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16. Abstract

A detailed examination was made of 34 bridge decks, 11 to 30 years old, containing uncoated reinforcing steel. These bridges are located throughout Virginia. Cores were taken from each to evaluate the quality of the concrete with the objective of determining the relationship of concrete properties with the long-term performance of such bridge decks. It was shown that the greatest deterioration in these decks results from the ingress of chloride ions into the concrete, thus confirming the need for concretes with low permeabilities to be used in bridge decks. Low permeability is especially important where uncoated reinforcing steel is present.

Some of the bridges examined in this study were constructed prior to 1966 when changes were made in the Virginia Department of Transportation's specifications. However, other than entrained air content, there is a relatively narrow range of measured quality parameters for these concretes, and most are considered to be of acceptable quality. Accordingly, specific numerical relationships between the concrete properties studied and the environmental and traffic conditions or the deterioration of the bridge decks were not established.

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FINAL REPORT

FACTORS AFFECTING THE PERFORMANCE OF BRIDGE DECKS IN VIRGINIA

Celik Ozyildirim
Research Scientist

and

Woodrow J. Halstead
Research Consultant

(The opinions, findings, and conclusions expressed in this report are those of the authors and not necessarily those of the sponsoring agencies.)

Virginia Transportation Research Council
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ABSTRACT

A detailed examination was made of 34 bridge decks, 11 to 30 years old, containing uncoated reinforcing steel. These bridges are located throughout Virginia. Cores were taken from each to evaluate the quality of the concrete with the objective of determining the relationship of concrete properties with the long-term performance of such bridge decks. It was shown that the greatest deterioration in these decks results from the ingress of chloride ions into the concrete, thus confirming the need for concretes with low permeabilities to be used in bridge decks. Low permeability is especially important where uncoated reinforcing steel is present.

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INTRODUCTION

The results of previous studies of the relationships between various properties of hardened concrete and the long-term performance of concrete bridge decks in Virginia showed a number of connections between concrete properties and performance under different environmental and traffic conditions.[1,2]

On the basis of in-depth studies of 17 randomly selected bridges throughout the state, Newlon and Walker suggested that a prediction of the long-term performance of the concrete could be based on petrographic examinations of the concrete combined with a knowledge of the service conditions.[2] Accordingly, this study was undertaken to determine whether the trends noted in the earlier studies could be verified and whether a numerical concrete performance index could be devised that would be useful for predicting performance under different environmental and loading conditions. Such an index would make a significant contribution to the overall determination of priorities for bridge maintenance and rehabilitation.

SCOPE

Thirty-four bridges were selected for this study. The selection among bridges built between 1966 and 1974 was on a random basis. However, to obtain as wide a spread in data as possible, some bridges built prior to 1966 were selected on the basis of visible evidence of deterioration. At the time the evaluations were made (1986 through 1988), the ages of these bridges ranged from 11 to 30 years. The reinforcing steel in these bridges was not coated with epoxy resins, nor were other protective compounds used; thus, potential deterioration from corrosion of the reinforcing bars was a factor in all cases. The identification of each of these bridges (including the year each was built) is given in Table 1. Some of these bridges were constructed prior to 1966 when the Virginia Department of Transportation's (VDOT) specifica-

TABLE 1

LOCATION AND DESCRIPTION OF THE TEST BRIDGES

Bridge No.	Year Built	Type[a]	Length[b] (ft)	City or County	Location
1	1971	SBM	60	Shenandoah	Rt. 211/Smith Creek(EBL)
2	1967	SBM	43	Wythe	Rt. 52/I-81(EBL)
3	1971	SBM	108*	Rockingham	Rt. 33/Shenandoah River(EBL)
4	1971	SPG	43*	Frederick	Rt. 37 Bypass/ Penn. RR(NBL)
5	1969	SBM	72	Page	Rt. 675/Shenandoah River(EBL)
6	1969	SBM	96*	Albermarle	I-64/Mechunk Creek(EBL)
7	1969	SBM	55*	Madison	Rt. 29/Robinson River(SBL)
8[c]	1961	PC	60	Wythe	Rt. 618/I-81(EBL)
9[c]	1961	PC	64	Smyth	Rt. 660/I-81(EBL)
10[c]	1962	SBM	66	Smyth	Rt. 730/I-81(EBL)
11	1969	SBM	59	Louisa	I-64/Rt. 615(WBL)
12	1970	SBM	46	Fluvanna	I-64/Rt. 799(EBL)
13	1969	SBM	80	Fluvanna	I-64/Buck Creek(EBL)
14	1970	SBM	60	Amherst	Rt. 29/RR(SBL)
15	1970	SBM	40	Amherst	Rt. 29/659(NBL)
16[c]	1961	SBM	53	Pulaski	Rt. 99/Peak Creek(NBL)
17[c]	1960	SBM	48	Pulaski	Rt. 99/N & W RR(NBL)
18[c]	1960	RC	45	Franklin	Rt. 220 Business/220(EBL)
19	1970	SBM	46*	Culpeper	Rt. 29 Bypass/Southern RR(SBL)
20	1974	SBM	65*	Fauquier	Rt. 29/Rappahannock(SBL)
21[c]	1962	SBM	57	Stafford	Rt. 628/I-95(WBL)
22	1971	SBM	112	Culpeper	Rt. 29/29 Bypass(SBL)
23	1969	PC	60	Clarke	Rt. 7/Opequon Creek(WBL)
24	1966	PC	62	Spotsylvania	Rt. 612/Ni River Reservoir(EBL)
25	1971	RC	37	Prince Edward	Rt. 460/Bush Creek(WBL)
26	1973	SPG	117	Cumberland	Rt. 45/James River(NBL)
27	1969	PC	65	Essex	Rt. 17/Mt. Landing Creek(NBL)
28	1970	SBM	40	Essex	Rt. 360/Piscataway Creek(EBL)
29	1969	SPG	86	Clifton Forge[d]	I-64/Rt. 606(WBL)
30	1970	SBM	39*	Mecklenburg	Rt. 58/Buffalo Creek(WBL)
31	1972	SPG	86	Bland	Rt. 77/Clear Fork Creek(SBL)
32	1969	SPG	45*	Scott	Rt. 23/Clinch River(NBL)
33	1968	SBM	72	Brunswick	Rt. 712/I-85(EBL)
34	1966	SBM	50	Virginia Beach[d]	Rt. 60/Lynn Haven Inlet(EBL)

