

Evaluation of Bridges Constructed in Chromite Ore Processing Residue

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A field investigation was conducted to assess the structural and durability implications of bridges constructed in chromite ore processing residue (COPR) fill. The findings are based on visual inspection and various testing of 31 bridges, located in known chromium and nonchromium sites in and around Hudson County, New Jersey. Various types of observed deterioration of the concrete bridge substructures are documented for each of the study bridges. Of the 31 bridges, approximately half (15) of the bridges underwent more extensive nondestructive testing, including determination of chloride ion content and pH, determination of the presence of alkali-silica reactivity, impact-echo for assessment of integrity, and measurement of half-cell corrosion potentials where exposed reinforcement was available to document the causes of observed deterioration. On the basis of the study results, it is concluded that observed deterioration of the bridges is a result of classical causes, such as chloride-induced corrosion and alkali-silica reactivity and clearly not a result of exposure to COPR.

From the turn of the century until the early 1970s, several industrial concerns in and around Hudson County, New Jersey, operated chromite ore processing facilities. A by-product of the kiln firing of chromite ore was a soil-like material similar in appearance and properties to a brown sandy silt soil with pebble-sized particles but with a lower density than soil. Because of its similarity to soil, this by-product material, commonly referred to as "slag" or "mud," was used by builders and public agencies as a fill material for reclamation of swamp lands known locally as the meadowlands (1).

Chromium, like other metals, may exist in several physicochemical states. Trivalent chromium [Cr(III)] is a naturally occurring stable form of chromium. Hexavalent chromium [Cr(VI)], less stable, generally is not naturally occurring but is produced by industrial processes. Total chromium refers to the total amount of chromium in all valent forms including Cr(III) and Cr(VI). A particular site has been informally deemed to be a chromium site in New Jersey if the Cr(VI) concentration in soil is 10 ppm or greater or the total Cr concentration is in excess of 500 ppm, or both.

In 1984, the U.S. Environmental Protection Agency classified Cr(VI) as a respiratory carcinogen to humans. Hexavalent chromium has been found at various sites in the Hudson County area where chromite ore processing residue (COPR) has been used as fill. More recently, the New Jersey Department of Environmental Protection and Energy and the New Jersey Turnpike Authority have alleged that COPR causes structural deterioration of roadways, bridges, and other structures in contact with COPR (2). Even though no specific mechanisms have been identified for deterioration of highway structures as a result of chromium exposure, the postulated mechanism apparently is related to crystallization of chromium salts.

This paper reports the results of a research study that was conducted to obtain data related to performance of bridges in Hudson County, New Jersey. This included a literature search to identify previous research and knowledge of any deleterious effects of chromium on construction materials and structural behavior. To assess the possible deleterious structural and durability implications of COPR fill on bridge structures in Hudson County, New Jersey, a comparative evaluation of bridges constructed both within and outside of chromium sites was conducted. The comparative evaluation was based on visual inspection of selected bridges augmented by nondestructive testing of bridge substructure components that included determination of chloride ion content and pH, determination of the presence of alkali-silica reactivity, and measurement of half-cell corrosion potentials. In addition, impact-echo testing was conducted to assess the overall structural integrity of selected bridge piers.

PROJECT DESCRIPTION

The study bridges were of two principal types of structural systems. Most bridge superstructures were conventional multigirder, steel or concrete, with a composite reinforced concrete deck supported on concrete pier bents. The other major superstructure system was steel truss with a concrete deck supported by steel stringers. The truss bridges were also supported on concrete pier columns or bents.

The exposure of the pier bents of the study bridges varied, depending on the site, from relatively dry fill and groundwater at depth to more brackish groundwater for those bridges in low-lying marsh areas. In all cases, the study bridge decks have likely been subjected to chemical deicers as evidenced by staining at expansion joints and drains.

FIELD INVESTIGATION

Initially, a reconnaissance of bridges located on and over major highways in Hudson County and surrounding areas was conducted. This was done to identify potential sites to be included in the overall survey. On the basis of this reconnaissance, 31 bridges, representing a broad range of structural systems and ages, were selected for study in the overall evaluation. The bridges generally were 40 plus years of age, as determined from date markers at the site and historical factors, such as bridge type and on which highway they were located. Table 1 summarizes the 31 bridges included in the overall evaluation and their general location in the northern New Jersey area.

