

Research Report
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SPECIALLY CONSTRUCTED BRIDGES: ACTIVITIES FOR
FISCAL YEAR 1983

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16. Abstract The field performance of bridge features including masonry coatings, galvanized steel, weathering steel, conventional deck reinforcing steel, and epoxy-coated reinforcing steel were examined either visually or nondestructively on various pre-selected bridges throughout Kentucky. Additionally, a nationwide survey of state highway authorities was conducted on the application and service performance of stay-in-place forms. All of the bridge features inspected appeared to be performing satisfactorily, except for the masonry coating failure on the I-471 twin bridges over the Ohio River at Newport. Results of the stay-in-place form survey support their application as a cost-saving feature.					
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INTRODUCTION

Seven topics were designated for investigation in fiscal year 1983 by the study advisory committee:

1. Continue comparative long-term corrosion-potential testing of bridge decks containing epoxy-coated reinforcing steel and of those having uncoated reinforcing steel.
2. Inspect and monitor segmental construction practices, weathering steel bridges, zinc-coated steel bridges and overlays (bituminous, low-slump, and latex).
3. Evaluate masonry coatings on concrete bridge elements.
4. Survey other states to determine current practices and attitudes relative to the use of stay-in-place forms.
5. Determine cost comparisons between modular expansion joints and other expansion joint details.
6. Monitor construction work on bridges employing reinforced-concrete approach slabs.
7. Monitor construction work involving super water-reducers.

The first five topics were investigated during the 1983 fiscal year. However, activity was deferred on the other items. The following is a description of work performed on each major topic.

CORROSION POTENTIAL TESTS OF BRIDGE DECKS

Since the mid-1960's, a number of steps have been taken in Kentucky to prevent premature bridge-deck failure due to corrosion of the reinforcing steel. Those methods were directed toward improving barrier effects to prevent deicing salts from penetrating from the deck surface into the concrete and contacting the reinforcing steel. Salts promote the electrochemical corrosion of steel and subsequently cause bridge deck failures (1). Those methods included the use of 2.0 - 3.0 inches of Class AA concrete cover over conventional reinforcement as well as latex and low-slump concrete overlays. In the early 1970's, FHWA-sponsored work led to the use of epoxy as a protective coating for steel reinforcing bars. Presently, some 40 states, including Kentucky, have accepted epoxy coating as the prime method to prevent corrosion of reinforcing steel.

Kentucky has a number of experimental bridges constructed within a few years of each other that employ different types of corrosion protection. The conventional type of protection incorporates the use of the epoxy-coated reinforcing steel. The earlier type uses 2.0 inches of AA concrete over uncoated reinforcing steel. Also, corrosion potential records exist on several experimental bridges (2). It is desirable to take advantage of those factors and

continue corrosion potential tests of those bridges with and without epoxy-coated reinforcement on a comparative basis. The long-term performance capabilities of both approaches to deck reliability have not been determined, and notable cost differences exist between the two corrosion-protection schemes.

Two bridges of each corrosion-protection type were selected for continued corrosion potential testing. One bridge of each type carries substantial traffic volumes and loadings. The other bridges are subject to rural low-volume traffic and carried relatively light loads. The high-volume bridges were chosen because deck cracking was more severe on those types of bridges. Localized reinforcement corrosion may be expected to be prevalent at points under cracks since deicing salts would readily penetrate cracked concrete. Therefore, those bridges would be the most likely to show the earliest signs of bridge-deck deterioration. The rural bridges were subject to fewer deicing treatments and possessed fewer noticeable deck cracks. Those bridges would be expected to reveal the durability of those corrosion-protection schemes under the most favorable service conditions (atmosphere dependent, service independent).

Experimental bridges that employed conventional reinforcing steel and a 2-inch concrete cover were the I-64 bridge over Elkhorn Creek, completed in 1971, in Scott County (heavy service) and the Yarnalton Road bridge over I 64 completed in 1972, in Fayette County (light rural service). The experimental bridges using epoxy-coated reinforcing steel were the KY-80 bridge over Buck Creek, completed in 1978, in Pulaski County (heavy service) and the County Road 5381 bridge over Panther Creek, completed in 1976, in Daviess County (light rural service).

Corrosion potential tests were first conducted on the I-64 bridge over Elkhorn Creek in 1973. The Yarnalton Road bridge also was tested initially in 1973. The KY-80 bridge was tested first in 1978. The KY-56 bridge was originally tested in 1976. None of those tests revealed any incipient corrosion problems in the reinforcing steel.

Follow-up corrosion potential tests performed in fiscal year 1983 revealed no signs of active ferrous corrosion on the reinforcing steel of any of the bridges (Figures 1-4). However, corrosion potential measurements for all of the bridges had increased over values originally measured.

In the future, some reinforcing-steel corrosion may be anticipated on one or both types of bridges. It is desirable to determine when it occurs and also the relation between subsequent reinforcement corrosion and associated pop-out problems.

Epoxy coatings are subject to undercutting failures by rust creep after experiencing initial corrosion. When corrosion once begins on epoxy-coated reinforcing steel, the rate of rust creep and the magnitude of corrosion of the underlying steel will strongly affect the subsequent amount of deck damage. Figure 5 shows a segment of epoxy-coated reinforcing steel on the Illinois approach of the Irvin S. Cobb Memorial Bridge (US 45 at Brookport, Illinois) that was exposed by a vehicle collision with the bridge. Dark spots on the reinforcing bar are locations where the epoxy has stripped from the steel. Rust also was evident on the reinforcing bar.

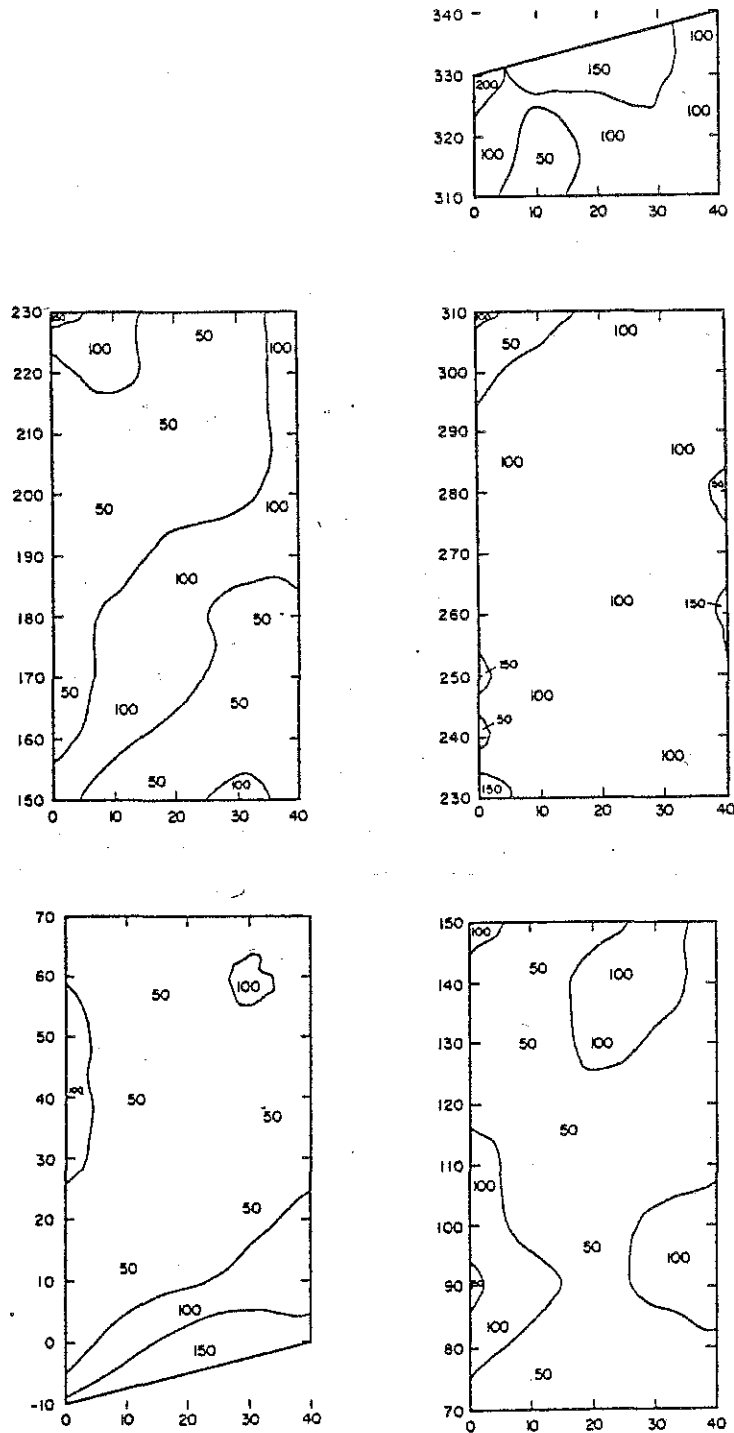


Figure 1. Corrosion Potential Test of the KY-80 Bridge over Buck Creek, February 21, 1983.

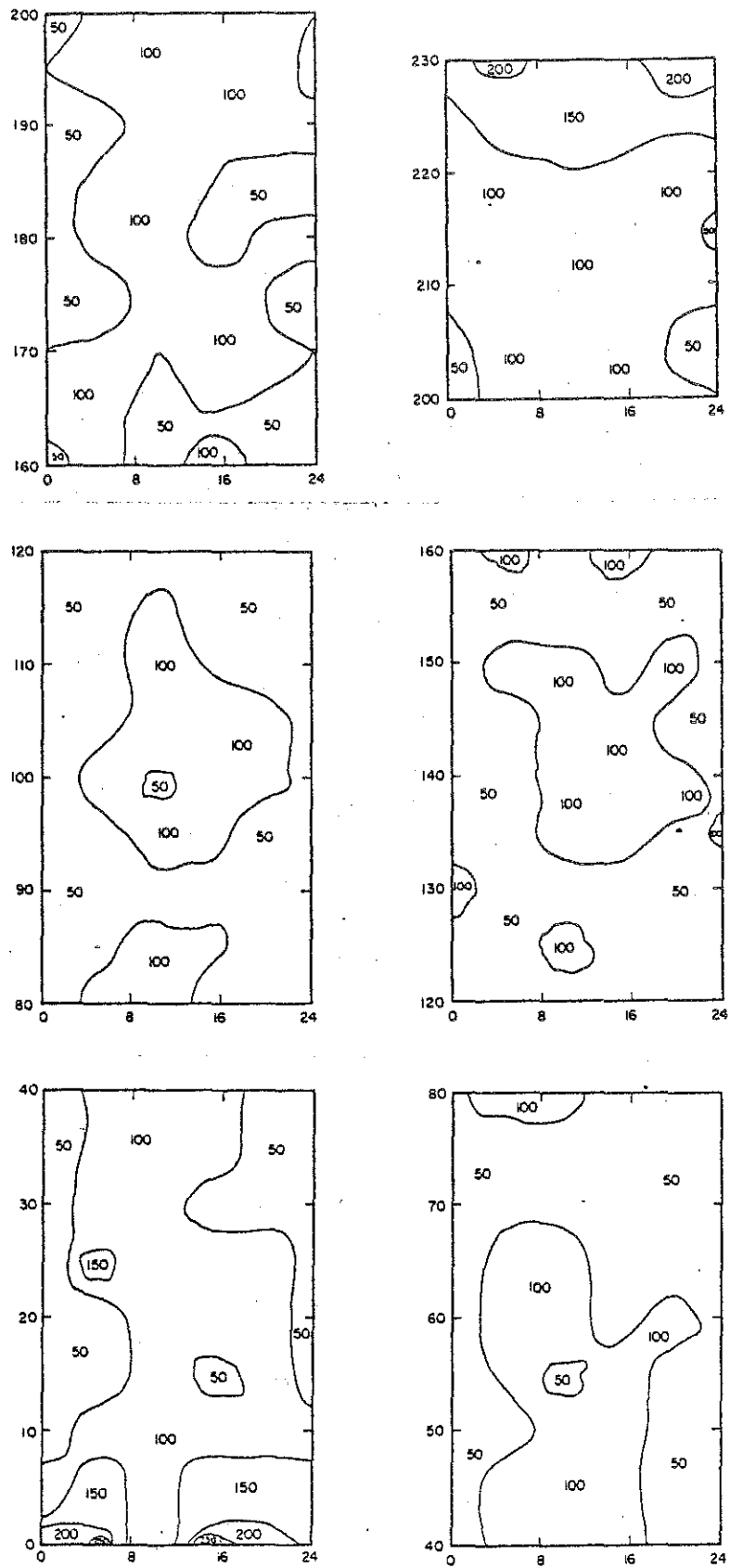


Figure 2. Corrosion Potential Test of the KY-56 Bridge over Panther Creek, October 13, 1982.

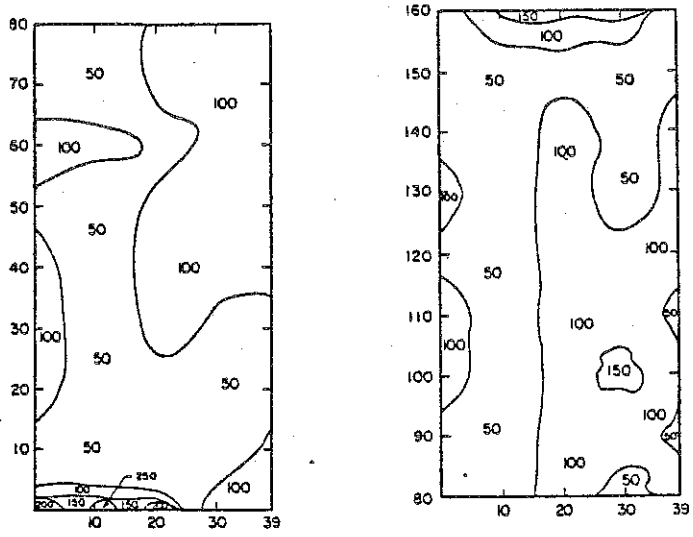


Figure 3. Corrosion Potential Test of the US-460 Bridge over Elkhorn Creek, February 20, 1983.

