to 2 in. of slump, but plans to change over to penetration limits in the near future.)

The Bidwell strike-off machine (Fig. 3) was adopted as a control machine. Others included: Trueline (Fig. 4), Borges (Fig. 5), and Clarey (Fig. 6).

The control finishing float was a wooden 16-ft longitudinal plow handle float (Fig. 7) applied approximately 45 min behind the strike-off machine. Variables included one floating as close behind the strike-off machine as possible, one floating as late as the workability of the concrete would permit, and a combination of both an early and late floating, all with the wooden float; floating at the standard time with two 6-in. diameter, 10-ft long aluminum pipes in tandem (Fig. 8); and floating at the standard time with a single 4-in. diameter 10-ft long aluminum pipe equipped with a handle for full floating control (Fig. 9). All floating was transversely applied.

The standard texturing was achieved with a stiff bristle broom (Fig. 10). The texturing variables were burlap drag (Fig. 11) and natural texturing by the longitudinal wooden float (Fig. 12). All texturing was transversely applied.

Figure 3. Bidwell strike-off machine.

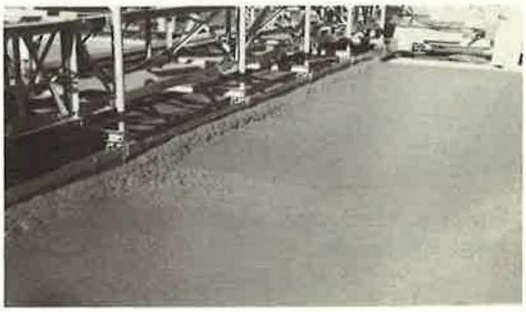


Figure 4. Trueline strike-off machine.

Figure 5. Borges strike-off machine.

Figure 6. Clarey strike-off machine.

Figure 7. Sixteen-foot longitudinal plow handle float.

Figure 8. Aluminum pipes (6-in. diameter, 10 ft long) float in tandem.

Figure 9. Aluminum float (4-in. diameter, 10 ft long) with handle.

Figure 10. Surface texturing with stiff bristle broom.

Figure 11. Surface texturing with burlap drag.

Figure 12. Natural texturing by longitudinal wooden float.

Figure 13. Placing rugs for a wet rug cure.

Figure 14. Applying liquid membrane-type curing compound.

The standard 7-day cure was provided by wet rugs (Fig. 13) with variations of delayed cures and liquid membrane-type curing compounds (Fig. 14). The membrane cures were supplemented with the wet rugs the day after the cure began. A monomolecular film evaporation retarder was used on 4 placements before the standard cure.

DATA COLLECTION

Previous experience has shown that normal construction records do not contain enough detail to correlate final results with placement conditions. The records reflect average construction conditions, whereas the final results are most often affected by conditions which vary from the average. To furnish a more complete picture of placement conditions, an unprecedented quantity of data were collected during these placements.

During each placement, a minimum of 7 men were engaged in either collecting data or assisting in maintaining construction control. Two men at the batch plant checked the batch proportions, obtained cement and aggregate samples, and recorded the quantity of water added to the mix from cleaning operations. One man at the job site controlled the water added to produce the desired slump, recorded slump (Fig. 15) and concrete temperature measurements, made unit weight tests (Fig. 16), and fabricated test specimens (Fig. 17). Two men conducted normal inspection duties, coordinated control of operational timing variables, and placed grid reference points in the finished concrete (Fig. 18).

A majority of the construction data was collected by a 2-man observation team. A typical packet used by this team to record events during each placement is included as an Appendix. The collected data include:

1. Condition of forms, support and tying of reinforcing steel, and depth of cover over the steel.
2. The in-place location of each batch of concrete (Fig. 19).
3. A time history on each batch of concrete: total batch time, and the time when each batch was placed in the deck, vibrated, struck-off, finished, textured, and cured; also, the number of passes made by the strike-off machine and finish float was recorded.
4. Irregularities, such as over or under vibration, excessive bleed water areas (Fig. 20), premature drying areas, and areas where excessive walking in the fresh concrete occurred (Fig. 21).
5. Weather history of temperature, humidity, wind direction and velocity, and rate of evaporation (Fig. 22).

Figure 15. Measuring concrete slump with a Kelly ball.

Figure 16. Making unit weight test.

Figure 17. Fabricating concrete cylinder test samples.

Figure 18. Placing grid reference points.

Figure 19. Recording concrete placement.

Figure 20. Excessive bleed water.

Figure 21. Excessive walking in the fresh concrete.

Figure 22. Recording climatological data.

6. Concrete temperature at various time intervals after placement.

Time placement plots of each operation furnished a visible history of the respective placement (Fig. 23).

To reference events during construction with final results, the decks were laid out in a grid pattern at 10-ft intervals along girder lines. For easy reference, most data were recorded on duplicate grid sheets (Appendix).

