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Inv. 59-2-A
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1965

A STUDY OF DETERIORATION IN CONCRETE BRIDGE DECKS

Division of Materials and Research
Missouri State Highway Department

MISSOURI STATE HIGHWAY DEPARTMENT
DIVISION OF MATERIALS AND RESEARCH
RESEARCH SECTION

in
cooperation
with

BUREAU OF PUBLIC ROADS

A Progress Report On

Investigation 59-2A
HPR 1(4)-30

A STUDY OF DETERIORATION IN CONCRETE BRIDGE DECKS

A report of the findings from several independent studies into the problem of bridge deck deterioration.

Jefferson City, Missouri
October 1965

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SYNOPSIS

This investigation into the problem of bridge deck deterioration in Missouri was initiated in 1959. On August 1, 1961, the investigation was approved by the Bureau of Public Roads as a cooperative research project. This approval made Federal funds available for continuation of the investigation.

Several phases of the research have been in progress simultaneously, and progress reports on some of these phases have been distributed. Further discussion of data from these phases is not presented in this report. The data contained herein represents data on other completed (with one exception) phases. The one exception, Phase B, is divided into two parts; part one is completed, and data from it are presented in this report.

The report is divided into several parts. First, a description of the different types of bridge deck deterioration is given. This is followed by a detailed discussion of four phases of the investigation. These four phases include (A) a detailed survey of deterioration on 620 bridge decks throughout Missouri, (B) the observation of the construction procedures followed and the testing of fresh concrete used on eleven bridge deck pours, (C) the analysis of data from locations where concrete cores were drilled to determine the relationship between the depth of cover over the top reinforcing steel, the occurrence of rust on the reinforcing steel, and the occurrence of fracture plane deterioration, and (D) the physical testing of concrete cores to determine the differences between the concrete in deteriorated and undeteriorated areas.

Also, a hypothesis is presented to explain the mechanics of the formation of fracture plane deterioration; measures to prevent or reduce deterioration are suggested; and the indications from each of the several parts of the report are summarized.

The following are the main indications found from the parts of the investigation included in this report:

1. Fracture plane and surface mortar deterioration are the most serious types of bridge deck deterioration in Missouri.

2. Fracture plane is associated with (a) a built-in plane of weakness occurring in the plane of the top mat of reinforcing steel, (b) use of de-icing salts, (c) surface cracking, and (d) depth of cover over the top steel.

3. Surface mortar deterioration is associated with an excessive amount of mixing water, an insufficient amount of entrained air, the use of de-icing salt, and the severity of the frost action.

Progress Report of

A STUDY OF DETERIORATION IN CONCRETE BRIDGE DECKS

Introduction

This is a progress report* of an investigation into the deterioration of concrete in the decks of bridges in the state of Missouri. The investigation was initiated in late fall of 1959 as the result of extensive deterioration which occurred on a fairly new bridge (9 years old) in Kansas City. During initial studies of concrete in the deck of this bridge, reports of similar deterioration on bridge decks throughout the state were received, and, ultimately, an investigational outline was prepared for further study of the problem of bridge deck deterioration. Because of wide interest in the subject, the Bureau of Public Roads on August 1, 1961, made research funds available for continuation of the investigation.

Most of the phases (parts of the whole investigation) listed in the original outline constitute minor investigations in themselves, and each phase reported herein is treated separately. The indications from each of these phases and from other parts of the report are summarized in a section at the end of the report.

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* It gives and discusses results of studies described in the paper, "Investigative Techniques Used or Contemplated", by E. O. Axon, Don E. Gotham, and R. W. Couch. This paper was published in Highway Research Board Bulletin 323.

It should be stated here that while there has been considerable delay in preparation of this report, much of the data and ideas contained herein has already been distributed to interested persons. This has, in some cases, already effected changes in specifications and construction practices. For example, previously, the specified air content was designated as a range with a specified minimum and a specified maximum. Tests on hardened concrete indicated that, for some reason, the air contents tended to be on the low side of this range, and, with the normal variation in air content usually encountered, too high a percentage of field concrete contained air contents below the specified minimum. Consequently, the specifications were rewritten first to specify a higher air content and later to specify the average desired air content with a plus and minus operating tolerance. The latter step was taken to emphasize that the midpoint rather than the low side of the allowable range was most desirable. Other specification changes were to reduce the maximum allowable slump from five to four inches, to increase the depth of cover over the top reinforcing steel from 1½ to 2 inches on bridges in the interstate or primary systems, and to require ties at every reinforcing bar crossing. Other changes included tighter control through increased inspection and more thorough training of both materials and construction inspectors through schools of instruction.

Some phases or parts of phases of the investigation are not included in this report because they have been reported previously or because they are not presently completed. Those phases which have not been included are as follows:

1. An investigation to determine the merit of insulating the underside of a concrete bridge deck. Three progress reports and a paper on this phase have been written. The paper was prepared by E. O. Axon and R. W. Couch for presentation at the annual meeting of the Highway Research Board in January 1963, and it was published in Highway Research Record Number 14. The third progress report (the final report of temperature data) was completed in January 1965, and has been given only limited circulation.

2. An investigation to determine the change in chlorine content in the top two inches of concrete bridge decks. This phase has not been completed.

3. An investigation to determine the age of a bridge at the time deterioration begins to occur and the rate of development of the deterioration. This investigation is a projection of the study reported in Phase B of this report. This projected phase is still in progress.

4. An investigation of the service performance of bridge decks treated with an epoxy resin seal coat. This phase is still in progress.

5. Continuation of attempts in the laboratory to develop fracture plane deterioration in small reinforced specimens provided a new or revised hypothesis should be developed. A previous attempt to develop fracture plane deterioration in the laboratory was not fully successful; this attempt is described on page 91 of this report.

6. Additional investigation of unusually poor service performance of bridge deck concrete or of the service performance of bridge decks constructed since some changes in construction practices and specifications have taken place. This additional work would not be undertaken unless it gave promise of providing new information.

7. An investigation of the service performance of bridge deck concrete wherein Dow Corning 777 was used as an admixture. The experimental decks in this study were built during 1964 and the first progress report was completed in January 1965.

DESCRIPTION OF DETERIORATION

A description of the defects or types of deterioration encountered on bridge decks in the state of Missouri was presented as part of a Highway Research Board paper* published in 1962. Nine types of deterioration were discussed in this paper, but the presence of only four of them on Missouri's bridge decks is believed to have made the problem of bridge deck deterioration in Missouri critical. These four types of deterioration are (1) fracture plane, (2) pot hole, (3) surface mortar deterioration, and (4) cracking. These four types have been intensively investigated, and the results are presented in this report.

Of the four above types of deterioration, two are of greater concern. They are fracture plane and surface mortar deterioration. Pot holing is a highly undesirable type of deterioration, usually requiring immediate and costly repairs. However, it actually assumes somewhat of a secondary position in its relationship to these first two because it is by definition (see definition below) an advanced, or secondary stage of fracture plane development. The occurrence of cracking is also undesirable, but its importance too is subordinated somewhat to the importance of the first two mentioned types of deterioration.

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* See footnote bottom of page 3.

Cracking by itself, while unsightly, was not generally found to affect the service life or performance of the concrete in a bridge deck. It was found, however, to be associated with other types of harmful deterioration. An association between the presence of cracking and the occurrence of fracture plane deterioration, for example, was found to exist. It is this associative type of relationship with the occurrence of the other types of harmful deterioration which precludes omission of a study of cracking in an investigation of this kind.

The four types of deterioration investigated are defined as follows:

1. Fracture Plane - This term is used to identify a defect, prevalent in many bridge decks. The defect is essentially a separation (fracture) of the upper layer of concrete from the lower layer along a horizontal plane in the upper portion of a bridge deck. The plane roughly parallels the surface of the deck, and undulates from a maximum depth of about 2 inches to a minimum of 1/4 inch or less below the top surface of the deck. This defect has been found to be the predecessor of a splitting off or shelling out of the surface layer of the deck resulting in the formation of first, "pot holes", and ultimately the breakup and removal of the surface over large areas of the deck. The maximum depth is commonly found at the upper re-bars and the minimum

depth between the upper re-bars. In general, the concrete above the fracture plane has the appearance of being sound and of good quality. Sometimes it can be readily removed as unbroken slabs up to several square feet in area.

2. Pot Holes - The depressions left in the surface of the deck when the concrete above the fracture plane has been removed. Their size, depth, and appearance depends upon the amount of concrete removed. Two stages of development have been described as follows:

   A. Incipient - In the incipient stage this defect may appear to be a small irregularly-shaped area of scale. As the development progresses half-moon-shaped areas are common. The area produced by the breaking up of the surface above the fracture plane may assume meandering outlines, but the depression characteristically is much deeper along one edge and feathers off along the opposite edge. The defect in its early stages often resembles a partially developed popout.

   B. Advanced - In the advanced stages, the pot holes enlarge and merge into one another resulting in large areas wherein, through traffic action, the surface layer of the deck is completely broken off and removed above the fracture plane. After this has occurred, the undulating characteristics of the fracture plane are evident; the exposed surface appears as a series of ridges and valleys paralleling the upper reinforcing bars which are exposed in the valleys while the ridges can be seen, located in general, midway between consecutive upper reinforcing bars.

3. <u>Surface Mortar Deterioration</u> - (Similar to what has often been called "progressive scale" by some observers or "salt scale" by others.) Progressive eroding away of the surface due to deterioration of paste. The mortar in affected areas is not sound and is generally weak and friable. Three stages of development are described as follows:
<u>Stage 1</u> - Areas where surface mortar has disappeared from spots, ranging in size from that of a nickel to a silver dollar, interspersed with large areas where surface mortar is still intact. Depth of spots lacking mortar is only 1/8 to 1/4 inch; coarse aggregate is not protruding.
<u>Stage 2</u> - Surface mortar disappearance is no longer spotty, but fairly well covers an entire area, and has progressed in depth so that coarse aggregate is definitely protruding 1/8 to 1/2 inch above surface of remaining mortar.
<u>Stage 3</u> - There has been sufficient progression in depth from Stage 2 that the top most layer of coarse aggregate has shelled out, and the underlying mortar is deteriorating.

4. <u>Cracking</u> - A crack is a fissure or cleavage visible in the surface. The depth of a crack may vary, but it generally

extends into the concrete in a direction approximately perpendicular to the surface. The cracks are generally longitudinal or transverse and parallel to and occurring over the stress steel. Diagonal cracks have been observed over bents.

The above four types of deterioration are illustrated in the following photographs.

Figure 1

ILLUSTRATIONS OF DETERIORATION

A. Pot holes and outline of fracture plane areas. Straight lines marked on surface indicate cracking over top transverse steel.

B. Incipient pot holes and cracking over top transverse steel.

C. Stage 2 surface mortar deterioration grading into stage 1 at edges and in foreground.

D. Core hole in fracture plane area showing fracture in wall of hole.

Phase A

RESULTS OF SURVEYS OF THE DETERIORATION ON DECKS OF IN-SERVICE BRIDGES

As the planning of this investigation advanced through preliminary stages, it was deemed desirable to determine the severity and distribution of bridge deck deterioration throughout the state. This was accomplished by surveying the deterioration on several bridges in each district. These surveys were made by the Materials personnel in the district. They were assisted by two additional survey teams from the Research Section. The two teams from the Research Section were assigned to work statewide to help balance the progress of the surveying between the districts.

Survey forms with instructions for recording observed deterioration, a dictionary of terminology, and photographs of each type of deterioration were provided each survey team. Each district survey team was requested to survey bridges on primary routes in their district by starting at district boundary lines and proceeding across the district on that route and surveying each bridge as it was encountered.

In addition to surveying the bridge deck deterioration, it was requested that obvious structural features of each of the bridges surveyed be also indicated on the survey forms. These structural features included the type of deck support (I-beam, deck girder, truss, etc.), span length, type of end supports for each span (simple or continuous), joint location, etc.

As the surveys were completed, the completed survey forms were sent to the Research Section where the data on each form was tabulated. By the time the surveying was terminated, data on a total of 953 bridge decks had been received. Of these bridge decks, the surfaces of 288 could not be seen because they had been treated with bituminous resurfacing*, and 45 others were eliminated because of reconstruction, use of lightweight aggregate or other reasons. This left a final sample size of 620 bridge decks (the steel bridges were predominately of non-composite design). The number of bridges from each district which became a part of this final sample is shown in Table A-1. Many multiple span bridges consisted of spans of more than one type. That is, for example, a bridge may have been made up of both truss spans and I-beam spans, etc. The spans on most of the bridges surveyed were usually of the following five types: concrete slabs, concrete deck girders (T-beam design), I-beams, plate girders, and trusses. Other types of spans encountered were placed into a miscellaneous group. The miscellaneous spans include through and hinged plate girders, cantilevered I-beams, concrete arches, voided slabs, and rigid frame structures. The number of each of the five main types of spans and the number of miscellaneous spans surveyed in each district are also shown in Table A-1. The state

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*These bituminous surfaced decks could not be surveyed, but the structural features of the bridge were recorded. No analysis was made of this structural data. The fact that the bridges were resurfaced indicates that some form of deterioration was probably present on many of them. The number of resurfaced bridge decks, therefore, provides a rough indication of the extent of bridge deck deterioration in Missouri.

was divided into three areas — the Northern area, the Central area, and the Southern area. These areas (made up of Highway Department districts) are indicated in Figure 2, and the number of bridges and spans in each area is shown in Table A-1.

Sample Distribution

There was considerable variation from district to district in the number of bridges surveyed with respect to age, span length, air entrainment, and traffic intensity. The sample was not stratified with respect to these variables. Such an attempt to stratify would have become very involved when the number of sub-classifications for each of these categories is considered. For example, consider the following classifications and sub-classifications:

Area of State (3 sub-classes)

Northern (Districts 1, 2, & 3)
Central (Districts 4, 5, & 6)
Southern (Districts 7, 8, 9, & 10)

Air Entrainment (2 sub-classes)

Air Entrained
Non Air Entrained

Traffic Intensity (3 sub-classes)

Light (less than 350 VPD)
Moderate (351 to 650 VPD)
Heavy (more than 650 VPD)

Length of Span (6 sub-classes)

0 - 25 ft.
26 - 50 ft.
51 - 75 ft.
76 - 100 ft.
100 - 150 ft.
151 ft. and greater

TABLE A-1

NUMBER OF BRIDGES AND TYPES OF SPANS SURVEYED IN EACH DISTRICT

	Number Surveyed														
		Type of Span													
		Concrete Slab		Concrete Deck Girder		I-Beam		Plate Girder		Truss		Misc.*		Total Simp. Spans	Total Cont. Spans
District	Bridges	Simp.	Cont.	Simp.	Cont.	Simp.	Cont.	Simp.	Cont.	Simp.	Cont.	Simp.	Cont.		
1	43	17	17	48	-	87	6	8	3	27	-	-	-	187	26
2	119	62	89	108	-	347	56	1	3	32	-	-	-	550	148
3	61	6	-	76	-	117	4	2	4	28	-	2	-	231	8
4	60	21	8	47	7	76	35	5	3	24	-	-	-	173	53
5	58	24	4	100	4	32	41	3	26	41	5	2	20	202	100
6	80	32	-	97	-	187	32	8	21	32	3	-	22	356	78
7	59	32	9	110	-	59	45	-	12	19	-	-	-	220	66
8	34	2	5	48	-	46	19	3	3	23	-	-	-	122	27
9	46	8	-	77	3	39	9	-	-	60	-	8	-	192	12
10	60	17	18	27	-	144	47	5	3	18	-	-	-	211	68
Area															
North	223	85	106	232	-	551	66	11	10	87	-	2	-	968	182
Central	198	77	12	244	11	295	108	16	50	97	8	2	42	731	231
South	199	59	32	262	3	288	120	8	18	120	-	8	-	745	173
Statewide	620	221	150	738	14	1134	294	35	78	304	8	12	42	2444	586

* Miscellaneous types include: Cantilever I-beam, through and hinged plate girder, concrete arch, voided slab, and rigid frame.

FIGURE 2

STATE OUTLINE MAP
SHOWING GEOGRAPHIC AREAS

Age of Bridge (8 sub-classes)

0 - 5 years
6 - 10 years
11 - 15 years
16 - 20 years
21 - 25 years
26 - 30 years
31 - 35 years
36 - 40 years

Stratification of a sample population with this many subdivisions results in a total of 864 categories. These categories still do not include a breakdown for type of deck (5 main sub-classes), aggregate type (limestone or gravel), design loading and other design variables. It can be seen that the task of stratifying does become very involved. Therefore, as stated, the sample was not selected on a stratified basis, but rather it consisted of bridges surveyed as they were encountered on primary routes in each district.