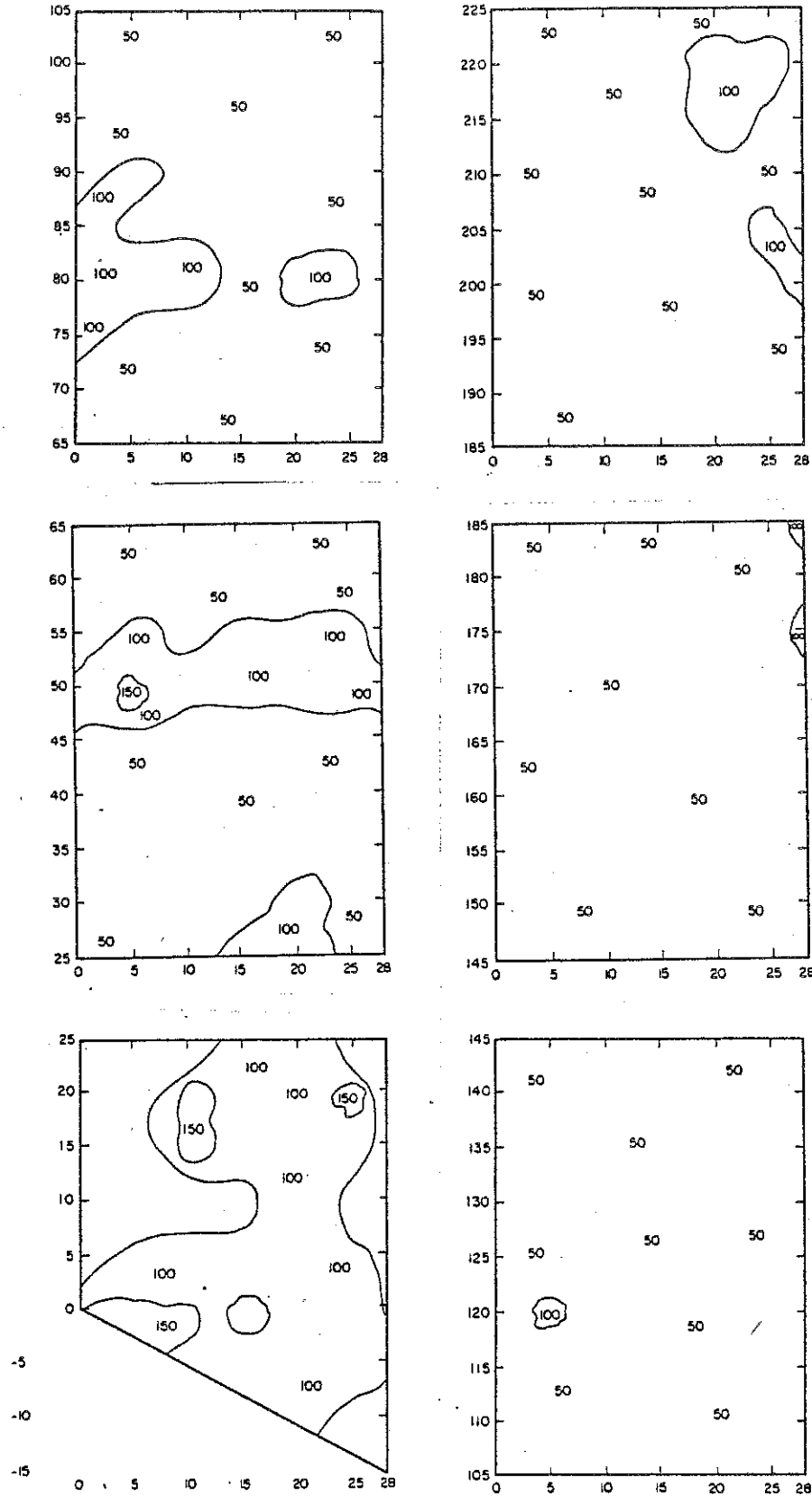


Figure 4. Corrosion Potential Test of the Yarnalton Bridge over I 64, February 20, 1983.

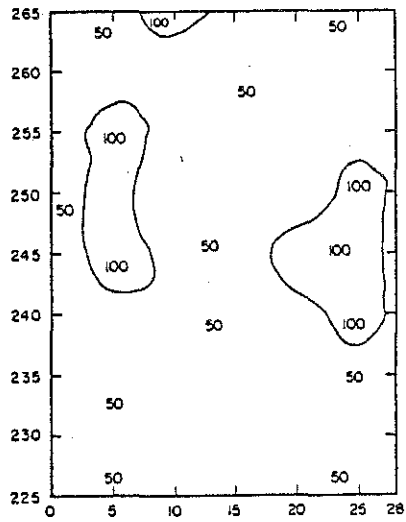
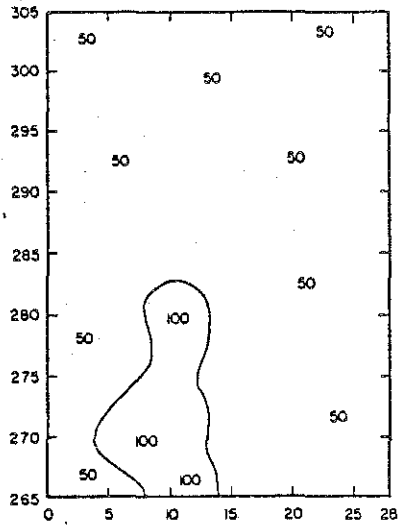
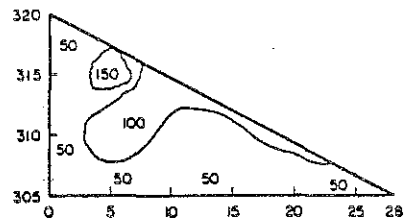


Figure 4. (continued)



Figure 5. Broken Epoxy and Rust on Exposed Reinforcing Steel, US-45 Bridge at Brookport, IL (1982).

While circumstances of exposure indicated that mechanical action might be involved in stripping the epoxy, it also was possible that the bond between the epoxy and steel may not be very strong in some instances. The relative newness of this type of corrosion-prevention treatment and lack of long-term service testing prior to its widespread application allow for the possibility of some eventual corrosion problems.

Thick concrete cover over conventional reinforcement may be mitigated by deck cracking, especially on heavy-service bridges where cracks open wider than on light-duty bridges. Also, heavy-service bridges may be subjected to more frequent deicing treatments, yielding greater salt concentrations in the deck concrete and subsequently more severe deck pop-out problems. Long-term monitoring between conventionally reinforced bridges of different service levels should reveal the effects of those factors. Comparative testing of decks employing epoxy-coated reinforcing steel having the same cover as uncoated reinforcing-steel decks will aid in determining whether the extra expense of epoxy coating is justified, especially on low-service bridges.

WEATHERING STEEL BRIDGES

Construction-grade, high-strength, low-alloy steels have been used on bridges since 1915. Weathering grades of that type of steel were achieved by adding copper in amounts varying from 0.20 to 1.00 percent, along with other alloying elements including phosphorous, chromium, and manganese. The ability of those steels to form a dense adherent rust that acts as a self-healing barrier against further corrosion allows those steels to be used without paint or other protective coatings. The weathering grades of ASTM high-strength, low-alloy steels have four to eight times the uniform corrosion resistance of normal low-carbon construction steels (3).

Weathering steels have been utilized in unpainted structures since the early 1960's. In those applications, extra section is sometimes provided to allow for the gradual uniform corrosion over the estimated life of the structure. The first unpainted weathering-steel bridge was constructed in 1968. Thereafter, numerous bridges were constructed using the feature. The more notable bridges include the world's longest steel-arch bridge (1,700-foot span), the New River Gorge Bridge in West Virginia, and the towers of a cable-stayed bridge (I 310) at Luling, Louisiana.

In March 1980, the Michigan Department of Transportation declared a statewide moratorium on the use of weathering steel in its highway program. Shortly thereafter, the American Iron and Steel Institute organized a task group to study the problem. Forty-nine bridges in Michigan, Illinois, Maryland, New York, North Carolina, Wisconsin, and New Jersey were inspected by state and federal engineers and steel industry experts. Of those bridges, 30 percent showed good performance in all areas, 58 percent indicated good performance with moderate corrosion in some areas, and 12 percent showed good overall performance with heavy corrosion in some areas (4).

Corrosion problems with weathering steels were caused by standing or pooled moisture and may be aggravated by the presence of deicing salts (5). An industry source stated that unpainted weathering steel should not be used in salt-water environments nor with open finger-type expansion joints. He mentioned that, on inland applications, moisture and salts leaking down from expansion joints had caused many of the more severe problems detected in the American Iron and Steel Institute (AISI) study. Ohio had successfully treated their problem areas by painting the weathering steel girders within five feet of the expansion joints.

The Kentucky Department of Highways has three unpainted weathering-steel bridges under its authority, including the Scenic Bridge on County Road 5418 over the Green River Parkway in Barren County, constructed in 1972; the equestrian bridge over I 64 in Louisville, constructed in 1970; and the KY-1893 railroad overpass bridge at Shawhan in Bourbon County, constructed in 1977.

On October 14, 1982, the Scenic Bridge was inspected (Figure 6). That bridge was a conventional deck-girder bridge (as were the other unpainted weathering-steel bridges owned by the Department). The surface corrosion product (rust) showed some slight pitting. However, the rust was tight. It was performing satisfactorily (Figure 7). The concrete deck had effloresced slightly and some faint streaks were visible on girders. Rust washed from the steel had stained the abutment pedestals. A loud knocking sound could be heard when traffic passed over the bridge; however, the source of the noise could not be located.

The equestrian bridge at Louisville was inspected on October 15, 1982 (Figure 8). As with the Scenic Bridge, the weathering steel was performing satisfactorily. A rolling seam was detected on the center column near the base of the column. One small spot was found inside the center column, near the base, where the weathering steel had performed like stainless steel and had not rusted (Figure 9). At another location, a small patch of corrosion product had a bright orange appearance similar to common ferrous rust (Figure 10). However, that rust spot was as tight as the normal dark rust on the remainder of the bridge. Some faint efflorescence stains and graffiti were present on exterior surfaces of the fascia girders. Also, rust stains were visible on faces of the abutment pedestals.

The weathering-steel bridge (KY 1893) at Shawhan was inspected on November 12, 1982 (Figure 11). The weathering steel was in good condition away from the bridge abutments (Figure 12). However, at both ends of the bridge, the girders, rockers, and bearing plates showed signs of abnormal corrosive damage in the form of scaly, loose rust. That corrosive activity was worse on interior girders and interior faces of fascia girders. On the beams, corrosive damage was worse on the lower portion of the webs and the upper horizontal faces on the lower flanges. That damage appeared to be limited to 5-foot portions on the ends of the beams (Figures 13 and 14). Stains on the webs of girders indicated water was leaking through the bridge deck and settling on the lower portions of the beams.

A second inspection was conducted on the bridge several weeks later. Rain had fallen in the area for several days and was occurring during the inspection. Water was observed seeping through



Figure 6. The Scenic Bridge over the Green River Parkway in Warren County.

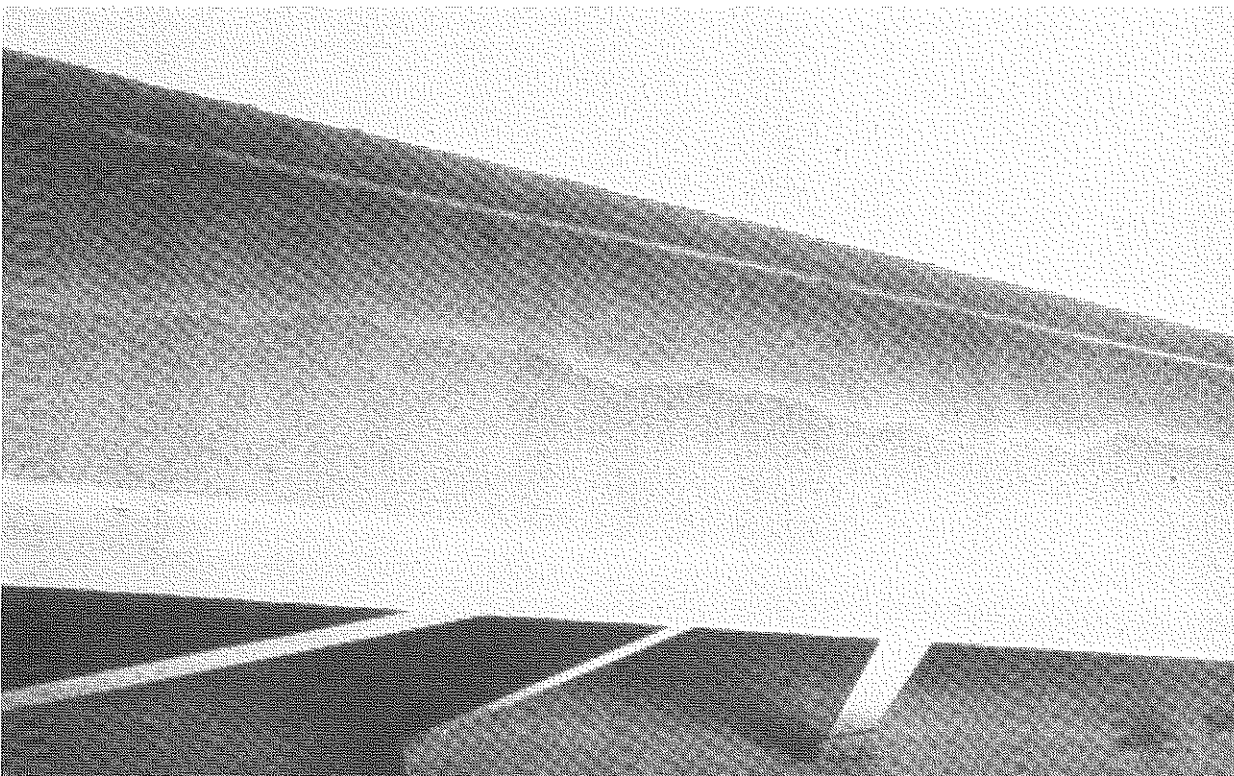


Figure 7. Weathering Steel on the Scenic Bridge, which Is Performing Satisfactorily.

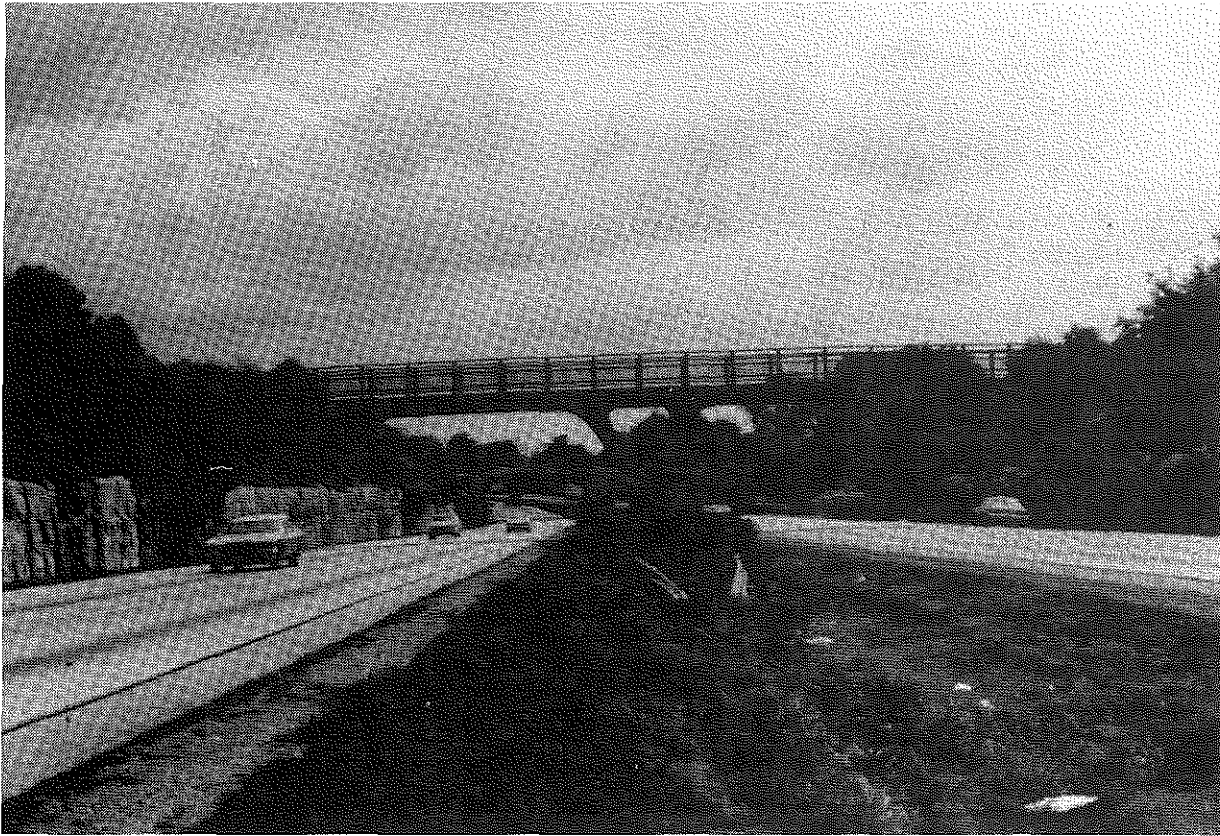


Figure 8. The Equestrian Bridge over I 65 in Louisville.



Figure 9. Alloy Segregation Spot on the Equestrian Bridge Showing No Visible Corrosion.