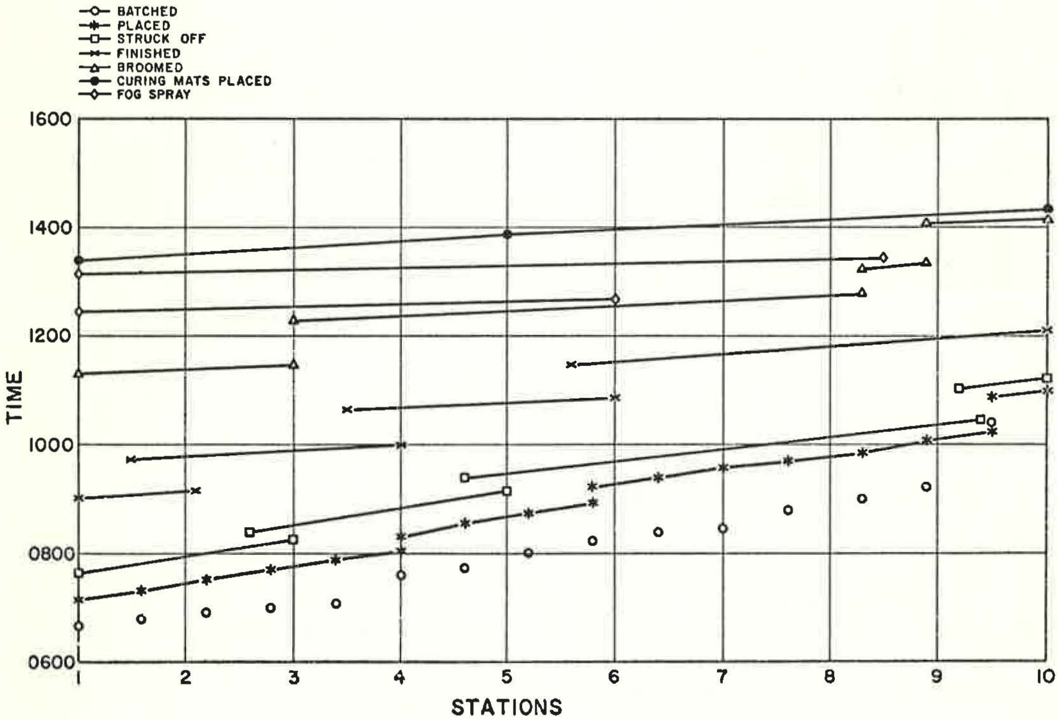


Figure 23. Time placement plots; time study, observation No. 4.

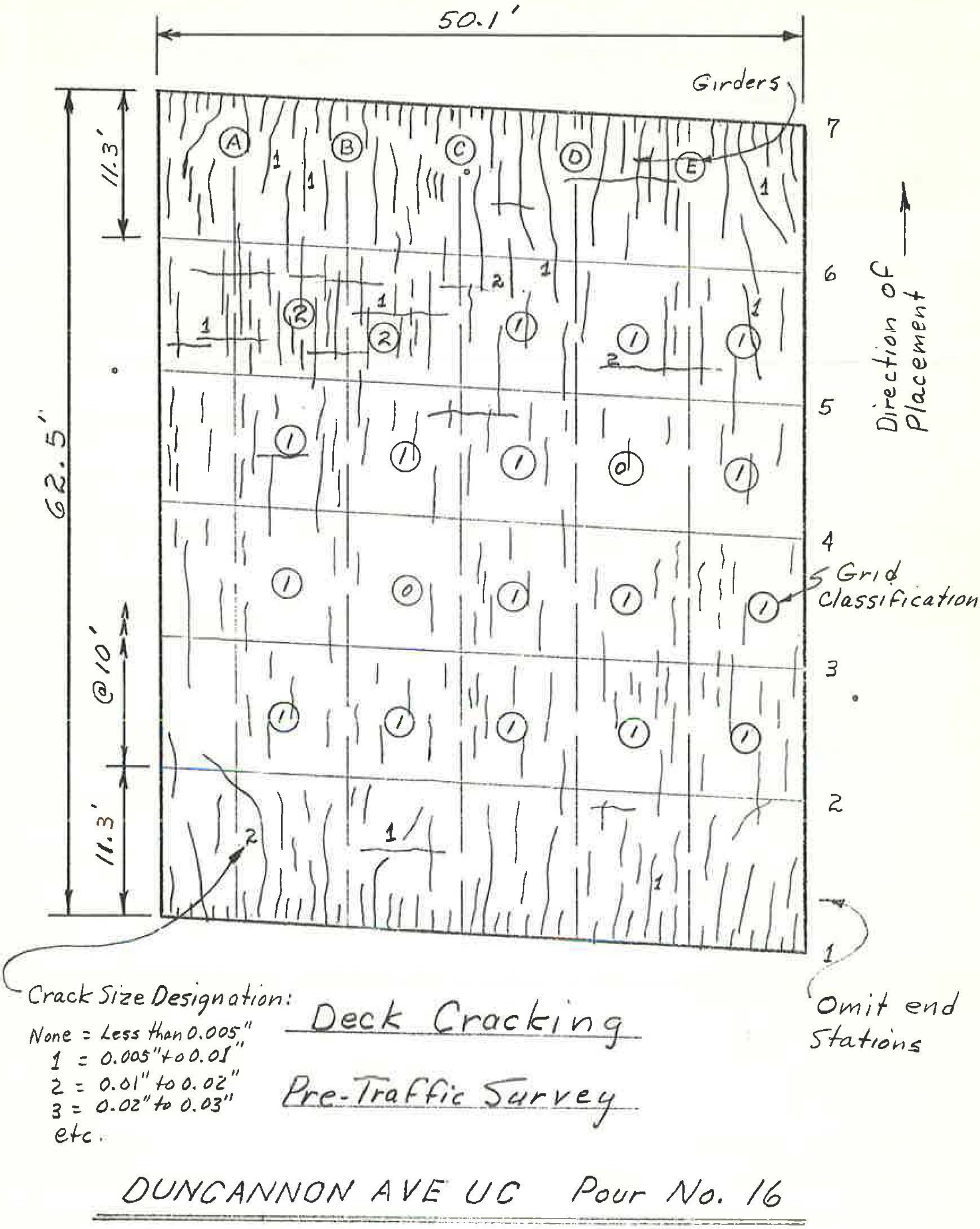


Figure 24. Crack severity rating.