[a] Type of Superstructure (SBM = steel beam; SPG = steel plate girder; RC = reinforced concrete beam; PC = prestressed concrete beam).
[b] All are simple spans except those marked with and asterisk, which are continuous span.
[c] Constructed prior to specification change.
[d] City

tions were changed to require minimum compressive strengths of 4,000 psi instead of 3,000 psi, an entrained air content of 6 $^1/_2 \pm 1\ ^1/_2$ percent instead of 3 to 6 percent, and a maximum water to cement ratio (w/c) of 0.47 instead of 0.49. However, other than the air content, these changes did not appear to have caused systematic differences in the measured characteristics of concrete supplied before and after 1966.

Traffic lanes on each of these bridges were randomly selected for detailed examination and coring. In simple-span bridges, the length given in Table 1 is the total span length surveyed. In continuous-span bridges, the length given is the length between construction joints in the deck. Indications of deterioration (spalling, cracking, delamination, and scaling) were observed; measurements of half-cell potential were taken; and tests were made on cores for concrete quality and salt contamination.

DATA COLLECTION

The observations made and data collected are given in Tables 2 through 7. Table 2 shows the data relating to the condition of the bridge decks. Spalls are shown as the total number of square inches of spalled area divided by the area of the deck surface in square feet. Delaminations are expressed as the percentage of the area of the deck examined. Cracking is shown as the number of feet of cracks per 100 ft of bridge length. Small cracks are those less than 1 mm wide. Large cracks are those wider than 1 mm. The scaling factor is the average of ratings assigned to each 4-ft x 4-ft section of the deck where the assigned number is in accordance with ASTM C672 as follows:

0 - no scaling

1 - very light scaling (1/8 in depth maximum, no coarse aggregate visible)

2 - slight to moderate scaling

3 - moderate scaling (some coarse aggregate visible)

4 - moderate to severe scaling

5 - severe scaling (coarse aggregate visible over entire surface).

Table 3 shows the data on half-cell potentials measured in accordance with ASTM C876. Measurements were taken at each corner of a 4-ft by 4-ft section of the area being examined. The values shown are the percentage of the readings in each of the indicated ranges. In accordance with the ASTM method, when readings are more positive than -0.20 volts, there is a 90 percent probability that no corrosion of the reinforcing steel is occurring. When the readings are between -0.20 and -0.35 it is uncertain whether corrosion is occurring. When the readings are more negative than -0.35, there is more than a 90 percent probability that corrosion is occurring.

Table 4 includes cover depths over reinforcing steel, salt applications, freeze-thaw cycles, and traffic. The latter three represent deteriorating forces af-

TABLE 2

RESULTS OF BRIDGE DECK CONDITION SURVEY

Bridge No.	Spalls (in^2/ft^2)	Delaminations (% of Area)	Cracks (ft/100 ft) Small	Large	Average Scaling
1	0.00	0.0	0	0	1.2
2	0.42	11.1	51	74	1.5
3	0.02	0.3	150	105	1.2
4	0.00	0.0	152	55	2.1
5	0.00	0.0	268	99	1.1
6	0.00	0.0	0	83	1.4
7	0.04	0.0	165	161	2.0
8	2.36	41.7	10	37	2.2
9	0.02	8.2	8	14	1.6
10	4.40	65.2	94	30	1.7
11	0.00	0.0	43	215	1.9
12	0.00	0.0	0	33	1.7
13	0.00	0.0	28	56	2.1
14	0.00	0.0	0	61	0.3
15	0.00	0.0	0	0	1.4
16	0.89	40.5	0	196	1.8
17	0.15	19.4	15	261	2.3
18	0.96	57.6	11	167	2.2
19	0.00	0.0	11	11	2.7
20	0.00	0.0	51	154	2.4
21	0.00	0.0	0	113	2.2
22	0.01	0.3	0	65	1.5
23	0.00	0.0	0	0	2.5
24	0.00	0.0	98	323	1.8
25	0.00	0.0	0	0	2.3
26	0.00	0.0	0	70	2.0
27	0.00	0.0	0	2	2.2
28	0.00	0.0	0	229	2.5
29	0.00	0.0	0	0	4.5
30	0.00	0.0	0	31	3.0
31	0.00	0.8	39	32	3.1
32	0.00	13.2	111	91	3.8
33	0.00	0.1	82	20	2.4
34	0.07	0.0	20	62	2.8

fecting performance. The cover depth of the concrete is a measure of the degree of protection for the steel. The salt applications per year are estimated from records compiled by the appropriate area headquarters. Some application rates appear to be very high, but the areas represented in such cases are those having severe weather conditions, dangerous locations, steep slopes, heavy truck traffic, or proximity to the area headquarters. The freeze-thaw cycles are estimated from weather records from nearby airfields or weather stations. A freeze-thaw cycle is considered a drop in air temperature below 28°F with a subsequent rise to 32°F or above. The

