DURABILITY OF BRIDGE CONCRETE
I 75 - US 23 Over the Flint River (B1 of 25-7-3)

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In response to a letter from H. J. Rathfoot, Chief Maintenance Engineer, on August 6, 1959, expressing concern about severe scaling on bridges in the area of Midland and Flint after only two seasons of winter maintenance treatment, a party of Office and Division heads visited the projects concerned on August 26. Subsequently, Howard E. Hill, Managing Director, requested an investigation of one of the bridges, B1 of 25-7-3, a divided structure on I 75 - US 23 over the Flint River. The Research Laboratory Division started its study with a condition survey in November 1959. Additional observations were conducted in the following months, with a final condition survey in August 1960.

This bridge was constructed in 1957 by C. A. Hull Co., Inc., of Bloomfield Hills, Mich. Concrete for the piers and abutments was poured between March 14 and September 12, and the superstructure concrete between October 8 and December 3.

FIELD OBSERVATIONS

The plan view of the superstructure in Fig. 1 shows the deterioration observed in the 1959 condition survey, along with further scaling, cracking, and spalling through the survey of August 1960. In the following discussion of bridge concrete condition, the superstructure and substructure are each considered in turn, with special attention to such subjects as scaling, cracking, and popouts.

Superstructure Concrete

Scaling and Patching. The extent of scaling for the various pours is summarized in Table 1. On the northbound roadway, the surfaces of five of the six spans have scaled (Figs. 2-4), generally in limited areas near the curbs. Some additional scaling developed in the 1959-60 winter, but the original scaling does not appear to be progressive--most seems to have occurred in the first months after the structure was completed.

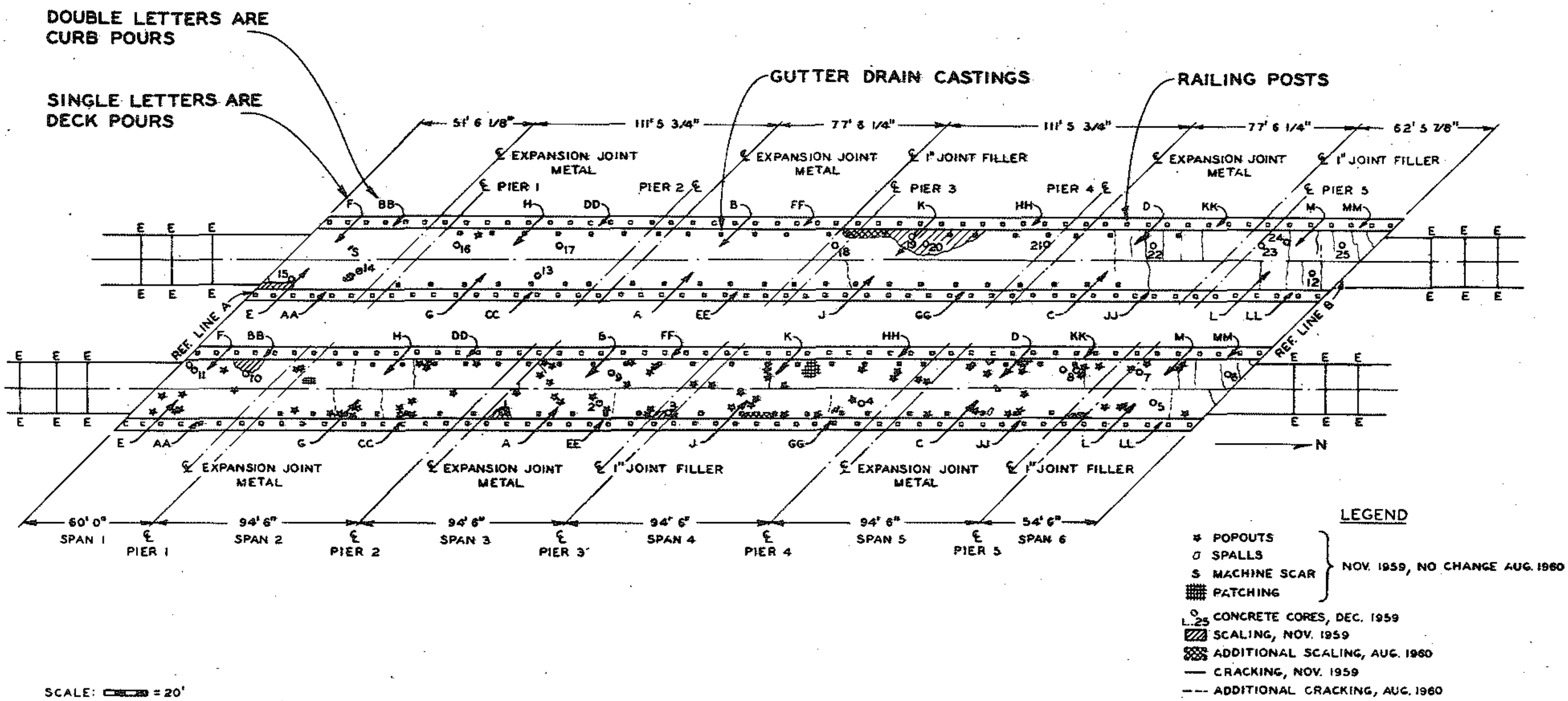


Figure 1. Plan view and condition survey of superstructure of B1 of 25-7-3.

Some concrete patching or concrete mortar spillage appears on the deck surface of the northbound roadway. Fig. 5 shows that some of this material has broken away from the underlying concrete.

TABLE 1
BRIDGE DECK SCALING
(Parenthesized letters indicate deck pours)

Span	Northbound Roadway				Southbound Roadway			
	Traffic Lane Area Scaled		Passing Lane Area Scaled		Traffic Lane Area Scaled		Passing Lane Area Scaled	
	percent	sq ft	percent	sq ft	percent	sq ft	percent	sq ft
1	0.0	0	7.8	70(F)	0.0	0	7.8	70(E)
2	0.0	0	0.0	0	0.0	0	0.0	0
3	6.4	90(A)	0.0	0	1.4	20(K)*	0.0	0
4	2.8	40(J)	0.0	0	29.6	420(K)	0.0	0
5	0.0	0	0.7	10(D)	0.0	0	0.0	0
6	3.7	30(L)	0.0	0	0.0	0	0.0	0
TOTAL	2.2**	160	1.1**	80	6.0**	440	0.9**	70

* The 20 sq ft of scaled concrete on Span 3 (Southbound roadway) is part of Pour K, which is primarily Span 4.
** Percentage values for entire roadway lane area.

On the southbound roadway, some scale appears on Span 1 (Fig. 6), and the most extensive and severe scaling on the project is found in the traffic lane of Span 4 (Pour K). Although scaling here is deeper than at any other point on the bridge, penetration is not so severe that any large coarse aggregate has been dislodged (Fig. 7). There appears to be no prospect of any substantial future increase in depth of scaling on Pour K.

Eight railing posts on Span 4 of the northbound roadway, all on the traffic lane side, have fairly extensive pitting and scaling on their approach and traffic faces, first noted in the inspection of November 1959 (Fig. 8). In addition, similar scaling has developed since then on two posts on Span 1 of the southbound roadway (Fig. 8). Some other posts have small areas of scaling, but these are less extensive than on the ten posts mentioned. Fig. 9 illustrates the marked difference in the separate faces of deteriorated railing posts, with most marking on the traffic and approach faces.