The distribution of the sample (620 bridges) between districts and areas of the state with respect to age, span length, air entrainment, and traffic intensity was examined. These distributions are shown in Tables A-2, A-3, A-4, and A-5, respectively. The number of spans surveyed varies considerably from district to district for each sub-classification in each of the tables. There appears to be no reason, however, to suspect that the bridges surveyed are not a representative sample of the bridges available for survey.

TABLE A-2

AGE DISTRIBUTION OF ALL SPANS SURVEYED

		Age Group (Years)																Total
	No. of	0-5		6-10		11-15		16-20		21-25		26-30		31-35		36-40		No.
District	Bridges	No.	%	No.	%	No.	%	No.	%	No.	%	No.	%	No.	%	No.	%	Spans
1	43	18	8.5	16	7.5	5	2.3	0	-	20	9.4	106	50	41	19	7	3.3	213
2	119	148	21	43	6.2	34	4.9	8	1.2	30	4.3	272	39	156	22	7	1	698
3	61	0	-	6	2.5	16	6.7	0	-	49	20	104	44	52	22	12	5	239
4	60	13	5.8	75	33	10	4.5	27	12	0	-	45	20	36	16	20	8.9	226
5	58	67	22	31	10	20	6.6	30	9.9	19	6.2	43	14	56	18	36	12	302
6	80	34	7.8	56	13	9	2.1	54	12	81	19	164	38	7	1.6	29	6.7	434
7	59	47	16	34	12	39	14	3	1	22	7.7	82	29	59	21	0	-	286
8	34	34	23	34	5	5	3.4	4	2.7	20	13	42	28	10	6.7	0	-	149
9	46	21	10	18	8.8	7	3.4	14	6.9	12	5.9	90	44	16	7.9	26	13	204
10	60	126	45	16	5.7	6	2.2	21	7.5	26	9.3	46	16	27	9.7	11	4	279
Area																		
North	223	166	14	65	6	55	5	8	0.7	99	8	482	42	249	22	26	2	1150
Central	198	114	12	162	17	39	4	111	12	100	10	252	26	99	10	85	9	962
South	199	228	25	102	11	57	6	42	5	80	9	260	28	112	12	37	4	918
Statewide	620	508	17	329	11	151	5	161	5	279	9	994	33	460	15	148	5	3030

TABLE A-3

SPAN LENGTH DISTRIBUTION OF ALL SPANS SURVEYED

District	No. of Bridges	Length Groups (feet) 0-25 No.	%	26-50 No.	%	51-75 No.	%	76-100 No.	%	101-150 No.	%	>150 No.	%	Total No. Spans
1	43	42	20	117	55	23	11	18	8	12	6	1	.5	213
2	119	262	38	344	49	57	8	22	3	10	1	3	.4	698
3	61	20	8	181	76	12	5	15	6	9	4	2	.8	239
4	60	36	16	115	51	50	22	11	5	10	4	4	2	226
5	58	39	13	133	43	60	20	34	11	18	6	18	6	302
6	80	64	15	237	55	72	17	20	4	27	6	14	3	434
7	59	48	17	162	57	46	16	19	7	10	3	1	.3	286
8	34	16	11	87	58	21	14	11	7	4	3	10	7	149
9	46	14	7	104	51	20	10	48	23	6	3	12	6	204
10	60	51	18	165	60	37	13	11	4	11	4	4	1	279
Area														
North	223	324	28	642	56	92	8	55	5	31	2	6	1	1150
Central	198	139	14	485	50	182	19	65	7	55	6	36	4	962
South	199	129	14	518	56	124	14	89	10	31	3	27	3	918
Statewide	620	592	20	1645	54	398	13	209	7	117	4	69	2	3030

TABLE A-4

DISTRIBUTION WITH REGARD TO USE OF
AIR ENTRAINMENT OF ALL SPANS SURVEYED

District	No. of Bridges	Air-Entrained No.	Air-Entrained %	Non Air-Entrained No.	Non Air-Entrained %	Total Number Spans
1	43	31	15	182	85	213
2	119	143	21	555	79	698
3	61	3	1	236	99	239
4	60	58	26	168	74	226
5	58	84	28	218	71	302
6	80	67	15	367	85	434
7	59	97	34	189	66	286
8	34	58	39	91	61	149
9	46	27	13	177	87	204
10	60	78	28	201	72	279
Area						
North	223	177	15	973	85	1150
Central	198	209	22	753	78	962
South	199	260	28	658	72	918
Statewide	620	646	21	2384	79	3030

TABLE A-5

DISTRIBUTION WITH REGARD TO
TRAFFIC INTENSITY ON ALL SPANS SURVEYED

District	No. of Bridges	Traffic Intensity: Light No.	Light %	Moderate No.	Moderate %	Heavy No.	Heavy %	Total No. Spans
1	43	173	81	29	14	11	5	213
2	119	462	66	236	34	0	0	698
3	61	222	93	17	7	0	0	239
4	60	128	57	13	5	85	38	226
5	58	170	56	16	5	116	38	302
6	80	225	52	5	1	204	47	434
7	59	81	28	97	34	108	38	286
8	34	94	63	11	7	44	30	149
9	46	176	86	28	14	0	0	204
10	60	157	56	104	37	18	7	279
Area								
North	223	857	74	282	25	11	1	1150
Central	198	523	54	34	4	405	42	962
South	199	508	55	240	26	170	19	918
Statewide	620	1888	63	556	18	586	19	3030

Surveying Procedure

Before surveying each bridge, it was marked off into subsections. This procedure was in accordance with the instructions prescribed for use of the prepared survey forms. Each subsection consisted of an area 10 feet long by one traffic lane wide. The 10-foot length was arbitrarily chosen as a convenient size for surveying in the field.

Each survey record sheet was designed to accommodate the survey results of 100 lineal feet of bridge deck two lanes wide (a total of 20 subsections). As deterioration was encountered on a bridge deck, it was recorded on the sheet in a place which appropriately described the location of the subsection in which the deterioration was found. Ultimately, a total of 30,808 subsections was surveyed. Of these subsections 29,992 were on the five main types of spans.

To summarize the deterioration and the structural features reported on the survey forms, each bridge was broken down into its component spans, and the pertinent information for each span was itemized. This itemized information included the age, span length, traffic intensity, type of end support (simple or continuous), deck design type, total number of subsections surveyed, and the number of subsections with each type of deterioration. The items on this summary sheet were then coded and an even more condensed summary was prepared. These summaries provided a convenient source from which to pick off various types of data for analysis.

It was interesting to see just how many of the bridges surveyed contained each of the four main types of deterioration. Data in Table A-6 show the number of bridges containing cracking, fracture plane, pot holes, surface mortar deterioration, and none of these defects. Data in Tables A-7 and A-8 show similar information for the number of spans and the number of subsections, respectively, with each of these types of deterioration (and none of the defects). The percent of bridges, spans, and subsections with no defects is 9.5, 18.6, and 39, respectively. It is obvious that the percent of bridge deck area with <u>no</u> defects should increase as the size of section used for analysis is decreased (from the whole bridge, to spans, to subsections) because smaller areas of bridge deck are indicated to be affected by each type of deterioration. Likewise, the area of bridge deck <u>with</u> each defect decreased as the size of survey section is decreased. Thus, using the small survey section, a more accurate estimate is obtained of the area of bridge deck surface which is affected by each type of deterioration. These small survey sections (10 feet long by one traffic lane wide) were the units used in determining the relationships between age, span length, traffic intensity, etc. and the different types of deterioration. The relationships found for each type of deterioration are presented in the following discussion.

TABLE A-6

NUMBER AND PERCENT OF BRIDGE DECKS AFFECTED BY EACH TYPE OF DETERIORATION

District	No.of Bridge Decks	Type of Deterioration: Cracking No.	%	Fracture Plane No.	%	Pot Holes No.	%	S.M.D. No.	%	No Defects No.	%
1	43	31	72	1	2.3	9	21.0	38	88	2	4.6
2	119	78	65	8	6.7	18	15.0	113	95	5	4.2
3	61	34	56	1	1.6	2	3.3	46	75	7	11.0
4	60	49	82	23	38.0	24	40.0	52	87	1	1.7
5	58	43	74	5	8.6	10	17.0	42	72	5	8.6
6	80	66	83	12	15.0	54	67.0	77	96	1	1.3
7	59	42	71	2	3.4	5	8.5	37	63	8	14.0
8	34	26	77	3	9.0	2	6.0	20	59	6	18.0
9	46	40	87	2	4.3	7	15.0	41	89	2	4.3
10	60	29	48	5	8.3	4	6.7	32	53	22	37.0
Area											
North	223	143	64	10	4.5	29	13.0	197	89	14	6.5
Central	198	158	80	40	20.0	88	40.0	171	86	7	3.5
South	199	137	69	12	6.0	18	9.0	130	65	38	19.0
Statewide	620	438	71	62	10.0	135	22.0	498	80	59	9.5

TABLE A-7

NUMBER AND PERCENT OF SPANS
AFFECTED BY EACH TYPE OF DETERIORATION

		Type of Deterioration									
		Cracking		Fracture Plane		Pot Holes		S.M.D.		No Defects	
District	No. of Spans	No.	%	No.	%	No.	%	No.	%	No.	%
1	213	80	37.6	1	0.5	10	4.7	122	57.3	63	29.6
2	698	281	40.3	17	2.4	32	4.6	611	87.5	67	9.6
3	239	86	36.0	4	1.7	3	1.3	154	64.4	49	20.5
4	226	135	59.7	42	18.6	42	18.6	153	67.7	27	11.9
5	302	186	61.6	20	6.6	23	7.6	197	65.2	45	14.9
6	434	294	67.7	29	6.7	179	41.2	403	92.9	18	4.1
7	286	131	45.8	2	0.7	8	2.8	97	33.9	113	39.5
8	149	78	52.3	3	2.0	3	2.0	60	40.3	53	35.6
9	204	156	76.5	5	2.5	9	4.4	150	73.5	18	8.8
10	279	129	46.2	9	3.2	6	2.2	76	27.2	112	40.1
Area											
North	1150	447	38.9	22	1.9	45	3.9	887	77.1	179	15.6
Central	962	615	63.9	91	9.5	244	25.4	753	78.3	90	9.4
South	918	494	53.8	19	2.1	26	2.8	383	41.7	296	32.4
Statewide	3030	1556	51.4	133	4.4	315	10.4	2023	66.8	565	18.6

TABLE A-8

NUMBER AND PERCENT OF 10' SURVEY SUBSECTIONS AFFECTED BY EACH TYPE OF DETERIORATION

District	No. of Bridge Decks	No. of Sub-sections	Type of Deterioration: Cracking No.	Cracking %	Fracture Plane No.	Fracture Plane %	Pot Holes No.	Pot Holes %	S.M.D. No.	S.M.D. %	No Defects No.	No Defects %
1	43	1985	429	22	1	0+	13	0.6	708	36	1042	52
2	119	5311	1279	24	37	0.7	53	1.0	4155	78	900	17
3	61	2342	437	19	4	0.2	2	0.1	997	43	1018	43
4	60	2344	795	34	193	8.2	118	5.0	1001	43	859	39
5	58	3885	1501	39	107	2.8	61	1.6	1325	34	1573	40
6	80	5159	1920	37	79	1.5	543	10.5	3363	65	1120	22
7	59	2844	669	24	3	0.1	17	0.6	437	15	1890	66
8	34	1739	390	22	4	0.2	5	0.3	409	23	1066	61
9	46	2649	1034	39	13	0.5	16	0.6	1542	58	889	34
10	60	2550	662	26	29	1.1	29	1.1	472	18	1573	62
Area												
North	223	9638	2145	22	42	0.4	68	0.7	5860	61	2960	31
Central	198	11,388	4216	37	379	3.3	722	7.0	5689	50	3552	31
South	199	9782	2755	28	49	0.5	67	0.7	2860	29	5418	55
Statewide	620	30,808	9116	30	470	1.5	857	2.8	14,409	47	11,930	39

Analysis of Cracking

The following is a list of tables giving the relationship between cracking and other variables. Opposite each table number is shown the variables presented in the table.

Table A-9: Age, type of deck support, and type of end support for the span.

Table A-10: Span length, type of deck support, and type of end support for the span.

Table A-11: Air-entrainment by district.

Table A-12: Traffic intensity by district.

Table A-13: District and area, type of deck support, and type of end support for the span.

A question arose after all the field surveys were completed as to what was the actual slab span-length on some of the plate girders and trusses. As recorded on our field survey sheets, the slab span-length is shown as the distance between the end supports for the span (the distance between bents). However, the floor system on some of the plate girders and trusses consists of various combinations of floor beams and stringers; in some cases the stringer rests on the floor beam, and in other cases the top flange of both the stringer and the floor beam are at the same elevation. Both of these types of floor systems support the concrete slab. It is obvious that the slab span-length must be less than the distance between the ends of the truss or plate girder. It appears that the proper slab span-length for these types of floor systems is probably the distance between floor beams. Attempts to further break down the data did not clarify this condition because of inadequacies in the sampling distribution. Therefore, to avoid the possibility of mis-classifying the plate girders and trusses, these two types of bridges were not broken down into the simple or continuous types.

<u>Discussion of Data</u>

Effect of Age - It would appear logical that the cracking in a bridge deck should increase as the bridge grows older. The percent of subsections exhibiting cracking, on different types of spans classed in different age groups, is shown in Table A-9. The age groupings in this table are at 5-year intervals. The effect of age on the occurrence of cracking can best be seen by the percents in the totals columns at the right side of the figure. These percents indicate no significant variation in cracking for the different age groups. This itself, however, is an indication that most of the cracking occurs during the early life of the bridge. A smaller age group interval, particularly in the younger bridges, may have indicated some affect of age, but this was not investigated.

Effect of Span Length - The percent of subsections exhibiting cracking on different types of spans classed in different span length groups is shown in Table A-10. The span length groupings in this table are at 25-foot intervals. The effect of span length is best shown, again, by the data in the totals columns at the right side of the figure. The percents in these columns indicate that cracking increases with increasing span length. For the plate girder and truss spans, the amount of cracking is relatively constant for span lengths greater than 76 feet.

Effect of Air-Entrainment - The percent of subsections exhibiting cracking on the air-entrained and the non-air-entrained bridges surveyed in each district is shown in Table A-11. The data in this table includes the data for subsections on the miscellaneous spans. The percents shown at the bottom of the table (statewide basis) indicate no effect of air entrainment on the occurrence of cracking.

Effect of Traffic Intensity - The percent of subsections exhibiting cracking in each district on bridges with different traffic intensities is shown in Table A-12. The number of vehicles per day for each category of traffic intensity was given on page 15. The data in this table also includes the data for subsections on the miscellaneous spans. The percents shown at the bottom of the table (statewide basis) indicate no effect of traffic intensity on the occurrence of cracking.

Effect of Design Type (Simple or Continuous End Supports) - In Tables A-9 and A-10, the slab, deck girder, and I-beam design types are each subdivided into either simple or continuous design types. This design type refers to the type of slab support at the bents. For each of the design types shown, a greater percent of subsections exhibiting cracking is indicated to occur on the slabs running continuously over the bents.

Effect of Geographic Area - The percent of subsections exhibiting cracking on the five main types of spans in each district is shown in Table A-13. The three totals columns at the right side of the figure show how cracking is distributed between the districts. Below the data for districts is shown the percent of cracking in each area of the state. The areas of the state were defined on page 15. The percents shown in the totals columns for the area rows indicate a slight tendency for more cracking to occur in the central area.

Analysis of Fracture Plane

Presentation of Data

The following is a list of tables giving the relationship between fracture plane deterioration and other variables. Opposite each table number is shown the variable discussed in the table.

Table A-14: Cracking
Table A-15: Contingency Table (Cracking vs Fracture Plane)
Table A-16: Amount of Sodium Chloride Purchased

There are fewer tables presented here on fracture plane deterioration than were presented on cracking because of the indicated association between the two. This association is discussed below.