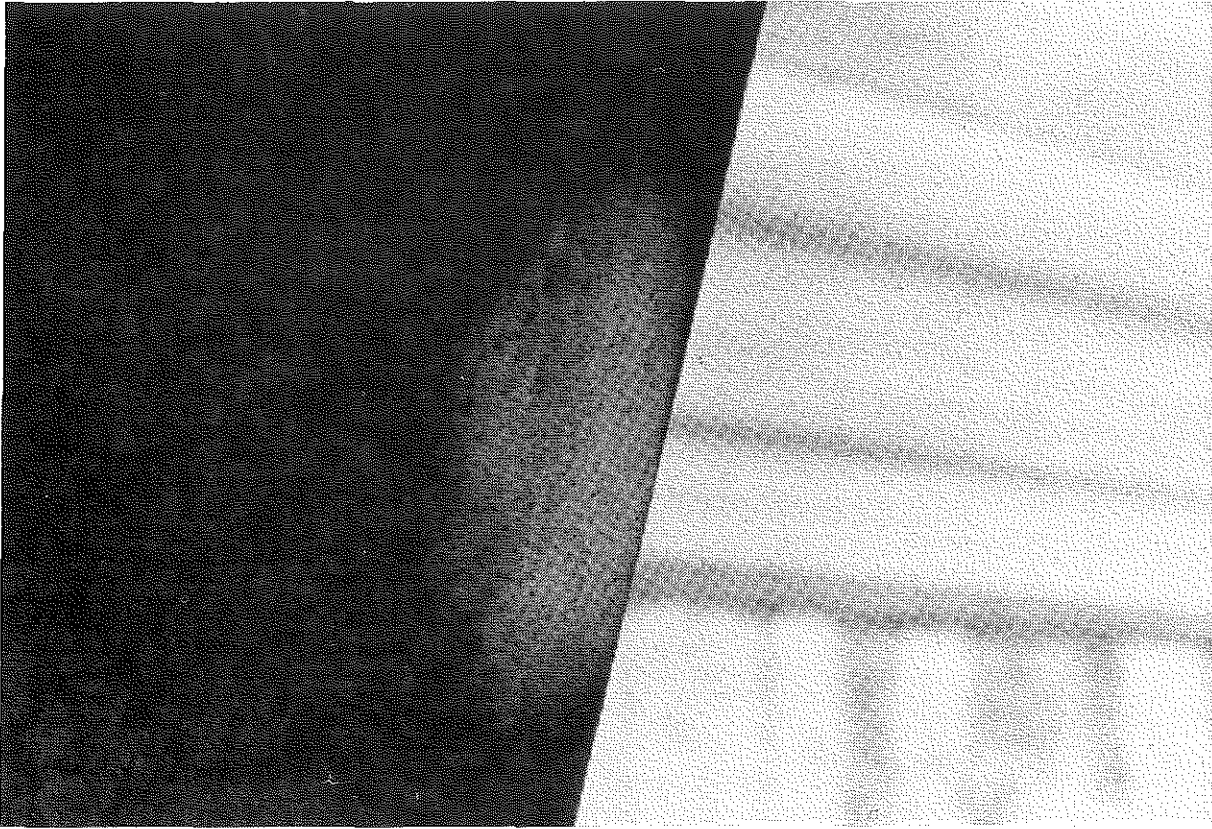


Figure 10. Alloy Segregation Spot on the Equestrian Bridge Showing Apparent Ferrous Corrosion (Rust).

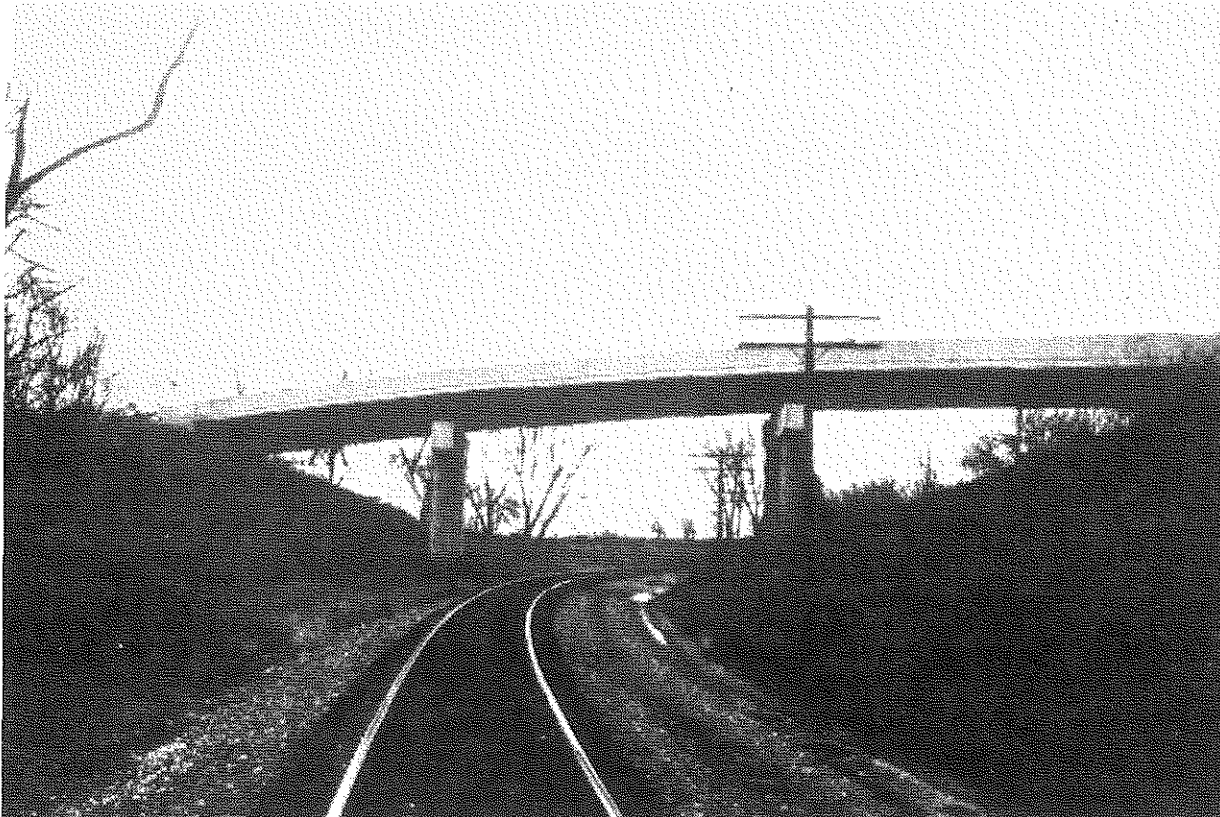


Figure 11. KY-1893 Bridge at Shawhan.

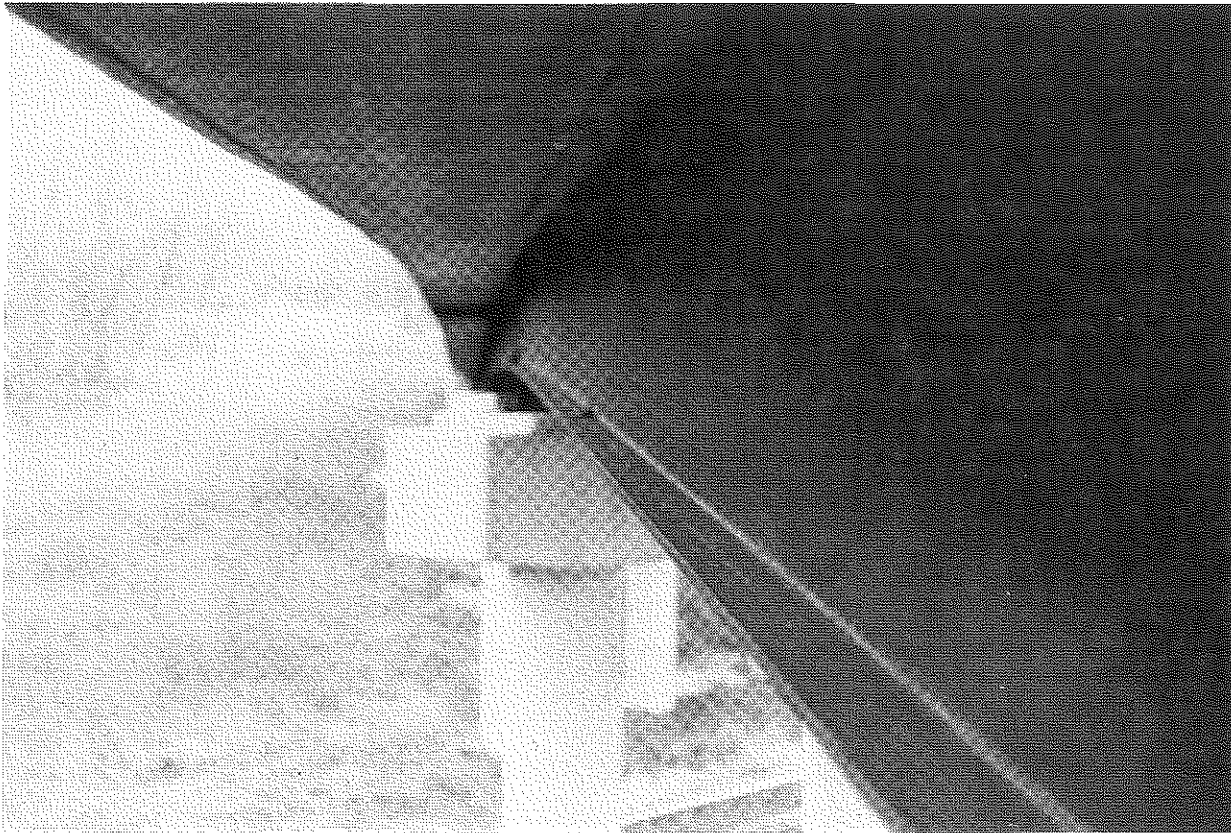


Figure 12. Fascia Girder of KY-1893 Bridge, which Is Weathering Satisfactorily.



Figure 13. Abnormal Corrosion on Interior Girder at East End of KY-1893 Bridge.

the deck and ponding on upper faces of the lower flanges (Figure 15). Also, water was seeping through expansion joints on both ends of the bridge and was being deposited on the extreme ends of the beams and rockers. Those expansion joints were simple units consisting of neoprene and cork.

This inspection confirmed the early suspicions about the entry of water through the bridge deck. The other two weathering steel bridges have similar vertical stains, which ran from the deck-to-beam interface down the web to the bottom flange. However, on those bridges, there were no signs of the unstable corrosion activity detected at Shawhan. One obvious reason is that the other two bridges were more subject to "bold exposure." This term describes the premise to which the weathering steel bridges were constructed. Originators of the steel assumed that wetting and drying cycles would occur in short intervals. Also, they assumed that wind and rain would wash harmful corrodants (usually salts) from the surfaces of the weathering steel. In locations like the Scenic Bridge, that could be anticipated. The good condition of that bridge indicated "bold exposure" conditions were prevalent.

In the AISI study, researchers found that bridge overpasses crossing depressed roadways were subject to corrosion problems due to salt sprays from the roadways. That was especially true when the roadway clearance was less than 20 feet. In many cases, depressed roadways were closely bounded by vertical walls that may have affected the "bold exposure" factor by reducing washing and wind flow around the bridge. Both the Scenic and equestrian bridges abut vertical walls. However, the exposed steel on both bridges was sufficiently distant from the road in both the vertical and horizontal directions to prevent contact with deicing salt sprays. Also, the openings under those bridges were sufficiently wide to promote "bold exposure."

The bridge steel at Shawhan shows "bold exposure" weathering on exterior portions of fascia girders. However, at the abutments, especially on interior girders, conditions for "bold exposure" were not present. Wind flow around interior girders and girder ends, rockers, and bearing plates was insufficient to flush or rapidly dry those components.

The possibility of interaction with deicing chlorides was investigated. Three samples of loose scale from several different locations on the KY-1893 bridge were taken to the University of Kentucky Metallurgical Engineering Laboratory for spectographic analysis. Spectographic examination in a scanning electron microscope revealed the presence of chlorine on the surfaces of each specimen of scale. One specimen was sectioned and chlorine was detected in the interior portion. Sodium was detected in one specimen. Calcium was detected in two other specimens (though that element may be expected to be present in steel). That examination indicated the possible interaction of deicing salts with retained moisture in promoting unstable corrosion.

The Wisconsin Department of Transportation has placed restrictions on the use of weathering steels (6). Weathering-steel bridges are not permitted over roadways. As with the Ohio Department of Transportation, weathering-steel girders are painted within 6 feet of the ends of the structure and expansion joints.



Figure 14. Abnormal Corrosion on Interior Girder at West End of KY-1893 Bridge.

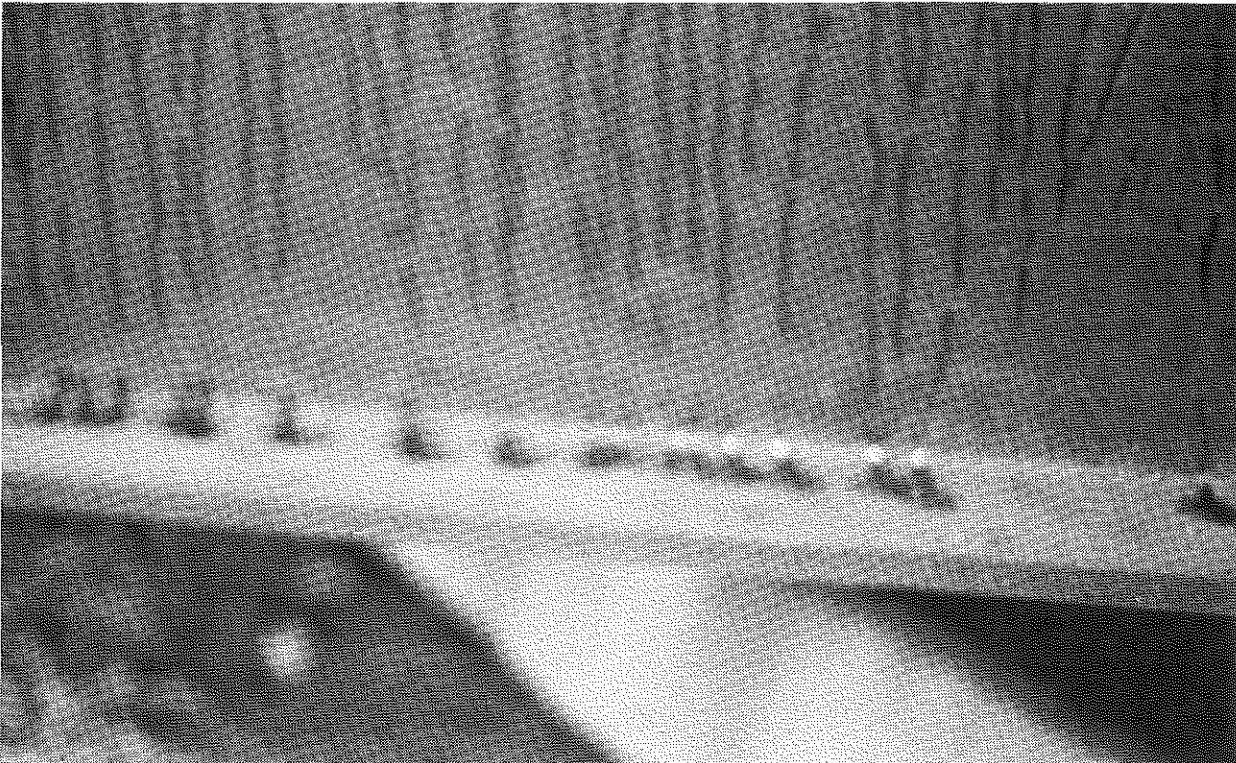


Figure 15. Water Ponding on Interior Girder of KY-1893 Bridge.

Northern states such as Wisconsin and Michigan may be expected to use a greater amount of deicing salts on both underlying roadways and bridge decks. It is doubtful that the equestrian bridge or the Scenic Bridge (in Western Kentucky) are salted frequently. There also is some question as to whether a bridge over a secondary road like the one at Shawhan is salted frequently, if ever.