Observation No. 16

Crack Width 0.005 In. or Less

Classification	No. of Grids Appearing	Classification Times Number of Grids
0	2	0
1	16	16
2	2	4
		Sum 20

$$\frac{\text{Sum}}{\text{Total grids}} = \frac{20}{20} = 1.0$$

Crack Width Greater Than 0.005 In. but Less Than 0.02 In.

$$\frac{\text{Sum}}{\text{Total grids}} = \frac{4}{20} = 0.2$$

Crack Width 0.02 In. or Greater

$$\frac{\text{Sum (1.5)}}{\text{Total grids}} = \frac{2(1.5)}{20} = 0.2$$

CRACK SEVERITY RATING: 1.4

Figure 25. Deck crack severity rating.

EVALUATION

The variables in this study are to be evaluated by comparing the collected construction data with properties of the finished deck. These properties will include cracking and other defects, and abrasion and skid resistance.

A number of cores will be taken both before and after traffic is allowed on the decks. Of primary interest are those taken through the same cracked areas at different times. These cores will be examined visually and microscopically in the anticipation that some measure of progression can be determined of both the macrocracks and the microcracks. In addition, some cores will be tested for abrasion resistance to see if any correlation can be found between this property and deck durability.

The most practical way to compare the influence of controlled study variables on deck durability appears to be in reducing the various durability parameters (cracking, abrasion resistance, etc.) into a single quantitative value. Before the single value can be obtained, however, each of the parameters must be evaluated quantitatively. Developing a system whereby this can be done is one objective of the study. So far, all efforts in this direction have been concentrated on a deck-cracking rating system, and one has been developed which appears to be a useful tool in comparing overall cracking severity in concrete decks.

CRACK RATING

Rating the crack severity of concrete decks is highly subjective. There is not always agreement on which should be given the greatest weight, in respect to detrimental effect on a deck, crack size or total number. Generally, crack size is considered to be more harmful, and the rating system developed reflects this. However, a large number of small cracks could eventually cause deterioration, hence the system promotes the assigned weight of this condition.

Observations are referenced to a grid system; the girder lines and 10-ft longitudinal stations, for instance. The cracks are then located, marked and sized, and the information recorded on a grid sheet (Fig. 24).

In making the rating from the plotted information, the cracks are grouped, or counted, according to their width: < 0.005 in., > 0.005 but < 0.02 in., and 0.02 in. or greater. The groups are then treated as follows:

<u>Widths 0.005 In. or Less</u>

1. Classify each grid according to the number of < 0.005 in. cracks that appear in it: "0" for 0 to 3 cracks, "1" for 4 to 10 cracks and "2" for 11 or more.
2. Multiply each classification number by the number of grids in which it appears.
3. Divide the sum of the products in Step 2 by the total number of grids. This is the small crack numerical rating.

Figure 26. Conducting a deck crack survey.

Widths Greater Than 0.005 In., but Less Than 0.02 In.

The middle size cracks are rated by dividing the total number appearing in all of the grids by the total number of grids.

Widths 0.02 In. or Greater

Before rating the larger cracks, their weight is promoted by multiplying the total number appearing in all the grids by 1.5. They are then rated by dividing this product by the total number of grids. The sum of the three ratings gives a crack severity rating for the deck. A sample is shown in Figure 25.

Concrete construction practices at the beginning and ending areas of deck placements generally differ from the central area, both in placing and finishing. Furthermore, the underlying support (usually rigid end diaphragms) is different. These local factors appear to create different cracking patterns at the bridge ends from those manifested in the central deck area. Therefore, end areas are excluded in the rating determination.

CRACK SURVEY

Two crack surveys have been made: initial and pre-traffic. Age of the concrete varied from 21 to 202 days for the initial and from 295 to 492 for the pre-traffic. For each survey, the deck was thoroughly washed (Fig. 26), and a 4-man team systematically examined the deck for cracks. As cracks were found, a keel mark was placed alongside them. The larger ones were measured and coded according to their width. The location and width of each was later indicated on the respective grid sheet.

DEFERMENT

From the wealth of data collected, there is ample reason to believe that considerable knowledge will be gained regarding the effect of certain construction practices on

Figure 27. Comparison of initial and pre-traffic crack surveys.