Table A-9

SUBSECTIONS EXHIBITING CRACKING BY AGE GROUPS FOR EACH OF THE DECK TYPES SHOWN
(Data Shown is on a Statewide Basis)

Age Group	Concrete Slabs Simple No.*	%	Concrete Slabs Cont. No.*	%	Concrete Deck Girders Simple No.*	%	Concrete Deck Girders Cont. No.*	%	I-Beams Simple No.*	%	I-Beams Cont. No.*	%	Plate Girders No.*	%	Trusses No.*	%	Totals: Slabs, Deck Girders, I-Beams Simple No.*	%	Totals: Slabs, Deck Girders, I-Beams Cont. No.*	%	Totals: Plate Girders & Trusses No.*	%
0-5	-	-	96/631	15	13/432	3	25/64	39	251/962	26	402/1821	22	493/804	61	0/24	0	264/1394	19	523/2516	21	493/828	60
6-10	5/8	63	18/50	36	77/650	12	30/46	66	165/1002	17	515/842	61	295/532	55	126/706	18	247/1660	15	563/938	60	421/1238	34
11-15	0/4	0	11/98	11	20/396	5	21/30	70	102/334	31	221/450	49	-	-	30/84	36	122/734	17	253/578	44	30/84	36
16-20	1/28	4	0/16	0	46/164	28	-	-	45/622	6	193/432	45	273/533	51	114/164	70	92/814	11	193/448	43	387/697	56
21-25	4/58	7	-	-	88/714	12	-	-	218/1026	21	46/142	32	48/343	14	231/464	50	310/1798	17	46/142	32	279/807	35
26-30	18/154	12	-	-	336/2306	15	12/40	30	785/4087	19	-	-	0/86	0	1390/2841	49	1139/6547	17	12/40	30	1390/2927	47
31-35	24/562	4	-	-	232/1412	16	-	-	93/474	20	-	-	-	-	626/1232	51	349/2448	14	-	-	626/1232	51
36-40	18/160	11	-	-	48/166	29	-	-	11/78	14	-	-	-	-	961/1718	56	77/404	19	-	-	961/1718	56
Totals	70/974	7	125/795	16	860/6240	14	88/180	49	1670/8585	19	1377/3687	37	1109/2298	48	3478/7233	48	2600/15,799	16	1590/4662	34	4587/9531	48

*Numerator indicates cracked subsections. Denominator indicates total subsections.

Table A-10

SUBSECTIONS EXHIBITING CRACKING BY SPAN LENGTH GROUPS FOR EACH OF THE DECK TYPES SHOWN
(Data Shown is on a Statewide Basis)

Span Length Group	Concrete Slabs				Concrete Deck Girders				I-Beams				Plate Girders		Trusses		Totals: Slabs, Deck Girders, I-Beams				Totals: Plate Girders & Trusses	
	Simple		Cont.		Simple		Cont.		Simple		Cont.						Simple		Cont.			
	No.*	%	No.*	%	No.*	%	No.*	%	No.*	%	No.*	%	No.*	%	No.*	%	No.*	%	No.*	%	No.*	%
0-25	68/942	7	122/662	18	41/208	20	-	-	96/961	10	-	-	-	-	-	-	205/2111	10	122/662	18	-	-
26-50	2/32	6	3/133	2	750/5498	14	51/118	43	1458/6993	21	112/600	19	0/52	0	8/22	36	2210/12,523	18	166/851	20	8/74	11
51-75	-	-	-	-	69/534	13	37/62	60	116/631	18	963/2547	38	102/347	29	194/593	33	185/1165	16	1000/2609	38	296/940	31
76-100	-	-	-	-	-	-	-	-	-	-	302/540	56	310/682	45	1211/2364	51	-	-	302/540	56	1521/3046	50
101-150	-	-	-	-	-	-	-	-	-	-	-	-	599/1119	54	809/1726	47	-	-	-	-	1408/2845	49
$>$ 150	-	-	-	-	-	-	-	-	-	-	-	-	98/98	100	1207/2528	48	-	-	-	-	1305/2626	50
Totals	70/974	7	125/795	16	860/6240	14	88/180	49	1670/8585	19	1377/3687	37	1109/2298	48	3429/7233	47	2600/15,799	16	1590/4662	34	4538/9531	48

*Numerator indicates cracked subsections. Denominator indicates total subsections.

Table A-11

SUBSECTIONS EXHIBITING CRACKING BY DISTRICTS ON AIR-ENTRAINED AND NON-AIR-ENTRAINED SPANS

District	Air-Entrained Concrete		Non-Air-Entrained Concrete	
	No.*	%	No.*	%
1	42/306	14	387/1679	23
2	377/1096	34	902/4215	21
3	14/34	41	423/2308	18
4	252/631	40	543/1713	32
5	504/1042	48	997/2843	35
6	296/782	38	1624/4377	37
7	327/1186	28	342/1658	20
8	140/774	18	250/965	26
9	40/522	8	994/2127	47
10	1/844	0.1	661/1706	39
Area				
North	433/1436	30	1712/8202	21
Central	1052/2455	43	3164/8933	35
South	508/3326	15	2247/6456	35
Statewide	1993/7217	28	7123/23,591	30

*Numerator indicates cracked subsections.
Denominator indicates total subsections.

Table A-12

SUBSECTIONS EXHIBITING CRACKING BY DISTRICTS
FOR DIFFERENT TRAFFIC INTENSITIES

	Traffic Intensity					
	Light		Moderate		Heavy	
District	No.*	%	No.*	%	No.*	%
1	358/1637	22	31/246	13	40/102	39
2	571/3482	16	708/1829	39	-	-
3	392/2166	18	45/176	26	-	-
4	394/1313	30	24/98	24	377/933	40
5	807/2292	35	317/324	98	377/1269	30
6	965/2516	38	0/32	0	955/2611	37
7	181/772	23	192/826	23	296/1246	24
8	308/1177	26	23/166	14	59/396	15
9	965/2421	40	69/228	30	-	-
10	375/1326	28	286/1076	27	1/148	0.7
Area						
North	1321/7285	18	784/2251	35	40/102	39
Central	2166/6121	35	341/454	75	1709/4813	35
South	1829/5696	32	570/2296	25	356/1790	19
Statewide	5316/19,102	28	1695/5001	34	2105/6705	31

*Numerator indicates cracked subsections.
Denominator indicates total subsections.

Table A-13 SUBSECTIONS EXHIBITING CRACKING BY DISTRICT FOR EACH DECK TYPE SHOWN

	Slabs				Deck Girders				I-Beams				Plate Girders		Trusses		Totals: Slabs, Dk.Gird. & I-Beams				Totals: Pl.Gird. &Trusses	
	Simple		Cont.		Simple		Cont.		Simple		Cont.						Simple		Cont.			
Dist.	No.*	%	No.*	%	No.*	%	No.*	%	No.*	%	No.*	%	No.*	%	No.*	%	No.*	%	No.*	%	No.*	%
1	0/78	0	8/85	9	8/398	2	-	-	62/594	10	26/78	33	40/248	16	285/504	57	70/1070	7	34/163	21	325/752	43
2	18/266	7	87/446	20	54/855	6	-	-	383/2329	16	369/705	52	79/108	73	289/602	48	455/3450	13	456/1151	40	368/710	52
3	0/26	0	-	-	96/607	16	-	-	109/945	12	21/54	39	0/60	0	201/608	33	205/1578	13	21/54	39	201/668	30
4	8/90	9	7/44	16	67/393	17	55/110	50	80/597	13	270/458	59	60/128	47	248/524	47	155/1080	14	332/612	54	308/652	47
5	8/106	8	16/20	80	139/894	16	12/40	30	55/232	24	123/508	24	454/557	82	624/1304	48	202/1232	16	151/568	27	1078/1861	58
6	17/138	12	-	-	168/866	19	-	-	388/1684	23	198/380	52	369/673	55	508/1026	50	573/2688	21	198/380	52	877/1699	52
7	15/144	10	1/46	2	66/952	7	-	-	126/502	25	201/616	33	103/240	43	157/344	46	207/1598	13	202/662	31	260/584	45
8	0/10	0	6/26	23	30/394	8	-	-	40/386	10	68/206	33	3/120	3	243/597	41	70/790	9	74/232	32	246/717	34
9	2/34	6	-	-	201/665	30	21/30	70	99/342	29	31/128	24	-	-	644/1292	50	302/1041	29	52/158	33	644/1292	50
10	1/72	1	0/128	0	32/226	14	-	-	328/974	34	70/554	13	1/164	0.6	230/432	53	361/1272	28	70/682	10	231/596	39
Area																						
North	18/370	5	95/531	18	158/1860	9	-	-	554/3868	14	416/837	50	119/416	29	775/1714	45	730/6098	12	511/1368	37	894/2130	42
Central	34/344	10	23/64	36	373/2143	17	67/150	45	523/2513	21	591/1346	44	883/1358	65	1380/2854	48	930/5000	19	681/1560	44	2263/4212	54
South	18/260	7	7/200	4	329/2237	15	21/30	70	593/2204	27	370/1504	25	107/524	20	1274/2665	48	940/4701	20	398/1734	23	1381/3189	43
State-wide	70/974	7	125/795	16	860/6240	14	88/180	49	1670/8585	19	1377/3687	37	1109/2298	48	3429/7233	47	2600/15,799	16	1590/4662	34	4538/9531	48

*Numerator indicates cracked subsections. Denominator indicates total subsections.

Discussion of Data

Relationship of Fracture Plane Deterioration with Cracking - While making the surveys of deterioration on bridge decks throughout the state, it was observed that much of the fracture plane deterioration was located near cracking. The data presented in Table A-14 was prepared for use in determining if there actually was an association between cracking and the occurrence of fracture plane deterioration. The table shows the number of subsections containing cracking deterioration only, fracture plane deterioration only, both cracking and fracture plane deterioration, and neither cracking nor fracture plane deterioration. The number of subsections in each of these classifications for each district is shown in the table. The data is combined to show the area distribution and the statewide distribution. The data shown for the statewide distribution is used in the contingency table presented in Table A-15.

The contingency table shown in Table A-15 is a statistical means of determining if an association exists between the two variables -- fracture plane and cracking. A more detailed explanation of the test is given in the two paragraphs below the table. The results of the test indicate that there is an association between the two variables. Therefore, the factors which promote the occurrence of cracking would be also expected to promote the occurrence of fracture plane deterioration.

TABLE A-14

NUMBER OF SUBSECTIONS WITH CRACKS BUT WITHOUT AREAS OF FRACTURE PLANE, WITH AREAS OF FRACTURE PLANE BUT WITHOUT CRACKS, WITH BOTH CRACKING AND AREAS OF FRACTURE PLANE, AND WITH NEITHER CRACKING NOR AREAS OF FRACTURE PLANE

District	Total Sub-Sections	Number of Subsections With: Cracks Only	Fracture Plane Only	Both Fracture Plane and Cracks	Neither Fracture Plane nor Cracks
1	1985	429	1	0	1555
2	5311	1264	22	15	4010
3	2342	433	0	4	1905
4	2344	630	27	165	1522
5	3885	1400	6	101	2378
6	5159	1857	27	63	3212
7	2844	667	1	2	2174
8	1739	389	3	1	1346
9	2649	1021	0	13	1615
10	2550	638	5	24	1883
Area					
North	9638	2126	23	19	7470
Central	11,388	3887	60	329	7112
South	9782	2715	9	40	7018
Statewide	30,808	8728	92	388	21,600

TABLE A-15

CONTINGENCY TABLE TO DETERMINE THE ASSOCIATION BETWEEN CRACKING AND THE OCCURRENCE OF FRACTURE PLANE DETERIORATION

	Number of Subsections		Marginal
	With Cracks	Without Cracks	Totals
With Fracture Plane	388 (142)	92 (338)	480
Without Fracture Plane	8728 (8974)	21,600 (21,354)	30,328
Totals	9116	21,692	30,808

In each cell of the table the unparenthesized value is the number of test units having the attributes described by the row and column headings in which the cell occurs; the parenthesized value is the "expected" value, calculated from the marginal totals under the hypothesis that the attributes "cracking" and "fracture plane" are not associated, i.e., are mutually independent.

A statistical "Goodness of Fit" test, comparing the "expected" values with those actually observed, demonstrates that there is less than one chance in 100 that the observed distribution of test results could have emanated from a population in which the measured attributes were actually independent. Therefore, it must be concluded that the attributes are dependent. Inspection of the tabular values shows that the relation between the attributes is direct, i.e., the number of subsections having fracture planes increases as the number of subsections having cracks increases.

Amount of Sodium Chloride Purchased - In Table A-16 is shown the amount of sodium chloride purchased for ice and snow control for the winters from 1956-57 through 1961-62. The data in the table show how salt purchases have steadily increased, and they show that the greatest amount of salt is used in the central area of the state. The use of de-icing salt is thought to be a factor affecting the development of fracture plane deterioration. (A discussion of factors affecting fracture plane deterioration is presented in the section beginning on page 86.) The amount of de-icing salt used may partially account for the high percent of fracture plane deterioration in some areas (Table A-8).

Analysis and Discussion of Pot Hole Deterioration

Pot holes are defined as an advanced stage of fracture plane development, and elimination of the pot hole problem should be automatic with the solution of the fracture plane problem.

The percent of subsections affected by pot hole deterioration, indicated in Tables A-6, A-7 and A-8, is in many instances greater than the corresponding percent of subsections affected by fracture plane deterioration. This is inconsistent with the definition for pot holes. The discrepancy is attributed to the following:

1. Fracture plane areas may have been completely broken out, leaving a pot hole, but no note was made on the survey form of the fracture plane which existed first. This type

Table A-16

SODIUM CHLORIDE PURCHASED
BY THE MISSOURI HIGHWAY DEPARTMENT
FOR THE WINTER SEASONS FROM 1956 TO 1962

	Tons of Salt Purchased for Winter						Total
District	1956-57	1957-58	1958-59	1959-60	1960-61	1961-62	Tonnage
1	150	100	100	56	3523	221	4150
2	160	80	200	160	200	240	1040
3	70	140	711	1064	888	1402	4275
4	1443	1690	2510	2363	4893	3904	16,803
5	380	799	1565	1821	2708	2400	9673
6	555	1016	4085	7904	9081	16,681	39,322
7	40	40	80	80	120	162	522
8	70	17	90	86	172	113	548
9	0	0	0	0	360	105	465
10	37	15	10	17	111	127	317
Area							
North	380	320	1011	1280	4611	1863	9465
Central	2378	3505	8160	12,088	16,682	22,985	65,798
South	147	72	180	183	763	507	1852
Statewide	2905	3897	9351	13,551	22,056	25,355	77,115

This table includes information for the two years after the year of completion of our reconnaissance surveys (1960). The data for these two years should not be considered when making comparisons with the surveys. This data is included here to show how salt purchases have increased from year to year. It must be remembered that these are total quantities purchased for all uses (pavements as well as bridges, etc.).

of condition erroneously was not discussed in the instructions for making the surveys.

2. Other types of deterioration such as advanced surface mortar deterioration or large pits or popouts might have been misclassified as pot holes.

Analysis of Surface Mortar Deterioration

Presentation of Data

The following is a list of tables giving the relationship between surface mortar deterioration and other variables. Opposite each table number is shown the variable discussed in the table.

Table A-17: Air-Entrainment

Table A-18: Age

Table A-19: Traffic Intensity

Discussion of Data

Effect of Area of State - The occurrence of surface mortar deterioration (SMD) is indicated to be greatest in the Northern area of the state, next greatest in the Central area, and least in the Southern area (Table A-8). This would indicate that the occurrence of SMD is associated with freezing temperature.

Effect of Air-Entrainment - The data in Table A-17 show that a considerably smaller percent of SMD occurred on bridges made with air entrained concrete. It is obvious, however, that even with air entrainment SMD still remains a problem.

Effect of Age - The data in Table A-18 show the effect of age on the occurrence of SMD. Considering the data on a statewide basis, they show that this deterioration occurs at a rapid rate on new bridges, and then begins to level off after 15 years of age. A similar trend is found on the bridges surveyed in the Northern area of the state. In the central area, the deterioration rate increases to about 25 years of age, and then declines for the older bridges. In the Southern area, the trend is considerably different from the trends in the Northern and central areas; the rate of deterioration is relatively slow in the younger bridges, and after 25 years of age it increases, almost approaching that of the younger bridges in the other two areas. The reasons for these varying trends from area to area were not apparent from the data, but the age of the concrete at time of initial use of de-icing salts is undoubtedly a factor. The data in Table A-16 indicate that extensive use of de-icing salts in Missouri is rather recent.

Effect of Traffic - The data in Table A-19 show the percent of subsections with SMD on bridges carrying light, moderate, and heavy traffic. These percents are shown for the three areas of the state and for the state as a whole. None of these data indicate that SMD is associated with traffic intensity.