Cracking. The most extensive cracking of the bridge deck has occurred on Spans 5 and 6 of the southbound roadway and Span 6 of the northbound roadway (Fig. 1). In all three areas, there are six or more cracks per span (Fig. 10). Winter maintenance chemicals entering these cracks are passing completely through the deck, in large enough quantities to stain the deck bottom surface.

Popouts. The northbound roadway has numerous large popouts of the deck, curbs, and railing posts of all six spans (Fig. 11). The southbound roadway, however, is relatively free of popouts. This difference in frequency and its relation to materials changes is discussed later.

Substructure Concrete

The only scaling found on the substructure concrete surfaces is between the roadways on the south parapet wall (Fig. 12). This scale seems to have resulted from splashing of chemical-laden slush and snow from the decks above.

Water leakage through the north abutment wall has left extensive stains on the inside face. This leakage is occurring along vertical cracks in the stub abutment and through the bridge seat construction joint between the back wall and the abutment pour (Fig. 13). The latter leakage may be the result of failure of joint waterproofing material at certain points along this joint.

Another minor imperfection noted in the substructure was extensive, fine hairline cracking (Fig. 14). This deterioration was most pronounced on the Pier 4 pedestal, but was also noted on some columns of Pier 1. This type of cracking has also been noted on other structures, particularly on pier columns, and does not appear to be a serious defect.

A large concrete spall was found on the deck undersurface at the expansion joint at the south end of Span 5 on the southbound roadway (Fig. 15).

MATERIALS AND CONSTRUCTION

The 59 days of concrete pouring for the substructure occurred between March 14 and September 12, 1957. The cements were Aetna and Peerless

TABLE 2
SUPERSTRUCTURE CONCRETE POURS

Concrete Pour				Coarse Aggregate*	Field Air Content, percent	Temperature, deg F					Remarks
						At Time of Pour		Air for 7 Days After Pour			
Date	Designation	Roadway	Location			Mix	Air	Max.	Min.	Avg	
10-8-57	B, F	NB	Spans 3 & 1	Groveland 6A	5.2	72	70	69	23	46	
10-9-57	A, E, H,	NB	Spans 3, 1, & 2	Groveland 6A	5.0	68-76	62-68	69	23	46	
10-10-57	D, G	NB	Spans 5 & 2	Groveland 6A	6.0	64-66	56-58	69	23	48	
10-11-57	C, K, AA, BB, DD	NB	Spans 5, 4, & curbs	Groveland 6A	5.9	60-66	46-53	69	23	48	
10-14-57	J, CC, EE, FF	NB	Span 4 & curbs	Groveland 6A	---	64-70	56-62	69	32	49	
10-15-57	M, HH	NB	Span 6 & curbs	Groveland 6A	6.4	66	62	62	32	50	
10-28-57	L, KK, MM	NB	Span 6, Curbs, & Railing Posts	Green Oak 6B**	---	68-74	48-52	60	32	46	Mix water heated
10-29-57	GG, JJ, LL	NB	Curbs	Green Oak 6B**	6.0	68	54	60	33	45	
11-1-57			Railing posts and south parapet wall	Green Oak 6B**	---	68	56	57	27	42	
11-4-57	A, E	SB	Spans 3, 1, & railing posts	Green Oak 6B**	6.5	60-64	44-50	52	22	36	Mix water heated. Straw-burlap curing.
11-5-57	B, G, F	SB	Spans 3, 2, & 1	Green Oak 6B**	6.4	60-58	40-46	53	22	37	Mix water heated. Straw-burlap curing. Slump: 3-3/4 in.
11-6-57	C, H, AA	SB	Spans 5, 2, & curbs	Killins 6B	5.6	60-58	46-50	53	22	37	Mix water heated.
11-7-57	CC, EE	SB	Curbs & railing posts	Killins 6B	---	58	40	57	22	39	Mix water heated.
11-11-57	D, J	SB	Spans 5 & 4	Killins 6B	---	56-63	36-50	59	30	45	Mix water heated.
11-12-57	BB, DD, FF	SB	Curbs	Killins 6B	5.6	72-68	56-48	59	32	46	Aggregates & water heated. Straw-canvas curing.
11-15-57	L	SB	Span 6 & railing posts	Green Oak 6B**	---	72-68	45-49	59	25	39	Mix water heated.
11-18-57	K	SB	Span 4	Groveland 6A	4.9	65-68	38-40	58	17	32	Aggregates & water heated. Burlap-canvas-straw curing. Slump: about 5 in. Heavy rain during most of pour.
11-20-57	M, GG, JJ	SB	Span 6, curbs, & railing posts	Green Oak 6B**	---	76-84	32-38	57	17	31	Water heated with 2 lb $CaCl_2$ per sack. Straw curing. Slump: 4 in.
11-25-57	LL, MM, KK, HH	SB	Curbs and railing posts	Green Oak 6B**	---	58-59	34	57	16	33	Burlap-2 ft straw-canvas curing.
11-27-57			Railing posts and north parapet wall	Green Oak 6B**	---	72	55	53	15	30	
12-3-57			Railing posts and north parapet wall	Green Oak 6B**	---	70	34	51	13	28	

* Throughout superstructure the cement was Aetna Type 1A, sand was Groveland 2NS.
** American Aggregates Green Oak pit.
NB = Northbound
SB = Southbound

type 1A; the fine aggregates were from A. S. Leffler Gravel Co., Groveland Gravel Co., and Holly Sand and Gravel Co.; the coarse aggregate was Groveland 6A, American Aggregates Green Oak 6A, and Drummond Dolomite 6B; and the concrete was from the Catsman transit-mix plant and the contractor's batch plant.

The 21 days of concrete pouring for the superstructure are itemized in Table 2, which gives locations, materials information, field air checks, and temperatures. All superstructure concrete was obtained from the Catsman transit-mix plant.

Laboratory Tests of Deck Cores

In connection with the scaling of the bridge deck, it should be noted in Table 2 that all field checks of air percentage of the fresh concrete showed satisfactory levels. In Table 3, air and cement contents are