TABLE 1 Summary of Bridges Included in Study

Bridge	Location	Description	Approx. Age
1	Western NJTP near Exit 18W	Welded plate girders with concrete pier cap and 2 column pier bents	40
2	Eastern NJTP near Exit 17	Welded plate girders with concrete pier cap and 2 column pier bents	40
3	Western NJTP near Rt. 506	Welded plate girders with concrete pier cap and 4 column pier bents	40
4	Eastern NJTP near Rt. 508	Riveted plate girders with concrete pier caps and 2 column pier bents	50
5	Eastern NJTP near Rt. 506	Riveted plate girders with concrete pier caps and 2 column pier bents	50
6	Eastern NJTP near Hackensack River	Riveted plate girders with concrete piers and 2 column pier bents	50
7	Eastern NJTP	Riveted plate girders with hammerhead wall piers	50
8	Pulaski Skyway	Steel arch bridge with diagonally braced concrete, 2 column piers	60
9	Pulaski Skyway	Steel arch bridge with diagonally braced concrete, 2 column piers	60
10	Pulaski Skyway	Steel arch bridge with diagonally braced concrete, 2 column piers	60
11	Pulaski Skyway	Steel arch bridge with 4 column piers	60
12	Pulaski Skyway	Steel arch bridge with 2 column piers	60
13	Pulaski Skyway	Steel arch bridge with concrete wall piers	60
14	Pulaski Skyway Viaduct	Concrete arch bridge with 4 column piers	60
15	NJTP Hudson County Extension	Welded plate girders with 2 column hammerhead piers	40
16	NJTP Hudson County Extension	Welded plate girders with 2 column hammerhead piers	40
17	NJTP Hudson County Extension	Welded plate girders with steel pier caps and 2 column pier bents	40
18	NJTP Hudson County Extension	Riveted plate girders with steel pier caps and 3 column pier bents	50
19	NJTP Hudson County Extension	Welded plate girders with concrete pier caps and 2 column pier bents	40
20	NJTP Hudson County Extension	Rolled steel girders with concrete hammerhead wall piers	40
21	NJTP Hudson County Extension	Riveted plate girders with concrete hammerhead wall pier	50
22	NJTP Hudson County Extension	Welded plate girders with 2 column hammerhead pier	40
23	NJTP Hudson County Extension	Riveted plate girders with concrete pier cap and 2 column pier bents	50
24	NJTP Hudson County Extension	Riveted plate girders with concrete pier cap and 2 column pier bents	50
25	NJTP near Exit 12	Welded plate girder with concrete pier cap and 2 column pier bents	40
26	NJTP near Exit 11	Welded plate girder with concrete hammerhead pier	40
27	Garden State Parkway near Exit 137	Riveted plate girders with concrete wall piers	50
28	NJTP Hudson County Extension	Riveted plate girders with concrete pier cap and 2 column pier bents	50
29	NJTP Hudson County Extension	Riveted plate girder with steel pier cap and 3 column piers	50
30	NJTP Hudson County Extension	Welded plate girder with concrete pier cap and 2 column pier bents	40
31	Western NJTP near Hackensack River	Welded plate girder with concrete pier cap and 2 column pier bents	40

For each of the 31 bridges included in the study, a limited visual condition survey was conducted from the underside of the bridge, with particular emphasis on the condition of the supporting pier substructures in direct contact with the soil. Various forms and signs of distress and deterioration, such as cracking, efflorescence, presence of corrosion products, delamination, spalling, and freeze-thaw damage were documented and recorded for each bridge.

After completing the initial limited condition evaluation of the 31 selected bridges, 15 bridges (approximately half the total) were selected for more in-depth evaluation and nondestructive testing. The testing of these 15 sites was conducted to assess the general integrity of the supporting piers at selected locations and to determine the probable cause of visible distress and deterioration. Non-destructive testing is testing that does not require any significant invasive probing or sampling of structural materials. At each site, soil samples were collected at approximately 0.3 m below grade adjacent to selected bridge piers. These soil samples were tested for total and hexavalent chromium concentrations, chlorides, sulfates, pH, and the percentage of solids in the soil. On the basis of prevailing health-based guidance levels of 10 ppm Cr(VI) and 500 ppm total Cr, 7 of these 15 bridge sites would be classified as chromium sites.

A summary of the various types of nondestructive testing conducted at selected bridge piers of the 15 specific sites follows.

Alkali-Silica Reactivity

The visual survey of the 31 bridges indicated that in some cases observed cracking and distress in concrete bridge piers may have

been caused by alkali-silica reactivity (ASR). ASR reaction products form in concrete when sufficient alkalis, such as potassium and sodium, in the cement react with silica and silicates in the aggregates. In the presence of moisture, ASR gel reaction products expand, which may lead to cracking.

The presence of ASR was determined using the uranyl acetate fluorescence method (3). This procedure was developed in the recent government-sponsored Strategic Highway Research Program (SHRP), which focused on the development of advanced technology for improving and rehabilitating the nation's highway infrastructure. The results from each ASR test were classified into one of four categories—none, meager, moderate, and abundant—depending on the amount of gel reaction products and brilliance of the background fluorescence.