Table A-17

SUBSECTIONS AFFECTED BY SURFACE MORTAR DETERIORATION ON AIR-ENTRAINED AND NON-AIR-ENTRAINED BRIDGES

	Type of Concrete					
	Air-Entrained			Non-Air-Entrained		
District	Total Sub-sections	Sub-sections with SMD*	Percent of Subsections with SMD*	Total Sub-sections	Sub-sections with SMD*	Percent of Subsections with SMD*
1	306	11	4	1679	697	42
2	1096	320	29	4215	3835	91
3	34	14	41	2308	983	43
4	631	65	10	1713	936	55
5	1042	150	14	2843	1175	41
6	782	322	41	4377	3041	69
7	1186	163	14	1658	274	17
8	774	181	23	965	228	24
9	522	102	20	2127	1440	68
10	844	0	0	1706	472	28
Area						
North	1436	345	24	8202	5515	67
Central	2455	537	22	8933	5152	58
South	3326	446	13	6456	2414	37
Statewide	7217	1328	18	23,591	13,081	55

*Surface Mortar Deterioration

TABLE A-18

NUMBER AND PERCENT OF SUBSECTIONS IN EACH AGE GROUP AFFECTED BY SURFACE MORTAR DETERIORATION

	Age Group (Years)															
	0 - 5		6 - 10		11 - 15		16 - 20		21 - 25		26 - 30		31 - 35		36 - 40	
District	No.	%	No.	%	No.	%	No.	%	No.	%	No.	%	No.	%	No.	%
1	11	9	33	15	23	88	-	-	67	23	454	49	70	19	50	100
2	303	27	295	74	319	89	55	76	214	72	1921	97	940	97	108	100
3	-	-	42	68	55	38	-	-	253	56	500	45	66	15	81	59
4	15	7	313	43	27	34	227	78	-	-	129	29	144	45	146	58
5	70	8	162	53	151	75	245	67	148	78	137	20	177	30	235	33
6	98	38	326	44	139	90	651	63	546	69	1390	79	31	46	182	66
7	81	12	79	24	18	5	2	6	47	19	87	11	123	29	-	-
8	49	15	139	26	0	0	0	0	7	4	153	30	61	64	-	-
9	84	38	40	9	47	76	147	98	16	17	620	58	190	100	398	92
10	11	1	45	32	0	0	18	8	43	20	156	34	144	61	55	38
Area																
North	314	25	370	55	397	75	55	76	534	52	2875	72	1076	60	239	81
Central	183	13	801	45	317	73	1123	67	694	71	1656	58	352	36	563	45
South	225	10	303	21	65	13	167	38	113	15	1016	36	518	55	453	78
Statewide	722	14	1474	38	779	54	1345	61	1341	49	5547	57	1946	53	1255	59

TABLE A-19

NUMBER AND PERCENT OF SUBSECTIONS AFFECTED BY SURFACE MORTAR DETERIORATION IN LIGHT, MODERATE, AND HEAVY TRAFFIC

	Traffic Intensity					
	Light		Moderate		Heavy	
Area	No.	%	No.	%	No.	%
North	4444	61	1314	58	102	100
Central	3101	51	113	25	2475	51
South	2227	39	365	16	268	15
Statewide	9772	51	1792	36	2845	42

Phase B

A STUDY OF THE EFFECT OF CONSTRUCTION PROCEDURES AND PRACTICES UPON THE DETERIORATION OF CONCRETE IN BRIDGE DECKS

The objective of this study was to observe the construction procedures and practices followed during bridge deck construction and to evaluate those procedures and practices in terms of the future service performance of the bridge deck. A total of eleven bridge deck pours on 6 bridges was observed during the late summer and early fall of 1960.

The first part of this two-fold objective was accomplished at the time of construction of each deck pour. At this time, extensive study of construction details and comprehensive sampling and testing of the concrete was made. These observations and tests provide information regarding the following variables and the location of the concrete in the deck affected by them:

1. Air and concrete temperatures;
2. The slump or consistency of the concrete;
3. Air content of the concrete;
4. Elapsed time between start of mixing and placement of concrete on deck;
5. Elapsed time between start of mixing and the various finishing operations, including start of wet curing;
6. Elapsed time between placement of contiguous portions of concrete;
7. Elapsed time between placement of portions of concrete in any transverse section;

8. Positioning and tying of reinforcing steel;
9. Amount and difficulty of finishing the concrete;
10. Time and rate of bleeding; and
11. Unusual construction practices or procedures that might affect the serviceability of the concrete.

The data obtained from these observations and tests provide a means of (a) comparing the procedures and practices followed with those specified and accepted for sound concreting practice, and (b) determining if any relationship exists between variations in construction practices and deterioration which may subsequently occur.

Determining if any relationship exists between construction practices and subsequent deterioration requires that the bridge decks (where the pours were observed) be surveyed periodically. These surveys have been made at least annually since the bridge decks were constructed. Observation of one pour was discontinued in 1962 because the bridge deck was given a bituminous seal treatment. As of the last survey (summer 1965), no deterioration other than cracking has occurred on these bridges. However, if other types of deterioration should occur, these detailed construction records might provide a clue as to the cause.

The results from some of the observations and tests made at the time of the construction of the deck pours indicate the need for more careful control if a concrete meeting all specified requirements is to be produced. These results are presented in the following discussion and tables.

1. Variation in Slump - The data in Table B-1 show that 178 slump tests were made and that 14.1 percent exceeded the specified maximum of five inches. The range of all the slumps varied from 0.2 inches to 8.5 inches. Wide variations in slump on single pours result in concrete that is difficult to finish and highly variable in strength and durability.

2. Variation in Air Content - The specified air content at the time these bridge deck pours were constructed was 3 to 6 percent. The results of the air tests made during the construction of each of the eleven bridge deck pours is shown in Table B-2. The data presented in this table show that 177 air tests were made, that 8.5 percent of these air tests were less than the minimum specified amount, and that 3.4 percent were more than the maximum specified amount. The range of all the air contents varied from 1.0 to 7.9 percent. In every pour the aggregate correction factor used by the field forces was less than that determined in the laboratory (see Table B-2). This partially accounts for the high percent of low air contents (and the low percent of high air contents) mentioned above. The areas of bridge deck with low air content are the areas with questionable protection against possible future deterioration due to applications of de-icing salts.

3. Mixing Time - The data in Table B-3 show that on two of the pours the elapsed time for some of the batches between the start of mixing and the placement of the concrete on the

deck exceeded the specified time of 90 minutes. This resulted in prolonged agitation. Prolonged agitation results in stiffening concrete mixtures which are difficult to place and which may become unworkable unless water is added. Similar results may occur even with shorter periods of agitation when the conditions are unfavorable, such as when unusually high air temperatures are encountered.

4. Continuity of Placement - From the data presented in Table B-4, it can be seen that there were six pours where the elapsed time between placement of some of the contiguous portions of concrete was greater than 45 minutes (the specified limit). Also shown in this table is the percent of transverse sections completely filled with concrete within the time limits indicated at the head of each column. There were five pours where some of these transverse sections were not filled with concrete after a period of 45 minutes. The wide variations in time for both the placement of contiguous batches of concrete and the filling of transverse sections with concrete indicate that portions of the bridge deck, containing concrete varying considerably in plasticity and/or workability, are being finished simultaneously. If the finishing operations are timed (final floating takes place after cessation of bleeding) to the most plastic portion of concrete in a transverse section, it is likely that the concrete will be adequately compacted. If, however, the final compactive effort is stopped before bleeding (or internal

subsidence) is completed, there is a possibility that built-in structural weaknesses (causing fracture plane and/or cracking) will result. However, the timing of the finishing operation in a transverse section to the most plastic portion of concrete, may disturb some of the older concrete in that section, particularly if some initial setting has already taken place. Thus, timing the finishing to the younger concrete could disturb the partially set older concrete and make it more susceptible to deterioration (scaling).

5. <u>Curing</u> - Information on the time of application of the curing mats, the wetness of the curing mats, and the length of time the mats were used in wet curing is given in Table B-5. It can be seen that there is considerable variation (not only between the batches in a single pour but also between the pours) in the time between placement of concrete on a deck and the application of wet curing, and in the duration of curing. In two of the pours (I and J) dry mats were layed on the concrete and then wet down. Our specifications (1955) require that wet mats be placed on the deck and that they be kept in this wet condition for 72 hours.

6. <u>Concrete Placement</u> - Concrete was occasionally deposited into pockets of water which had collected ahead of concrete placement when excessive amounts of water were used to wet down the steel and forms. This water then became mixed with the concrete.

7. Placing Reinforcement - The placement of the reinforcing bars was not consistent from pour to pour, in that the number of ties used to secure the steel varied from a tie at every bar crossing to as few as one tie in 16 bar crossings. This shows that there is disagreement as to the number of ties needed to prevent displacement of the bars during placement of concrete. No direct evidence was obtained during these observations to indicate that an insufficient number of ties was being made, however, evidence has been obtained in observing bridge deck repairs which demonstrates that bars can and do get displaced during placement and finishing of concrete. In addition to variation in number of ties, there was also variation in the spacing of bars within the same pours.

Specifications have been written to control the above items, but these data show that in some cases the actual variations exceed those specified. Large variations in control are definitely undesirable, and permissible variations within the specification requirements may also be undesirably large. Actually, in an effort to obtain a more durable concrete, some specifications have already been rewritten. The specifications were rewritten first to specify a higher air content and later to specify the average desired air content with a plus and minus operating tolerance. The latter step was taken to emphasize that the midpoint rather than the low side of the allowable range was most desirable. Other specification changes were to reduce the maximum allowable slump from five to four inches, to increase the depth of cover over the top reinforcing steel from 1-1/2 to 2 inches on bridges in the interstate or primary systems and to require ties at every reinforcing bar crossing. Other changes included tighter control through increased inspection and more thorough training of both materials and construction inspectors through schools of instruction. It may also be desirable to make additional changes and to add some new ones.

TABLE B-1

STUDY OF SLUMP TEST RESULTS ON ELEVEN POURS

Pour	No. of Tests	Slump Avg.	Slump Range	% of Tests >5"	% of Tests <2"	% of Tests >4"	% of Tests <1"
A	16	3.8"	2.6" to 5.0"	0.0	0.0	37.5	0.0
B	22	3.9"	2.6" to 6.0"	9.1	0.0	45.4	0.0
C	29	3.6"	1.4" to 8.5"	13.8	10.3	20.7	0.0
D	11	4.8"	2.3" to 7.8"	45.5	0.0	63.6	0.0
E	16	3.5"	0.2" to 8.3"	12.5	12.5	25.0	6.25
F	20	4.0"	0.8" to 8.0"	25.0	15.0	45.0	5.0
G	13	3.6"	2.5" to 7.0"	7.7	0.0	23.1	0.0
H	9	2.8"	0.3" to 5.0"	0.0	22.2	22.2	11.1
I	8	4.5"	3.1" to 5.1"	12.5	0.0	75.0	0.0
J	16	3.7"	1.5" to 7.0"	12.5	12.5	25.0	0.0
K	18	4.2"	2.0" to 6.3"	16.7	0.0	55.6	0.0
All Pours	178			14.1	6.7	37.6	1.7

TABLE B-2

STUDY OF AIR TEST RESULTS ON ELEVEN POURS

Pour	No. of Tests	Air Content Percent Range	Percent of Tests >6%	Percent of Tests <3%	Aggr. Correction Factor, % T-704 Report	Aggr. Correction Factor, % Determined
A	16	3.9 to 6.6	6.2	0.0	0.15	0.20
B	22	4.0 to 5.1	0.0	0.0	0.15	0.35
C	29	3.2 to 7.9	13.8	0.0	0.20	0.44
D	10	3.7 to 5.2	0.0	0.0	0.20	0.33
E	16	2.2 to 4.7	0.0	25.0	0.20	0.50
F	20	1.0 to 6.1	5.0	10.0	0.20	0.45
G	13	3.1 to 5.4	0.0	0.0	*	0.50
H	9	2.7 to 4.0	0.0	11.1	0.20	0.35
I	8	2.6 to 3.4	0.0	75.0	0.15	0.50
J	16	3.4 to 5.0	0.0	0.0	0.40	0.45
K	18	1.6 to 5.5	0.0	11.1	0.20	0.50
All Pours	177		3.4	8.5		

*Not known, but probably 0.20%.

TABLE B-3

STUDY OF MIXING AND AGITATING TIME ON ELEVEN POURS

	Elapsed Time, in Minutes, Between							
	Batching Plant and Bridge Site		Arrival at Bridge Site and Start of Unloading		Start and Completion of Unloading		Start of Mixing and Placement on Deck	
Pour	Average	Range	Average	Range	Average	Range	Average	Range
A	13.8	9 to 29	12.8	0 to 39	7.6	4 to 13	44.9	25 to 78
B	14.0	10 to 25	11.8	0 to 31	8.3	5 to 14	44.8	29 to 69
C	24.1	20 to 36	10.7	0 to 32	12.3	5 to 29	47.1	27 to 75
D**							21.5	13 to 46
E**							2.3	1 to 7
F**							10.5	6 to 24
G**							14.3	9 to 27
H**							2.9	2 to 10
I	31.9*	22 to 47*	14.2*	0 to 44*	12.4	2 to 39	73.8	35 to 116
J**							4.6	3 to 14
K	29.3	23 to 37	19.0	0 to 79	10.7	5 to 36	66.5	40 to 129

* For 10 of 15 Batches

** Job mixed concrete.

TABLE B-4

STUDY OF CONCRETE PLACEMENT ON ELEVEN POURS

Pour	Percent of Transverse Sections where the Maximum Difference in Elapsed Time Between Placement of Portions of Concrete in the Section was				Percent of Elapsed Times Between Placement of Contiguous Portions of Concrete which were:			
	<10 Min.	<30 Min.	<45 Min.	>45 Min.	<10 Min.	<30 Min.	<45 Min.	>45 Min.
A	72.3	97.9	100.0	0.0	67.3	99.0	100.0	0.0
B	20.5	94.9	100.0	0.0	61.5	94.0	100.0	0.0
C	35.3	70.6	88.2	11.8	31.2	49.9	89.6	10.4
D	11.1	66.7	88.9	11.1	21.7	61.0	87.0	13.0
E	30.0	100.0	100.0	0.0	40.5	92.8	100.0	0.0
F	7.7	53.9	92.3	7.7	21.6	75.7	89.2	10.8
G	0.0	90.0	100.0	0.0	26.9	88.5	100.0	0.0
H	0.0	90.0	100.0	0.0	37.2	100.0	100.0	0.0
I	11.1	61.1	72.2	27.8	58.5	82.7	89.6	10.4
J	0.0	50.0	100.0	0.0	30.0	75.0	85.0	15.0
K	33.4	93.4	97.8	2.2	42.2	92.2	95.6	4.4

TABLE B-5

APPLICATION, THOROUGHNESS, AND DURATION OF WET CURING

Pour	Elapsed Time Between Placement of Batches and Start of Wet Curing, in Minutes		Curing Water Applied by	Mats Kept Continuously Saturated	Curing Interrupted or Discontinued on Portions of Pour Because of Work on Succeeding Pours	Number of Days Mats Remained on Surface
	Average	Range				
A	107(1)	67 to 147	Lawn Sprinklers	Unknown	Probable, sidewalk poured on 2nd day.	Unknown
B	99	78 to 126	"	Unknown	- "	Unknown
C	202	122 to 281	Periodic Sprinkling	No	On 4th day mats removed along gutter	4+
D	180	136 to 218	"	No	No	4+
E	151	119 to 167	"	No	Interrupted on W. end on 2nd day	4+
F	183	131 to 230	"	No	"	Unknown
G	164	131 to 195	"	No	Unknown	Unknown
H	192	154 to 219	"	No	Yes and because of poor coverage	4+
I	230(2)	161 to 290	"	Unknown	Unknown	Unknown
J	325(2)	214 to 426	"	Unknown	Unknown	Unknown
K	248	176 to 296	"	No	Along each gutter on 3rd day	3+

(1) First 25 of 30 batches.
(2) Mats previously dry, wet down at time indicated.

Phase C

A STUDY TO DETERMINE IF AN ASSOCIATION EXISTS BETWEEN THE OCCURRENCE OF FRACTURE PLANE DETERIORATION, THE OCCURRENCE OF RUST ON REINFORCING STEEL, AND THE DEPTH OF COVER OVER THE TOP REINFORCING STEEL

The objective of this study was to determine if a relationship exists between any combination of the following variables.

1. The occurrence of fracture plane areas.
2. The occurrence of rust on the top reinforcing steel.
3. The depth of the concrete cover over the top reinforcing steel.

Sampling Procedure

The sample for this study was taken from sound areas (no fracture plane deterioration) of bridge deck and from areas containing fracture plane deterioration. The soundness of each sample area was determined by tapping the deck surface with a steel rod. On sound concrete, this tapping results in a "solid" sound; in areas of fracture plane deterioration, the tapping results in a "hollow" sound. The difference in these two sounds is obvious once each has been encountered in the field. Each sample area was drilled to the depth of the top reinforcing steel, and the presence or absence of rust on the reinforcing steel and the clear depth-of-cover over the reinforcing steel was recorded. A total of 148 sample areas on 22 bridges was investigated.

Presentation of Data

In Table C-1, an itemized listing of each of the sample areas is presented. Each sample area is described for the presence of fracture plane deterioration, rusting of reinforcing steel, and depth of cover. The bridges from which the samples were taken are also shown. The frequency of occurrence for any combination of categories for any two of these variables is presented in Tables C-2, C-3, and C-4. The specific variables presented in each of these tables are as follows:

Table C-2: Fracture plane deterioration (present or not present) and the condition of the reinforcing steel (rusted or not rusted).