TABLE 3

MEASURED HALF-CELL POTENTIALS

Bridge No.	Percentage of Area in Each Range A[a]	B[b]	C[c]
1	100	0	0
2	0	33	67
3	37	60	3
4	85	15	0
5	100	0	0
6	0	90	10
7	28	72	0
8	0	6	94
9	0	34	66
10	0	0	100
11	83	14	3
12	100	0	0
13	99	1	0
14	80	20	0
15	100	0	0
16	0	16	84
17	0	4	96
18	10	29	61
19	71	27	2
20	93	7	0
21	61	37	2
22	70	27	3
23	92	8	0
24	100	0	0
25	95	5	0
26	98	2	0
27	56	40	4
28	16	84	0
29	47	53	0
30	2	98	0
31	79	19	2
32	0	0	100
33	96	3	1
34	84	16	0

[a] More positive than –0.20 V, indicative of no corrosion.
[b] In the range of –0.20 to –0.35 V, presence of corrosion uncertain.
[c] More negative than –0.35 V, indicative of corrosion.

TABLE 4

PERFORMANCE AND ENVIRONMENTAL FACTORS

Bridge No.	Cover Depth (in)	Salt (app/yr)	Freeze-Thaw (cyc/yr)	ESAL daily avg
1	2.27	35	30	165
2	1.98	80	40	79
3	1.79	35	30	241
4	1.85	35	22	780
5	1.31	25	30	9
6	2.69	80	35	1157
7	2.56	25	29	1106
8	2.28	80	40	71
9	2.71	120	40	91
10	2.00	35	40	14
11	2.42	80	35	1093
12	2.41	80	35	1126
13	2.72	80	35	1093
14	3.06	9	35	1432
15	2.92	9	35	473
16	2.38	20	40	226
17	2.36	20	40	226
18	2.83	25	37	166
19	2.66	25	29	1194
20	2.18	25	29	1349
21	2.23	5	29	23
22	2.39	8	29	1266
23	2.45	35	22	551
24	2.10	5	29	116
25	2.23	13	34	930
26	2.59	10	35	44
27	2.08	10	20	431
28	2.29	35	20	359
29	2.52	35	36	611
30	2.23	50	26	302
31	2.66	80	40	1593
32	1.83	20	44	423
33	2.18	16	26	17
34	2.38	6	18	1230

estimates of yearly traffic have been converted to equivalent 18,000 pound single axle loads (ESAL) using the following weighting factors for each kind of vehicle: 0.0006 for cars, 0.24 for trucks, and 0.88 for tractor trailers.[3]

Tables 5 and 6 are the results of laboratory and petrographic tests on the bridge-deck concrete. At the time of the inspection and evaluation, one 4-in-diameter core was taken from each of the wheel paths, and a third core was taken from between the wheel paths. The cores were cut into sections and tested (see Figure 1). The laboratory tests for absorption were made in accordance with ASTM C642, and the chloride permeability was determined in accordance with AASHTO T277. In the absorption test, pieces of concrete cores are weighed after oven-drying. They are then saturated by immersion in water and the absorption is recorded as the difference between the dry and saturated surface-dry weights expressed as a percentage of the dry weight. In the rapid chloride permeability test, the top 2 in of cores are vacuum saturated and then subjected to a constant 60v D.C. potential for a 6-hr period. The total charge in coulombs passing through the specimen is a measure of the relative chloride permeability of the concrete as described in the test procedure.

Pulverized samples for the determination of chlorides were also obtained by drilling with a 2-in drill bit at areas adjacent to one of the cores from the wheel path and adjacent to another core between the wheel paths. Material was collected from two depths. One sample includes the material from the 0.25-in to 0.75-in depth, reported as the value at 0.5 in. The other includes the material between the depths of 1.5-in and 2.0-in. This is reported as the value at 1.75 in. At each depth, chloride samples from each location (wheel path or between wheel path) were analyzed separately according to AASHTO T260. The differences for results of samples taken in the wheel path and between wheel paths were statistically evaluated. At the 5 percent level, differences for chlorides at the 0.5-in level were significant, but the differences at the 1.75-in level were not. Because the latter are those of interest for the purpose of comparison with permeability values and other parameters, the results for the chloride content at each depth were averaged. These are reported in Table 5. These figures have been corrected for base chloride content by determining the chloride content of one core per deck at a depth of 5 in.

Table 6 shows the measured air-void parameters (air content, spacing factor, and specific surface) of the cores. These determinations were made in accordance with the linear traverse method of ASTM C457.

Table 7 is the summary of the available condition ratings assigned by field inspectors to each bridge over its lifetime. The basis of these conditions ratings is explained in the analysis of data, which follows. Where ratings for an interval are not shown, the information for that period was not available. None of these decks were replaced or completely overlaid during the period indicated, but some minor repairs were made as needed.