TABLE 3
AIR AND CEMENT CONTENT OF BRIDGE CORES

	Core No.	Pour Designation	Core Location		Condition of Surface	Air Content, percent[2]		Coarse Aggregate	Cement Content, sacks/cu yd	
			Span	Lane[1]		Field Tests	Laboratory Tests		Design	Lab. Analysis *
Northbound Roadway	1	A	3	TL	scaled	5.0	4.7	Groveland 6A	5.5	5.8 (1, 3)
	2	A	3	TL	unscaled		4.1	Groveland 6A	5.5	5.3 (2, 4)
	3	J	4	TL	scaled	---	5.4	Groveland 6A	5.5	
	4	J	4	TL	unscaled		4.2	Groveland 6A	5.5	
	5	L	6	TL	unscaled	---	5.3	Green Oak 6B	5.9	
	6	M	6	PL	cracked area	6.4	2.1	Groveland 6A	5.5	
	7	M	6	PL	uncracked		3.5	Groveland 6A	5.5	
	8	D	5	PL	cracked	6.0	3.6	Groveland 6A	5.5	
	9	B	3	PL	unscaled		2.2	Groveland 6A	5.5	
	10	F	1	PL	scaled	5.2	3.4	Groveland 6A	5.5	
	11	F	1	PL	unscaled		2.7	Groveland 6A	5.5	
Southbound Roadway	12	L	6	PL	cracked	---	2.9	Green Oak 6B	5.9	
	13	G	2	PL	unscaled	6.4	4.3	Green Oak 6B	5.9	
	14	E	1	PL	unscaled	6.5	3.8	Green Oak 6B	5.9	5.4
	15	E	1	PL	scaled		4.5	Green Oak 6B	5.9	6.4
	16	H	2	TL	unscaled	5.6	3.7	Killins 6B	5.9	
	17	H	2	TL	unscaled		3.8	Killins 6B	5.9	
	18	K	4	TL	unscaled		4.9 5.7**	Groveland 6A	5.9	7.0 (18, 21)
	19	K	4	TL	scaled	4.9	2.9 5.4	Groveland 6A	5.9	5.5
	20	K	4	TL	scaled		3.1 6.3	Groveland 6A	5.9	
	21	K	4	TL	unscaled		3.3 5.1	Groveland 6A	5.9	
	22	D	5	TL	cracked area	---	3.6	Groveland 6A	5.9	4.8
	23	M	6	TL	uncracked		---	Green Oak 6B	5.9	
	24	M	6	TL	cracked	---	2.9	Green Oak 6B	5.9	5.5
	25	M	6	TL	cracked		3.9	Green Oak 6B	5.9	

[1] TL=Traffic Lane; PL=Passing Lane
[2] Field checks are an average of three locations in a day's pour; core locations are not necessarily from the same areas.
* Numbers in parentheses or brackets indicate cores combined for cement determinations; pulverized cores were analyzed according to ASTM Method C85-54.
** Second set of values for Cores 18-21 were obtained on slices 1-1/2 in. from top surface; all other values were 3/4 in. from top surface.

shown for 25 deck cores taken on December 9, 1959, from points selected to include scaled, unscaled, and cracked areas. Air content was determined on the concrete cores at a level 3/4-in. from the deck surface by the linear traverse method, and averaged 4.1 percent for the scaled areas as compared to 3.7 percent for the unscaled areas. The air content of the concrete at 3/4-in., even in the scaled areas, appears to indicate that the concrete is basically satisfactory, and that the scaling is a surface phenomenon which probably will not progress much deeper. This confirms the Laboratory's observation in November 1959, that the light scaling which had occurred at an early age should not become progressively more serious. It is true that some new areas of shallow scaling developed during the third winter (1959-60), but the older areas have not deteriorated appreciably. Since the concrete below the surface in the scaled areas has an air content consistent with good durability, it appears that the lack of surface durability at some locations is probably associated with overworking, overfinishing, and/or wet mixes.

In the area of most extensive and severe scaling, Pour K in the traffic lane of the southbound roadway on Span 4, the cores were checked for air content both at the 3/4-in. level and at 1-1/2-in. by the linear traverse method. It was known that heavy rain fell as this pour was in progress and the double check of the Pour K cores was intended to determine if air content close to the surface had been specially affected by this rain. At the 3/4-in. level, average air content was 3.6 percent, as compared to 5.6 percent at the 1-1/2-in. level. This indicates a loss of air near the surface, with satisfactory air content at the lower level. Scaling did not deepen significantly during the third winter. It appears that the poor surface condition on this span is due to the high fluidity of the concrete--estimated slump: 5 in. according to Testing and Research field reports--and to the heavy rain during pouring and finishing operations.

Comparison of the field and laboratory air determinations indicates that the field average was 5.6 and the laboratory average was 3.6. Statistical analysis of the data indicates no significant correlation between field air content of the fresh concrete and air content as determined by the linear traverse method from sample cores. The correlation coefficient was 0.13, with 0.48 or more required for significance. This is to be expected as the exact location of field checks is not known and cores were undoubtedly from different locations in the same pour.

Core cement contents in Table 3 were determined by grouping certain cores to get a sufficiently large sample for a silicon dioxide determination according to ASTM Method C85-54. Only one cement determination was low enough to be significant, 4.8 sacks per cu yd for Pour D of the southbound roadway. The difference between this value and the design cement content of 5.9 sacks per cu yd, is greater than the experimental error inherent in this determination, which is approximately 0.5 sack per cu yd.

Aggregates and Popout Frequency

Study of the materials used in the various pours (Table 2) helps explain why aggregate popouts are numerous in the northbound roadway and infrequent in the southbound roadway. The coarse aggregate on the northbound deck was Groveland 6A, except for Green Oak 6B on Pour L (Span 6). On the southbound, it was 6B, either from American Aggregates Green Oak or from Killins, with the exception of Pour K, Span 4--the worst scaled area--where Groveland 6A was used. The difference between allowable percentages of deleterious materials in the 6A and 6B, according to MSHD specifications for gravel aggregate, is as follows:

	6A, percent	6B, percent
(1) Maximum soft particles	3	1
(2) Maximum chert	4	1
(3) Maximum hard absorbent particles	5	2
Maximum of (1) + (2) + (3)	9	3

The specification for 6A coarse aggregate permits a higher percent of poorer aggregate constitutents with results which are apparent in the much greater frequency of aggregate popouts on the northbound roadway. However, use of 6A aggregate was in accordance with the specifications for this bridge.

Temperature and Concrete Surface Deterioration

Scaling of particular railing posts cannot be explained specifically since construction records do not show which posts were constructed from a given pour. However, records do show that all railing posts were poured on eight days: October 29, November 4, 5, 7, 15, 20, 25, 27, and December 3. During the first three days after each of these pours, low air temperatures dropped to freezing or below. After each of the final four post-pour days, daily low temperatures dropped considerably below freezing--24, 24, 15, and 20 F, respectively--so that surface weakening of the concrete mortar might be expected, even when protected by construction forms.

Although records are not available giving concrete temperatures for these pours, there are certain data from which such temperatures may be projected. In 1958 the Laboratory determined concrete temperature within 3/4-in. of the surface of a bridge abutment protected by insulated forms. For three days after pouring, the temperature at this point, 3/4-in. beneath the concrete surface, averaged 9.5 deg above the average air temperature, which was 34 F. In the case of the Flint bridge posts, the average air and concrete temperatures would differ even less, because the heat of hydration would be much less in a small post than in an abutment, and the railing post forms were not insulated. Average daily air temperature for the final railing post pours was so low that a concrete surface temperature near freezing would be expected.