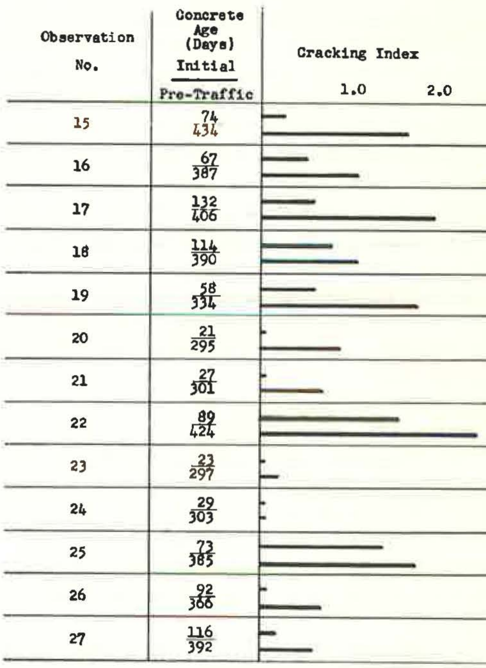


Figure 28. Comparison of initial and pre-traffic crack surveys.

concrete deck durability. However, the only yardsticks available for comparing the practices at this time are the pre-traffic crack surveys. These surveys show concrete age to have a significant effect on deck cracking for several months after construction. The older pre-traffic cracking generally increased substantially above the initial cracking level (Figs. 27 and 28). The greatest change occurred in the number of cracks and the widening of smaller cracks; the width of larger cracks changed little. The surveys also show the pre-traffic cracking pattern to be unlike that found on structures after they have been under traffic a few years. (Pre-traffic cracking has a longitudinal orientation, whereas, post-traffic cracking usually has a transverse orientation.)

Because the cracking pattern and cracking intensity are expected to be markedly different after traffic uses the decks, it appears that conclusions based on the pre-traffic surveys would be premature. Consequently, conclusions will be deferred until after the post-traffic crack survey.

RESEARCH PROBLEMS

Instead of giving conclusions, this report will discuss some of the problems encountered during the project. These problems may be of interest to those concerned with bridge deck construction, particularly to those contemplating a similar research project.

From the beginning, it was considered important that an accurate accounting be maintained on the amount of water in each batch of concrete. Unfortunately, the accuracy of some of the accumulated data is not as good as desired. The water introduced at the plant or added at the site was metered, and the indicated amounts are probably reliable. But, water used to wash the mixing drum after discharging concrete was not entirely removed prior to charging the subsequent batch, and the amount present could be only roughly estimated. Also, variations in moisture content of the sand probably were not always accurately measured by the moisture meter. Another source of error was the practice of hosing off all cement dust and sand from the trucks after charging. The water entering the drums from hosing had to be estimated. Thus, the data accumulated to show total water and water-cement ratio in each batch of concrete have some margin of error.

Close observation of water is also needed in curing. It is well known that improperly cured concrete leads to cracking. Consequently, if uniformity in curing is not maintained during a research project with an objective of determining effect of other variables on cracking, the results could be greatly altered by the curing variable, thereby defeating the objective. Control of curing was delegated to regular construction personnel, but it was found that at times the curing did not receive the attention it deserved for research purposes. In future studies, an inspection form will be provided that is to be filled in periodically in order to act as a reminder to the inspector, draw attention to the importance of curing uniformity, and provide a record of curing irregularities that might occur.

Visual evaluation of the properties and behavior of the fresh concrete did not always agree with the consistency as determined by Kelly ball "slump." Certain batches of concrete appeared extremely fluid when discharged from the placing bucket onto the deck forms. Other batches exhibited a large amount of free water at the front of the placing and finishing operation. In spite of this, slump recorded for these batches is about 4 in. No explanation for this anomaly is apparent; however, it is believed that variation in aggregate gradation or caliber of slump measurements could be factors.

The biggest problem during the construction phase was controlling, or scheduling, the planned variables and avoiding unplanned variables. Planned variables were often disrupted by either equipment breakdown, the weather (remaining mild when variables to combat hot or windy conditions were being used), unavailable machinery, or insufficient finishing personnel. In most cases, it was possible to work around the disruptions by changing a variable during placement or by reclassifying the variable after studying the placement data. There was one occasion, however, when it was impossible to do either and the placement was declared unsuitable for any of the variable classifications. Unfortunately, most of the events that disrupted the scheduling were common problems in construction, and as such were unavoidable. Scheduling several repetitions of single variable groups will mitigate this problem.

An anticipated problem is how to isolate the numerous variables introduced. The variables are grouped into so many combinations that isolation will be very difficult. For some, about all that can be expected is an indication of their effect on deck durability. In retrospect, the number of variables should have been decreased and the number of placements of selected combinations increased.

The problems encountered will no doubt reduce the overall effectiveness of the study. Nevertheless, considerable knowledge is expected to be gained, not only on the study's objectives, but also in more clearly defined directions for further research.

SUMMARY

1. Execution of planned variables that relate to weather conditions or coordinating variable timing with contractor's operations are difficult problems.
2. Numerous repetitions are needed of each planned variable combination to minimize conflict with weather and normal construction variables.
3. Accurate accounting of total water in a transit mix delivery is very difficult, but essential to a research project.
4. Concrete age is a significant factor in the cracking pattern during the first months after placement.

ACKNOWLEDGMENTS

It took many people to plan and execute the construction phase of this research project. Each served a vital part and deserves recognition. However, it is not practical to list all those who took part. The authors, therefore, wish to thank as a group all who contributed to the project, and give special recognition to a few: W. Ames, J. Woodstrom, and C. Sundquist of the Materials and Research Department; A. Rossing, the Bridge Resident Engineer, and his assistant H. Wolfe; and W. Egloff, of the Special Studies Section of the Bridge Department.