A more important question arises in reference to the structural integrity of weathering bridges regardless of the existence of unstable corrosion. A recent report (7) by the Maryland Department of Transportation revealed that weathering-steel weldments exhibited lower fatigue lives than anticipated in the AASHTO codes. The decrease in the fatigue lives were less for the more severe category of fatigue detail. While much significant work was contained in that report, the conclusions should be considered tentative, pending further reviews and research.

Major bridges utilizing unpainted weathering steel should not be constructed, based on conclusions of the Maryland Department of Transportation report. Also, it is recommended that bridges having fracture-critical members not be fabricated from unpainted weathering steel. Unpainted weathering-steel bridges carrying major roads should not be constructed until further AASHTO or AISI directives are issued. Unpainted weathering-steel bridges may be considered suitable for the following circumstances:

1. the design is load-redundant,
2. the road should be a rural or secondary highway,
3. heavy loads or frequent loading would be unlikely,
4. the bridge would not tend to be treated frequently with deicing salts,
5. bridges should not be constructed over other roadways unless the weathering-steel members are at least 25 feet from the roadway in the horizontal and vertical directions,
6. weathering-steel bridge members under the bridge deck should be painted within 6 feet of the girder ends and any expansion devices (modular expansion joints or finger dams), and
7. all dimensional clearances about new weathering-steel bridges should be sufficiently large to promote "bold exposure," especially near abutments.

Several minor problems were discovered during inspections of the weathering-steel bridges. Washing of the steel surface deposits unsightly stains on piers and abutments (Figure 16) was noted. On bridges over waterways, that is not important. However, on bridges over roadways, that gives the impression the structure is not well tended. The central pier of the equestrian bridge has a catch basin filled with sand or soil that retains rust and maintains the proper appearance of the pier (Figure 17). That detail should be considered for construction of future weathering-steel bridges that

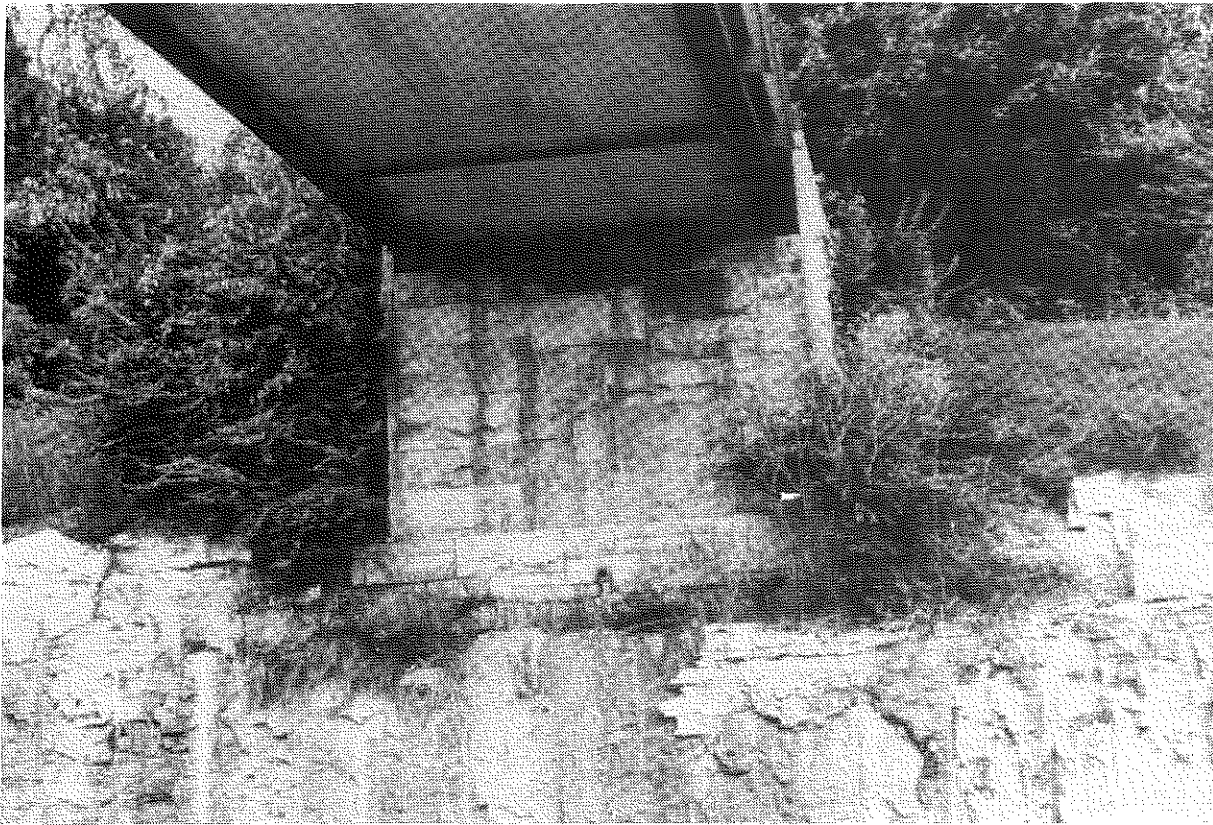


Figure 16. Rust Stain on Abutment of Equestrian Bridge at Louisville

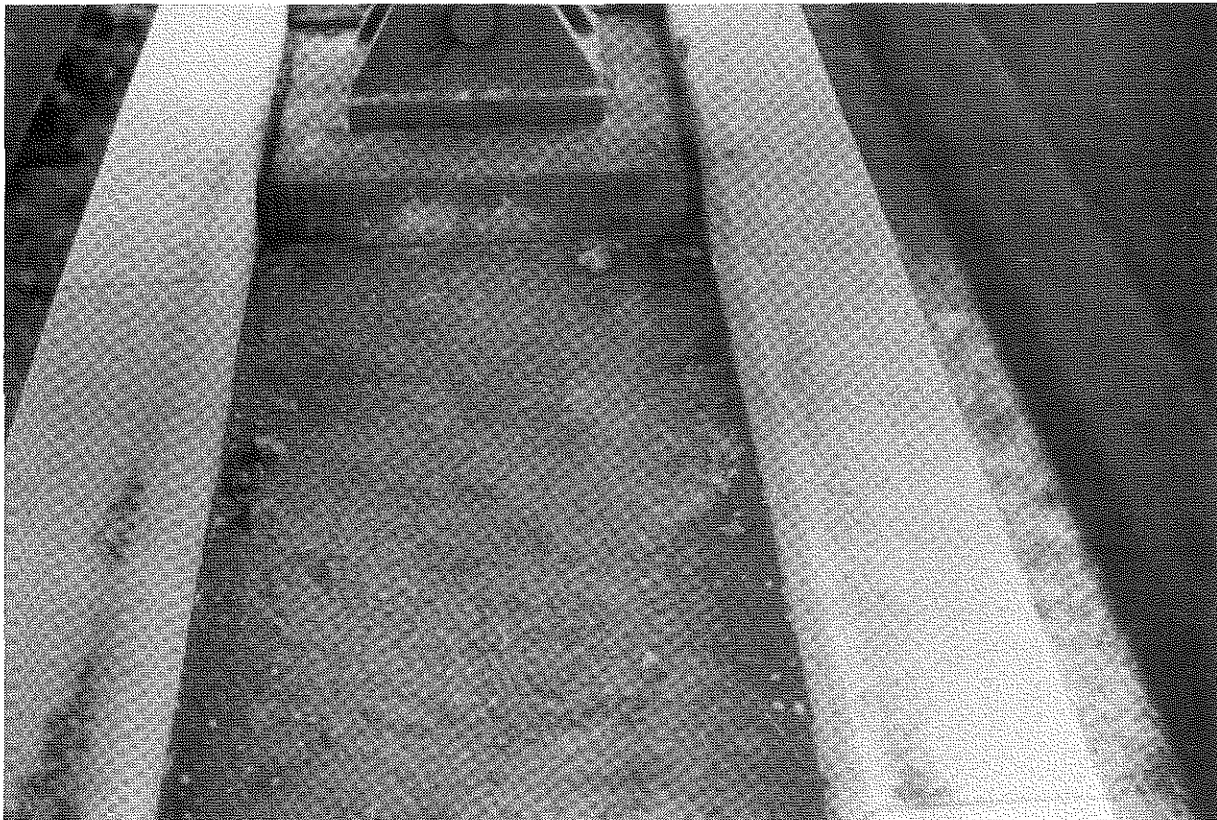


Figure 17. Central Pier on Equestrian Bridge at Louisville.

are visible to the public.

The weathering performance of the Scenic Bridge and the equestrian bridge steel is good at all locations, and those bridges do not require painting for normal service. However, the KY-1893 bridge at Shawhan needs remedial painting at the abutments for a distance of at least 6 feet from the girder ends. That includes the girders, rockers, and bearing plates. The paint should match the color and texture of the weathering steel for appearance. Also, the equestrian bridge at Louisville has spray-paint graffiti on the girders (Figure 18). It is desirable to find a way of obliterating the graffiti without damaging the underlying stable rust. Two possibilities exist: 1) use a chemical paint remover or 2) top coating the graffiti with a weathering-steel-compatible paint.

ZINC-COATED BRIDGES

The I-24 bridge (westbound) over KY 93 near Paducah is a zinc-coated steel deck-girder bridge constructed in 1977. The companion eastbound bridge is a similar but conventionally painted structure. The bridges were inspected on October 13, 1982.

The paint on the eastbound bridge was in very good condition (Figure 19). The only coating defects were some slight rust stains on splice plates on the bottom flanges of the girders.

The galvanized coating on the westbound bridge was generally in good condition (Figure 20). A white zinc corrosion product was observed infrequently at random sites on the bridge. However, at flange-deck interfaces, it was difficult to determine if the white stains were efflorescent stains from the concrete deck or a white zinc corrosion product (Figure 21). One diaphragm showed what appeared to be initial ferrous corrosion (Figure 22). Apparent rust stains were visible on splice plates on the lower flange, similar to the eastbound bridge (Figure 23).

The galvanized coating had several construction nicks that were not overcoated with sprayed-on zinc-rich paint. Also, numerous repair marks indicated some difficulties were encountered in erecting the zinc-coated members (Figure 24). Some apparent dirt was visible on the lower portions of girders. If the dirt was construction-related, it should have been removed prior to erection.

Also, there was a wide variance in the size of zinc spangles from approximately two square inches down to microscopic size (Figure 25). The pattern of zinc spangles on the girders indicated they were too large for the hot-dipping tank and had to be dipped on both ends for complete coverage.

The quality of the paint on the eastbound bridge appeared to be better than the quality of the galvanizing on the westbound bridge. This should be taken into account when future comparisons are made of the conditions of the two bridges.

MASONRY COATINGS

Masonry coatings have been used in Kentucky as a top coating on concrete bridge sidewalls, abutments, wingwalls, and piers since



Figure 18. Graffiti (Spray Paint) on Girder of the Equestrian Bridge at Louisville.

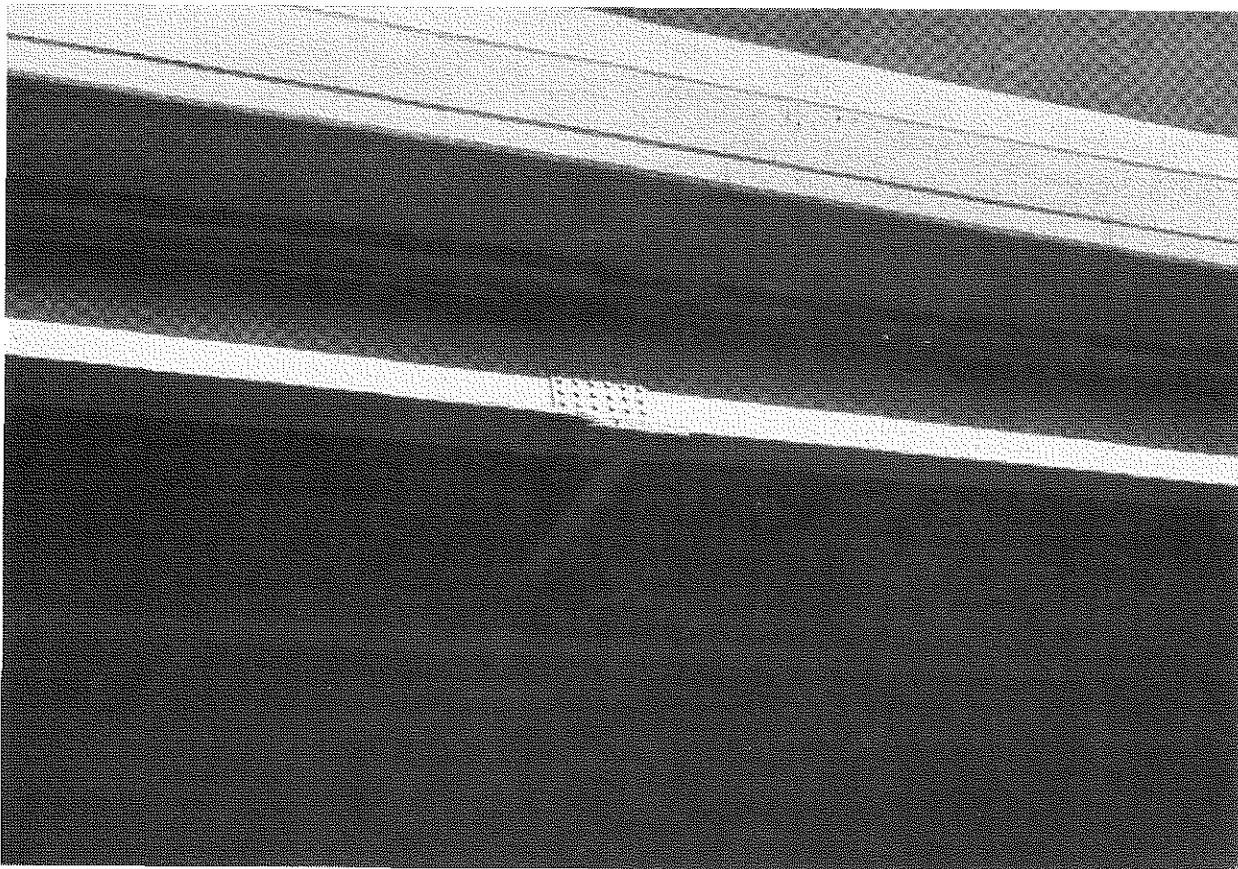


Figure 19. I-24 Eastbound Bridge over KY 93 (Painted Girders).