TABLE 4
MODULUS OF RUPTURE OF CONCRETE BEAMS
Based on Field Reports of Construction Personnel

Pour Date	Pour Location or Designation	Modulus of Rupture, psi*		Curing Method		Slump, in.
		7-Day	28-Day	Structure	Beams	
3-18-57	Footing Pier 4L	517	---	Wet burlap	Wet burlap	2-1/2
3-27-57	Pedestal Pier 4	508	641	Wet burlap	Wet burlap	2
4-19-57	Abutment A Wall	458	525	Wet burlap	Wet burlap	3
5-15-57	Pier Cap 1R	567	667	Wet burlap	Moist sand	3-1/2
6-4-57	Pedestal Pier 2R	575	675	Wet burlap	Moist sand	2-1/2
6-13-57	Pier Cap 1L	617	758	Wet burlap	Moist sand	2
8-14-57	Pier Cap 2L	550	650	Wet burlap	Wet burlap	3
8-23-57	Pedestal Pier 3R	608	725	Wet burlap	Moist sand	2
9-4-57	G & H Column Pier 3L	575	692	Wet burlap	Moist sand	2-1/2
10-11-57	Pours C & K--NB Roadway	609	---	Wet burlap	Moist sand	3
10-28-57	Pour L--NB Roadway	559	608	Wet burlap	Wet burlap	3
11-5-57	Pours B, G, & F--SB Roadway	550	625	Wet burlap, straw	Wet burlap, straw	3
11-11-57	Pours D & J--SB Roadway	513	624	Wet burlap, straw	Wet burlap, straw	3-1/2

* MSHD Specifications call for 550 psi at 7 days and 650 psi at 28 days

The hairline cracking, most prevalent on the Pier 4 pedestal wall (Fig. 14), also appears to be associated with cold temperature pouring. The Pier 4 pours were made on March 23, 27, and 28. All other exposed substructure concrete was poured after April 10, when temperatures were generally more moderate.

Field reports on modulus of rupture values for the concrete mixes also illustrate the problem of curing temperatures during the early substructure pours and the final superstructure pours. The first three sets of beams (Table 4) failed to meet the 7- or 28-day requirements for modulus of rupture. These beams appear to be representative of all those poured on or before April 19. From May 15 through October 11, every set of beams met specification values, but from October 28 no beams cast met both the 7- and 28-day requirements. Six superstructure pours were made prior to October 28 and 15 pours on or after that date.

SUMMARY

Laboratory tests on cores taken from the bridge deck indicate that lack of air content of the concrete does not explain the concrete surface problems on this bridge deck. Cement content of bridge cores also generally tested as satisfactory, with only one test out of eight significantly below the designed cement content. It appears that only the finished surface of the deck was of inferior quality, as illustrated by scaling at certain locations, and this may be due to overfinishing in these locations, or in the case of Span 4 (Pour K) of the southbound roadway, due to a combination of a wet mix and rain saturation during placing of the concrete. The difference in the performance of 6A and 6B coarse aggregate is well illustrated by the difference in frequency of popouts between the northbound and southbound roadways. Concrete containing 6B aggregate was significantly less liable to popouts. Unfavorable curing temperatures during most of the pouring of superstructure concrete may have had a very significant effect on surface durability of the deck and railing posts, in reducing resistance to scaling during the first winter.

Figure 2. Scaled deck pavement on northbound roadway -- passing lane at center of Span 1 (left) and traffic lane at south end of Span 3 (right). Sept. 1960.

Figure 3. Scaled deck pavement on Span 4, northbound roadway traffic lane -- south end (left), center (center) and north end (right). Sept. 1960.

Figure 4. Scaled deck pavement on northbound roadway--passing lane at center of Span 5 (left) and traffic lane at south end of Span 6 (right). Sept. 1960.

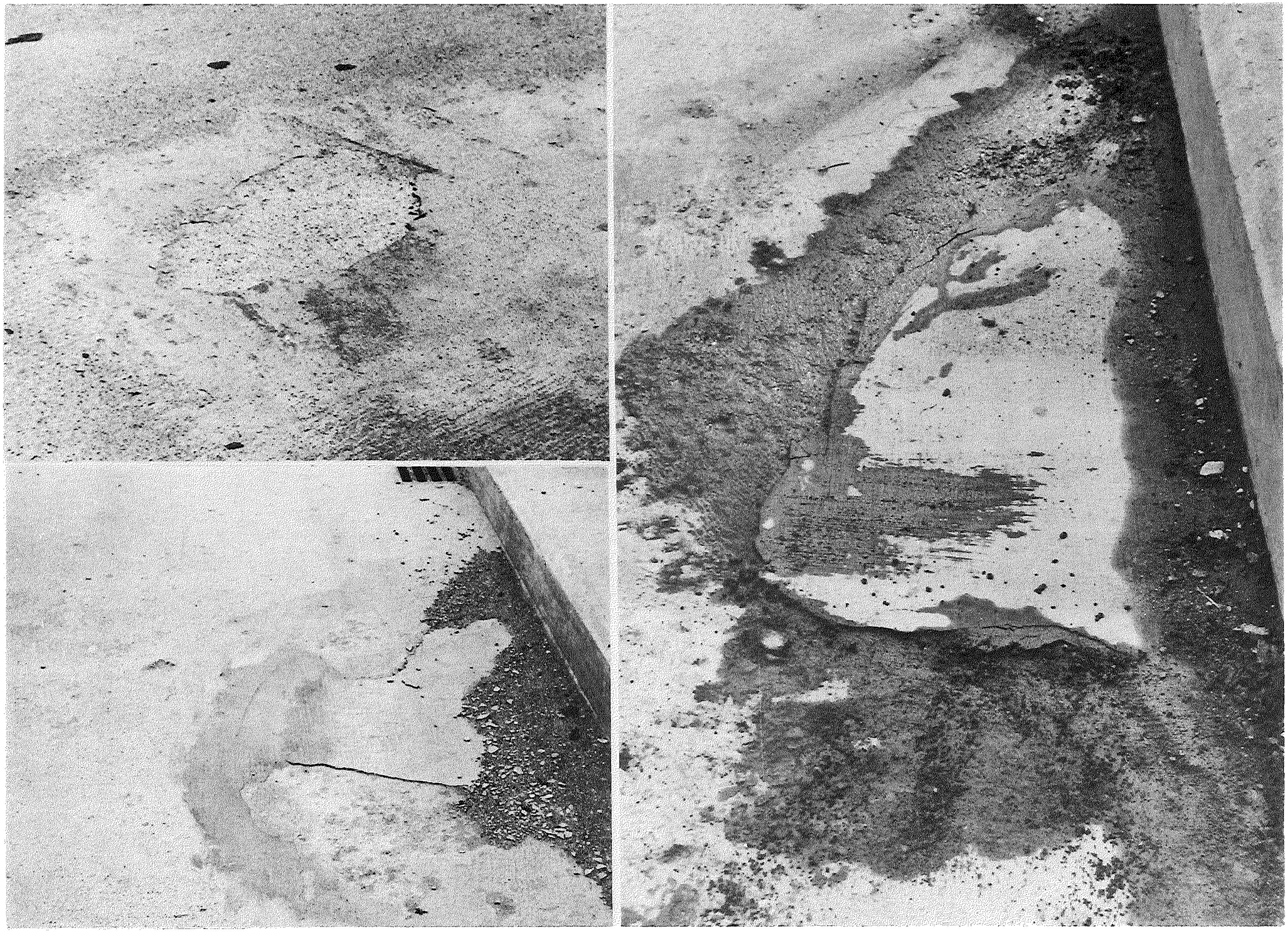


Figure 5. At three locations in the passing lane of the northbound roadway, local concrete patching or mortar spillage has broken away from the original pavement surface -- south end of Span 2 (top left --Sept. 1960), center of Span 4 (lower left -- Nov. 1959), and center of Span 5 (right -- Sept. 1960).