Appendix

FACTORS AFFECTING THE
DURABILITY OF CONCRETE BRIDGE DECKS
Construction Observations

PROJECT INFORMATION

Construction Variable: Late Finish

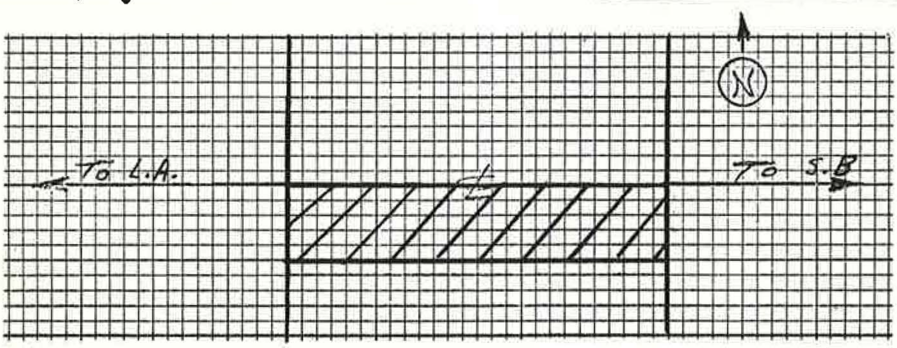
Date of Placement: 2-2-66 | Bridge No.: 53-1824

Contract No.: 14-042134 | Bridge Name: Highland Ave. OC

Road: 07-LA-210 - R33.7/R35.9 | Bridge Type: Box Girder ~ S.S.

Limits: Magnolia Ave. to Highland Ave.

Section Placed: First 1/2 of S. Half



Contractor: Peter Kiewit

Contractor (Structures): Same

Resident Engineer: Al Rossing

Bridge Dept. Repr.: Same

Bridge Inspector: H. Wolf & F. Bartley

Research Investigators: W. Egloff, B. Neal, C Sundquist

Comments:

14030 - 951128
19503 - 762500 - 35145

Observation No. Sample

MATERIALS

Concrete Supplier Consolidated Rock ~ Irwindale

Aggregate Source San Gabriel Wash ~ Irwindale

Cement (Brand, Source and Type) SW Portland Cement ~ Majave ~ Type II

Mixing Water (Source) City of Irwindale

Admixture None

Type of Mixing (Plant and Truck) 2 1/6 Yd Batch Plant - 3 batches per transit mix truck

Mix Design

	Wts.		Sp.Gr.	Abs.	SE/CV	LART 500 rev.	NaSO4	Mortar Strength
Sand	1312	(SSD)	262	1.2	82/	-	1.0	1.25
3/4x#4	970	(SSD)	265	1.2	/88	29%	1.0	-
$1\frac{1}{2}$ x#4	1050	(SSD)	266	1.0	/86	31%	1.0	-

Cement 564

Water 282

Admixture None

Notes:

FALSEWORK AND FORMS DESCRIPTION

(Pictures/sketches) Generally there is a 1/8" gap between the lost deck forms and girder stems. Lost deck forms appear to be solid enough.

Date falsework removed 2-23-66.

REINFORCING STEEL

(Ties, supports, etc.) Top mat tied every other lap. Is supported by plastic chairs setting on concrete blocks which are spaced @ 5'± C-C, longitudinally between girders. 1 3/4" Cover. Bottom mat tied every 1 in 5 laps. Is supported by plastic chairs spaced at 12"± along & between girders.

Observation No. Sample

SUMMARY SHEET

Concrete Delivery Data
and Test Results

Length of Haul 6.2 Miles, 15-20 Minutes

Type of Haul Roads AC & PCC Pavement

(Batch Data (Obtain all pertinent batch data from Resident Engineer)

Load No.	Ticket Number	Truck Number	Depart. Time	Arrival Time	Begin Disch.	End Disch.	No. of Revs.	W/C* (#/C.Y.)	Slump Inches	Temp. Degrees	Unit Weight	Air Content	Cement Factor
1	996-38	4501	0646	0708	0717	0728	177	286					
2	39	4518	0651	0710	0732	0742	200	283	3-4-3½	64°	153.2		6.10
3	40	4506	0702	0728	0745	0800	250	302					
4	41	4509	0715	0734	0800	0812	195	283					
5	42	08	0725		0816	0825	-	312					
6	43	01	0758		0847	0854	-	302	4-5-4½	63°	151.9		6.02
7	44	18	0802		0856	0904	-	295					
8	45	06	0813		0909	0916	-	287					
9	46	08	0830		0917	0927	-	292					
10	47	01	0848		0937	0945	-	283					
11	48	06	0908		1002	1009	-	283					
12	49	05	0938	0958	1010	1018	-	283	3-2½-3½	67°	152.6		6.07
13	50	04	0950	1008	1019	1025	-	277					
14													
15													
16													
17													
18													
19													
20													

*Obtain all necessary data to determine W/C per batch.
Obtain 9 cylinders on 2nd load, 3 for 7, 14, and 28-day strengths.
Obtain 3 cylinders near center of pour and near end of pour for 28-day strengths.