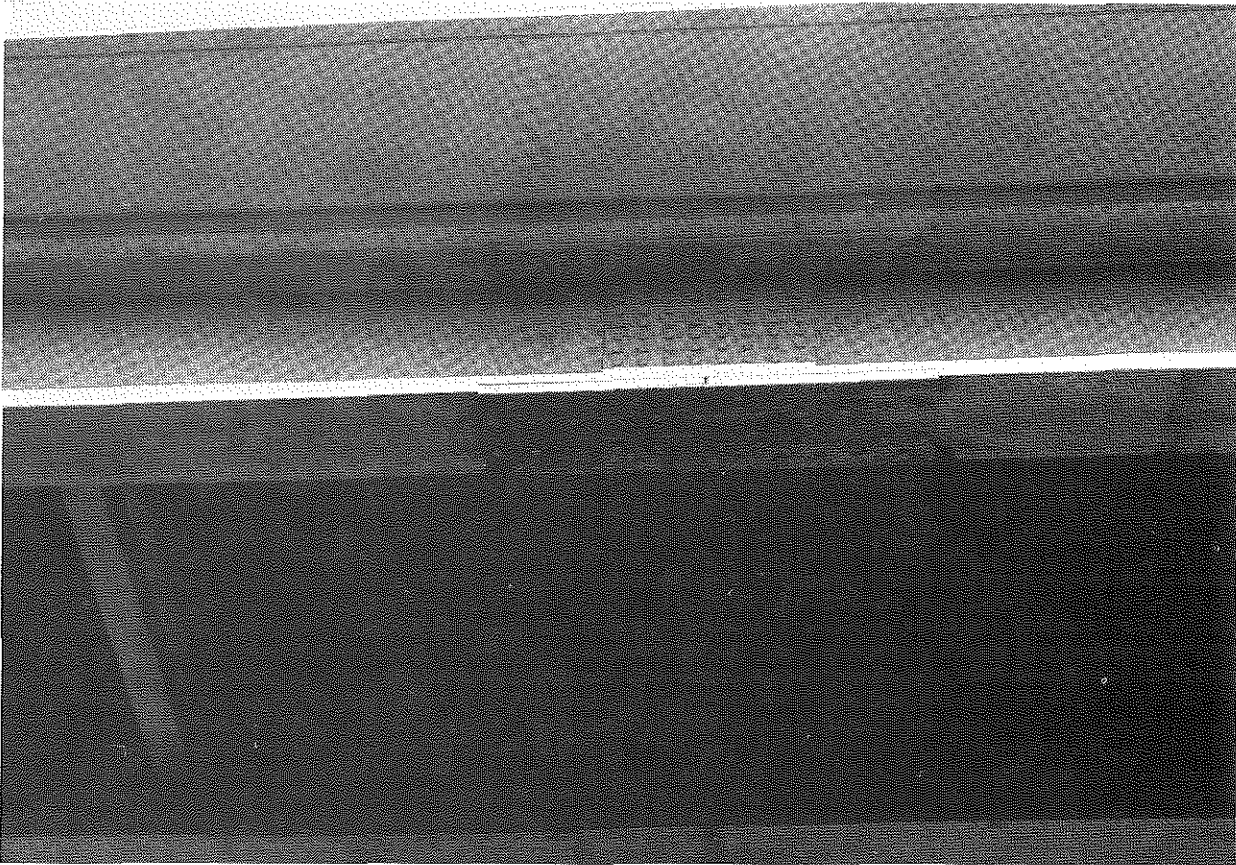


Figure 20. Typical Galvanized Girder on the Westbound, I-24 Bridge.

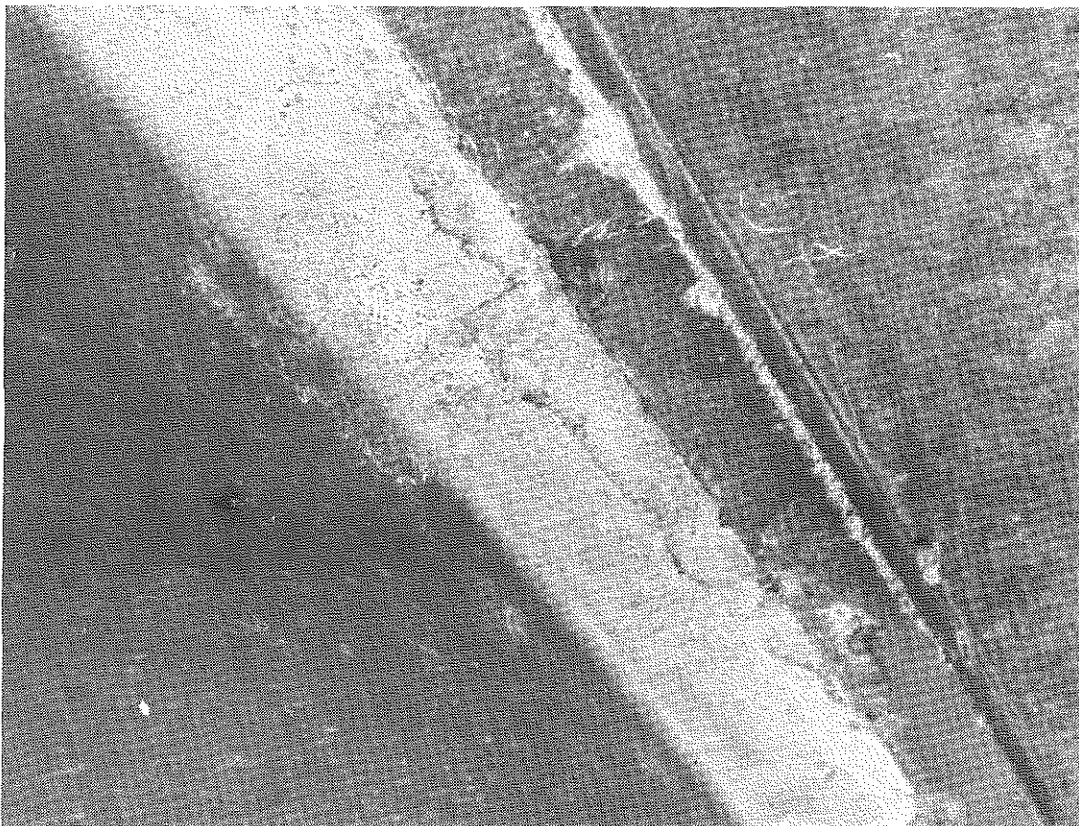


Figure 21. Possible Zinc Corrosion Product or Efflorescent Stain on Galvanized Girder.

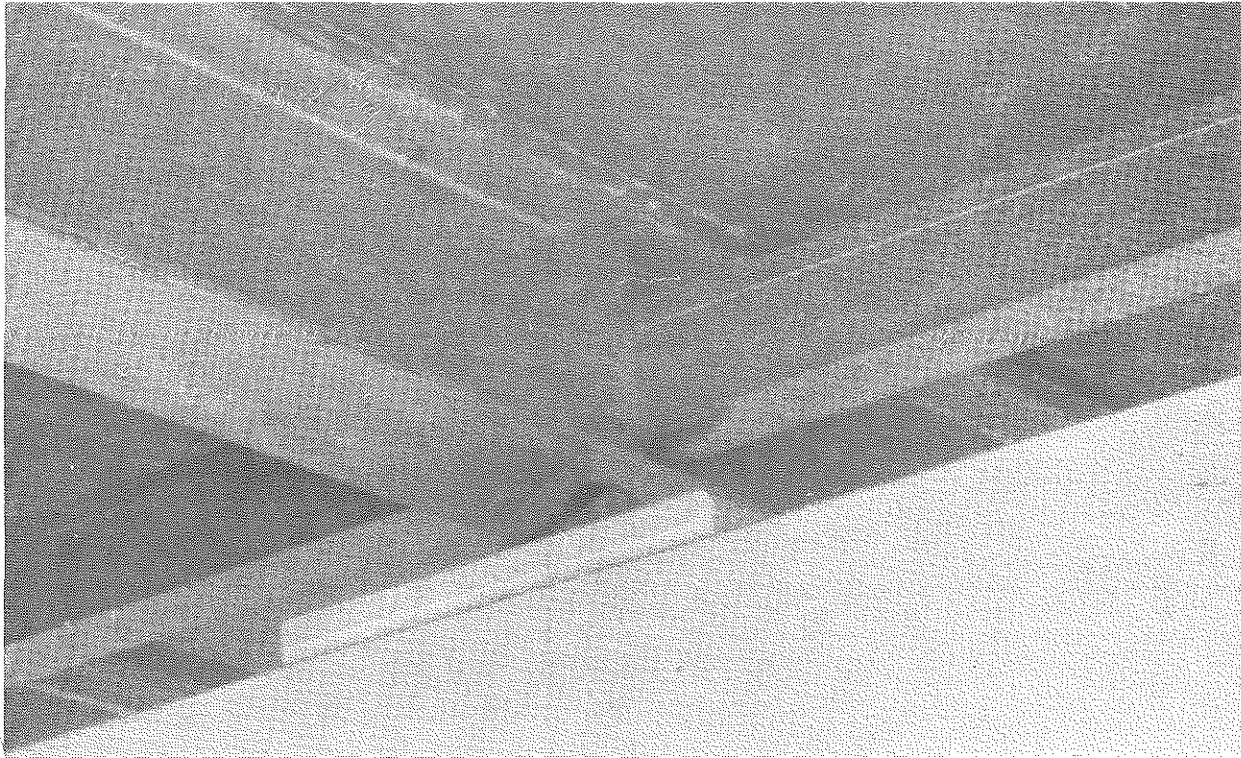


Figure 22. Possible Steel Corrosion Product on Diaphragm of I-24 Westbound Bridge.

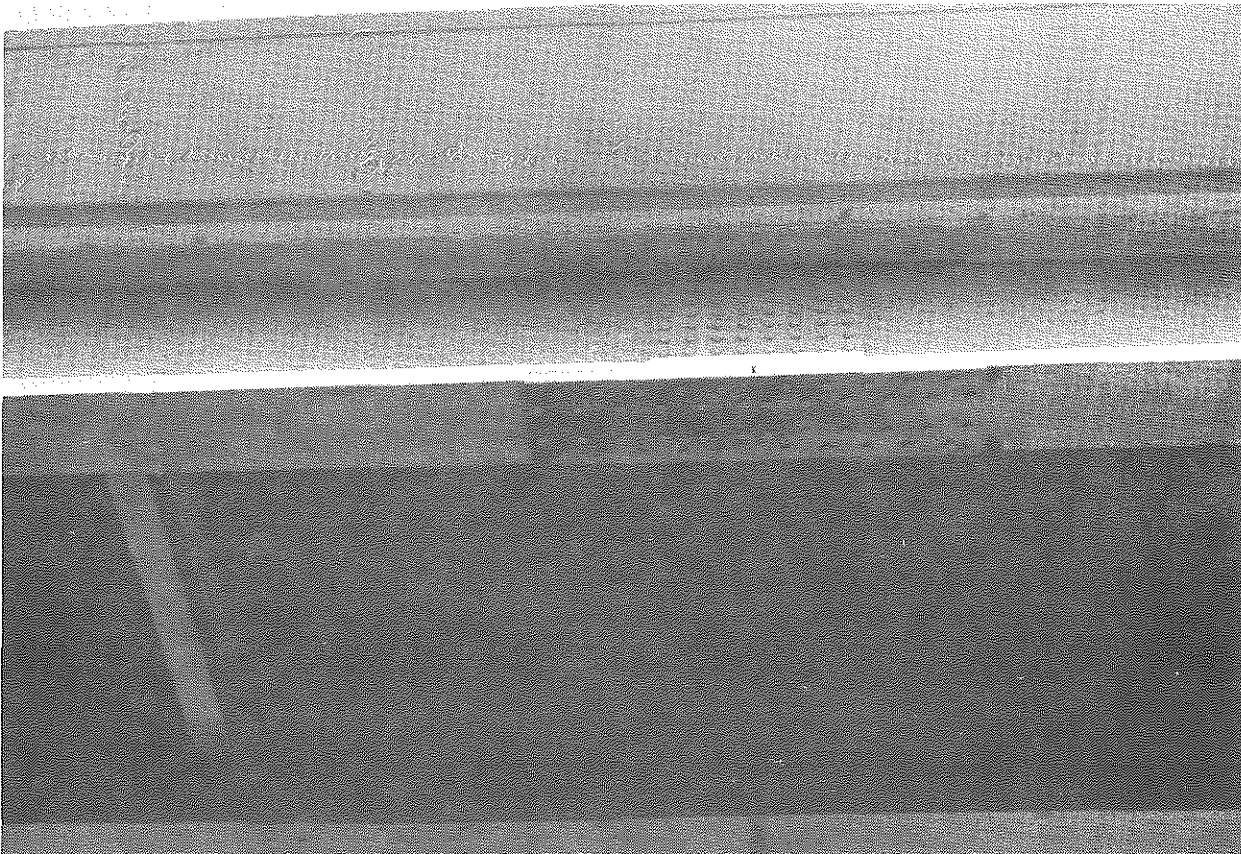


Figure 23. Apparent Rust Stain on Splice Plate and Girder of I-24 Westbound Bridge.

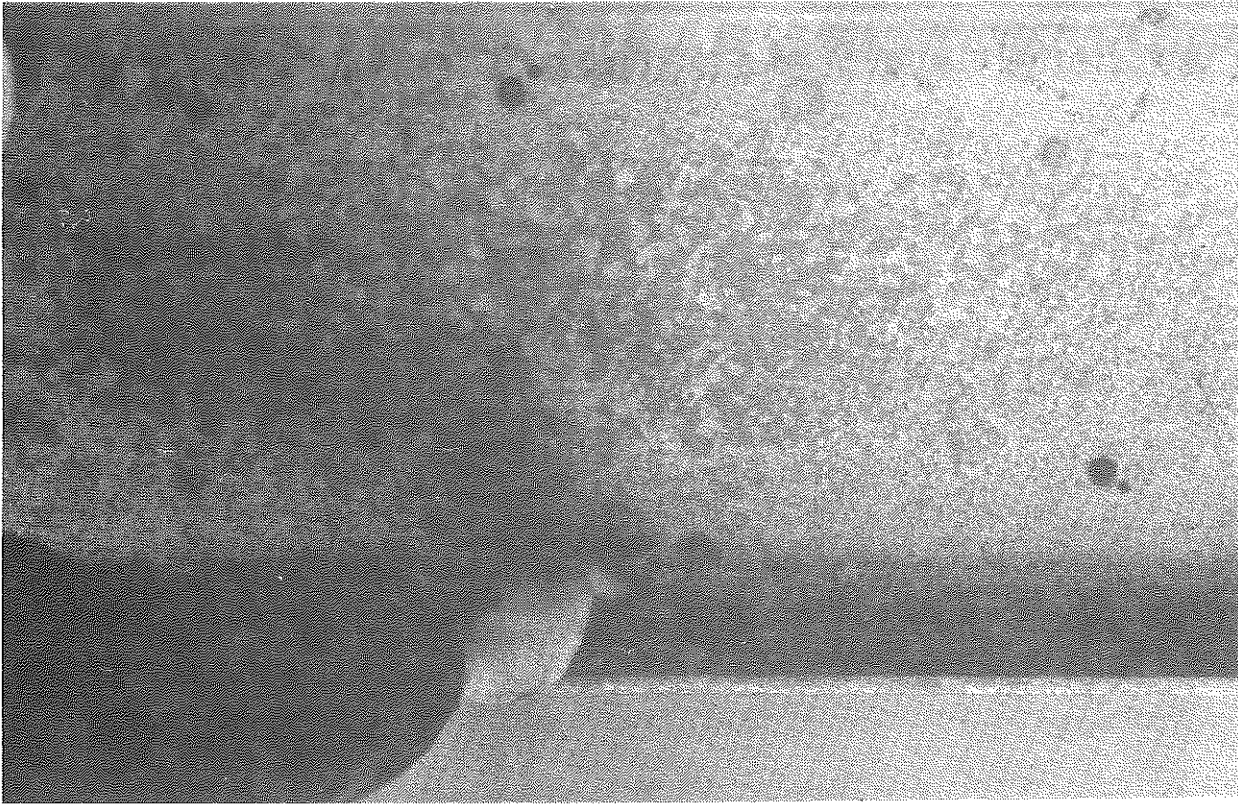


Figure 24. Galvanizing Repair Marks on I-24 Westbound Bridge.