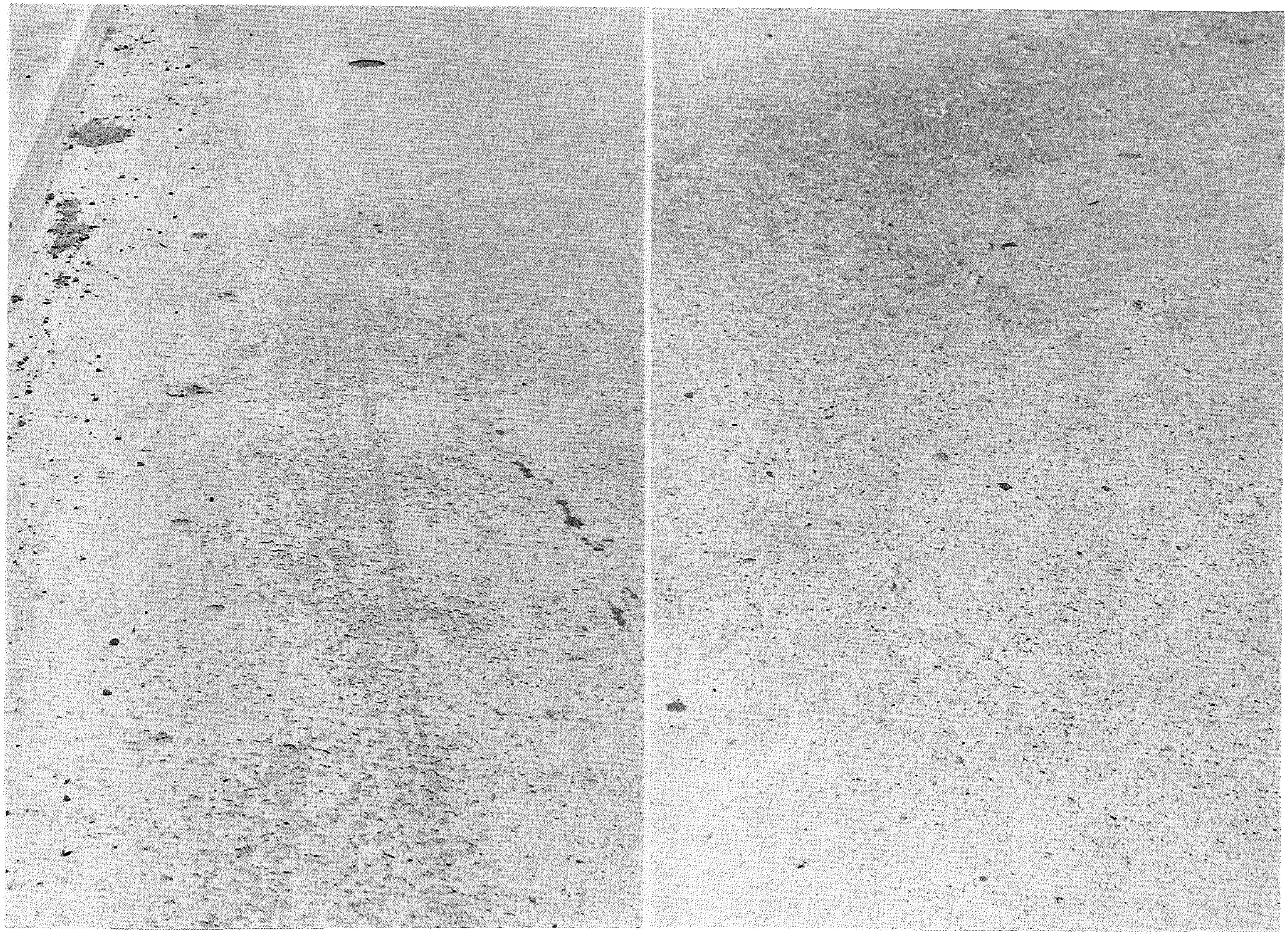
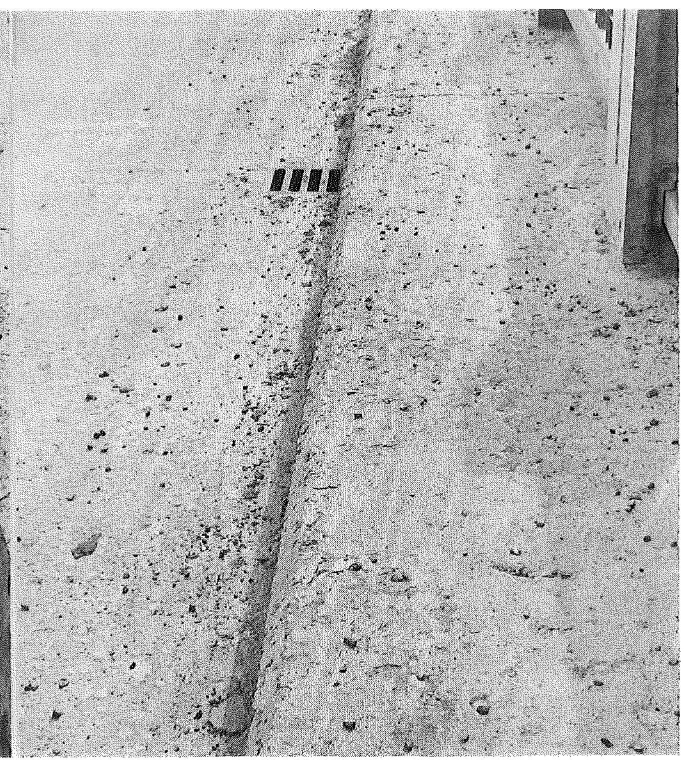


Figure 6. Scaled deck pavement on Span 1 southbound roadway passing lane -- south end (left) and center (right). Sept. 1960.

Center looking toward south

Center



North end

Figure 7. Scaled deck pavement and curb of Span 4 southbound roadway traffic lane--the region of most severe and extensive scaling on the project (Sept. 1960).

Figure 8. In November 1959, two of the posts on Span 4, northbound roadway (top), showed considerable exfoliation and scale from splashed winter maintenance chemicals; note snow or slush deposits remaining on post faces. In September 1960, two posts on Span 1, southbound roadway (bottom), show similar deterioration which developed during the 1959-1960 winter.