TABLE 5

RESULTS OF LABORATORY TESTS

Bridge No.	Absorption (% by wt.)	Permeability (coulombs[a])	Chloride 0.5 in (lb/cu yd[b])	Chloride 1.75 in (lb/cu yd[c])
1	4.98	2160	2.77	0.00
2	4.85	2900	9.88	1.48
3	4.69	1900	1.85	0.13
4	4.86	2230	2.15	0.07
5	4.67	1630	0.43	0.10
6	4.91	2130	2.41	0.83
7	5.49	2270	4.35	1.53
8	5.17	3330	11.44	6.48
9	4.03	2700	14.47	6.41
10	4.64	3770	10.56	8.44
11	4.27	2590	3.40	0.52
12	4.10	1920	0.55	0.00
13	4.10	2130	3.72	0.05
14	4.30	1900	2.38	0.08
15	4.13	2180	3.13	0.00
16	4.03	1870	8.81	6.59
17	4.50	2340	12.24	4.19
18	4.53	2270	9.18	6.02
19	5.13	2460	6.45	1.83
20	5.30	1630	4.15	1.41
21	4.27	4290	5.79	1.78
22	3.90	2510	6.07	1.66
23	3.60	1910	8.97	1.70
24	3.40	1980	2.07	0.40
25	3.63	1730	3.99	0.19
26	3.73	1910	2.00	0.53
27	3.53	2750	5.33	0.61
28	3.60	1530	2.16	0.71
29	5.11	2350	4.82	0.87
30	5.38	2860	3.30	1.73
31	4.17	2800	14.14	1.39
32	5.10	2980	10.09	7.42
33	4.34	4530	1.62	0.52
34	5.45	2750	1.94	0.02

[a] AASHTO T277

[b] Sampled from 0.25 to 0.75 in depth.

[c] Sampled from 1.50 to 2.00 in depth.

TABLE 6

PETROGRAPHIC ANALYSIS OF BRIDGE DECK CONCRETE

Bridge No.	Air Content (Percent)			Spacing Factor (in)	Specific Surface (in^2/in^3)
	Entrapped[a]	Entrained[b]	Total		
1	2.5	8.5	11.0	0.0047	614
2	3.2	4.7	7.9	0.0069	524
3	1.9	5.8	7.7	0.0042	900
4	2.6	7.0	9.6	0.0045	711
5	2.2	4.7	6.9	0.0061	667
6	2.5	6.5	9.0	0.0055	598
7	2.7	7.5	10.2	0.0046	663
8	1.8	5.4	7.2	0.0068	570
9	1.7	1.8	3.5	0.0132	446
10	1.1	1.3	2.4	0.0155	440
11	2.2	7.9	10.1	0.0049	651
12	2.9	5.4	8.3	0.0060	592
13	2.9	5.1	8.0	0.0066	558
14	2.1	5.0	7.1	0.0058	703
15	2.1	4.8	6.9	0.0067	599
16	0.8	1.7	2.5	0.0106	581
17	0.7	3.5	4.2	0.0083	649
18	1.6	2.7	4.3	0.0092	549
19	1.3	7.9	9.2	0.0049	684
20	1.5	6.5	8.0	0.0051	715
21	2.4	5.5	7.9	0.0076	477
22	3.2	7.0	10.2	0.0067	457
23	3.4	6.5	9.9	0.0055	593
24	1.7	3.3	5.0	0.0091	523
25	1.9	5.7	7.6	0.0052	731
26	3.5	8.6	12.1	0.0060	462
27	1.7	3.7	5.4	0.0077	601
28	2.6	5.0	7.6	0.0065	581
29	3.0	5.5	8.5	0.0044	787
30	1.8	5.3	7.1	0.0062	639
31	1.8	6.4	8.2	0.0046	816
32	2.4	5.5	7.9	0.0065	561
33	2.1	5.6	7.7	0.0054	687
34	1.3	5.6	6.9	0.0056	720

[a] Air bubbles greater than 1 mm diameter.
[b] Air bubbles 1 mm diameter or less.

TABLE 7

CONDITION OF BRIDGE DECK CONCRETE

Bridge No.	Ratings Reported in Designated Interval									
	71–72	73–74	75–76	77–78	79–80	81–82	83–84	85–86	87–88	89–90
1	8		8		8	8	8	8	7	
2	8						7	6	5	5
3	8		8		8	7	7	7	6	
4	8	8	8		8	8	6	7	7	7
5	9	8	8	8	8	6	6	6	7	6
6	7	8	8	7	8	7	7	5	6	7
7	8	7		7	7		7		6	
8	8					6	5			7[a]
9	8					7		5		8[a]
10	8					7	6	5		6[a]
11		8	7	7	7	6	7	6	6	
12	7	7	7	7	7	7	7		7	7
13	8	7	7	6	6	6	6		7	7
14		7	7	7	7	7	6	6	7	
15	9			7	7	7	6	6	7	
16	7	7		8	8	8	8	6	4	6[a]
17	7	7	7	7	7	6	6	6	5	5
18	7	7	7	7	7	8	8	6	5	4
19	8		8	8	8	7	7	7	6	6
20			7	7	7	7	7	7	6	7
21	7	7	7	7	7	7	7	6		
21	8		7	7	7	7	7	7	6	
23	9	8	8	8	8		6	6	7	7
24	8			7	7		7	7	7	7
25	9		7	7	7	6		6	6	
26	8	7	7	7	7	6	6		6	
27	8	7	7	7	7	7	7	7	7	8[b]
28	8	8	7	7	7	7	7	7	7	7
29		8	8		7	7	6	6	6	
30	9	8		8	8	8	8	7	6	
31			8			7			5	5[c]
32	8					7	5		5	5[c]
33			8	8	8	8	8	8	8	
34									6	

[a] Repairs made during interval between inspections.
[b] Change results from change in guidelines.
[c] Repairs under way.