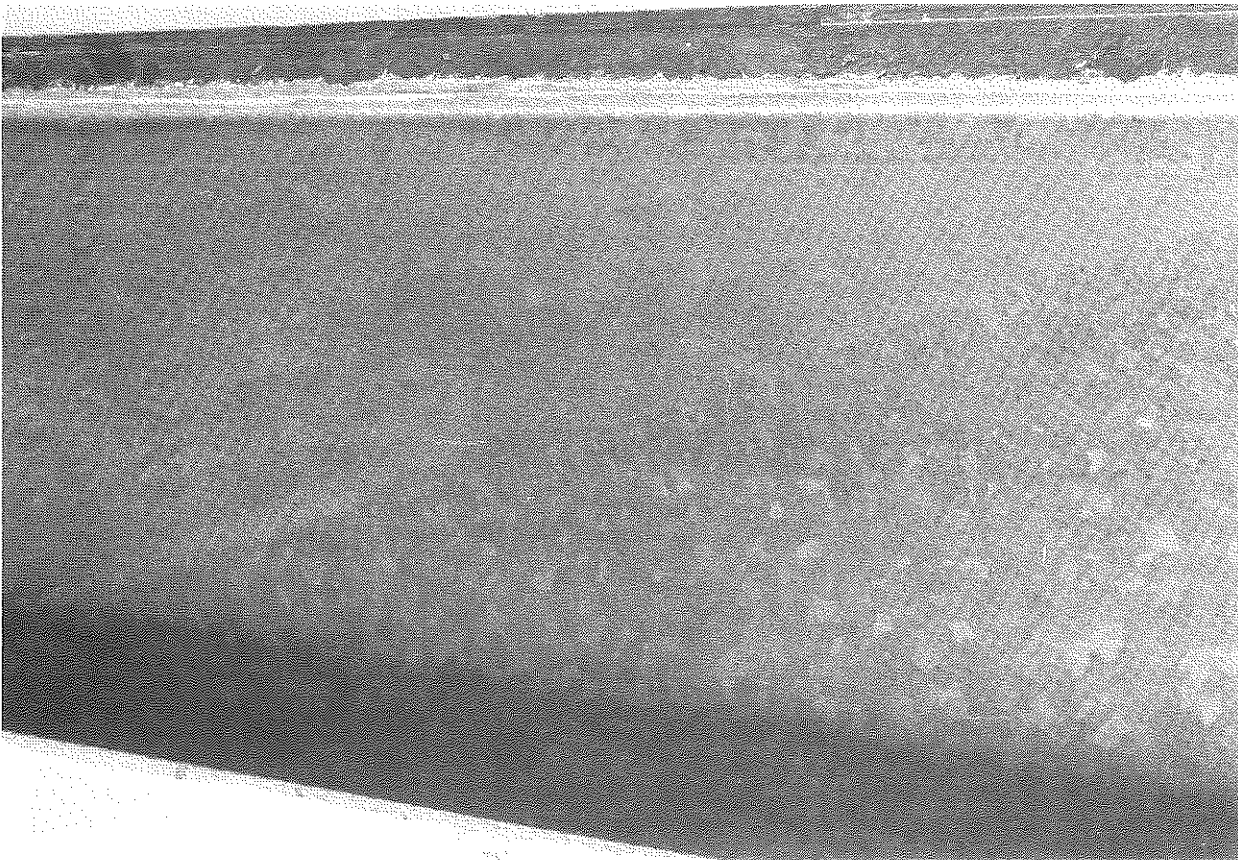


Figure 25. Galvanized Coating on a Girder of I-24 Westbound Bridge Showing a Variance in the Size of Zinc Spangles.

1970. Those coatings have supplanted rubbing as a means of finishing concrete surfaces that have occasional surface imperfections, created during the placement operation. While those coatings were intended to be cosmetic, protective properties (primarily weathering resistance) are sometimes attributed to them (8).

Masonry coatings used on Kentucky bridges are primarily TEX-COTE manufactured by Textured Coatings of America, Inc. and THOROSEAL manufactured by Standard Dry Wall Products, Inc. Purportedly, different formulations of those products are available; however, in general, those products may be described as follows:

1. TEX-COTE -- A synthetic elastomer-polymer base containing fiberglass, asbestos, pearlite, and mica. A pigment is added to produce the desired colors. The material is furnished in containers, pre-mixed, and ready to use. Application is by brushing or spraying.

2. THOROSEAL -- A non-reamulsifying acrylic resin binder used with a cement-base powder with other non-metallic additives. The material is furnished with the binder separate from the base mixture. Water is added to the binder prior to mixing with the base. Application is by brushing or spraying.

One notable failure of a textured coating occurred on the concrete sidewalls of the I-471 (Dan Beard) twin bridges over the Ohio River near Newport, Kentucky. The coating was a pearl-grey THOROSEAL having ACRYL-60 as the bonding agent. The contractor applied the coating by brush in September 1976.

On September 29, 1982, the sidewalls/guardrails were inspected. The sidewall section is shown in Figure 26. The following observations were made.

Northbound Bridge:

A. KENTUCKY APPROACH -- Many failures (delaminations) occurred on both the horizontal (top) face and vertical (inner) face (Figure 27). Failures on the outer faces of both upstream and downstream sidewalls were limited to small spots, approximately 1.0 inch in diameter, apparently aggregate pop-outs (Figures 28 and 29). The upstream inner face showed signs of frequent vehicle impacts. In some instances, the coating suffered only slight mechanical damage. In other cases, the impacts removed large pieces of the finish coating (Figure 30). At some delamination locations on the inner face and curb, rust stains and exposed rusted reinforcing steel were visible. That also occurred infrequently at other locations on the bridge, but was more pronounced on the northbound upstream approach sidewall.

B. MAINSPAN -- On the average, the appearance of the northbound mainspan sidewalls was better than the southbound mainspan sidewalls. There were fewer points where the coating had separated from the concrete. However, at many locations on the northbound mainspan sidewalls, the coating had debonded, superficially cracked,

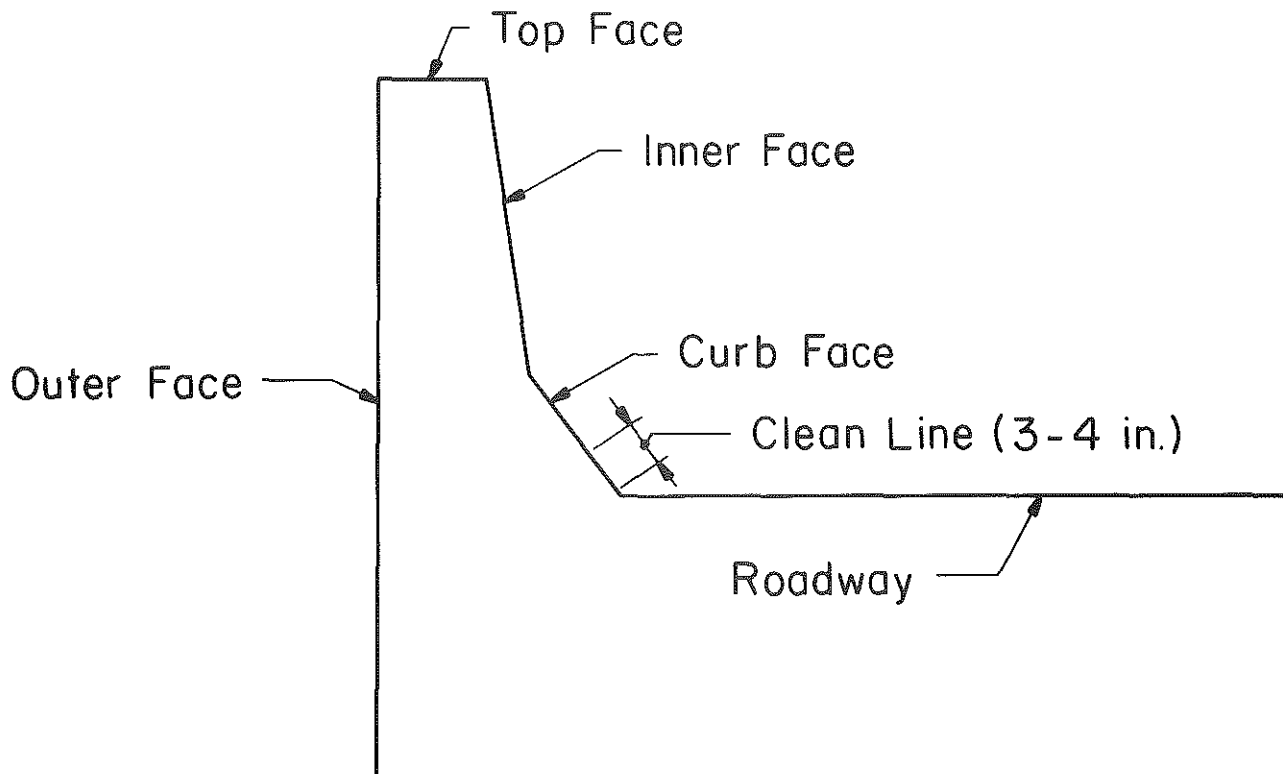


Figure 26. Cross Section of I-471 Bridge Sidewall.

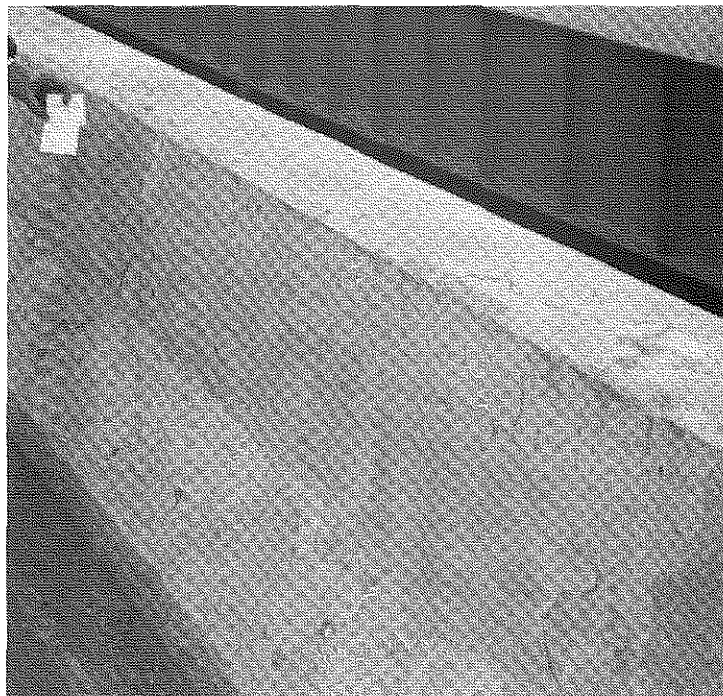


Figure 27. Northbound Kentucky Approach of the I-471 Bridge Showing Delamination of the Masonry Coating.

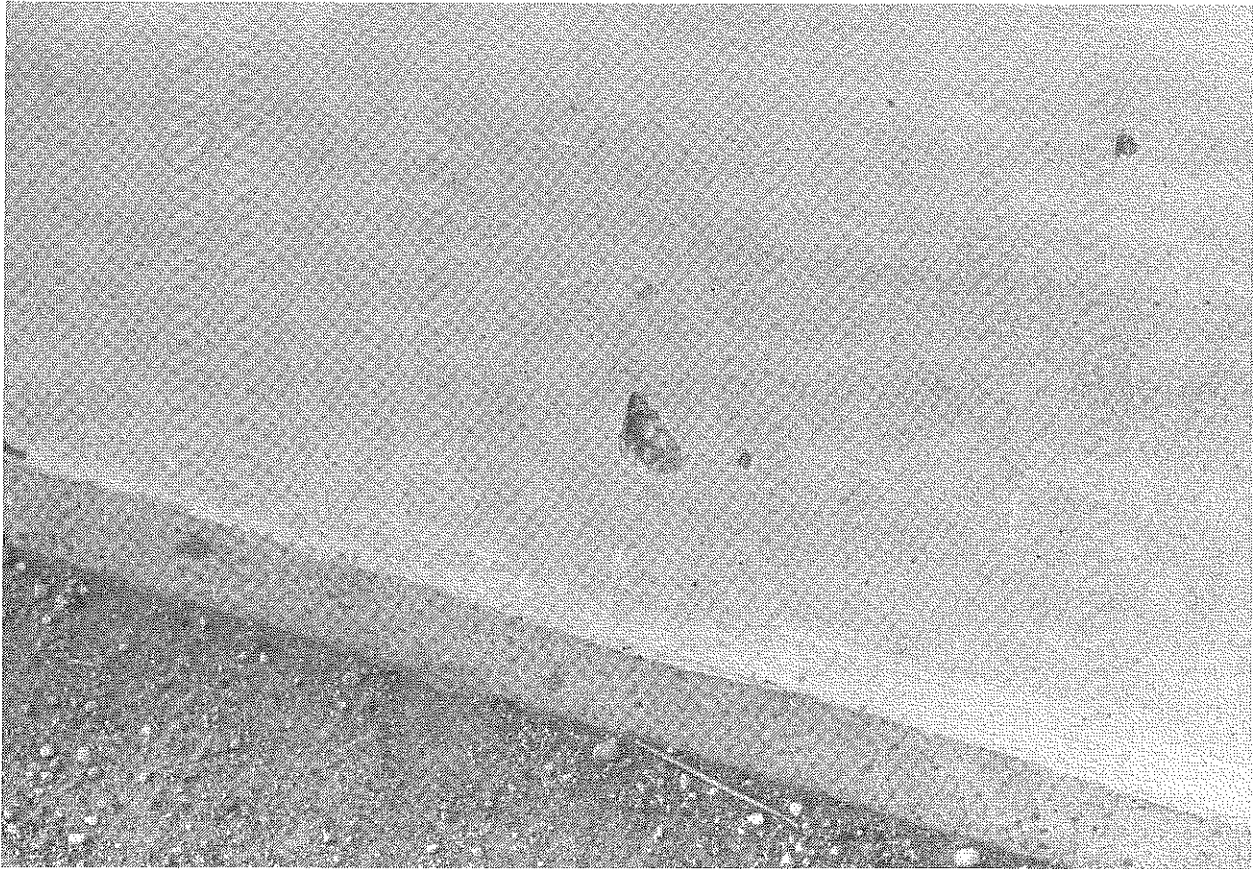


Figure 28. Pop-out Caused by Porous Aggregate.

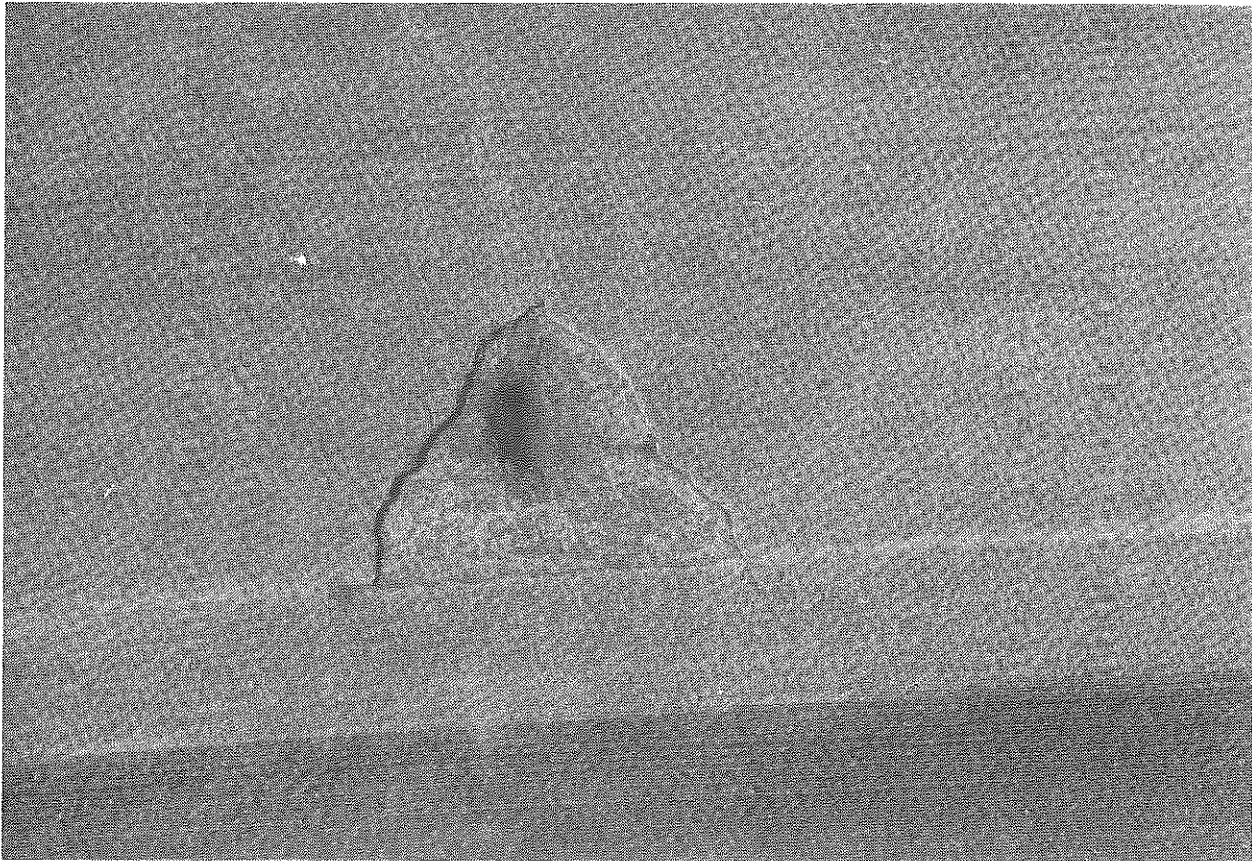


Figure 29. Pop-out Caused by Shale.

and required only slight force to pull from the concrete. The upstream (traffic lane) sidewall showed slightly less exposed concrete (less complete coating failures) than the downstream (passing lane) sidewall. Compared to other locations on the northbound bridge mainspan, more of the coating was missing from the top face than from the curb or inner faces. Massive failures on the vertical curb and inner faces appeared to be clustered in adjacent panels. Many failures on vertical faces were more evident at lower locations, especially on the curb face that was only partially covered by the finish coating. Many small failures ran along the coating parting lines, even on panels where the coating was, as a whole, well bonded. A crudely brushed-on coating patch was evident on the upstream sidewall (Figure 31). The bond of the patch was good. The outer faces were in good condition, except for random aggregate pop-outs.