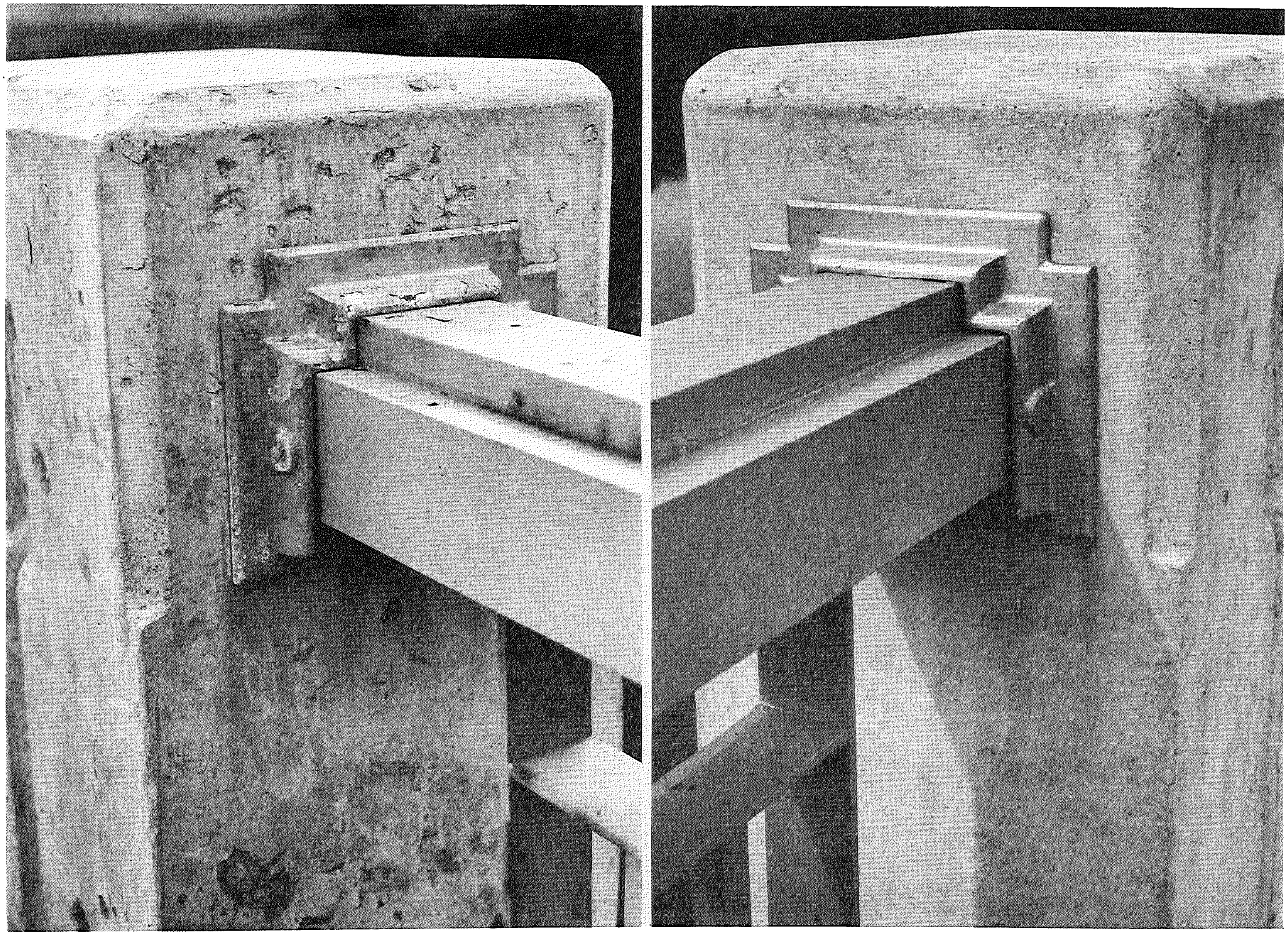


Figure 9. Marked difference in performance was noted on various faces of the railing posts. On a typical post the approach and traffic faces (left) have deteriorated, while the trailing face (right) has a clear, unmarked surface. Note railing corrosion on the approach side and uncorroded coating on the other side (Sept. 1960).

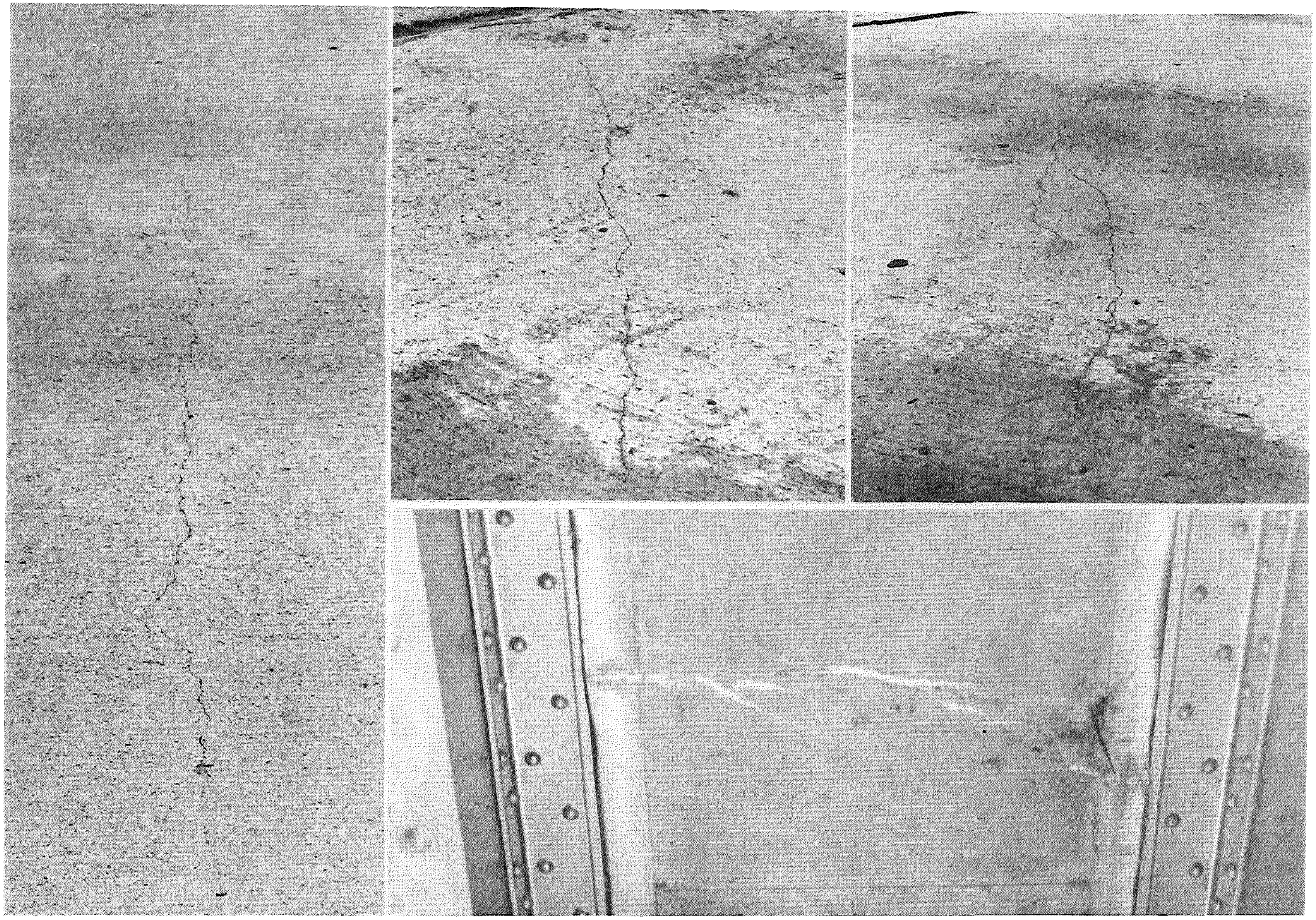


Figure 10. Cracking in the traffic lane of the southbound roadway. The deck undersurface (lower right) is marked by ice removal and chemicals leaching through deck cracks--center of Span 5 (left), north end of Span 6 (top center and right), and undersurface at north end of Span 6 (lower right). Nov. 1959.