Chloride Ion and pH

Concrete powder samples from select piers of the 15 bridges were obtained by percussion drill and subsequently analyzed for chloride ion (Cl^-) content and pH. This testing was conducted to assess the susceptibility of the embedded reinforcing steel to chloride- and carbonation-induced corrosion.

The generally accepted chloride corrosion threshold, the value above which corrosion of reinforcing steel will occur in the presence of sufficient amounts of oxygen and moisture, is 300 ppm. In addition, corrosion will not likely occur as long as the concrete maintains a pH in excess of 12 (4). If pH is reduced to below 11 by the presence of high chloride levels or if calcium hydroxide (CaOH) is converted to calcium carbonate (CaCO_3) by the presence of

atmospheric carbonation (CO_2), corrosion can occur if oxygen and moisture are available (4).

The Cl^- content in the concrete powder samples taken from selected bridge piers at each of the 15 sites was measured by the recently SHRP-developed procedure, which uses direct reading in a chemical digestion solution using a specific chloride ion probe (5). The pH was determined by using an analytic chemistry procedure of placing a pH electrode in solution.

Half-Cell Corrosion Potentials

Measurement of corrosion potentials was possible only on bridge piers at two sites where reinforcing steel was already exposed because of deterioration. Corrosion potentials were measured using a copper/copper-sulfate half cell following the standard ASTM method (6). Half-cell potential measurement does not provide information about corrosion rate but does provide an indication of on-going corrosion activity; half-cell corrosion potentials more negative than -350 mV indicate ongoing active corrosion.

Impact-Echo Testing

The initial reconnaissance revealed the presence of pier cracking on some of the study bridges. In some cases, the cracks have been epoxy injected presumably to restore integrity. At other study sites, the concrete piers had little if any signs of distress or deterioration despite being in service for more than 40 years. To quantify to some degree the present structural condition of selected bridge piers at the 15-in.-depth study sites, as well as to assess the extent of cracking beyond that which could be visually observed, impact-echo testing was conducted. In addition, impact-echo testing was conducted to be able to directly compare the integrity of selected piers constructed in chromium and nonchromium sites. The theory, experimental, and field techniques using the impact-echo method for testing platelike (slabs) and beamlike (beams, columns, and bridge piers) concrete structures are well documented (7-10).

In the impact-echo technique, a transient stress pulse is introduced into the test object by mechanical impact on the surface. The stress pulse propagates into the object along spherical wavefronts as *P*- and *S*-waves and along the surface of the object as an *R*-wave. The *P*- and *S*-waves are reflected by internal cracks or interfaces and by the external boundaries of the object. The arrival of these reflected waves at the surface where the impact was generated produces displacements that are monitored by a transducer. If the transducer is placed close to the impact point, the waveform is dominated by displacements caused by *P*-wave arrivals.

In plates, wave reflections from the side boundaries do not have a significant effect on the response. In contrast, transient wave propagation in linear (barlike) elements, such as bridge piers, subjected to transverse elastic impact is more complex because the effects caused by the close proximity of the side boundaries are significant. The response of a linear member subjected to transverse impact is dominated by cross-sectional modes of vibration set up by multiple reflections of waves in a cross section (9). The geometry and dimensions of a cross section determine the shape and frequency of each cross-sectional mode. The length is not a factor as long as it is greater than about three times the width. The presence of a flaw within a linear element disrupts the frequency pattern created by the cross-sectional modes.

RESULTS

Literature Review

A review of the technical literature related to effects of chromium on concrete found few papers on the subject. One reference (11) referred to a 1969 study by Craig (12) in which potassium chromate was used as a corrosion inhibitor. Although Craig (12) reported that concrete compressive strength decreases with increasing levels of admixed potassium chromate, it is important to emphasize that a careful review of the data shows some scatter. Moreover, the strength decrease is related to the percentage of chromate salt, but it is not time dependent. In other words, for a given percentage of admixed chromate salt, the strength does not decrease with time.

Mehta (13) reports a possible deterioration mechanism of porous materials subjected to sulfate salts. He states that crystallized salts inherent in the pores of materials in critically saturated solutions may create stresses large enough to cause cracking. However, Mehta (13) does not specifically mention that chromium salts cause this type of deterioration.

The possible deterioration mechanism identified by Mehta (13) is analogous in effect to deterioration associated with freeze-thaw cycles. Porous concrete that is critically saturated can freeze and an associated expansion can occur. If the tension stresses developed from the expansion exceed the tensile strength of the concrete, cracking can occur. Thus, concrete that might be susceptible to deterioration from crystallization of salts would likely also be susceptible to deterioration by freeze thaw.