Observation No. Sample

METHODS AND EQUIPMENT

Placement Crane & Bucket

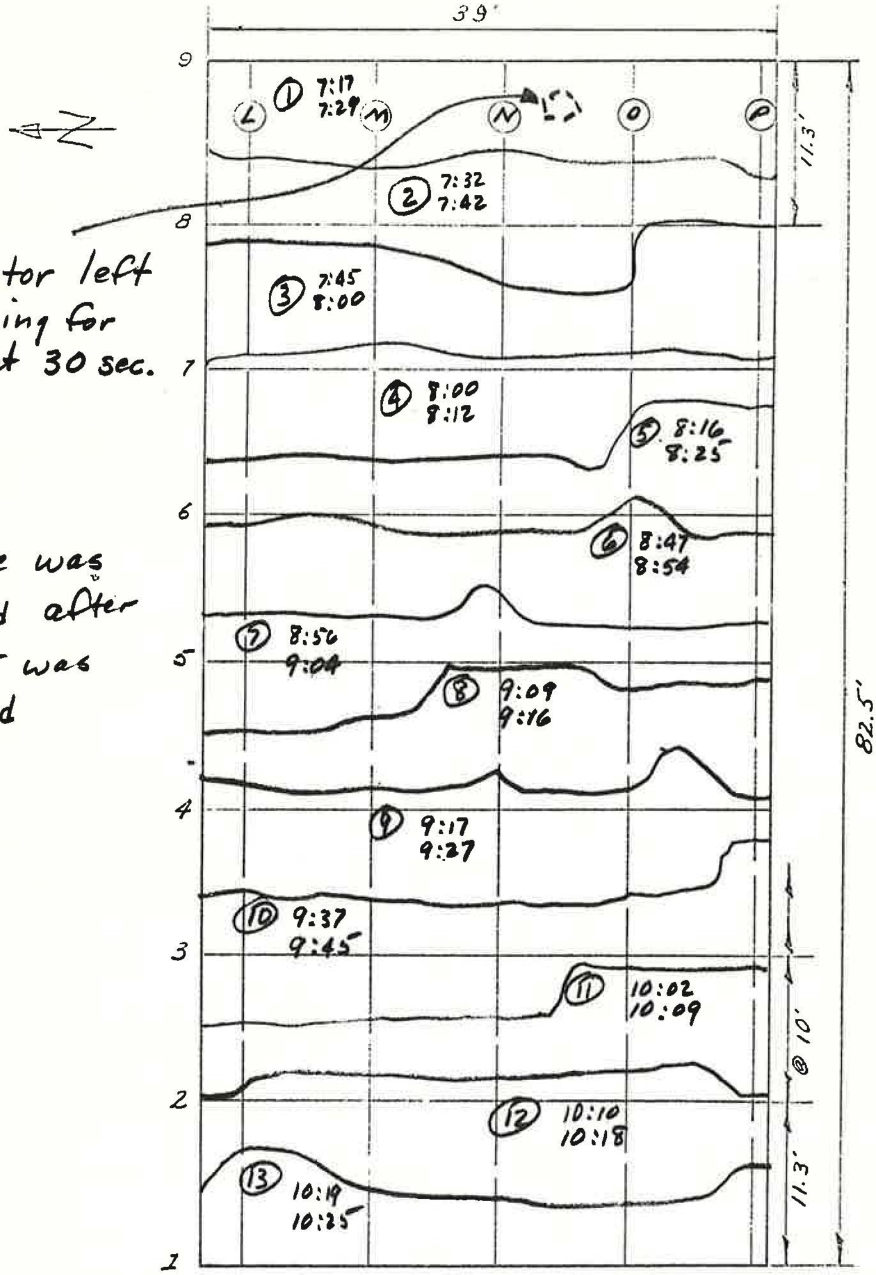
Vibration 2 1/2" McGinnis ~ 10,000 cycles/min