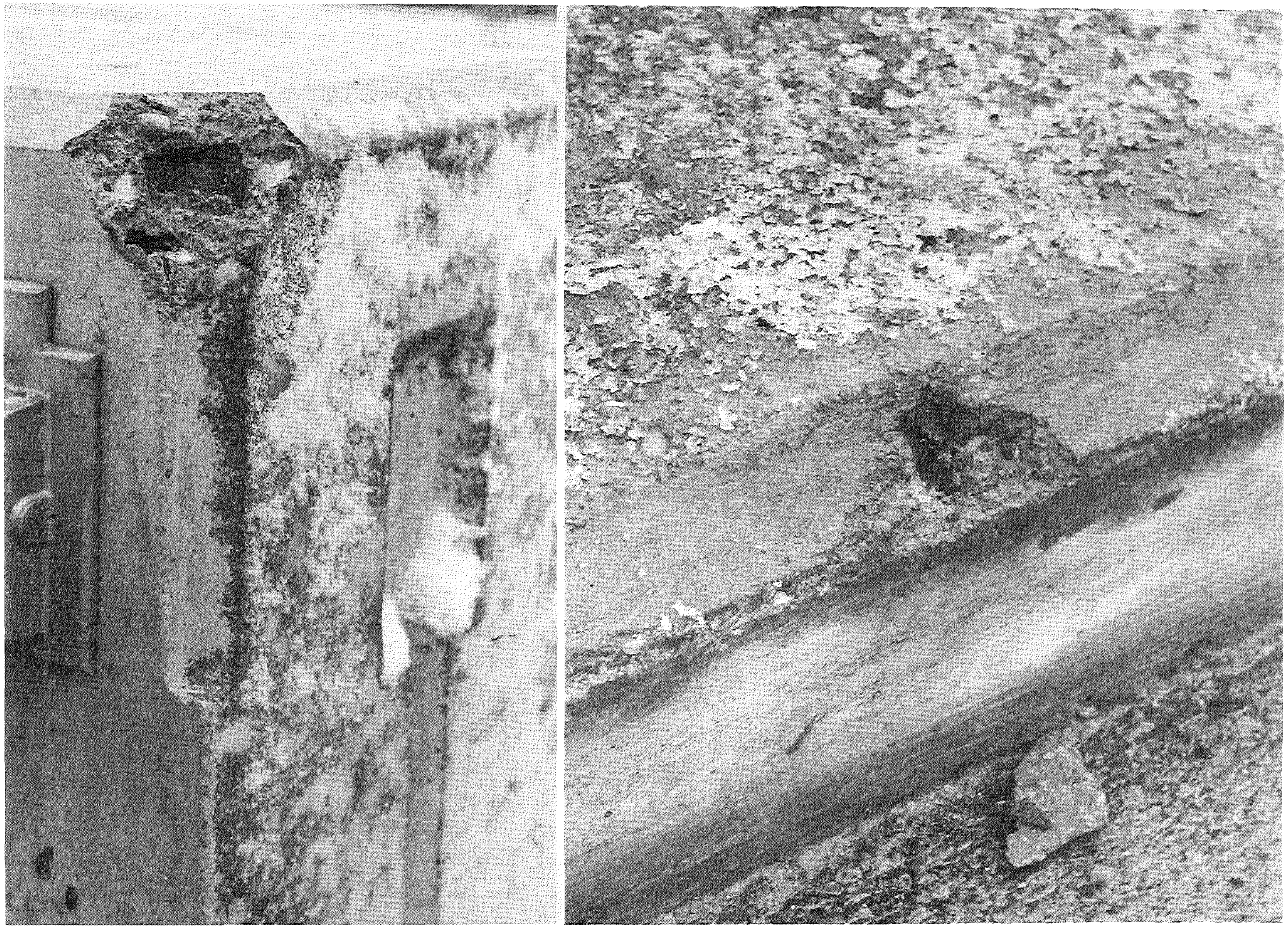


Figure 11. Aggregate popouts on a railing post (Span 2, southbound roadway) and on a curb edge (Span 3, northbound roadway). Nov. 1959.

Figure 12. Scaling or pitting similar to that on the railing posts has occurred on the south retaining wall between the separate roadways, probably due to splashing of chemical-laden slush and snow by passing vehicles and winter maintenance operations. Sept. 1960.

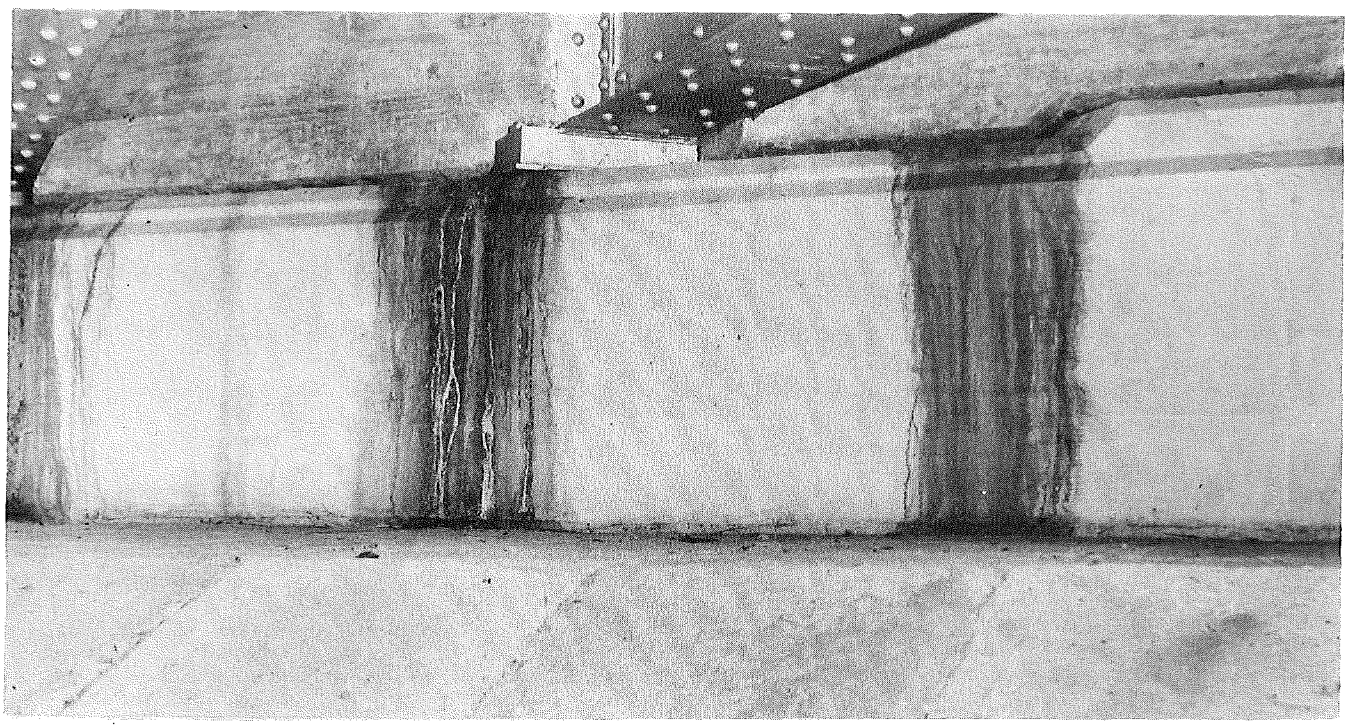


Figure 13. Water leakage has stained concrete on the inside face of the north abutment through cracks in the abutment wall and at various points along the bridge seat construction joint. Sept. 1960.