Table C-3: Fracture plane deterioration (present or not present) and the depth of cover (over 1.5 inches or 1.5 inches or less).

Table C-4: The occurrence of rust and the depth of cover (using the same categories for these variables as in Tables C-2 and C-3).

Statistical tests were made to determine if (1) a mutual association exists between the occurrence of fracture plane, rusting, and depth of cover (3 variables) and (2) an association exists between the combination of any two of these three variables (the possible combinations are shown in Tables C-2, C-3, and C-4). The results of these tests are presented in Table C-5.

The data in Figure 3 show how the depth of cover over the top reinforcing steel varies for sound areas of bridge deck and for areas showing fracture plane deterioration. The curves in this figure are cumulative frequency curves, and they indicate the proportion (percent) of the areas sampled which have a depth of cover equal to or less than a corresponding value indicated on the horizontal axis.

Discussion of Data

The data in Table C-2 indicate that there is an association between fracture plane deterioration and rusting of the reinforcing steel. That is, where one of these variables is present (rusting, for example) there is a strong tendency for the other (fracture plane) to be present, and if one is not present, the other is not likely to be present. The data do not indicate, however, which of these two variables is first to occur.

The data in Table C-3 indicate that there is an association between fracture plane deterioration and the depth of cover. Where the depth of cover is over 1.5 inches there is a tendency for less fracture plane deterioration to occur.

The data in Table C-4 indicate that there is a tendency for less rust to occur on the reinforcing steel when the depth of cover is over 1.5 inches.

The relationships just discussed are based on a visual examination of the data presented in each table. However, a statistical test of the relationship between variables provides a more accurate measure of any existing association. Statistical tests were made on data taken from Table C-1 to determine if there was a mutual association between the three variables listed in the table, and on the data in Tables C-2, C-3, and C-4 to check the validity of the relationships indicated by visual examinations of the data in these tables. For each combination of variables, the data was arranged into contingency table form, and a "Goodness of Fit" test was made. The conclusion of each test is shown in Table C-5, and a discussion of each test follows.

A 3-way contingency test was made to determine if there actually was a mutual association between the three subject variables. A definite association was found to exist. The association is that the tendency for fracture plane deterioration and rusting of reinforcing bars is greatest when the depth of cover is 1.5 inches or less.

Two-by-two contingency tests were used to find the association between the variables examined in Tables C-2, C-3, and C-4. The results of these tests verified the indications which were found by visual examination.

The original data used in preparing Table C-1 have been used in plotting Curves A and B in Figure 3. These curves indicate that the median depth of cover (50th percentile) in the fracture plane areas (54 measurements) is approximately 1.5 inches, and in the sound areas (94 measurements) is approximately 2.0 inches.

There is also indicated (by both curves) a considerable spread (variation) in the depth of cover (from 0.5 to 3.25 inches). It can be determined then (from this variation), that to maintain the average depth of the cover at 1-1/2" or greater in at least 84 percent or any newly placed deck concrete, the specified depth should be about 2 inches. (NOTE: It is indicated by Curve C that the specified depth should be approximately 2.375 inches if a minimum depth of 1.5 inches of cover is to be obtained in at least 95 percent of new deck concrete). The curves shown in Figure 3 are primarily based upon data obtained from bridge decks which were hand finished. Machine finishing methods used on newer decks should result in a smaller variation in the depth of cover, and for such finishing specified depths of cover less than those indicated above would probably be warranted.

Results of Study

There is a mutual association between the depth of cover, the occurrence of rusting, and the occurrence of fracture plane deterioration. There is also a mutual association between any combination of these variables. In the test for association between rusting and fracture plane, it was not indicated which of these two variables was first to occur. The indicated median depth of cover in areas with no fracture plane deterioration was 2.0 inches but, with the indicated variation, 16 percent of the reinforcing steel investigated still had a depth of cover less than 1.5 inches.

TABLE C-1

DESCRIPTION OF THE SAMPLE AREAS USED FOR DETERMINING THE ASSOCIATION BETWEEN FRACTURE PLANE DETERIORATION, DEPTH OF COVER, AND RUSTING OF RE-BARS

Sample Area	Fracture Plane Area		Rust on Re-Bars		Clear Depth of Cover	
	Yes	No	Yes	No	Over 1.5"	1.5" or less
			Bridge A-243			
1		X		X		X
2		X		X	X	
3		X		X	X	
			Bridge A-245			
4		X		X	X	
			Bridge A-247			
5		X		X	X	
			Bridge F-485R			
6	X		X			X
7	X		X			X
8		X		X	X	
9		X		X		X
10	X		X			X
			Bridge G-487			
11	X		X			X
12		X		X	X	
13	X		X			X
14	X		X		X	
15		X		X	X	
16		X		X	X	
			Bridge G-499R			
17		X		X	X	
18		X		X	X	
19		X		X	X	
20		X		X	X	
21		X		X	X	
			Bridge J-40R			
22	X		X			X
23	X		X			X
24		X		X		X
25	X		X			X
26		X	X			X
27	X		X			X

Table C-1, Cont'd.

Sample Area	Fracture Plane Area: Yes	Fracture Plane Area: No	Rust on Re-Bars: Yes	Rust on Re-Bars: No	Clear Depth of Cover: Over 1.5"	Clear Depth of Cover: 1.5" or less
			Bridge J-217			
28	X		X		X	
29	X			X	X	
30		X		X	X	
31		X		X	X	
32	X			X	X	
33		X		X	X	
34	X			X	X	
35		X		X	X	
36	X			X	X	
37		X		X	X	
			Bridge J-991			
38		X		X	X	
39		X		X	X	
40		X		X	X	
41	X		X		X	
			Bridge J-993			
42		X		X	X	
43	X			X	X	
44	X			X	X	
45		X		X	X	
46		X		X	X	
			Bridge K-294			
47		X		X	X	
48		X		X	X	
49		X		X	X	
50		X		X	X	
51		X		X	X	
			Bridge K-458			
52	X		X			X
53		X		X		X
54	X		X			X
55	X		X			X
56		X	X			X
57		X		X		X
58	X		X			X
59		X		X		X
60		X	X			X
61	X			X		X
			Bridge K-506			
62		X		X	X	
63		X		X	X	
			Bridge K-606			
64		X		X	X	

Table C-1, Cont'd.

Sample Area	Fracture Plane Area		Rust on Re-Bars		Clear Depth of Cover	
	Yes	No.	Yes	No	Over 1.5"	1.5" or less
Bridge K-790						
65	X		X		X	
66	X		X		X	
67	X		X		X	
68	X		X		X	
69		X		X	X	
70		X		X	X	
71		X		X	X	
72		X		X	X	
73		X		X	X	
74		X		X	X	
75		X		X	X	
76		X		X	X	
77		X		X		X
78		X		X	X	
79		X		X	X	
80		X		X	X	
Bridge K-795						
81		X	X			X
82		X	X		X	
83	X		X		X	
84		X	X		X	
85		X	X		X	
86	X		X		X	
87		X	X		X	
88	X		X		X	
89		X		X	X	
Bridge L-403						
90	X		X		X	
91	X		X			X
Bridge L-492						
92	X		X			X
93		X		X	X	
94	X		X		X	
95	X		X		X	
96	X		X		X	
97	X		X			X
98	X		X		X	
99		X		X	X	
100		X		X	X	
101		X		X	X	
102		X		X	X	
103		X		X	X	
104		X		X	X	
105		X		X	X	

Table C-1, Concluded

Sample Area	Fracture Plane Area: Yes	Fracture Plane Area: No	Rust on Re-Bars: Yes	Rust on Re-Bars: No	Clear Depth of Cover: Over 1.5"	Clear Depth of Cover: 1.5" or less
			Bridge L-493			
106	X		X			X
107		X		X	X	
108	X		X			X
109	X		X			X
110	X		X			X
111	X		X			X
112		X		X		X
113		X		X		X
114		X		X	X	
115		X		X	X	
116		X		X	X	
117	X		X			X
118		X		X	X	
			Bridge L-536			
119		X		X	X	
120		X		X		X
121		X		X	X	
			Bridge L-641			
122		X		X		X
123	X		X			X
124		X	X			X
125		X	X		X	
126	X		X			X
127	X		X			X
128	X		X			X
129		X		X		X
130	X		X		X	
131		X	X		X	
132		X		X		X
133		X		X		X
134		X		X		X
135		X		X	X	
			Bridge L-655			
136	X		X			X
137		X	X			X
138		X		X	X	
139	X		X			X
140		X		X		X
141	X		X			X
142		X		X		X
143	X		X			X
144		X	X		X	
145	X		X			X
146	X		X			X
147		X		X	X	
148		X		X	X	
Totals	55	93	61	87	92	56

Table C-1, Concluded

Sample Area	Fracture Plane Area		Rust on Re-Bars		Clear Depth of Cover	
	Yes	No	Yes	No	Over 1.5"	1.5" or less
			Bridge L-493			
106	X		X			X
107		X		X	X	
108	X		X			X
109	X		X			X
110	X		X			X
111	X		X			X
112		X		X		X
113		X		X		X
114		X		X	X	
115		X		X	X	
116		X		X	X	
117	X		X			X
118		X		X	X	
			Bridge L-536			
119		X		X	X	
120		X		X		X
121		X		X	X	
			Bridge L-641			
122		X		X		X
123	X		X			X
124		X	X			X
125		X	X		X	
126	X		X			X
127	X		X			X
128	X		X			X
129		X		X		X
130	X		X		X	
131		X	X		X	
132		X		X		X
133		X		X		X
134		X		X		X
135		X		X	X	
			Bridge L-655			
136	X		X			X
137		X	X			X
138		X		X	X	
139	X		X			X
140		X		X		X
141	X		X			X
142		X		X		X
143	X		X			X
144		X	X		X	
145	X		X			X
146	X		X			X
147		X		X	X	
148		X		X	X	
Totals	55	93	61	87	92	56

TABLE C-2

	Condition of Reinforcing Steel		
	Rusted	Not Rusted	Total
Fracture Plane Present	48	7	55
No Fracture Plane Present	13	80	93
Total	61	87	

TABLE C-3

	Depth of Cover		
	1.5" or less	Over 1.5"	Total
Fracture Plane Present	33	22	55
No Fracture Plane Present	23	70	93
Total	56	92	

TABLE C-4

	Depth of Cover		
	1.5" or less	Over 1.5"	Total
Rust	38	23	61
No Rust	18	69	87
Total	56	92	

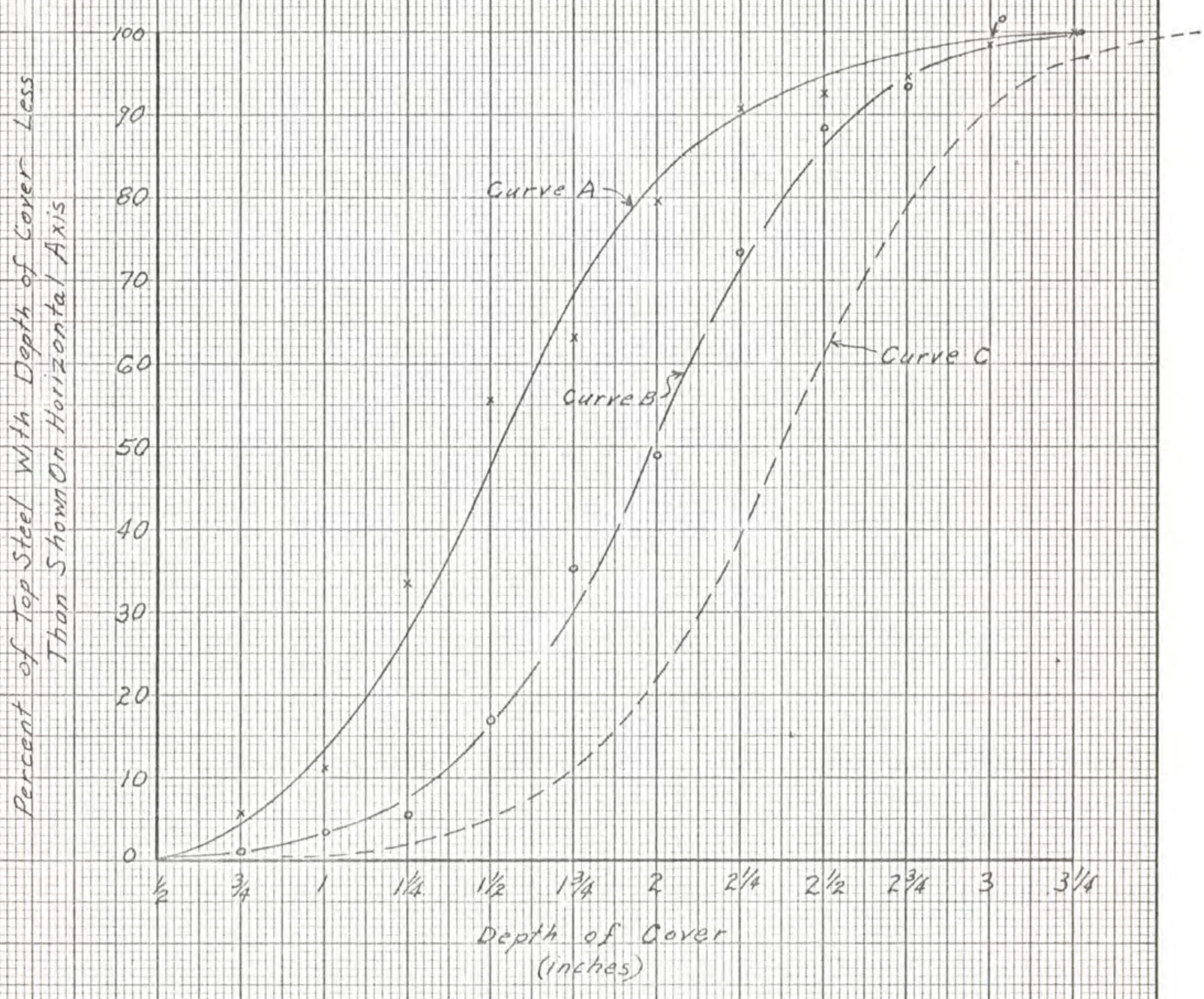
TABLE C-5

SUMMARY OF RESULTS OF STATISTICAL TESTS
MADE TO DETERMINE THE RELATIONSHIPS BETWEEN
FRACTURE PLANE, RUSTING OF RE-BARS, AND DEPTH OF COVER

Test	Type of Contingency Table	Variables	Variables Mutually Associated
A	3-way	Fracture Plane Depth of Cover Rusting	Yes
B	2 by 2	Fracture Plane Depth of Cover	Yes
C	2 by 2	Fracture Plane Rusting	Yes
D	2 by 2	Rusting Depth of Cover	Yes

Figure 3

Cumulative Proportion Distributions For Depth of Cover Over the Top Reinforcing Steel In Areas of Bridge Deck With and Without Fracture Plane Deterioration



Curve A- Distribution in areas with fracture plane deterioration.

Curve B- Distribution in areas without fracture plane deterioration.

Curve C- This curve is identical in shape to Curve B. It indicates that for a median depth-of-cover of 2 3/8 inches, 5 percent of the top steel will still have a depth-of-cover less than 1 1/2 inches.

Phase D

A STUDY TO DETERMINE THE PHYSICAL DIFFERENCES BETWEEN CONCRETE IN DETERIORATED AND UNDETERIORATED AREAS OF BRIDGE DECKS

The purpose of this study was to determine what physical differences exist between the concrete in deteriorated and adjacent undeteriorated (sound) areas of bridge decks. The study was confined to two types of deterioration — fracture plane and surface mortar deterioration.

Samples of concrete having each type of deterioration and samples of sound concrete adjacent to each type of deterioration were drilled from the decks of several bridges. A total of 61 core samples was obtained.* The cores were tested in the laboratory for cement factor, water-cement ratio, porosity, air content and spacing factor. Details of the testing procedures appear in the paper, "A Method of Estimating the Original Mix Composition of Hardened Concrete Using Physical Tests", by E. O. Axon, in Proc. Am. Soc. Testing and Mat., Vol. 62, pp. 1068 - 1080, 1962.

Presentation of Data

In Tables D-1, D-2, D-3, and D-4 is listed each of the cores tested and the bridge from which each was drilled.

*Two cores were drilled from Bridge K-795. Core number 89 was drilled in an area of both fracture plane and surface mortar deterioration, and core number 94 was drilled in an area of surface mortar deterioration only. In the tables showing results relative to fracture plane deterioration, core 89 was included in Table D-1 and core 94 was included in Table D-2 (no fracture plane was present). Both cores were included in Table D-3 because surface mortar deterioration was present on each.

Tables D-1 and D-2 list the cores drilled for the study of fracture plane deterioration and Tables D-3 and D-4 list the cores drilled for the study of surface mortar deterioration. The results of tests on each core are shown in these tables.