Visual Survey

The visual survey of the study bridges revealed that the concrete bridge substructures exhibited classical signs of deterioration associated with age and prolonged exposure to deicer chemicals, moisture, and freeze-thaw cycles. Figure 1 illustrates some of the more aggressive forms of deterioration observed on concrete bridge substructures of the study bridges. Table 2 documents the deterioration observed on the study bridges.

As described in Table 2, much of the deterioration is associated with corrosion, water penetration, and freeze thaw. This observed deterioration tended to be focused at the pier caps and upper por-



FIGURE 1 Freeze-thaw deterioration on pier.

TABLE 2 Summary of Visual Study

Bridge Site #	Description of Observed Deterioration and Distress
1	Cracks with efflorescence on pier cap and piers, 0.3 - 0.8mm in width; epoxy injection on pier caps and piers; rust stains on pier caps.
3	Good condition; rust stains on pier caps and piers from steel girders.
4	Cracks with efflorescence, spalls; cracking on pier caps and piers; corrosion products evident.
5	Cracks ; cracking on corners of piers. Piers in good condition otherwise.
6	Small cracks with efflorescence. Piers in good condition.
7	Cracks on pier cap, previous patching; cracks w/ efflorescence below bridge bearing shoes, patches on piers.
8	Heavy cracking with corrosion products and efflorescence; epoxy injection on piers; previous repair work.
9	Cracking with corrosion products and efflorescence; epoxy injection on piers; cracks with corrosion products and efflorescence; previous repair work.
10	Heavy cracking with corrosion products and efflorescence; epoxy injection on pier bases.
11	Cracks and spalling concrete on piers, staining on piers; cracks and spalls with corrosion products and efflorescence, previous repairs.
12	Cracks with efflorescence and corrosion products, epoxy injected cracks.
13	Cracks with efflorescence and staining; many epoxy injected cracks.
14	Cracks and spalls with exposed rebar, previous patches.
15	Cracks and spalls on pier caps; cracks with corrosion products; chipped out spalls; patches w/ efflorescence.
16	Stains from corrosion products; previous repairs with cracks; corrosion products.
17	Incipient spall on pier cap; staining and corrosion products on pier caps.
18	Cracking with corrosion products on pier cap; epoxy injection on concrete pier cap; corrosion on steel pier cap; staining.
19	Stains from corrosion products; stains around bridge shoes on pier caps.
20	Cracks with corrosion products and efflorescence on pier faces; incipient spall on pier face.
21	Heavy corrosion products at cracks, previous patches.
22	None
23	Cracks with corrosion products on pier cap bottoms.
24	Cracks with corrosion products and efflorescence on pier caps, incipient spalls on pier caps.
25	Cracks with corrosion products and stains on pier caps, previous patches.
26	Stains on piers, freeze-thaw damage.
27	Cracks with efflorescence; cracks with efflorescence below bridge shoe.
28	Incipient spalls on pier caps, previous repairs on pier caps.
29	Incipient spalls and cracks on piers, epoxy injection on piers, steel pier cap corrosion visible.
30	Spalled concrete on piers, corrosion and efflorescence present, epoxy injected areas.
31	Cracks and staining; vertical cracks in piers, corrosion products on piers.

Note: Site 2 did not have good access for visual survey and, therefore, is not included.

tions of the pier columns. This deterioration is likely caused by almost direct exposure of water and deicer chemicals through expansion joints in the bridge deck.

There were numerous cases in which it was apparent that observed cracking in pier bents had been previously epoxy injected. This cracking, which was generally prevalent throughout the entire pier, was likely caused by ASR, corrosion, or possibly sulfate attack. Clearly, because it was located well removed from any direct or indirect contact, this cracking could not have resulted from exposure to COPR.

For the piers examined in the 31 bridges included in the study, there was no visual indication of structural distress from foundation displacement or from vehicular impact.

Soil Sampling

Table 3 summarizes the results of the chemical analysis of the soils collected from the 15 bridge sites included in the more in-depth study. Of the 15 bridges 7 are considered to be chromium sites on

TABLE 3 Chemical Analysis of Soils from Sites of 15 Bridges Included in More In-Depth Study