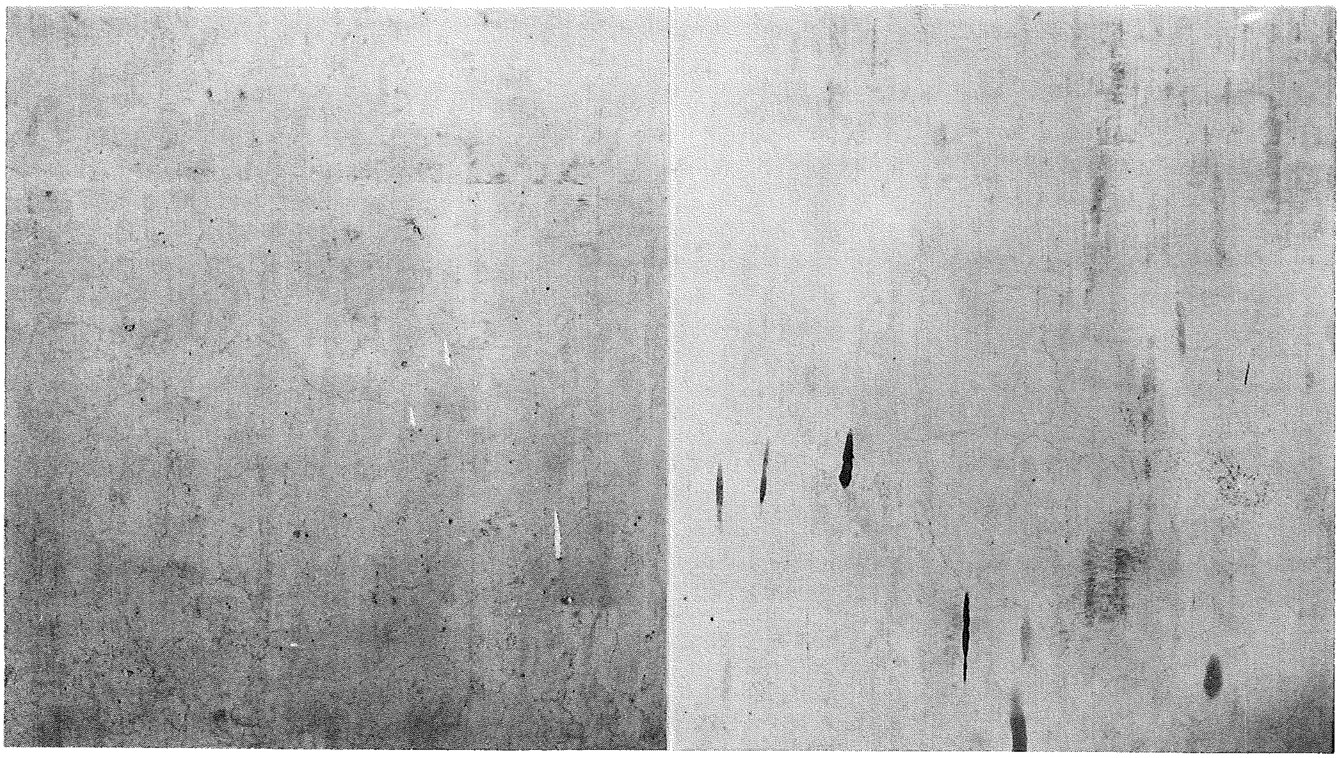


Figure 14. Hairline cracking of concrete surfaces on Pier 4 pedestal wall. Nov. 1959.

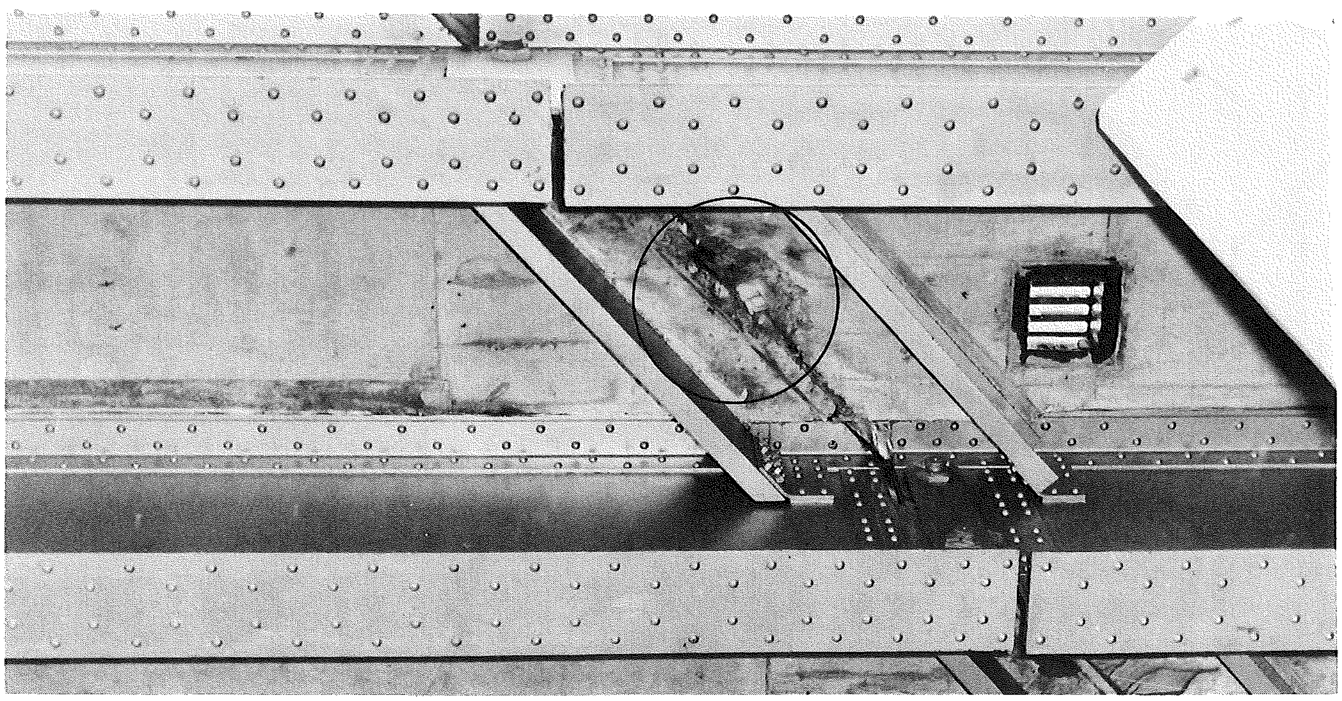


Figure 15. Concrete spall at expansion joint at the south end of Span 5 on the southbound roadway Nov. 1959.