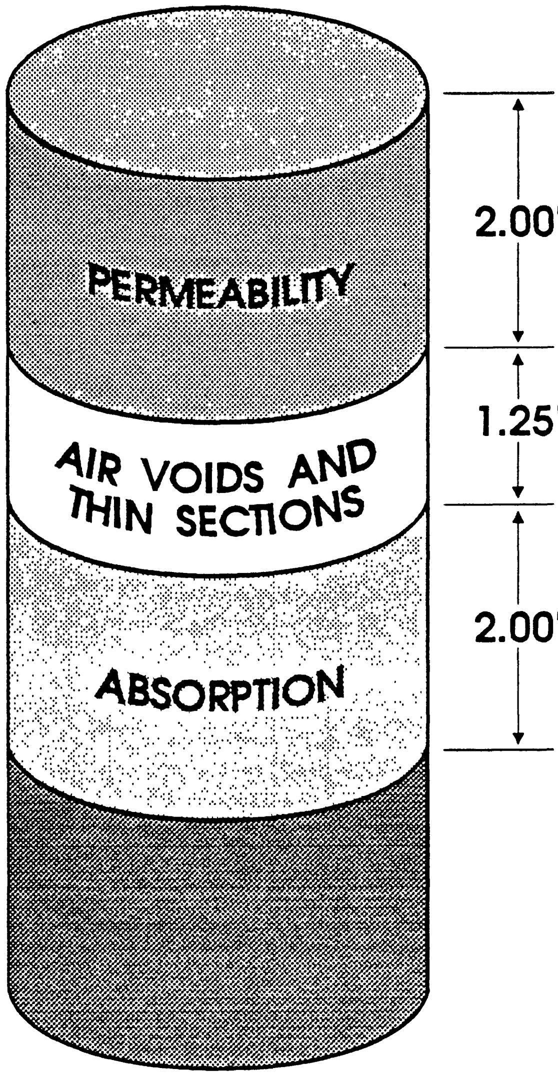

Figure 1. Sketch of sections of a core.

ANALYSIS OF DATA

Consideration of Individual Variables

The variables in the study were linearly correlated to determine whether one variable could be predicted from another. Such correlations would enable the establishment of relationships or equations from which deterioration could be related to the concrete quality parameters and the environmental and traffic conditions.

The variables examined were:

1. age
2. spalls
3. delaminations
4. crack lengths
   a. small
   b. large
   c. total
5. scaling
6. half-cell potentials
   a. percentage of areas with potentials more positive than -0.20V
   b. percentage of areas with potentials between -0.20V and -0.35V
   c. percentage of areas with potentials more negative than -0.35V
7. concrete cover over reinforcing bars
8. salt applications
   a. number per year
   b. total (yearly x age)
9. freeze-thaw cycles
   a. number per year
   b. total (yearly x age)
10. estimated average ESAL per day
11. absorption
12. permeability
13. chlorides
    a. at 0.5 in
    b. at 1.75 in
    c. difference between 0.5 in and 1.75 in.

A complete correlation matrix was run for all of the variables, and the correlation coefficients (r) higher than 0.60 are shown in Table 8. Because these values are not particularly high, probability values greater than 99 percent were calculated. They indicate that there is a strong probability that significant relationships between the selected pairs of variables exist.

The number of indicated relationships are limited, and the best correlation (r = 0.91) was established between the amount of salt at 1.75 in and the half-cell po-

TABLE 8

CORRELATION COEFFICIENTS AMONG VARIABLES

Variables[a]	Age	F–T	Delaminations	Spalls	Air	Salt
Age						
F–T						
Delaminations	0.68	0.72				
Spalls						
Air			0.64			
Salt		0.65	0.84	0.67	0.60	
V		0.71	0.80	0.63	–0.60	0.91

[a] Age is the current age of the bridge in years.
F–T is the total number of freeze-thaw cycles.
Delaminations is the percentage of area.
Spalls are in in^2/ft^2.
Air represents the total air content.
Salt is the amount of salt at the 1.75-in level.
V is the percentage of area more negative than –0.35 volts.

Note: All correlations had a probability exceeding 99%.

tentials more negative than –0.35 (see Table 8). Salt at 1.75 in and potentials more negative than –0.35 also had a fair to good relationship with the freeze-thaw cycles, delaminations, spalls, and the total air contents. The correlation (r = 0.68) between the age and delamination is expected since more time is given for the intrusion of chlorides and the action of the environment and the traffic. The correlation coefficients between the delaminations and the freeze-thaw cycles (r = 0.72) or the total air contents (r = 0.64) were fair. However, expected correlations were not established between the concrete quality parameters (absorption, spacing factor, and permeability) and the destructive forces or the evidence of deterioration. As previously noted, the VDOT specifications for bridge-deck concrete were expected to provide concretes of comparable quality. Thus, even though the air contents of the bridges constructed prior to 1966 are generally lower than those constructed after 1966, variations in other quality parameters are limited. The correlation coefficients were not sufficiently high to indicate correlations between the traffic density (ESAL) and the salt applications or between the traffic density and the parameters measuring deterioration.

Multiple regression analyses were also conducted for the relationships between each of the indicators of deterioration (half-cell potential, scaling, total cracks, and delaminations) and both the quality factors and the destructive forces. However, no additional useful information was obtained by this analysis.

Microscopic examination of thin sections from 10 bridges were made to determine the extent of microcracking, an estimation of paste quality, the amount of calcium hydroxide, the degree of hydration, and the quality of the bond between the paste and the aggregate. However, because the concretes were of different ages and had different histories, consistent interpretation of results and valid comparisons between different concretes could not be made using this technique. Also, indications of good paste quality by this technique would likely be related to permeability. Thus, because preparations for the permeability test are much less time consuming than those for the microscopic examinations, further tests on thin sections for the other bridges were not made.