Bridge Site #	Sample #	% Solids	Sulfates (ppm)	Chlorides (ppm)	Cr(VI) ^{1,2} (ppm)	Total Cr ¹ (ppm)	pH
1	A	90.6	113	80	< 4.0	18.4	7.84
	B	90.0	113	70	< 4.0	15.5	7.84
5	A	90.7	48	370	14.0	125.0	8.69
6	A	67.3	640	4500	< 6.0	21.3	7.14
7	A	83.3	9	69	< 5.0	18.0	7.86
9	A	-	-	-	< 2.5	2600.0	-
10	A	-	-	-	< 2.5	570.0	-
	B	82.8	1350	51	< 5.0	141.0	7.53
11	A	87.7	420	190	< 5.0	54.2	8.15
14	A	96.3	20	5	< 4.0	9.3	8.48
22	A	84.9	20	110	< 5.0	26.0	9.28
	B	83.7	32	260	< 5.0	41.6	8.88
25	A	90.3	47	700	< 4.0	9.7	8.75
27	A	90.0	180	12	< 4.0	17.2	5.17
28	A	74.3	51	200	12.0	132.0	7.71
29	A	93.1	620	3000	45.0	6680.0	8.25
	B	87.8	480	19000	32.0	2620.0	8.11
30	A	79.2	750	36000	248.0	2170.0	8.67
31	A	86.9	180	240	11.0	527.0	8.75

Notes:

1. Shaded cells indicate chromium sites based on health-based guidance levels in New Jersey of 10 ppm Cr(VI) and 500 ppm total Cr.
2. A < sign indicates concentrations less than the detection limits of the laboratory procedure.

the basis of the prevailing health-based guidance levels for Cr(VI) and total CR concentration in New Jersey.

The measured sulfate, chloride, and pH values in the sampled soils are shown in Table 3. There is no apparent trend between measured sulfate, percent solids in soil, pH, and chromium concentration.

A review of the chloride contents of the soil at the 15 bridge sites revealed some extraordinarily high chloride contents at two locations classified as chromium sites. It was assumed that these values were caused by outside chloride sources, such as chemical deicers, and not from the chromite ore processing residue apparently used as fill at these sites. Chemical deicers contain chlorides and are present in runoff water from the bridge decks that drains to the soil below through drains and expansion joints. To verify the validity of this assumption, the chloride and chromium contents from the bridge sites were compared with samples collected from a plant site where COPR was produced and used as fill. Results showed the plant site samples to have mean and median chloride contents of 236 and 50 ppm, respectively, with a standard deviation of 500. For the bridge site samples, the mean and median chloride contents were 3,815 and 200, respectively, with a standard deviation of 9,486. These results show that the chloride content of soil at bridge

sites is substantially higher than that of known COPR fill. Therefore, the assumption of an external chloride source is valid.

Figure 2 presents the measured chloride and total chromium concentrations of the soil at the bridge sites included in this study and that measured at plant sites where COPR was used as fill. The data shown in Figure 2 reflect the higher chloride concentrations at bridge sites and clearly indicate no correlation between chloride concentration and chromium concentration.

Concrete Chemical Analysis

The results from the various chemical analyses of selected bridge pier concrete are provided in Table 4. Several general trends are evident. At 13 of 15 sites, the Cl⁻ levels of at least one of the concrete samples exceeds the generally accepted chloride corrosion threshold of 300 ppm. Concomitantly, the pH of the concrete is generally less than 12. Both these factors suggest that steel reinforcement in the piers is at high risk to corrosion in the presence of moisture and oxygen.

The ASR testing revealed the presence of some ASR. Only in cases designated "moderate" and "abundant" is observed cracking

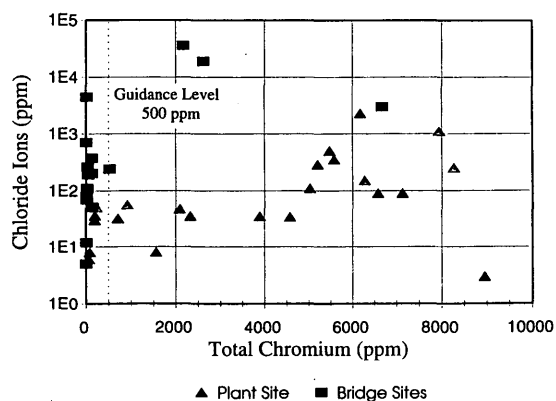


FIGURE 2 Comparison of chloride and total chromium concentration at bridge and plant sites.

directly attributable to ASR. This is the case for 8 piers of the over 30 examined at 15 different sites.

Corrosion Potentials

Half-cell corrosion potentials were measured at Sites 14 and 30 on piers in which there was exposed reinforcing steel. The measured corrosion potentials of the pier of Site 14, a nonchromium site, were between -60 and -280 mV, which indicates low probability of ongoing corrosion activity.

The corrosion potentials on the pier at Site 30 were measured around the pier at an area approximately 3 to 5 m off the ground. Electrical contact was made at an area of exposed reinforcing steel. On the basis of visual observations, there clearly had been past corrosion activity on the steel reinforcement. The measured corrosion potentials were between -450 and -575 mV, which indicated active corrosion. This area of exposed reinforcing steel and aggressive concrete deterioration of the concrete pier at this chromium site was well removed from possible contact with the COPR. In fact, the observed deterioration was in the trajectory of an open drain from the bridge deck above.