C. OHIO APPROACH -- The Ohio approach had fewer signs of rust and automotive impact failures. The upstream (traffic) lane sidewall showed more delamination failures on the inner face, while the downstream lane contained more spalls on the top face.

Southbound Bridge:

A. OHIO APPROACH -- The coating on the southbound Ohio approach sidewalls was in noticeably better condition than the southbound mainspan sidewalls. Some spalls were noticeable on the top faces, though that was not a predominant feature. The top faces of some panels showed good bonding between the coating and concrete. Others showed poor bonding but no spalls. Spalls were most prominent on the coating parting line running along the curb face (Figure 32). Failures were not necessarily panel dependent -- some panels had top-face spalls in one location and tight coating bonding in another. There were few signs of vehicular scars on the sidewalls. The outer faces had features similar to those of the outer faces of the northbound structure.

B. MAINSPAN - The downstream (traffic) sidewall showed more deterioration (spalls) than any other location (Figure 33). Along many panels, the top face had lost most of the masonry coating. Along the inner and curb faces, the coating was debonded but had yet to be mechanically broken from the sidewall. More severe failures were concentrated on the lower portions of the inner and curb faces. Some panels evidenced few failures, except for random pop-outs of underlying aggregates. Others exhibited massive spalls, most of which ran to the coating parting line. Some failures were associated with vertical cracks that ran the height of the sidewall. There were fewer spalls on the upstream (passing) lane. Most were either on the top face or were small failures along the parting line of the curb face. Some failures were near poured joints between panels. While visible spalls were less frequent on upstream panels, delaminations were probably as severe. The coating bulged in many locations. In some places, even where bond was good, the coating showed signs of fine crazing. The outer faces of both sidewalls exhibited a condition similar to the other locations.

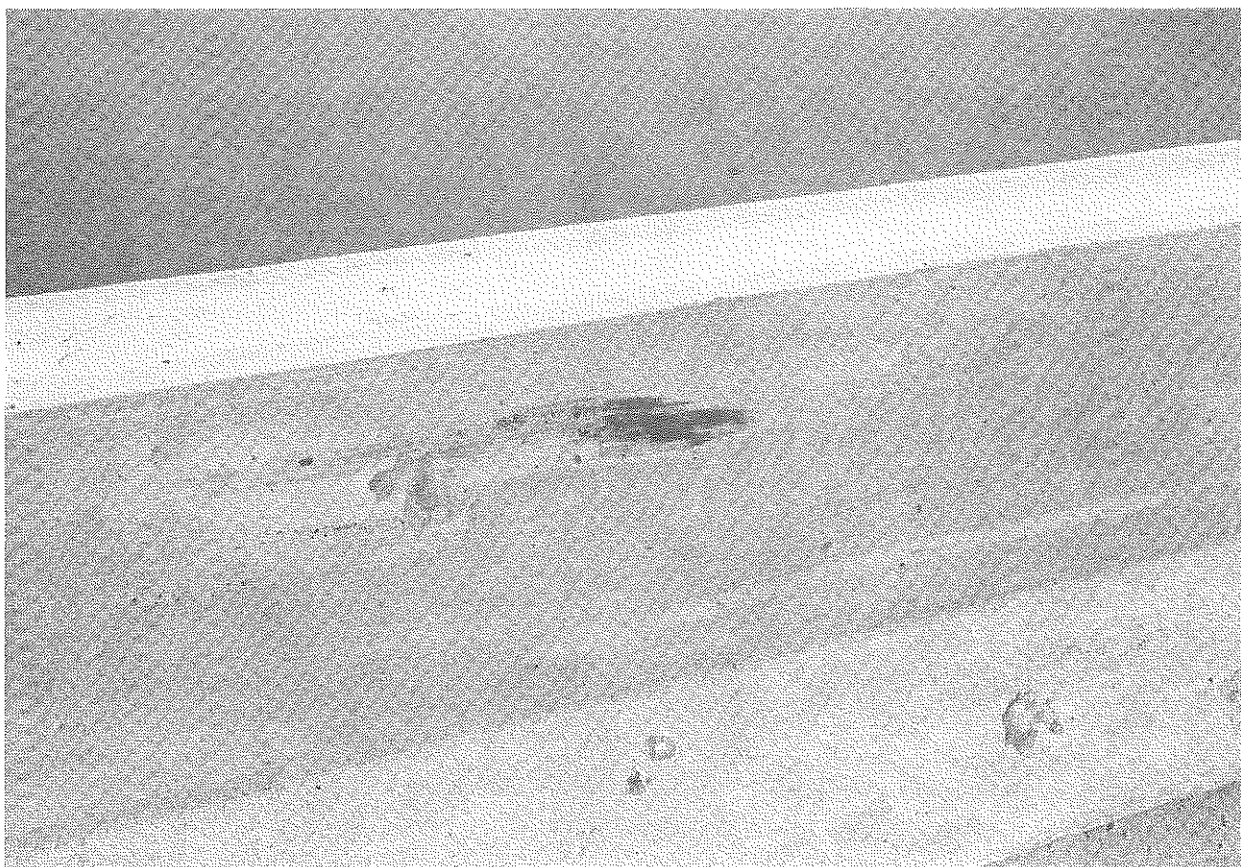


Figure 30. Delamination Caused by Vehicle Impact.

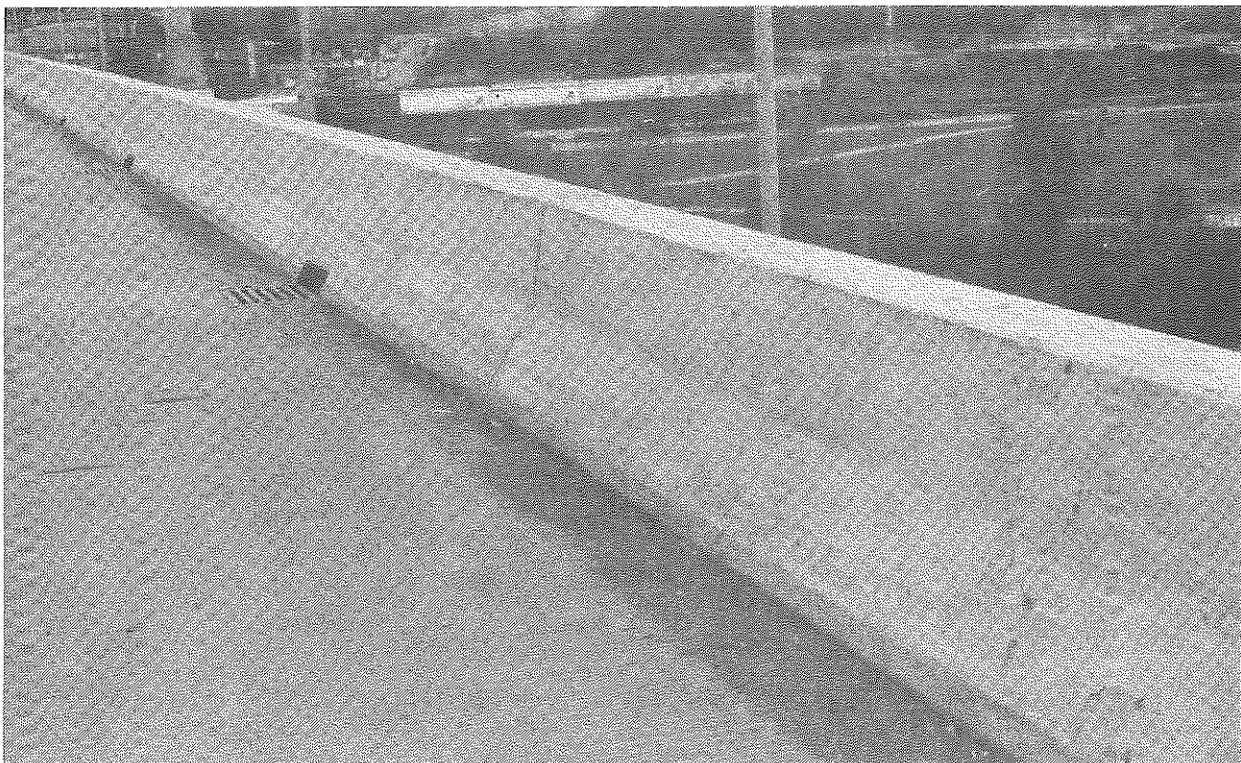


Figure 31. Patch on Northbound Main Span of the I-471 Bridge.

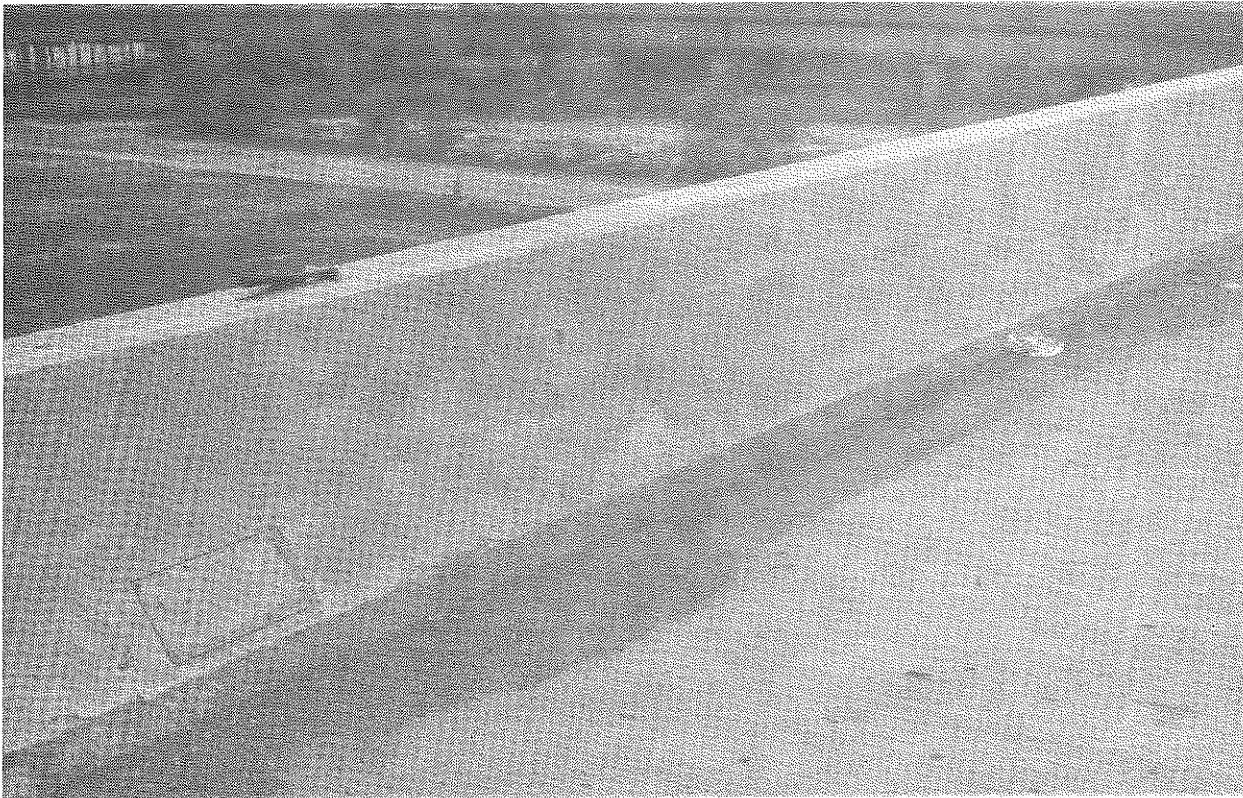


Figure 32. Delaminations on Sidewalls of Ohio Southbound Approach of the I-471 Bridge.

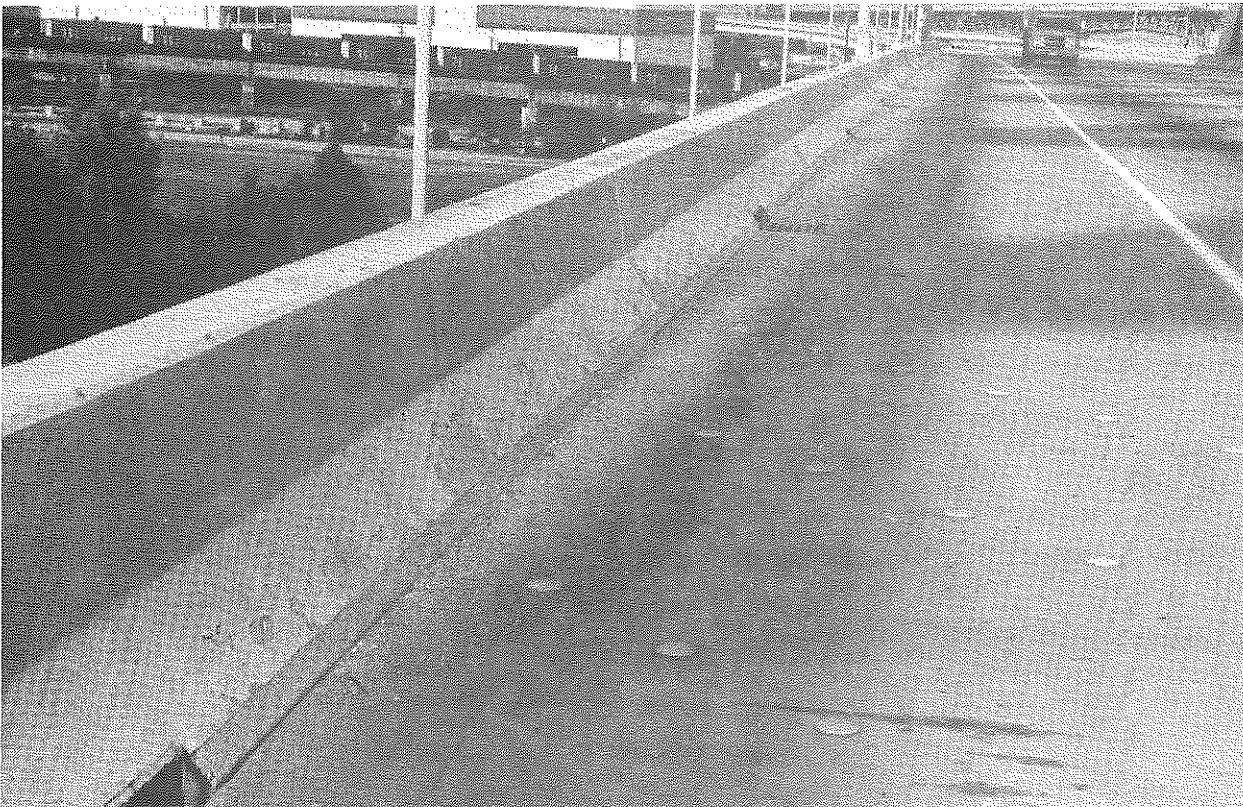


Figure 33. Traffic Lane Sidewall on the Southbound Main Span of the I-471 Bridge.