The data in Table 7 show the general condition ratings assigned by field inspectors to each bridge deck over its lifetime. Although early ratings predate its adoption, these ratings are in accordance with the "Structure Inventory and Appraisal Coding Guide" of the Virginia Department of Transportation, dated September 1, 1987. This guide is based on the "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges" published by the Federal Highway Administration (January 1979).

In accordance with the coding guide (see the Appendix), ratings 8 and 9 indicate that no repairs are needed on the decks. Decks with ratings of 7 are also considered to be in generally good condition, though they require some minor maintenance. Ratings of 5 and 6 indicate that the decks are in fair condition, and major maintenance (rating 6) or minor rehabilitation (rating 5) may be needed. Decks with a rating of 4 require posting while an analysis is made. Decks with a rating of 3 require immediate rehabilitation.

The usefulness of these inspection ratings is being considered by the Virginia Transportation Research Council as a means of assigning priorities to bridge rehabilitation as a component of a bridge management system. It has been reported that, despite the need for better records, the ratings were adequate for use in such systems.[4]

The deck condition ratings include a number of factors, only one of which (deck structural condition) pertains to the condition of the concrete. This structural condition rating takes into account cracking, spalls, delaminations, and scaling, which are part of the indications of deterioration used in this study. Concrete quality parameters, such as air content and permeability, are not addressed in the regular structural condition rating; thus, there is no complete equivalency between the parameters studied in this project and those evaluated in the field by the bridge inspectors. The field condition ratings for a given structure have remained essentially constant during most of its life (see Table 7). Thus, predictions of deterioration based on differences in quality of the concretes in these decks cannot be made.

In this study, the decks were sampled at only one age. Thus, it could not be determined whether the quality parameters of the deck when it was built affected the age at which deterioration began or the rate at which it progressed. However, there is general agreement between the deterioration of the concrete measured in this study and the field rating assigned to the bridge at the time the study was conducted.

Consideration of Grouped Data and Combined Rating

Efforts were made to judge the effect of various parameters by grouping the data in relative quality levels based on knowledge of general concrete technology. Four levels were established for each significant parameter, and numerical ratings from 0 to 3 were assigned to the levels as follows:

- Quality Factors
  - Air Content
    3 - >6.0
    2 - 5.1 - 6.0
    1 - 4.0 — 5.0
    0 - <4.0
  - Spacing factor, $\bar{L}$
    3 - <.0070
    2 - .0071 — .0100
    1 - .0101 — .0120
    0 - >.0120
  - Absorption (percent water absorbed by weight of specimen)
    3 - <4.00%
    2 - 4.01 — 4.50%
    1 - 4.51 — 5.00%
    0 - >5.00%
  - Chloride permeability (coulomb values, AASHTO T277)
    3 - < 1000
    2 - 1001 — 2000
    1 - 2001 — 4000
    0 - >4000
  - Cover depth (concrete cover over reinforcing steel)
    3 - >2.50 in
    2 - 2.01 — 2.50 in
    1 - 1.50 — 2.00 in
    0 - <1.50 in

- Destructive Forces
  - — Salt applications (per year)
    0 - <10
    1 - 10 — 25
    2 - 26 — 70
    3 - >70
  - — Freeze-thaw cycles (per year)
    0 - <25
    1 - 25 — 30
    2 - 31 — 37
    3 - >37
  - — Traffic (avg. ESAL per day)
    0 - <100
    1 - 100 — 500
    2 - 500 — 1200
    3 - >1200
  - — Salt intrusion (chloride contents in lb/yd^3 of concrete at 1.75 in)
    0 - <1.00
    1 - 1.00 — 1.27
    2 - 1.28 — 3.00
    3 - >3.00
- Evidence of deterioration
  - — Delaminations (percentage of area delaminated)
    0 - <1.0
    1 - 1.1 — 10.0
    2 - 10.1 — 40.0
    3 - >40.0
  - — Scaling (average scaling factor)
    0 - 0 — 1.4
    1 - 1.5 — 3.4
    2 - 3.5 — 4.4
    3 - >4.4
  - — Half-cell potential (percentage of area with half-cell voltages more negative than -0.35)
    0 - 0
    1 - 1 to 10.0%
    2 - 11.0 to 50.0%
    3 - >50.0%
  - — Cracking (sum of the large and small cracks as feet of cracks per 100 ft bridge length)
    0 - <10 ft
    1 - 11 — 150 ft
    2 - 151 — 250 ft
    3 - >250 ft

— Spalling (in^2 of spalled area per ft^2 of deck)

0 - 0
1 - 0.01 — 0.15
2 - 0.16 — 1.0
3 - >1.0

The results of this evaluation are shown in Tables 9, 10, and 11. Generally, a 3 rating for a quality parameter is considered to be indicative of excellent quality; level 2 good quality; level 1 fair quality; and level 0 poor quality. The data in Table 9 show that of the 27 bridges constructed after the change in the specifications, all but 2 have excellent spacing factors, and these 2 are rated good. Only 1 bridge (No. 8) constructed prior to 1966 had an excellent spacing factor; 3 others were rated good; 1 was fair; and 2 were poor. Essentially the same evaluations are indicated by the total air contents. However, the 1966 changes in the specification did not systematically change the quality ratings of the other measured parameters. Absorption values for 7 of the concretes are low, which is indicative of excellent quality; 11 are in the good quality range; 8 are in the fair range; and 8 are of poor quality because they have very high absorption. However, there is no general relationship between this factor and any of the deterioration parameters. Similarly, the chloride permeability (as indicated by the coulomb values) shows no significant relationship with other factors. In accordance with AASHTO T277, none of the bridges have very low permeability (excellent quality); 11 have low permeability (good quality); 21 have moderate permeability (fair quality); and 2 have high permeability (poor quality). Cover depths were less than 2.0 inches in 5 cases, but a correlation with spalling or corrosion of reinforcing bars was not indicated.