Discussion of Data

Fracture Plane Deterioration

The mechanism by which fracture plane deterioration occurs is not presently understood. However, a working hypothesis on the mechanics of fracture plane formation is presented in the next section of this report. This working hypothesis serves as a procedural guide for testing the cores drilled from areas of fracture plane deterioration.

Before briefly discussing this hypothesis, however, it may be well to first mention the mechanics of bleeding, particularly as it is expected to occur in normal slab construction. Concrete is a heterogeneous mixture of components of varying gravity. The bleeding results when the heavier components tend to settle and consolidate, which forces the water to rise. In this sedimentation process the maximum consolidation of the heavier material occurs first at the bottom of the deck and progresses upward, forcing the water toward the top. If the concrete remains plastic for a sufficient period of time, complete full depth consolidation may be attained and all of the excess water forced to the surface. In actuality, completion of this process would not be expected, and the water content and porosity of the concrete would reach a maximum at the surface and be at a minimum in the bottom portion of the deck.

The hypothesis presented in the next section postulates that normal sedimentation of the heavier components of the concrete above the reinforcing steel is interrupted by the rigidly supported steel and that a plane of weakness or of high water gain is built into the concrete in the upper portion of the bridge deck slab (in the plane of the top mat of reinforcing steel). Various factors cause this plane of weakness to change into a fracture plane. A characteristic of the plane of weakness is that it interrupts the normal bleeding process. That is, bleeding water originating in the portion of concrete below the plane of weakness will not reach the surface of the deck; bleeding water above the plane of weakness freely flows to the surface and evaporates or runs off. The water-cement ratio would then be expected to be greater in the concrete immediately below the plane of weakness than in the concrete above. Since the porosity of the paste in concrete is directly proportional to the water-cement ratio, a measure of the porosity of the concrete taken from below and from above the fracture plane will indicate if the water-cement ratio actually varies in the manner stated in the hypothesis. It should be stated here that no evidence was obtained which would indicate a difference (at time of placement) in the quality of the concrete appearing either above or below a fracture plane when fracture plane deterioration finally develops.

The porosity of the cement paste above the fracture plane (the cap) and immediately below the fracture plane (a section of the core approximately equal in volume to that of the cap) is shown in Table D-1 for each core drilled from areas of bridge decks containing fracture plane deterioration. In eighteen of the 20 cores, the porosity for the cement paste below the fracture plane was greater than for the cement paste above.

For determining the porosity of the top portions of cores drilled from sound appearing concrete (adjacent to areas of fracture plane), two disks each approximately 1 inch thick were sawed from the top of each core*. The results of the porosity tests on these disks are shown in Table D-2. For 11 of the 20 cores, the porosity of the cement paste in the disk adjacent to the top disk was greater than the porosity of the cement paste in the top disk. This was not expected for the sound concrete. However, since these cores were drilled close (adjacent) to areas of fracture plane, it is assumed that the results on these 11 cores indicate the presence of a plane of weakness which has not yet developed into a fracture plane.

From examination of the average porosities shown at the bottom of Tables D-1 and D-2, conformance (with the hypothesis) is indicated by their mode of variation for the cement paste in the cap and the cement paste adjacent to the

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*This thickness is the same as the average depth of the caps found on cores listed in Table D-1.

cap between cores drilled from deteriorated and from sound concrete. In sound concrete where normal bleeding takes place, the porosity (of the paste) should be greater in the top section of the slab; in deteriorated concrete (fracture plane) the porosity is greater in the section adjacent to the cap.

It can be seen (Tables D-1 and D-2) that the average cement factor for the cores from the deteriorated concrete and for the cores from the sound concrete was almost identical. However, the average water-cement ratio was less by 0.04 by volume, or 0.3 gallons per sack, for the cores drilled from the sound concrete than it was for the cores drilled from deteriorated concrete. This difference in water-cement ratios is not to be unexpected since the plane of weakness should be more pronounced in concretes with higher water-cement ratios (because there is more subsidence).

The relationship between water-cement ratio and difference in porosity of cement paste in concrete above and below a fracture plane for each core in Table D-1 is presented in Figure 4. The line of best fit indicates that this difference increases at the rate of 2.25 percent for each 0.10 cubic foot increase in water-cement ratio. The negative values on the ordinate (vertical axis) indicate that the porosity of the concrete below a fracture plane is greater than that above. The line of best fit intersects the line of zero difference at a water-cement ratio of 0.57 by volume (4.3 gallons per sack);

TABLE D-1

TEST DATA ON CORES DRILLED FROM AREAS OF FRACTURE PLANE DETERIORATION

Bridge No.	Core No.	Cement Factor Bbls/yd^3	Water-Cement Ratio by Vol.	- Porosity - Percent Absorption (by Vol.) of Paste, Without Air, in the: Top of Core (Cap)	Section Adjacent to Cap
L-232	5	1.48	0.75	40.3	38.3
"	7	1.39	0.75	39.5	40.5
"	9	1.30	0.77	41.9	44.1
"	10	1.35	0.79	42.8	49.1
"	16	1.54	0.68	35.1	39.4
"	17	1.50	0.78	43.6	42.8
"	18	1.36	0.75	38.2	43.4
"	19	1.51	0.72	41.0	43.6
"	20	1.52	0.74	41.3	48.0
"	23	1.50	0.77	39.1	43.8
"	32	1.27	0.89	43.9	50.7
"	37	1.62	0.62	34.6	36.2
L-428	6	1.26	0.77	43.4	48.9
"	2	1.37	0.77	37.2	45.8
K-790	38'	1.45	0.75	38.6	40.9
"	39'	1.51	0.72	39.6	49.3
G-487	161	1.66	0.70	36.3	39.0
L-493	242	1.44	0.85	40.2	50.5
K-500	33'	1.29	0.76	40.9	41.5
K-795*	89	1.35	0.84	39.0	46.5
Average		1.43	0.76	39.8	44.1

* Non Air-Entrained

TABLE D-2

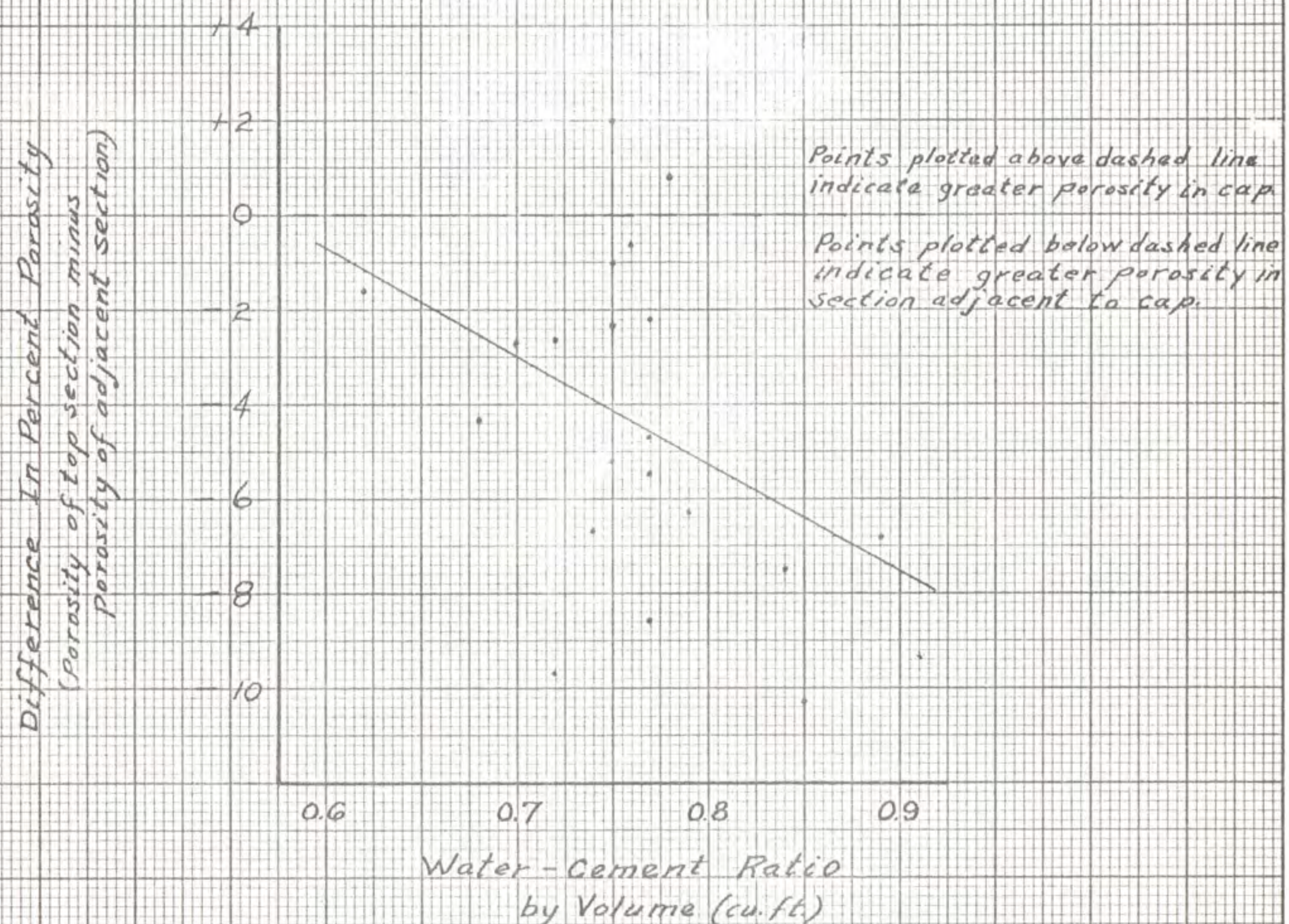
TEST DATA ON CORES DRILLED FROM
SOUND AREAS ADJACENT TO FRACTURE PLANE DETERIORATION

Bridge No.	Core No.	Cement Factor $Bbls/yd^3$	Water-Cement Ratio by Vol.	- Porosity - Percent Absorption (by Vol.) of Paste, Without Air, in the: Top of Core (Cap)	Section Adjacent to Cap
L-232	6	1.39	0.75	42.7	32.5
"	26	1.52	0.75	40.8	37.9
"	27	1.50	0.75	40.4	41.8
"	28	1.52	0.71	38.0	34.4
"	38	1.25	0.75	39.8	42.8
"	42	1.40	0.70	38.1	38.4
"	46	1.39	0.74	37.6	39.0
"	48	1.50	0.67	39.7	41.2
"	49	1.40	0.71	40.5	36.5
"	50	1.48	0.64	37.9	39.3
"	51	1.53	0.68	36.7	36.9
"	58	1.50	0.67	40.2	37.1
L-428	7	1.34	0.74	33.5	39.5
"	8	1.18	0.84	37.3	49.0
K-790	304	1.44	0.69	40.5	39.0
"	310	1.51	0.68	44.6	39.6
G-487	160	1.48	0.76	45.6	36.2
L-493	243	1.53	0.79	39.7	44.8
K-500	34'	1.47	0.68	39.8	40.4
K-795*	94	1.41	0.70	39.2	36.5
Average		1.44	0.72	39.6	39.1

*Non Air-Entrained

Figure 4

Difference In Porosity of Paste In Concrete Above and Below A Fracture Plane As A Function of the Water-Cement Ratio of the Core As A Whole.



this data indicates that there is a tendency for a greater occurrence of planes of weakness as the water-cement ratio increases above this amount.

Surface Mortar Deterioration

Ice and snow control procedures on bridge decks often result in deterioration of the deck concrete. Protection is afforded to the concrete if proper air entrainment is provided at the time the bridge deck is constructed. Proper air entrainment requires that minute air voids be uniformly dispersed throughout the cement paste. Proper dispersion is attained when the minute air voids are spaced closely enough that no portion of the cement paste is greater than 0.01 inches (or possibly less) from the periphery of an air void. Unless some minimum volume of uniformly dispersed air voids (bubbles) is entrained, this maximum desirable spacing will be exceeded.

In Figure 5 is shown the relationship between percent air and spacing factor for each of the 61 cores tested. The data in this figure show that the spacing factor decreases as the percent of air increases. For those cores with air contents of 3 percent and above, the spacing factors are 0.01 inches or less — with two exceptions. Exceptions such as these usually indicate the presence of an excessive amount of entrapped air.

In Table D-3 are shown the results of tests on cores drilled from areas of surface mortar deterioration. The top half of the table shows test results for whole cores, and the bottom half shows test results for one-inch disks cut from the top of each of these cores. The results in Table D-4 are arranged in like fashion but they are for tests on cores drilled from sound concrete adjacent to areas of surface mortar deterioration.

Of the 11 cores listed in Table D-3, 9 had high water content, low air content, or both.* (NOTE: Two of these 9 cores were drilled from a bridge which was constructed with non air-entrained concrete.) Of the remaining 2 cores (numbers 61-292 and 61-312), one, No. 61-292, had a high spacing factor which indicates that it contained high entrapped air.

Of the 12 cores listed in Table D-4, 6 had high water content, low air content, or both.* Seven of the 12 cores had adequate spacing factors. While these comparisons of results for cores in Tables D-3 and D-4 do not show a great difference, it must be remembered that the cores listed in Table D-4 were drilled adjacent to deteriorated areas. In fact, they may have come from the same batches which have already partially deteriorated.

The differences in the concrete from deteriorated areas and from undeteriorated areas can be more easily seen by comparing the average test results from each group of cores. The average results for whole cores and for top sections of cores from deteriorated and from undeteriorated areas are shown in the following table. Test values for the two non air-entrained cores (Bridge K-795) were not considered when calculating these averages.

*Assuming a specified water content of 6.25 gallons or less and a specified air content of 3 to 6 percent.

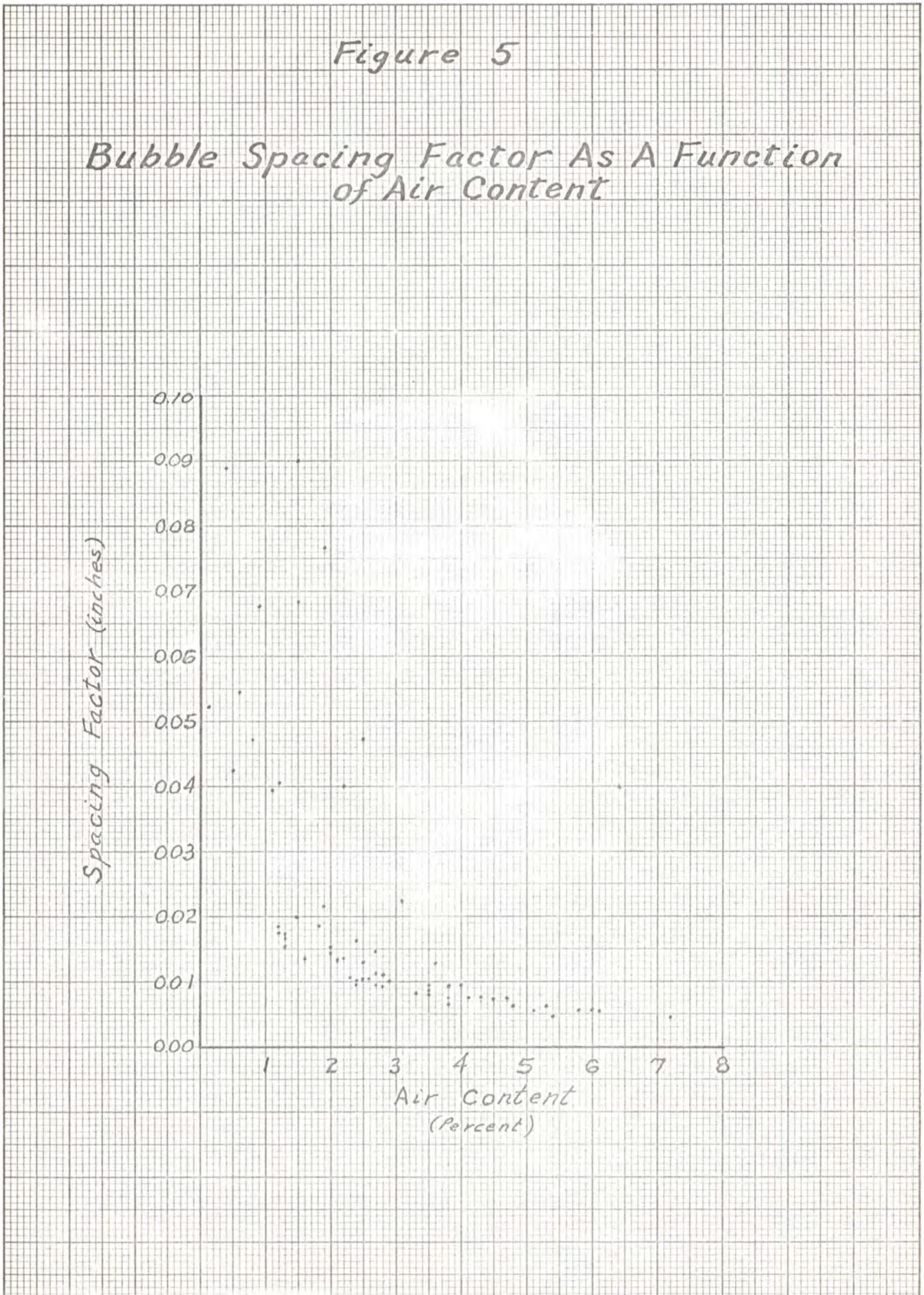


Figure 5
Bubble Spacing Factor As A Function of Air Content
Spacing Factor (inches)
0.10
0.09
0.08
0.07
0.06
0.05
0.04
0.03
0.02
0.01
0.00
1
2
3
4
5
6
7
8
Air Content
(Percent)