Impact-Echo

A summary of the impact-echo test results is also presented in Table 4. Many of the piers have flaws that generally were found to be at shallow depth, indicating that there is a basic core of concrete that is unaffected. A review of Table 4 indicates that the detected flaws are independent of whether the structure is located in a chromium or nonchromium site. This is the case because observed cracking is caused mainly by chloride-induced corrosion.

EVALUATION AND IMPLICATIONS

Raghu and Heieh (11) state that Mehta (13) offers a possible cause of the deterioration of concrete subjected to COPR as the pressure of crystallization of salts in pores of the concrete presumably caused by migration of chromium into the concrete. Mehta (13) discusses concrete deterioration caused by crystallization of salts but does not discuss chromium. In fact, Mehta's (13) discussion relates to sulfate salts inherent in the constituent concrete materials and not from ingress from external sources.

In other field investigations the authors observed deterioration by salt crystallization in masonry that contains surface glazing or a relatively impervious dye skin. This deterioration is caused by naturally occurring salts in the masonry units or mortar. There was no evidence in the present study that the observed deterioration in concrete, often well removed from contact areas of the chromium fill, could be related to salt crystallization. Moreover, as has been observed in masonry structures, if deterioration of concrete is caused by crystallization of salts inherent in the material, salts are clearly visible. There was no evidence of salts or salt residue at locations of deterioration on the bridges even though salts were clearly visible on the soil surface of some of the sites with COPR fill.

CHROMIUM VERSUS NONCHROMIUM SITES

The results presented in Table 3 reveal that 7 of the 15 sites included in the more in-depth study could be classified as chromium sites by the health-based guidance levels informally promulgated in New Jersey. Other sites adjacent to these chromium sites with bridges of similar age and structural system permitted direct comparison of the field investigation results.

Figures 3 and 4 are from Sites 10 and 11, which are on the Pulaski Skyway. As indicated in Table 3, Site 10 is classified as a chromium site, whereas Site 11 is not. Most of the deterioration of the large concrete piers was present just below the deck slab where water could accumulate and subsequently drip. The measured Cl^- levels in the concrete (see Table 4) were above the chloride corrosion threshold, indicating that much of the deterioration is caused by corrosion; this observation is supported by visible rust stains that emanate from cracks.

At both sites the large concrete piers also exhibited vertical cracks, generally on all four sides of the pier, rising a substantial height. These cracks tended to be near the center of the pier, indicating that they could be related to shrinkage early in the life of the structure. However, impact-echo results did not indicate that the cracks went through the entire section, a condition that could compromise the load capacity and thus the integrity of the piers.

Direct comparison of results was also possible between Sites 5 (chromium site) and 6 (nonchromium site) because they were adjacent to each other on the New Jersey Turnpike. In general, the concrete pier bents at these two sites were in fair condition, with limited signs of the usual cracking caused by reinforcing steel corrosion on the pier caps and at corners of piers. In both cases, the Cl^- levels in the concrete were above the threshold level, indicating that the observed cracking was likely caused by chloride-induced corrosion.

It was observed that the cracking in one of the piers examined at Site 6 was more extensive and pronounced. As noted in Table 4, the ASR test result for this pier was classified as "abundant," indicating that cracking was likely caused by ASR.

A comparison of test results between chromium and nonchromium sites reveals some general trends. For example, the average Cl^- contents in both chromium and nonchromium sites typically are above the generally accepted chloride corrosion threshold, indicating that the reinforcement in the piers is susceptible to corrosion in both chromium and nonchromium sites.

Analysis of the test results from Site 30 provides the most convincing evidence that observed deterioration is clearly not related to exposure to chromium. The soil data in Table 3 indicate that Site 30 is a chromium site. As indicated in Figure 5, many of the bridge piers at Site 30 are mounded by COPR: the visible pier cap in Figure 5 was previously epoxy injected, presumably to restore integrity.