The ratings for the destructive forces increase from 0 to 3 depending on the severity of the conditions. Similarly, the evidence of deterioration increases from 0 to 3 as the test values increase. A comparison of the level of destructive forces listed in Table 10 with the indications of deterioration in Table 11 show that neither the number of salt applications nor the traffic density showed a good relation with indications of deterioration. However, when the number of freeze-thaw cycles was very high, salt intrusion and corrosion also appeared to be very high in most cases. The condition ratings assigned by field personnel generally reflected the surface deterioration.

Correlation analyses were then made using a combined numerical rating for quality parameters (Table 9), destructive forces (Table 10), and indicators of deterioration (Table 11). This was done by using the sums of the individual ratings for all the parameters in each table. This analysis shows a fair correlation coefficient ($r = 0.70$) between the sum of the quality parameters and the sum of the deterioration factors, but the correlation coefficient between indications of deterioration and destructive forces was low ($r = 0.41$). This likely results from the fact that deterioration is not uniform throughout the life of the deck but accelerates rapidly once it begins. The sums of indications of deterioration showed a correlation coefficient with the field ratings of 0.72 at the time of the inspection, which is considered to be in the fair range.

TABLE 9

CONCRETE QUALITY PARAMETERS

Bridge No.	Air	$\bar{L}$[a]	Absorp	Perm	Cover	Sum
1	3[b]	3	1	1	2	10
2	3	3	1	1	1	9
3	3	3	1	2	1	10
4	3	3	1	1	1	9
5	3	3	1	2	0	9
6	3	3	1	1	3	11
7	3	3	0	1	3	10
8	3	3	0	1	2	9
9	0	0	2	1	3	6
10	0	0	1	1	2	4
11	3	3	2	1	2	11
12	3	3	2	2	2	12
13	3	3	2	1	3	12
14	3	3	2	2	3	13
15	3	3	2	1	3	12
16	0	1	2	2	2	7
17	1	2	2	1	2	8
18	1	2	1	1	3	8
19	3	3	0	1	3	10
20	3	3	0	2	2	10
21	3	2	2	0	2	9
22	3	3	3	1	2	12
23	3	3	3	2	2	13
24	2	2	3	2	2	11
25	3	3	3	2	2	13
26	3	3	3	2	3	14
27	2	2	3	1	2	10
28	3	3	3	2	2	13
29	3	3	0	1	3	10
30	3	3	0	1	2	9
31	3	3	2	1	3	12
32	3	3	0	1	1	8
33	3	3	2	0	2	10
34	3	3	0	1	2	9

[a] Spacing Factor
[b] 3 = excellent; 2 = good; 1 = fair; 0 = poor.

TABLE 10

LEVEL OF DESTRUCTIVE FORCES

Bridge No.	Salt app.	Freeze-thaw	ESAL	Salt at 1.75 in	Sum
1	2[a]	1	1	0	4
2	3	3	0	2	8
3	2	1	1	0	4
4	2	0	2	0	4
5	1	1	0	0	2
6	3	2	2	2	9
7	1	1	2	2	6
8	3	3	0	3	9
9	3	3	0	3	9
10	2	3	0	3	8
11	3	2	2	0	7
12	3	2	2	0	7
13	3	2	2	0	7
14	0	2	3	0	5
15	0	2	1	0	3
16	1	3	1	3	8
17	1	3	1	3	8
18	1	2	1	3	7
19	1	1	2	2	6
20	1	1	3	2	7
21	0	1	0	2	3
22	0	1	3	2	6
23	2	0	2	2	6
24	0	1	1	0	2
25	1	2	2	0	5
26	1	2	0	0	3
27	1	0	1	0	2
28	2	0	1	0	3
29	2	2	2	0	6
30	2	1	1	2	6
31	3	3	3	2	11
32	1	3	1	3	8
33	1	1	0	0	2
34	0	0	3	0	3

[a] 3 = very high level; 2 = high; 1 = moderate; 0 = low.

TABLE 11

LEVELS OF DETERIORATION

Bridge No.	Half-cell[b] Potential (% of area)	Delam. (% of area)	Scaling	Total Cracks (ft/100 ft)	Spalling (in^2/ft^2)	Sum	Field Rating
1	0[a]	0	0	0	0	0	7
2	3	2	1	1	2	9	5
3	1	0	0	3	1	5	6
4	0	0	1	2	0	3	7
5	0	0	0	3	0	3	7
6	1	0	0	1	0	2	7
7	0	0	1	3	1	5	7
8	3	3	1	1	3	11	5
9	3	1	1	1	1	7	5
10	3	3	1	1	3	11	5
11	1	0	1	3	0	5	6
12	0	0	1	1	0	2	7
13	0	0	1	1	0	2	7
14	0	0	0	1	0	1	6
15	0	0	0	0	0	0	7
16	3	3	1	2	2	11	4
17	3	2	1	3	1	10	5
18	3	3	1	2	2	11	5
19	1	0	1	1	0	3	6
20	0	0	1	2	0	3	6
21	1	0	1	1	0	3	7
22	1	0	1	1	1	4	6
23	0	0	1	0	0	1	7
24	0	0	1	3	0	4	7
25	0	0	1	0	0	1	6
26	0	0	1	1	0	2	6
27	1	0	1	0	0	2	7
28	0	0	1	2	0	3	7
29	0	0	3	0	0	3	6
30	0	0	1	1	0	2	6
31	1	0	1	1	0	3	5
32	3	2	2	2	0	9	5
33	1	0	1	3	0	5	8
34	0	0	1	1	1	3	6

[a] 3 = advanced deterioration; 2 = high deterioration; 1 = moderate deterioration; 0 = low or no visible deterioration.