TABLE D-3 - STUDY OF MIX COMPOSITION OF CONCRETE CORES DRILLED IN AREAS OF SURFACE MORTAR DETERIORATION

Bridge No.	Core No.	Age 1962 (Years)	Type Concrete	Cement Factor Bbls/yd^3	Water-Cement Ratio by Vol.	Percent Air	Spacing Factor (in.)	Percent Absorption of Paste Without Air (Porosity)
			Data for Whole Cores					
A-153	61-344	2	Air-Ent.	1.62	.89	1.9	.0213	48.8
L-555	61-292	7	Air-Ent.	1.48	.75	3.6	.0129	40.6
"	61-303	7	Air-Ent.	1.52	.86	2.8	.0110	45.3
"	61-312	7	Air-Ent.	1.62	.76	4.5	.0073	40.9
L-682	61-447	7	Air-Ent.	1.31	.69	2.2	.0404	37.7
"	61-457	7	Air-Ent.	1.39	.64	1.2	.0409	34.2
L-550	61-220	8	Air-Ent.	1.28	.99	2.7	.0112	50.3
"	61-224	8	Air-Ent.	1.29	.89	3.5	.0081	46.6
L-623	61-486	8	Air-Ent.	1.15	.88	4.1	.0076	46.2
K-795	89	21	Non A.E.	1.35	.84	0.[illegible]	.0471	44.5
"	94	21	Non A.E.	1.41	.70	0.5	.0426	37.7
		Average		1.402	.808	2.5	.0228	42.98
			Data for Top Sections					
A-153	61-344			1.47	.98	1.55	.0145	54.8
L-555	61-292			1.49	.77	2.97	.0115	41.4
"	61-303			1.65	.88	2.24	.0103	46.0
"	61-312			1.87	.66	4.64	.0072	35.4
L-682	61-447			1.36	.87	.36	.0301	45.4
"	61-457			1.37	.68	.65	.0325	36.3
L-550	61-220			1.21	1.03	1.61	.0099	52.5
"	61-224			1.40	.85	2.70	.0089	45.0
L-623	61-486			1.59	.70	4.81	.0073	37.7
K-795	89			1.69	.73	.55	.0337	39.0
"	94			1.71	.73	.91	.0343	39.2
		Average		1.53	.807	2.09	.0182	42.97

TABLE D-4 - STUDY OF MIX COMPOSITION OF CONCRETE CORES DRILLED IN SOUND CONCRETE ADJACENT TO AREAS OF SURFACE MORTAR DETERIORATION

Bridge No.	Core No.	Age 1962 (Years)	Type Concrete	Cement Factor $Bbls/yd^3$	Water-Cement Ratio by Vol.	Percent Air	Spacing Factor (in.)	Percent Absorption of Paste Without Air (Porosity)
			Data for Whole Cores					
A-153	61-340	2	Air-Ent.	1.53	.66	4.3	.0077	38.2
"	61-341	2	Air-Ent.	1.60	.72	3.8	.0066	41.1
"	61-342	2	Air-Ent.	1.47	.74	4.8	.0063	42.1
"	61-343	2	Air-Ent.	1.71	.71	2.5	.0130	40.6
"	61-345	2	Air-Ent.	1.62	.71	4.7	.0076	41.0
L-555	61-310	7	Air-Ent.	1.49	.81	3.3	.0080	43.2
"	61-297	7	Air-Ent.	1.43	.85	2.3	.0109	44.8
L-682	61-459	7	Air-Ent.	1.77	.60	1.1	.0395	31.6
"	61-450	7	Air-Ent.	1.79	.58	3.1	.0222	29.5
L-550	61-217	8	Air-Ent.	1.42	.74	2.7	.0096	39.9
L-623	61-476	8	Air-Ent.	1.52	.77	2.9	.0100	41.2
"	61-480	8	Air-Ent.	1.24	.83	2.5	.0103	44.1
		Average		1.549	.727	3.2	.0126	39.78
			Data for Top Sections					
A-153	61-340			1.97	.62	3.59	.0054	35.6
"	61-341			1.72	.76	3.42	.0056	42.9
"	61-342			1.94	.65	4.39	.0062	37.3
"	61-343			1.53	.81	2.23	.0094	46.1
"	61-345			1.10	.94	3.70	.0063	50.6
L-555	61-310			1.75	.72	2.79	.0079	42.1
"	61-297			1.80	.68	3.12	.0109	35.9
L-682	61-459			1.90	.61	.52	.0339	31.4
"	61-450			2.08	.58	2.53	.0139	30.3
L-550	61-217			1.41	.63	3.22	.0068	33.8
L-623	61-476			1.66	.71	2.18	.0096	38.2
"	61-480			1.52	.72	2.08	.0092	38.7
		Average		1.70	.702	2.81	.0104	38.58

	Water-Cement Ratio (by Vol.)	Water-Cement Ratio (Gal/Sk)	Percent Air	Spacing Factor
			Whole Cores	
Sound Areas	.727	5.45	3.2	.0126
Deteriorated Areas	.817	6.13	2.9	.0179
			Top Sections	
Sound Areas	.702	5.26	2.81	.0104
Deteriorated Areas	.824	6.18	2.39	.0147

The average values shown in this table for the whole cores and the top sections of cores indicate that high water-cement ratios, low air contents, and excessive spacing factors are characteristics of concrete showing surface mortar deterioration.

Results of Study

The tests on concrete in the study of fracture plane deterioration were directed toward authenticating the plane of weakness theory. While the results are not conclusive, they are in corroboration with the theory. They have shown that the porosity of the paste in the concrete immediately below a fracture plane is greater than in the concrete above. This is due to an interruption of the normal bleeding process caused by a plane of weakness which is postulated to be present before a fracture plane develops. The higher porosity is indicative of a higher water-cement ratio. The conflicting nature of the test results on some of the cores from sound (supposedly) concrete is attributed to the presence of undetected planes of weakness.

The tests on concrete in the study of surface mortar deterioration have indicated that high water-cement ratios and inadequate amounts of entrained air (resulting in excessive spacing factors) are characteristics of concrete exhibiting surface mortar deterioration. Elimination of these factors should tend to reduce (if not eliminate) the problem of surface mortar deterioration.

HYPOTHESIS ON THE MECHANICS OF THE FORMATION OF FRACTURE PLANE DETERIORATION

The mechanism by which fracture plane deterioration occurs is believed to involve the combined effect of many factors. The rather recent (since the mid 1950's) occurrence of this deterioration indicates that one or more of these factors is of recent origin, or the problem would have existed previously. The fact that bridges constructed prior to the mid 1950's are showing this type of deterioration (as are the bridges since constructed) suggests that some factors necessary for the mechanism to become operative may have been present but dormant in them. These factors are probably present in a bridge from the time it is constructed, and the mechanism is triggered by other influences impressed on the structure.

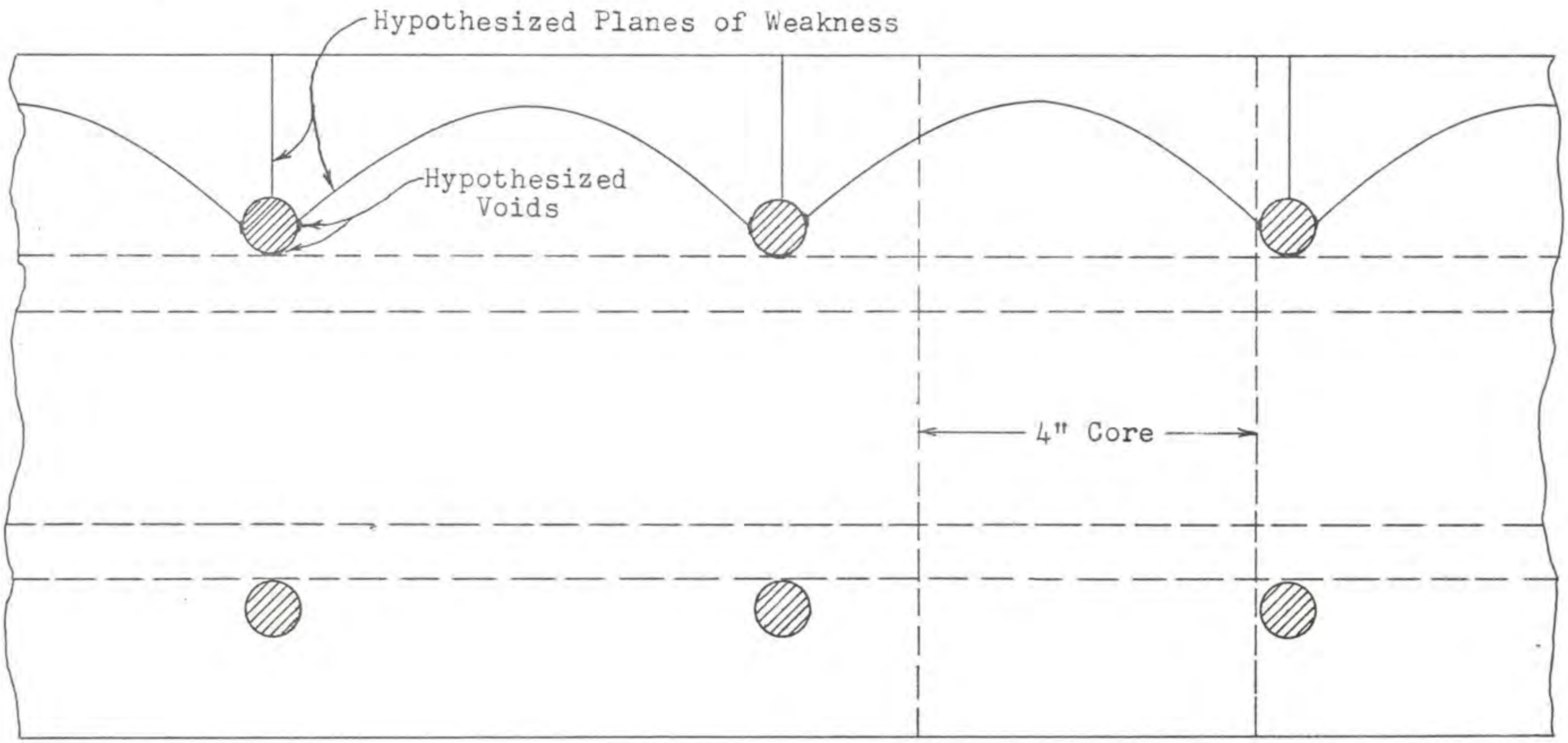
The definition of fracture plane deterioration was previously given (page 8). This definition states simply that fracture plane deterioration is a separation of upper and lower elements of a concrete slab, and the approximate location of the plane of separation relative to the surface of the slab was given. The definition does not state what forces cause the plane of separation or what determines the location of its occurrence. This hypothesis is presented to explain how the occurrence of a particular system of factors may result in fracture plane deterioration.

The central feature of the hypothesis is a plane (or zone) of weakness which is built into the deck at the time it is constructed. This plane of weakness (plane of high water gain) is located in the plane of the top mat of reinforcing steel. The plane is actually an undulating surface running between upper parallel reinforcing bars. Other weaknesses may also be built into the deck; they include a vertical plane of weakness over the top reinforcing bars and voids or zones of high water gain below and possibly along the sides of the top reinforcing bars. A section of bridge deck showing the approximate location of these hypothesized planes of weakness (both over and between each of the top reinforcing bars) and the voids at the bottom and the sides of the top bars is shown in Figure 6. The vertical planes of weakness may eventually develop into full fledged cracks and permit ready access of water to the interior of the slab. The voids around the steel provide little reservoirs in which the water and/or salt solutions may accumulate.

The undulating or horizontal plane of weakness (plane of high water gain) is due primarily to subsidence of the fresh concrete and to support given by the top reinforcing bars to the top layer of concrete. The magnitude or degree of weakness of this plane is dependent upon the slump of the concrete, the size and spacing of the reinforcing bars, the maximum size and angularity of the coarse aggregate, the depth of the concrete beneath the top reinforcing bars, the timing of the finishing operations, the temperature of the air and concrete, and the vibrational stresses caused by movement of the entire deck during or shortly after placement of the concrete.

FIGURE 6

HYPOTHESIZED VOIDS AND PLANES OF WEAKNESS SHOWN IN A PARTIAL CROSS-SECTION OF BRIDGE DECK



Scale: 1" = 2"

The partial underfilling of the forms during construction may result in the deposit of a scum (laitance) which may drain from the surface of surrounding concrete to underfilled areas, and this laitance can prevent a strong bond between the concrete in the bottom layer and the concrete placed on top. The amount of laitance is dependent upon the water-cement ratio, the elapsed time before final placement of the top layer of concrete in the underfilled areas, and the method of placing and finishing the top layer. If the top of this bottom layer of concrete coincides with the level where the plane of high water gain occurs, a deposit of laitance would accentuate the strength loss in the plane of weakness, and the effects of succeeding factors which would have changed the plane of weakness by itself into a fracture plane will be even more pronounced.

The vertical planes of weakness and the voids around the steel are caused by irregular subsidence of the fresh concrete about the rigidly held reinforcing bars. Under traffic, the vertical planes of weakness can develop into vertical cracks.

During periods when the pavement is wet, water or de-icing solutions can penetrate the cracks over the reinforcing steel, fill the voids around the reinforcing steel, and seep into the built-in horizontal plane of weakness. The use of de-icing salts during freezing temperatures would provide the solution and increase the time available for these solutions to penetrate. The decks of more flexible design, especially those that are heavily traveled, are more susceptible because of their flexing action to the penetration of water and de-icing solution. The varying concentrations of these solutions in the concrete cause osmotic pressures which also have a disruptive effect. The salt solutions, along with variations in the chemical properties of the steel, can form small electrochemical cells which contribute to corrosion of the steel. The corrosion causes pressures which are disruptive to the concrete. With decreasing temperatures, the solutions in the vertical cracks freeze and entrap the solutions in the built-in voids. As the temperature declines, the solution in the voids freezes and pressure is exerted in the horizontal plane of weakness. This pressure, the osmotic pressure, the pressure caused by corrosion of the reinforcing steel, plus stress resulting from the flexing action of the deck could, after many applications, change the horizontal plane of weakness into a fracture plane.

Available data also indicate that the daily minimum temperature in the surface of bridge decks tends to be lower than the minimum air temperature by up to 5°F. The low temperatures, then, increase the probability that some or all of the solution in the voids in the concrete will freeze. Accumulated salts in the plane of weakness should also tend to hold water and reduce the benefits of drying periods.

Attempt to Develop Fracture Plane Deterioration in Laboratory Specimens

Attempts to develop fracture plane deterioration in laboratory specimens have been made. These attempts involved reproducing (insofar as possible) the conditions which occur in the field and which are believed to contribute to fracture plane deterioration. All of the conditions occurring in the field could not be duplicated; for example, the stresses induced in a bridge deck by moving vehicles were not induced in the test specimens. However, the specimens were constructed by procedures similar to those followed in the field and they were subjected to different modes of exposure.

All of the laboratory specimens consisted of 8" x 8" x 18" reinforced concrete blocks. Each block contained three 5/8 inch round reinforcing bars (6" spacing) held rigidly in place with 1.5 inches clear depth of cover. All bars were brushed free of rust before being incorporated into the blocks. After the concrete had set, cracking was observed over the steel, and small horizontal separations were noted on the sides of the blocks running from the steel (about 0.5 inch long). To check this cracking, another block similarly constructed was sawed in half (lengthwise) through the steel, and the interior surface was examined; vertical cracks over the steel and short horizontal cracks starting at the bar were visible. There was, however, no evidence (even by microscopic examination) of the presence of additional cracking in the plane of the top reinforcing bars or of a plane of weakness.

The blocks were subjected to different types of exposure. Six blocks in all were tested. A neat cement-paste dike (0.5 inch high) was built around the edge of each test block so that different solutions could be impounded on the surface of the block. The data in Table E-1 show the type exposure given each test specimen.