TABLE 4 Chemical and Physical Analysis of Concrete from Selected Piers at 15 Bridges Included in More In-Depth Study

Bridge Site #	Sample	Chlorides(ppm)	pH	ASR	Impact-Echo
1	A	84	11.7	none	Solid
	B	311	11.7	meager	N/A
5	A	1958	12.2	meager	Solid
	B	4381	11.8	meager	Flaw
6	A	435	11.2	abundant	N/A
	B	1329	12.3	meager	N/A
7	A	808	12.0	meager	Solid
	B	334	11.9	meager	Flaw
9	A	2701	11.8	meager	Solid
	B	1585	11.8	moderate	N/A
10	A	1188	11.9	moderate	Solid
11	A	535	11.9	meager	Flaw
	B	1061	11.9	meager	Flaw
14	A	613	10.7	meager	Solid
	B	217	11.3	meager	Flaw
22	A	130	11.0	meager	Flaw
	B	208	11.9	moderate	Flaw
25	A	719	11.5	meager	Solid
	B	1976	11.7	meager	Solid
27	A	154	11.9	meager	Flaw
	B	208	11.3	meager	N/A
28	A	1922	12.1	none	Solid
	B	478	12.0	none	N/A
29	A	2826	11.4	moderate	Flaw
	B	898	11.9	meager	Flaw
30	A	417	11.2	meager	Flaw
	B	1144	11.7	meager	Flaw
	C	495	10.8	moderate	N/A
	D	1041	11.8	moderate	N/A
31	A	1373	12.1	moderate	Flaw
	B	1150	12.1	none	Solid

N/A - Not Available

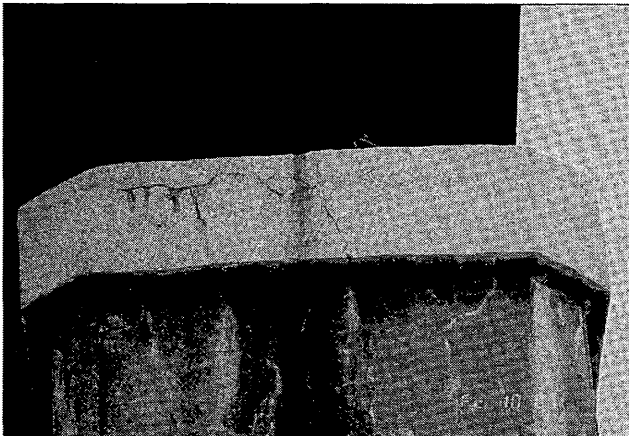


FIGURE 3 Observed deterioration of pier at Site 10 (chromium site).

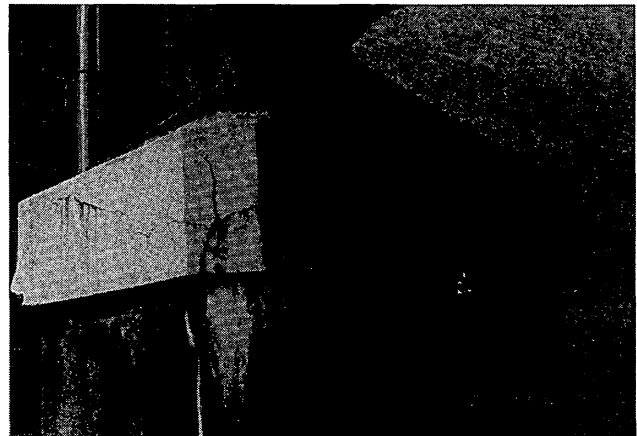


FIGURE 4 Observed deterioration of pier at Site 11 (nonchromium site).

This pier cap was directly under an expansion joint in the bridge deck, and the observed deterioration was clearly corrosion related, as evidenced by rust stains that emanated from the cracks.

Figure 6 indicates aggressive deterioration of the piers, approximately 3 to 5 m above grade, where cracking along the longitudinal corner bar of the pier and signs of previous epoxy injection occurred. Cl^- Sample B from Site 30 in Table 4 was taken from the location shown in Figure 6. This high Cl^- level concomitant with measured half-cell corrosion potentials up to -575 mV clearly indicates that the observed deterioration is from chloride-induced corrosion. There is no indication that the deterioration, by any measure, is related to contact with COPR.

It is again noted that the soil taken from Site 30 (see Table 3) indicates a high level of chloride as well as chromium. As previously discussed, the chlorides in the soil are in all probability from bridge deck runoff, which contained chemical deicers.

SUMMARY AND CONCLUSIONS

An evaluation of 31 selected bridges in and around Hudson County, New Jersey, was conducted to investigate the general condition of

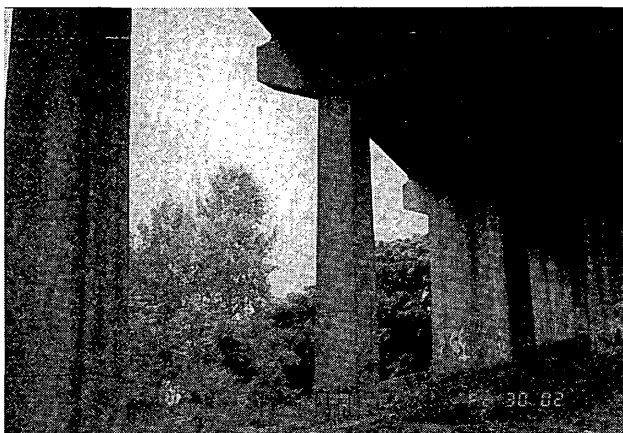


FIGURE 5 Site 30 showing piers mounded by COPR.