[b] More negative than –0.35 V.

Despite the lack of good quantitative statistical correlations, reviewing the data for a given structure provides some explanation of the observed behavior of that structure. The 6 bridges having the greatest deterioration are, in order of severity, numbers 10, 18, 8, 16, 17, and 32. All of these except number 32 were constructed prior to 1966; thus, age as well as low air content may have been a contributing factor to deterioration.

1. *Bridge 10.* The percentage of the area delaminated almost surely results from severe corrosion of the reinforcing bars. All of the half-cell potentials measured were more negative than -0.35. The air content and spacing factor of the concrete on this deck are poor, and both absorption and chloride permeability are only fair. Salt intrusion is very high. Although a direct relationship between the air-void system and permeability is not expected, the initial poor quality of the concrete apparently led to rapid deterioration and the consequent intrusion of salt to the levels of the reinforcing steel, which subsequently corroded.

2. *Bridge 18.* There has been very high salt intrusion into the concrete, which has resulted in severe delamination. Chloride permeability and absorption are in the fair range, and the number of salt applications has been moderate. The length of cracks is relatively high. The intrusion of the salt may have resulted from a combination of marginal concrete quality and a large number of cracks.

3. *Bridge 8.* The absorption on this bridge is high. As is true for bridge 18, the intrusion of salt accounts for the corrosion, and there have been very many salt applications. The indications are that the absorptive nature of the concrete, which allowed chlorides to penetrate it, contributed to the deterioration of this bridge. The air-void system of this bridge was in the excellent group. This shows that this parameter alone is not sufficient for good performance.

4. *Bridge 16.* This bridge has undergone a very high number of freeze-thaw cycles and a moderate number of salt applications. The large number of cracks may be related to the freeze-thaw cycles. The total air content is poor, and the spacing factor is only fair. The intrusion of the salt, which results in high corrosion (evidenced by half-cell potentials more negative than -0.35V, spalls, and delaminations), has caused the deterioration.

5. *Bridge 17.* This bridge has a fair rating for its total air content and a good spacing factor. Absorption is also in the good range. There are many feet of large cracks, which may be related to the large number of freeze-thaw cycles. This may be a factor in the large amount of salt that has penetrated into the deck.

6. *Bridge 32.* This bridge was constructed in 1969 after the specification change and had an excellent air void system. However, the absorption is high. Although only a moderate number of salt applications have been made, the intrusion of salt is very high with resulting corrosion of the reinforcing steel.

SUMMARY AND CONCLUSIONS

This study shows that present VDOT specifications and placement procedures generally provide good quality concrete from the standpoint of resistance to damage from freezing and thawing. Scaling of such concrete does not appear to be a major problem in Virginia. Most of the concretes are rated as having moderate or low permeability to chlorides as determined by AASHTO Test Method T277. However, there is evidence of high chloride penetration in some cases that cannot be explained by the differences in results of AASHTO T277.

This study provides useful confirmation of previously recognized trends and general qualitative guidelines for evaluating the performance of decks containing uncoated steel. These findings are not likely to be applicable to concrete constructed with epoxy-coated reinforcing steel, which is now generally specified by VDOT for bridge decks. The epoxy coating prevents the reaction between the aqueous solution of chlorides and the steel. It can be concluded that:

- The ingress of chloride ions to levels at or near reinforcing bars is a major cause of the deterioration of concrete bridge decks constructed with uncoated reinforcing steel. However, none of the characteristics of the concrete measured in this study provide a useful way of judging the likelihood of high rates of ingress of salt into the concrete.
- The results confirm previous findings that the measurement of half-cell potentials is a reliable measure of the presence of corrosive conditions. For these bridges, there are also strong correlations among the half-cell potential, the amount of salt at depths of 1.5 to 2.0 in, and the degree of delamination.

RECOMMENDATIONS

It is recommended that present practices of assessing the level of corrosion of bridge decks by measurement of half-cell potentials (ASTMC 876) be continued. Tests to determine the actual salt content of the concrete (AASHTO T260) at the 1.5 to 2.0 in level should also be made as an indication of the potential for accelerated deterioration.

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REFERENCES

1. Newlon, H. H., Jr. 1974. *A survey to determine the impact of changes in specifications and construction practices on the performance of concrete in bridge decks*. VHRC 73-R59. Charlottesville, Virginia: Virginia Transportation Research Council.

2. Newlon, H. H., Jr. and Walker, H. N. 1985. *Relationship between properties of hardened concrete and bridge deck performance in Virginia*. VHTRC 85-R23. Charlottesville, Virginia: Virginia Transportation Research Council.

3. Vaswani, N. K. and Thacker, D. E. 1972. *Estimation of 18-kip equivalent on primary and interstate road system in Virginia*. VHRC 71-R34. Charlottesville, Virginia: Virginia Transportation Research Council.

4. Allen, G. R. and W. T. McKeel, Jr. 1989. *Development of performance deterioration curves as a rational basis for a structure maintenance management system*. Pittsburgh, Pennsylvania: Presented at the Sixth Annual International Bridge Conference.

APPENDIX

Bridge Deck Condition Rating

Excerpts from:

VDOT "Structure Inventory and Appraised Coding Guide," September 1, 1987.

Based on: "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges," U.S. Department of Transportation, Federal Highway Administration, January 1979.