Strike-off Bidwell

Finishing 16' Longitudinal Wooden Plow Handle Float

Curing Wet Rugs

Observation No. Sample



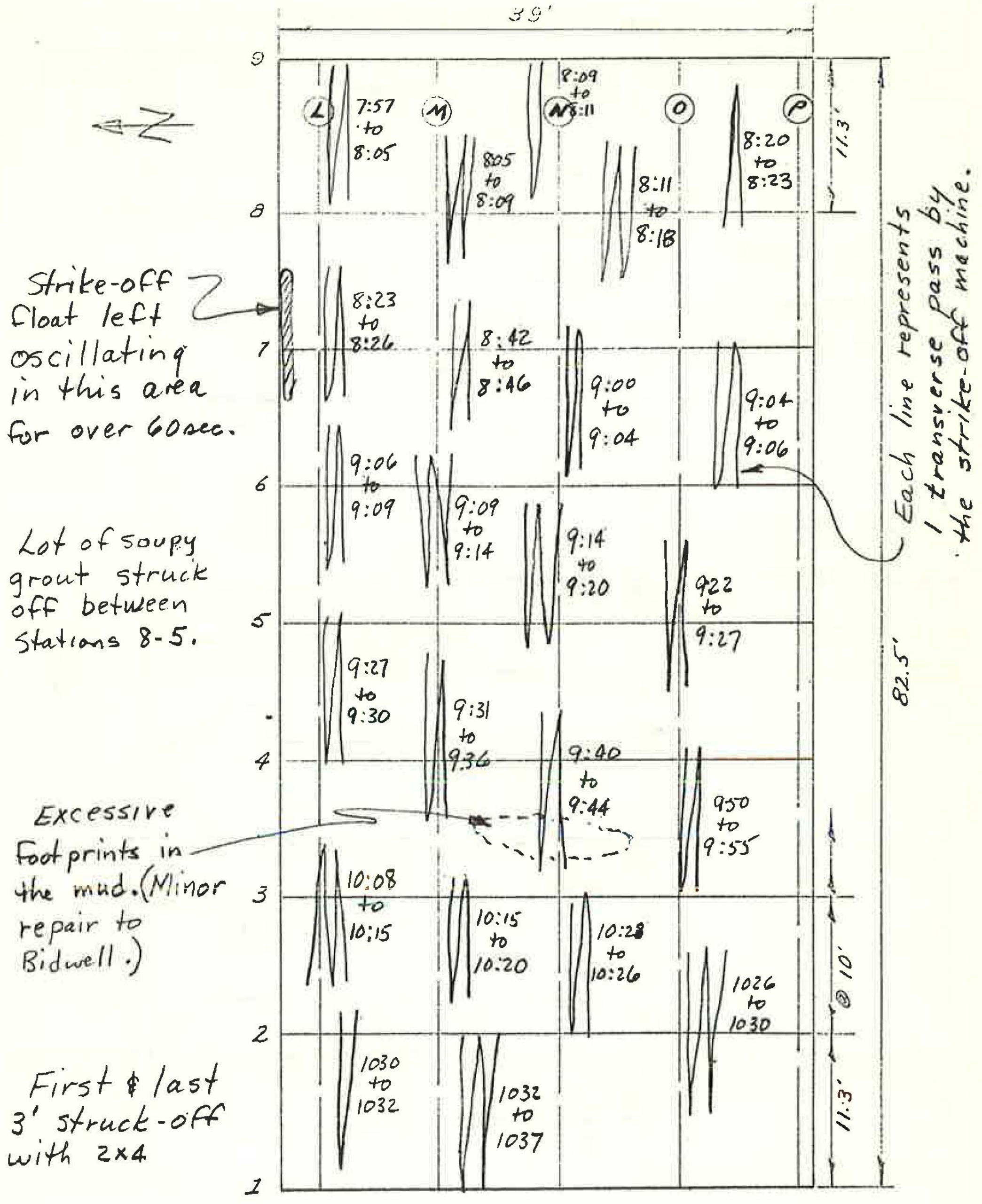
Vibrator left running for about 30 sec.

Crane was moved after load 5 was placed

Placing

Scale 1"=10'