Each specimen was tested for fracture plane by sounding the surface with a hammer; these soundings were made at the end of each cycle (whether freeze and thaw or wet and dry cycles). No fracture plane deterioration was detected.

Considerable surface mortar deterioration occurred on the blocks subjected to calcium chloride exposure. Those blocks with the highest concentrations of calcium chloride showed the severest deterioration. The reduced freeze and thaw exposure on the specimens with the highest concentrations of calcium chloride resulted from the advanced surface mortar deterioration on those blocks.

At the conclusion of the test, the reinforcing bars were removed from the test specimens and examined. Rusting was observed on several bars, and most of this rust was located on the bottom of the bar; this would indicate that water or de-icing solution passed through some of the vertical cracks over bars and accumulated in a void at the underside to cause the corrosion.

Even though a fracture plane was not developed in laboratory specimens, the data presented in Phase D, page 71, corroborates the hypothesis that a plane of weakness (or a plane

Table E-1

TYPES OF EXPOSURE GIVEN TO SPECIMENS MADE TO DEVELOP FRACTURE PLANE DETERIORATION IN THE LABORATORY

Number of Specimens	Type of Exposure	
1	Water	311 Freeze and thaw cycles
1	5%* $CaCl_2$	310 Freeze and thaw cycles
1	10%* $CaCl_2$	280 Freeze and thaw cycles
1	15%* $CaCl_2$	203 Freeze and thaw cycles
1	20%* $CaCl_2$	153 Freeze and thaw cycles
1	10%* $CaCl_2$ & water, 120 Wet and dry cycles**	

* Percent by weight

** Specimens were alternately exposed to 10 wet and dry cycles with 10% $CaCl_2$ and 10 wet and dry cycles with tap water. After a total of 100 wet and dry cycles (both $CaCl_2$ and water), three 1/8-inch diameter holes were drilled over the re-bars (at the ends and middle of each bar) and 10 more wet and dry cycles with 10% $CaCl_2$ and 10 more wet and dry cycles with tap water were applied.

of high water gain) is built into bridge decks. Consequently it would appear desirable to adopt the remedial measures suggested in the next section since they deal primarily with insistance on sound concreting practice. Adoption of the suggested maintenance seal would be helpful in preventing surface mortar deterioration as well as fracture plane deterioration. The suggested change in design to reduce vibration is worthy of consideration, although it may be of secondary importance.

MEASURES TO PREVENT OR REDUCE DETERIORATION IN NEW AND OLD BRIDGE DECKS

The two types of deterioration considered most detrimental in bridge decks are fracture plane and surface mortar deterioration. Surface cracking is considered detrimental mainly because there is some indicated tendency for fracture plane deterioration to be more prevalent when cracking is present.

The mechanics of the formation of fracture plane deterioration as postulated in the previous section is indicated to involve the cumulative effect of many factors. The control of many of these factors is subject to change or improvement. For example, the results of these studies indicate that:

1. The slump and water-cement ratio of the concrete should be kept as small as possible. Since a given slump can be maintained with a lower water-cement ratio in air-entrained concrete than in non-air-entrained concrete, the use of air-entrained concrete should be beneficial.

2. The air content of the concrete should be within the specification limits to insure both satisfactory strength and resistance to de-icers. To maintain a uniform air content, it is essential that fluctuation in the water-cement ratio and slump be minimized.

3. The spacing between the top reinforcing bars should be as large as sound design practices allow to help reduce the tendency for development of a plane of weakness. The support given to the top layer of concrete by the rigidly supported steel should vary inversely with the distance between bars. A greater bar spacing should require use of larger diameter bars. If the same depth of rust occurred on these larger bars as occurred on smaller bars, a greater area of effective steel would remain in the larger bars. This is desirable from the viewpoint of maintaining the design safety factor.
4. The finishing procedures should be so timed that the concrete is subjected to compaction at intervals throughout the bleeding or subsidence period. This will insure that the bleeding water rises to the surface of the deck and does not accumulate in the plane of the top reinforcing steel.

   The intent of the preceding is to regulate the timing of the finishing operations and not to increase the amount of finishing, as an increased amount of finishing could seriously reduce the percentage of entrained air in the surface of the deck.
5. None of the materials used (which met our specifications) were the primary cause of fracture plane deterioration. However, there may have been characteristics relative to some materials which resulted in other changes which may

have affected fracture plane deterioration. For example, consider crushed limestone versus gravel as a coarse aggregate; to obtain the same workability in concrete made with gravel coarse aggregate and with limestone coarse aggregate, slightly more water is required for the limestone mixtures; thus, somewhat more bleeding would be expected in concrete made with limestone coarse aggregate; bleeding is postulated to be an element involved in fracture plane formation. Another factor is the angularity of the aggregates used; this angularity may (through aggregate interlock) in some way be associated with the formation of a plane of weakness (bridging action). The use of a rounded coarse aggregate might be beneficial particularly with non-air-entrained concrete. The effect of the angularity of coarse aggregate would be expected, however, to be minor in our presently specified air-entrained concrete, and consequently a premium payment for rounded aggregate would not be expected to be warranted.

6. The minimum depth of cover over the top reinforcing steel should be approximately 1.5 inches. To attain this minimum depth of cover, the required specified depth is indicated to be from 2" to 2-3/8".

7. The reasons for using continuous span design in preference to simple span design should be re-evaluated in view of the probability that more fracture plane deterioration occurs

in the decks of continuous-design bridges. In the preceding statement continuous-design refers to the end supports for the deck, however, continuous-design can also occur in the slab of a bridge with simple end supports and this might also contribute to fracture plane deterioration. Continuous design is confounded with other factors in the occurrence of fracture plane deterioration and the weight of its influence is not known. At any rate, because of the association between fracture plane and cracking, deck designs which contribute to increased cracking in the surface of the deck should be considered as detrimental to the service life of the concrete.

8. To prevent the formation of plastic and drying shrinkage cracks in the surface of bridge decks, extreme care should be exercised to avoid delaying the start of wet curing and to avoid the sudden removal or stoppage of wet curing which (the latter) can cause extreme moisture gradients within the slab.

Some factors affecting the occurrence of surface mortar deterioration are also subject to improved control. They include particularly the water-cement ratio and the air content of the concrete. Data presented in this report strongly indicate that surface mortar deterioration could be largely eliminated by using low-slump, properly air entrained concrete. These data indicate that the minimum air content should definitely not be less than 3 percent and preferably not less than 4 percent (for normal weight concrete). The maximum amount of entrained air must also be closely controlled to avoid excessive loss of strength. Our current specifications cover the desired range of air content, but tests on samples of hardened concrete indicate that a much larger than expected percentage of this concrete has air contents outside the desired limits.

Wet curing for the full period specified is also highly desirable to help reduce surface mortar deterioration. The effect of moisture is to promote hydration of the cement, and the more cement hydrated in a concrete specimen, the less susceptible the concrete will be to deterioration.

From the preceding it can be seen that development of fracture plane and surface mortar deterioration should be reduced in future bridge decks by increased attention to contributing factors. To prevent or reduce the development of these types of deterioration on bridge decks already built, it would appear desirable to take steps designed to prevent the de-icing salts from coming into contact with the concrete. Experience has shown that some widely used maintenance measures have failed to keep the de-icers away from the concrete. In fact, some measures have tended to increase the concentration of salts in contact with the concrete. This happens when salt solutions penetrate a poorly bonded asphaltic overlay at joints and reflection cracks, and the salts accumulate underneath. These salts can react on the concrete the year around if they are not washed away. The following examples of resurfacing procedures reportedly used by two other agencies show the attention to construction detail needed to obtain sound bond between the concrete deck and the asphaltic overlay for thorough waterproofing of the deck.

The state of New Hampshire treats their bridge decks with a membrane waterproofing treatment covered by a 2-inch asphaltic concrete wearing surface. The membrane consists of a primer of "steep-roof pitch" having a melting point between 180°F to 200°F on which is layed two woven-glass fabrics each covered by a mopping of asphalt. The wearing surface consists of two one-inch layers of dense asphaltic mix. The first layer is applied by hand to avoid tearing the woven fabric, and the second is machine layed. Curbs are of granite stone and sidewalks are given a thin asphaltic wearing course. The joint between the roadway and curb is sealed by running the fabric up the side of the curb for the thickness of the asphalt surface and constructing a fillet of asphalt and asbestos flashing cement.

The New York State Throughway Authority uses a cutback tar as a primer on portland cement concrete, and a thin coating of a 3:2 mixture of coal tar pitch emulsion and medium gradation silica sand as a seal or waterproof coating beneath the asphaltic concrete resurfacing. When the asphaltic resurfacing has weathered through one winter, it is covered with the emulsion sealing material plus an addition of sand which is "rained on" the surface to provide skid resistance.

These agencies report very satisfactory service records on bridges given these treatments.

SUMMARY OF INDICATIONS

A summary of the findings or indications from each of the sections of this report follows.

Description of Deterioration

The four types of defects described are fracture plane, pot hole, surface mortar deterioration, and cracking. Of these defects, fracture plane and surface mortar deterioration are deemed to be the ones most critical in Missouri. Pot holes are a result of fracture plane development, and cracking is included because of its association with the occurrence of fracture plane.

Phase A - Results of Surveys of the Deterioration on Decks of In-Service Bridges

1. Although no effort was made to obtain a random sample of all the bridges in the state, there appears to be no reason to suspect that the bridges surveyed were not a representative sample of the bridges available for survey.

2. The data in Table A-8 in the report show that 1.5 percent and 47 percent of all 10-foot subsections surveyed exhibited areas of fracture plane and surface mortar deterioration respectively. In most of the subsections with reported deterioration, the entire subsection was not affected, therefore, the above percents must be considered somewhat high. It is estimated that the actual percent of total bridge deck surface affected by fracture plane is approximately one percent or less, and by surface mortar deterioration is approximately 10 to 20 percent.

3. Despite the large number of subsections surveyed, there were so many variables involved that, when the sample was broken down (stratified) to study the effect of each variable, the stratified sample was too small. Consequently, variables had to be combined to increase the sample size; this resulted in a confounding effect from other variables. However, even with this confounding effect some trends were indicated. These trends are as follows:

   a. Cracking was most severe on continuous spans, and most of it apparently developed during the first five years of the life of the bridge,

   b. The occurrence of fracture plane deterioration was significantly greater in subsections exhibiting cracking,

   c. The occurrence of fracture plane deterioration was greatest in areas where the greatest amount of de-icing salt was used,

   d. The occurrence of pot holes (by definition an advanced stage of fracture plane deterioration) was, despite an indicated disagreement between the percent of subsections affected by pot holes and fracture plane, related to factors affecting the occurrence of fracture plane.

   e. The occurrence of surface mortar deterioration was related to frost action in that its prevalence was greatest in the Northern area, intermediate in the Central area, and least in the Southern area,

f. Although the occurrence of surface mortar deterioration is apparently not directly related to the amount of de-icing salts used, the extensive development of this defect following extensive use of these salts strongly indicates a significant cause and effect relationship,

g. Use of air-entrained concrete decreased but did not eliminate the occurrence of surface mortar deterioration.

Phase B - A Study of the Effect of Construction Procedures and Practices Upon the Deterioration of the Concrete in Bridge Decks

1. The first part of this study was to observe and test the concrete being used in various bridge decks and to determine the extent of variation in the quality of this concrete and in the placement, finishing, and curing procedures. A total of 11 bridge deck pours was involved in this first part. This part has been concluded and the results show that:

   a. 14.1 percent of all slumps exceeded the specified maximum of 5 inches,

   b. 8.5 percent of the air contents were less than the specified minimum amount of 3 percent, and 3.4 percent were greater than the specified maximum of 6 percent,

   c. The elapsed time between the start of mixing and placement on deck of some of the batches exceeded the specified time of 90 minutes,

d. The elapsed time between placement of some of the contiguous portions of concrete was greater than the specified time limit of 45 minutes,

e. Considerable variation was noted in the time of application and the duration of wet curing,

f. Some concrete was placed in pockets of water on the deck forms, and

g. There was a variation in the number of ties used to secure the steel, and in the spacing of the reinforcing steel.

NOTE: Control of construction procedures has since been intensified, and revision of some specifications has been made (see either page 4 or 52 for specifics).

2. The second part of Phase B is to determine the effect of the above noted variations upon the development of defects. This part of the investigation is still in progress, and to date the only defect noted is cracking.

<u>Phace C</u> - <u>A Study to Determine if an Association Exists Between the Occurrence of Fracture Plane Deterioration, the Occurrence of Rust on Reinforcing Steel, and the Depth of Cover over the Top Reinforcing Steel</u>.

1. The possible occurrence of rust on the top reinforcing steel and the occurrence of fracture plane deterioration is shown to be significantly greater when the depth of the concrete over the steel is 1-1/2 inches or less.

2. Although there is a significant relationship between the occurrence of fracture plane deterioration and the occurrence of rust on the reinforcing steel, no positive evidence was obtained to indicate which of these types of deterioration occurs first.

<u>Phase D</u> - <u>A Study to Determine the Physical Differences Between Concrete in Deteriorated and Undeteriorated Areas</u>.

1. Examination of concrete samples from and adjacent to areas affected by fracture plane shows that:
   a. The average properties of these two groups of concretes are not radically different, and for both they are within specification requirements,
   b. The quality of the concrete above a fracture plane, however, is indicated to be slightly better than that immediately below the plane,
   c. The higher porosity of the concrete immediately below the fracture plane clearly shows that a plane of weakness or a plane of high water gain is built into bridge decks. This built-in plane of weakness is believed to be caused, during the bleeding or subsidence period, by support given to the upper layer of concrete by the rigidly supported mat of top reinforcing bars. These data support the theory that bleeding water is prevented, by the above supportive action, from rising to the surface of the deck, and that it tends to collect in voids in the plane of the top reinforcing bars,

d. The difference in the porosity of the cement paste in the sections of cores immediately above and immediately below a fracture plane is indicated to increase at the rate of 2.25 percent for each 0.10 increase in water-cement ratio (by volume). It is indicated, therefore, that the probability of the occurrence of a "built-in" plane of weakness and/or a fracture plane is directly related to the water-cement ratio of the concrete.

e. Factors other than water-cement ratio that might be expected to contribute to the development of this "built-in" plane of weakness or high water gain are:

(i) Size and spacing of the top reinforcing bars,

(ii) Maximum size and angularity of the coarse aggregate,

(iii) Depth of the concrete beneath the top reinforcing bars, and

(iiii) The extent to which the concrete is kept compacted during the bleeding or subsidence period.

2. Examination of concrete samples from and adjacent to areas affected by surface mortar deterioration shows that the average properties of these two groups of concrete are appreciably different, with a fairly high probability that the concrete in areas of surface mortar deterioration fails to meet the specification requirements for water or air

content or both. This indicates a relationship between the development of surface mortar deterioration and the use of too much water, too little air, or both.

Hypothesis on the Mechanics of the Formation of Fracture Plane Deterioration

This hypothesis will not be summarized here in every detail. Generally it postulates that a plane of weakness is built into the bridge deck because of the use of concrete having too high a slump, the improper placement of the concrete, and the improper timing of the finishing operations. Then, the built-in plane of weakness is changed into a fracture plane through the cumulative effect of several factors, such as the penetration of water and salt solution into the built-in plane of weakness, freezing temperatures, traffic, flexing of the deck, etc.

Measures to Prevent or Reduce Deterioration in New and Old Bridge Decks

Surface mortar deterioration and fracture plane areas are the two types of deterioration considered most serious in Missouri. Other types of deterioration occur, but they are considered detrimental mainly because of their effect on the occurrence of these two types.

The suggested procedures for preventing these two types of deterioration in new bridge decks are as follows:

1. Use and properly place a low slump, adequately air-entrained concrete.

2. Use as large a spacing between the top reinforcing bars as sound design practices allow to reduce the tendency for development of planes of weakness.
3. Obtain maximum density of the concrete by proper consolidation during the finishing operations.
4. Provide wet curing for the full period required by the specifications, and avoid sudden removal or stoppage of wet curing to avoid extreme moisture gradients within the slab.
5. Insure that the minimum depth of cover over the top reinforcing bars is 1-1/2".
6. Adjust the bridge design to decrease the amount of cracking in the surface of the deck, and possibly to decrease excessive vibrational stresses.

The prevention of deterioration in older bridge decks requires that de-icing salts be kept from coming in contact with the concrete. This means that the deck must be sealed or resurfaced. A problem encountered in applying resurfacing material is in the difficulty of securing bond between the concrete and the resurfacing material. If there is no bond, salts can accumulate between the concrete deck and the resurfacing material. There, the salts are not likely to be washed away by rain because of the protective overlay and can react with the concrete the year around.

A description of the construction details of waterproofing methods reportedly found to be effective by two other agencies is presented in this section.