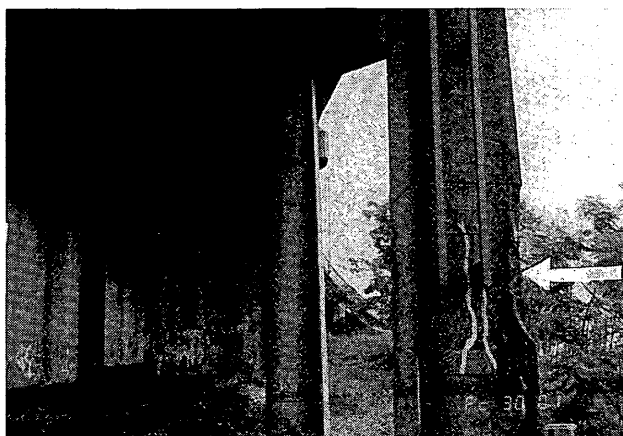


FIGURE 6 Corrosion-related deterioration of bridge pier at Site 30.

the structures as related to structural integrity and durability and to assess the causes of observed deterioration. The visual survey of these 31 bridges was followed by more in-depth testing of 15 of these bridges. Seven of the bridges are located in chromium sites as determined by measured Cr(VI) or total Cr concentration in the surrounding soil, exceeding health-based guidance levels in New Jersey. Testing of selected concrete bridge piers at 15 sites included determination of Cl^- content and pH, impact-echo testing to assess general structural integrity, determination of the presence of ASR, and, where possible, measurement of half-cell corrosion potentials.

On the basis of visual survey and results from the various types of nondestructive testing, it is concluded that deterioration of the concrete bridge substructures is clearly independent of site soil conditions and, in particular, the concentrations of Cr(VI) and total Cr. The general forms of observed deterioration were cracking and spalling caused by corrosion and freeze-thaw deterioration. There was also evidence of cracking at some piers caused by ASR.

The more salient specific conclusions from the investigation are summarized as follows:

1. A previously purported concrete deterioration mechanism associated with chromium is believed to be incorrect (11). That postulated mechanism, related to crystallization of salts, is believed to be appropriate for naturally occurring salts in the constituent materials of porous products such as masonry and, to a lesser degree, concrete. The migration of chromium from the surrounding soil into the concrete to a level associated with crystallization of salts and subsequent generation of tension stresses from expansion as a result of crystallization to a degree to cause cracking and deterioration is highly improbable. In any case, no deterioration was observed on the bridge structures included in this study that could be associated with this type of deterioration mechanism. Moreover, visible signs of chromium salt residue on the concrete, which would suggest that there had been chromium migration and deterioration caused by stresses generated by salt crystallization, were not present.

2. There is no evidence in the study bridges that crystallization damage associated with chromium intrusion into concrete has occurred in the concrete bridge substructures in contact with the soil. Even if chromium migration has occurred, it is evident from the lack of visible deterioration associated with crystallization that the pore size and pore structure of the substructure concrete accommodated any crystallization that occurred on evaporation at or near the surface of the concrete.

3. A comparison of the test results clearly indicates that most of the observed deterioration of the concrete piers was related to corrosion of embedded reinforcement, independent of soil conditions. On average, the measured Cl^- content in the concrete bridge piers in both chromium and nonchromium sites exceeded the chloride corrosion threshold. This is clear evidence that there has been significant Cl^- ingress over 40 years or more of service from salt water spray and runoff from the use of deicer chemicals on the bridge decks. This is also the reason for the relatively high Cl^- levels measured in some bridge site soil samples.

4. The observed deterioration of the concrete piers at the site with the highest concentrations of Cr(VI) (Site 30) was determined to be from chloride-induced corrosion. Chloride levels in some samples exceeded 1,000 ppm, and half-cell potentials on the order of -575 mV were measured.

5. To a lesser degree, some of the observed deterioration can be attributed to ASR. In several cases, the ASR was severe enough to cause cracking in the piers.

6. Neither distress nor deterioration that could be related to foundation displacement was observed in the bridge piers.

7. There was no evidence that observed deterioration of the bridge piers selected for study was severe enough to be of immediate structural concern. This was substantiated to a degree by the results from the impact-echo testing, which indicated that flaws were not deep rooted, leaving a basic core of sound concrete.

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Publication of this paper sponsored by Committee on Dynamics and Field Testing of Bridges.