HIGHLAND AVE U.C. Pour No. Sample

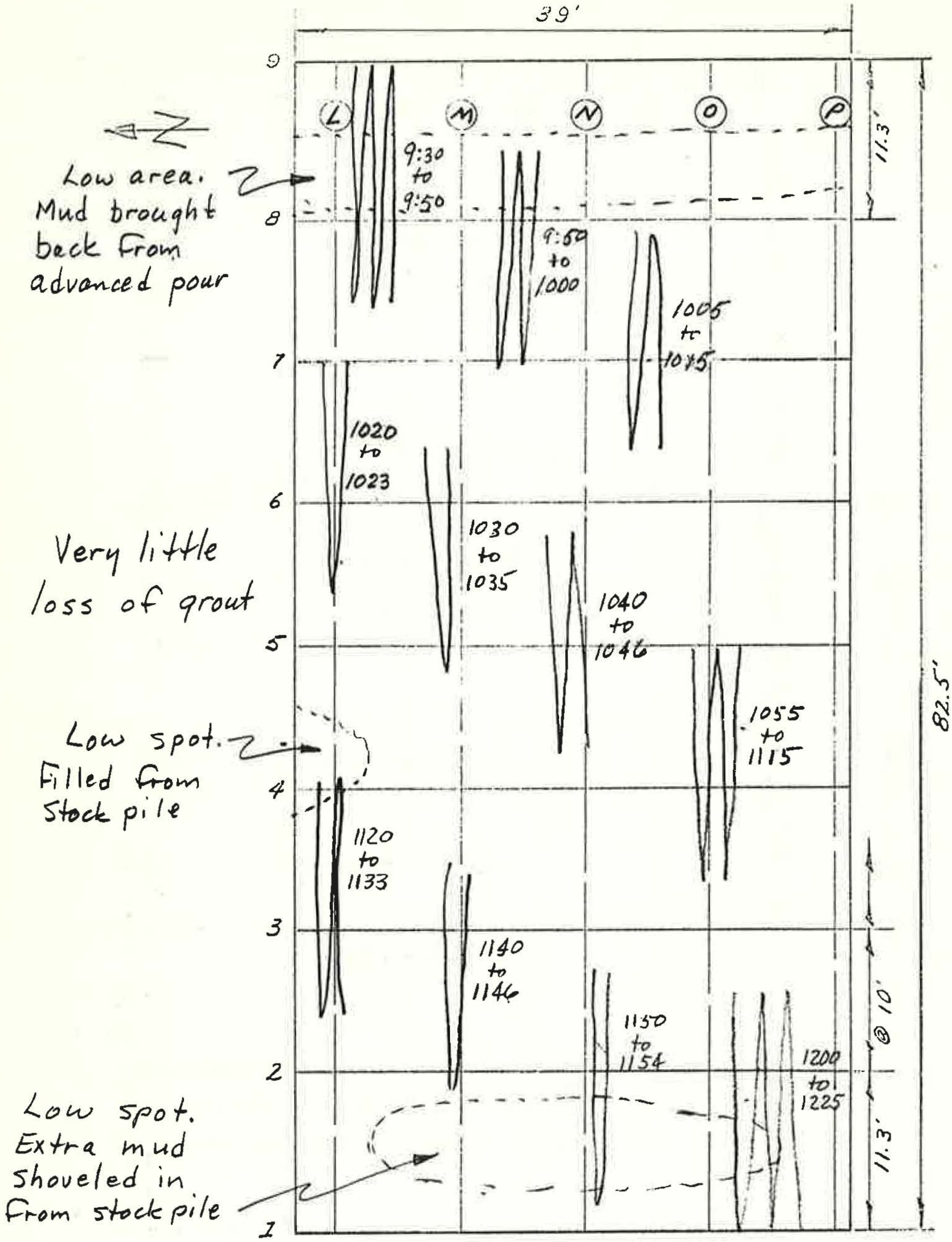


Strike-Off

Bidwell

Scale 1"=10'

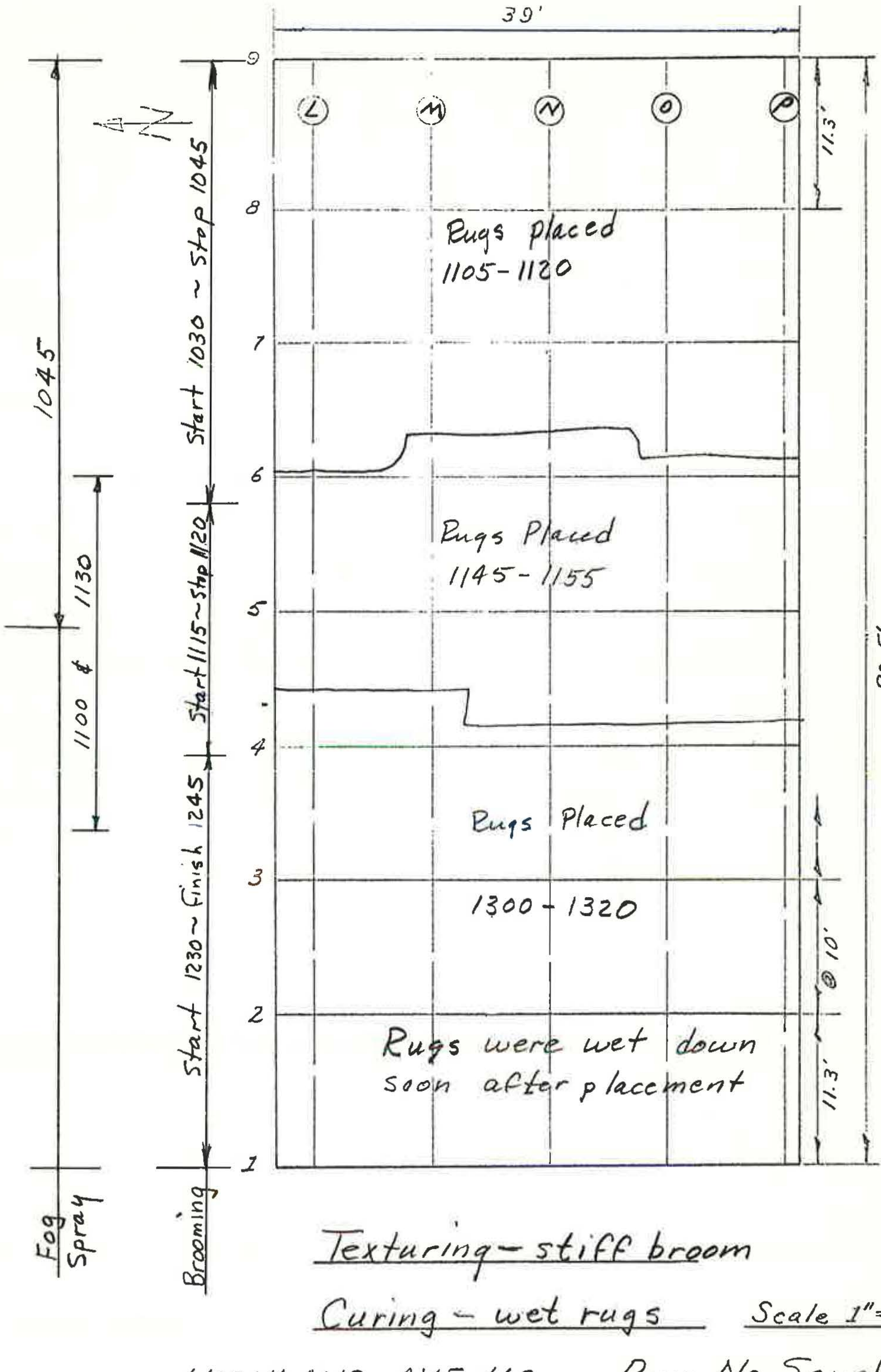
HIGHLAND AVE U.C. Pour No. Sample



Finishing (Late.)

16' Longitudinal-Wooden Scale 1"=10'

HIGHLAND AVE U.C. Pour No. Sample



Texturing – stiff broom

Curing – wet rugs   Scale 1"=10'

HIGHLAND AVE U.C.   Pour No. Sample