C. KENTUCKY APPROACH -- The Kentucky southbound approach sidewalls showed fewer spalls than the southbound Ohio and mainspans. Most of the visible spalls on both sidewalls were detected on the top faces rather than the inner faces. The condition of the outer faces of both sidewalls were similar to the other locations (Figure 34).

In summary, the following observations were made:

1. The condition of outer faces of the sidewalls was good, except for infrequent aggregate pop-outs.
2. The downstream southbound lane that had the more severe deterioration also had the most pedestrian traffic. There was good reason to believe that pedestrians tore off a large number of delaminations and tossed them into the river.
3. Many locations not showing spalls actually had delaminations that may easily be removed when subjected to light mechanical force.
4. Some sources of initial coating failure were
 - A. vehicle impacts,
 - B. pop-outs of porous aggregate and shales,
 - C. cracking of concrete sidewalls,
 - D. pop-outs due to corrosion of reinforcing steel, and
 - E. points of geometric discontinuity such as corners and drain edges.
5. Coating failures may have been promoted by
 - A. freezing-thawing at initial debonding sites,
 - B. failure to adequately clean the sidewalls prior to coating,
 - C. failure to adequately wet the sidewalls prior to coating,
 - D. inadequacies in some batches of the coating compound.

On March 3, 1983, the masonry coating on the KY-35 bridge at Sparta, Kentucky (Figure 35), was inspected. That was the second Kentucky bridge treated with a masonry coating (1970). The bridge was sprayed with TEX-COTE.

The downstream (southbound) sidewall masonry coating appeared to be in good condition. Close inspection revealed several pop-outs due to expansive materials. Some spalling was detected at the corner between the inner and top faces. The outer face was in good

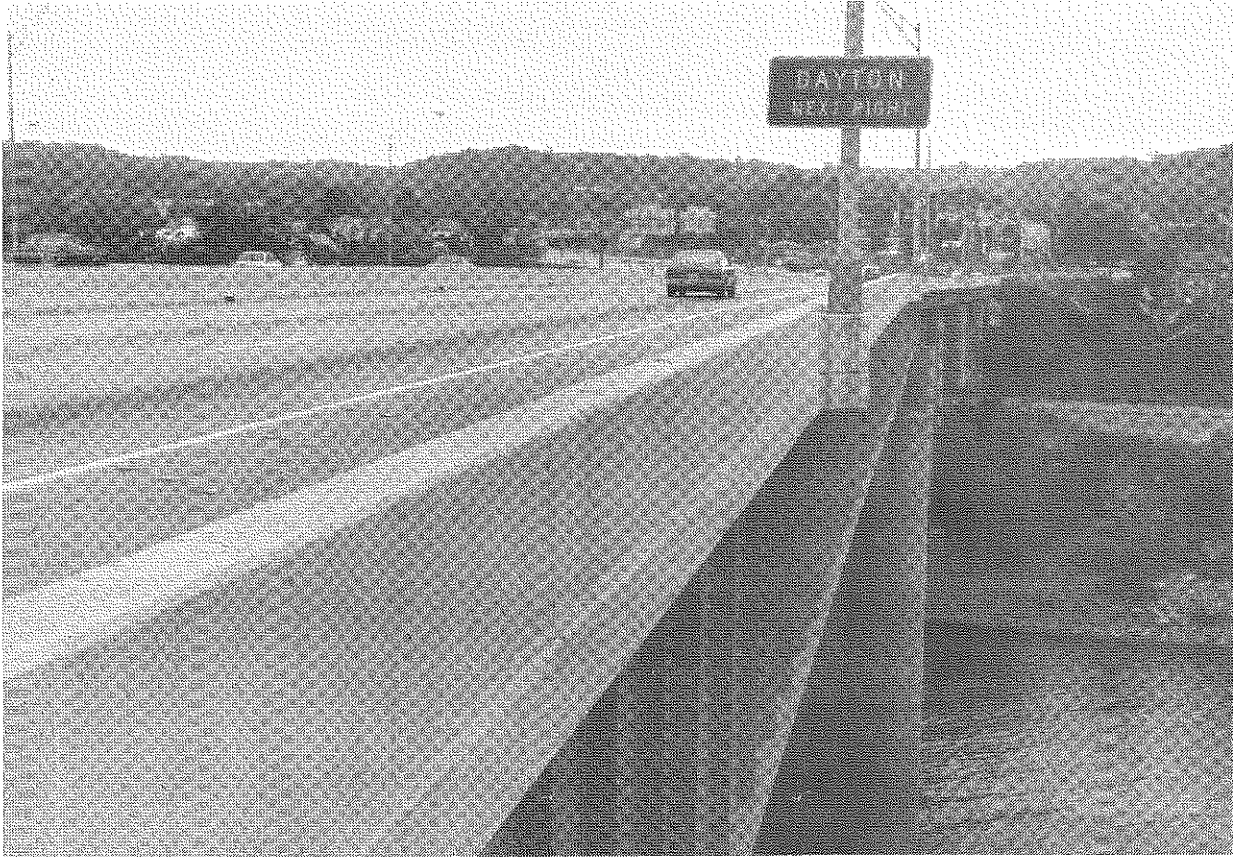


Figure 34. Southbound Kentucky Approach Showing the Condition of the Outer Walls.

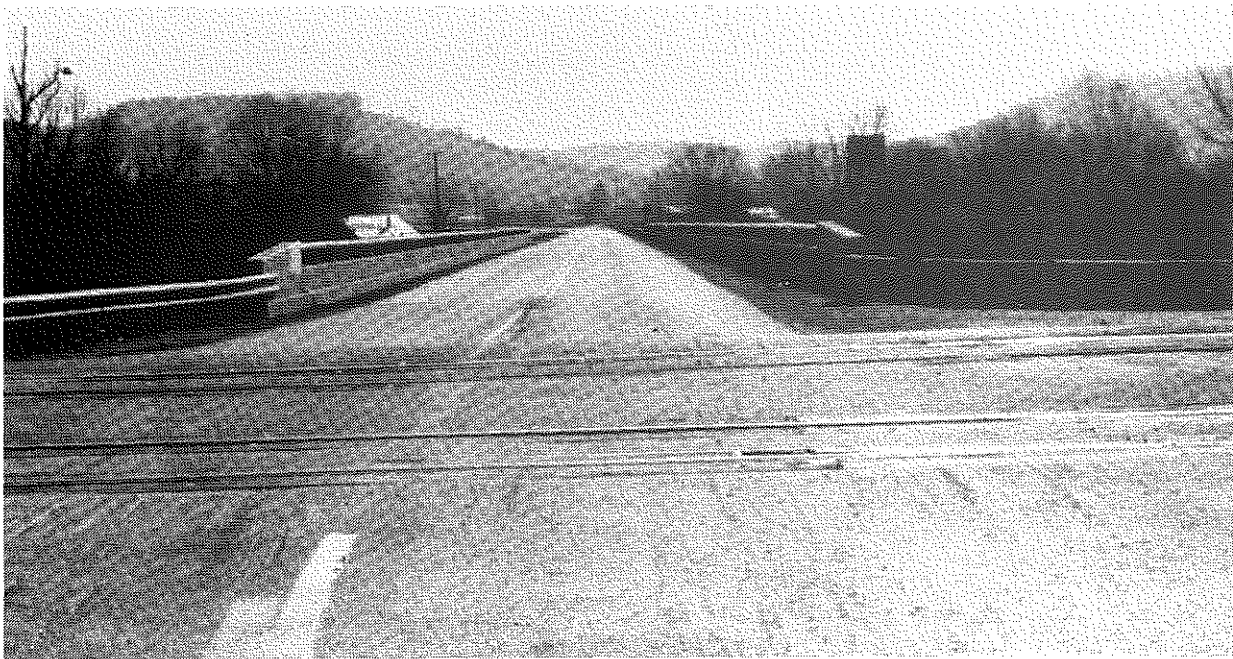


Figure 35. KY-35 Bridge at Sparta.

condition, with only a few pop-outs visible. Several cracks were visible in the sidewalls, but the bond was good in most of those locations.

On the south end of the downstream sidewall, the coating had debonded from the concrete on the horizontal face of the curb. Records indicated that debonding occurred no more than two years after the coating was applied (9). The vertical face of the curb showed signs of frequent vehicle impacts that stripped the masonry coating from the concrete. In locations below those impacts, the masonry coating was debonded from the concrete. Perhaps water settled in the interface created by the impact, and subsequent freeze-thaw action led to debonding.

On the upstream sidewall, rust stains and concrete pop-outs were visible (possibly due to inadequate concrete cover). There were fewer signs of vehicle impacts on the curb. However, in the horizontal portion of the curb, much of the masonry coating had spalled.

The masonry coating was applied much heavier to the top face of the sidewall than to the vertical faces or the horizontal face of the curb (Figure 36). At the guardrail bases on the top face, the TEX-COTE was debonding from the concrete. When the coating was applied, a small box was placed over guardrail studs to mask them during the spraying operation. That created an interface that moisture could penetrate and subsequently freeze, debonding the TEX-COTE. Also, the masonry coating did not completely fill many small voids in the surface of the concrete. Unexpectedly, the TEX-COTE performed very well in those locations.

Stains were detected on the innerface of the downstream sidewall and the outer face of the upstream sidewall (Figure 37). That could be expected, since the prevailing wind direction is west to east. Therefore, the outer face of the downstream sidewall and the inner face of the upstream sidewall would experience more washing. Parting failures at guardrail bases were less numerous on the upstream sidewall. That was probably due to decreased time-of-wetness on the windblown upstream sidewall.

Due to the light off-white color of TEX-COTE, stains were very prominent. That detracted from the effect of the masonry coating and made an otherwise good bridge appear shabby.

On November 11, 1982, several sample patches of coatings placed on the wingwall of a culvert under I 65 in Fayette County were inspected. Two acrylic latex solutions, E 330 and AC 35, furnished by the Rohm and Haas Company had been placed on the wingwall in 1967. The latexes were mixed with white portland cement and water. The wingwalls were wetted and covered with a tack coat of the latexes. Then, those areas were covered with a topcoat of the latex compounds. Also, a patch of THOROSEAL was placed next to the Rohm and Haas coatings.

By 1983, the E 330 and AC 35 were badly stained by sedimentation that spilled over the wingwall. Close examination of those coatings showed they were much thinner than the THOROSEAL and TEX-COTE coatings on the bridges. The latex coatings were showing signs of initial failure by flaking. The coatings contained many fine cracks. The THOROSEAL, being gray-tinted, did not show as much staining as the E 330 and AC 35 mixtures. The coatings were much



Figure 36. Uneven Masonry Coating on KY-35 Bridge.

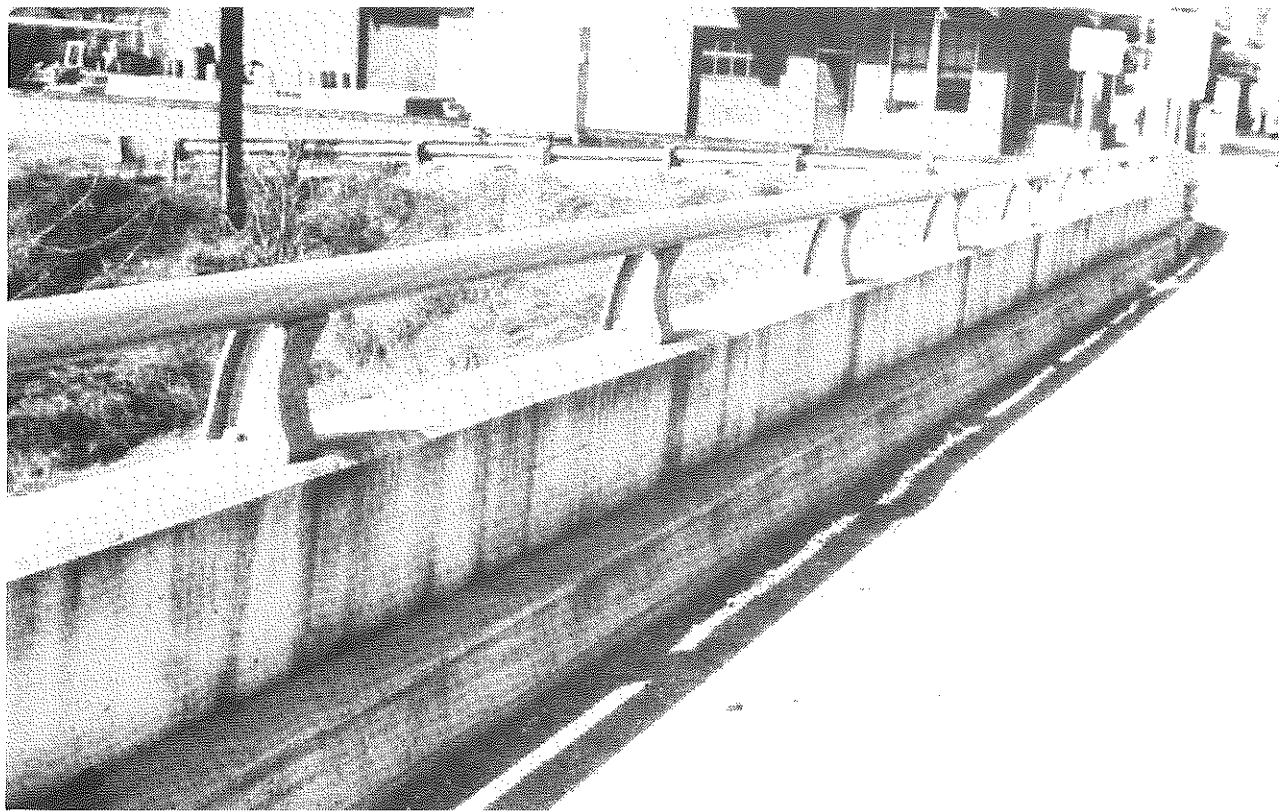


Figure 37. Stains on KY-35 Bridge Masonry Coating.

thinner than on the I-471 bridge. The coatings were tightly bonded and appeared to be in very good condition. A heavier patch of THOROSEAL sheltered in the culvert also was in good condition.

Masonry problems typical of those encountered on the I-471 bridges have not been frequent. According to Kentucky Department of Highways sources, few problems have been encountered in more recent bridges. Yet, there are several potential drawbacks to masonry coatings as they are presently applied. Horizontal surfaces will eventually spall, even when a coating is well bonded. Numerous chert and shale pop-outs on the I-471 bridges indicated those coatings will not prevent freeze-thaw failures. Indeed, the coatings, as presently applied, may promote such failures by retaining moisture in the concrete. Numerous small surface voids on the KY-35 bridge sidewalls, incompletely filled by the masonry coating, did not affect durability of either the concrete or masonry coating. The coatings, when tightly bonded, will withstand some vehicular impacts.

There is no compelling need for the large quantities of fillers in masonry coatings. Those escalate costs and provide negligible hiding benefits for scarred concrete surfaces. Hiding may be accomplished in less-expensive ways.

Obviously, existing coatings are not permanent. Staining, pop-outs, and spalling are problems that will eventually affect such coatings and will necessitate touch up or recoating (as with bridge paints). Use of thinner masonry coatings would reduce initial construction expenses and make maintenance work more viable (cheaper). Such coatings could be easily sprayed onto the concrete, possibly at the same time as bridge-steel maintenance painting by a painting contractor. As noted on the wingwalls, thin masonry coatings may perform satisfactorily for about 10 years (which corresponds to the life of many bridge steel paints). Thinner masonry coatings may "breathe" better than conventional ones and might prove to be more spall-resistant.

When masonry coatings are not substantially protective, they are only cosmetic. The question arises as to the need for fillers to hide surface defects. In cases such as massive honeycombing, a separate filler may be required, regardless of the thickness of the masonry coating. Also, dark or flat masonry coatings may provide sufficient hiding power for random voids. Use of white or off-white colors may make the sidewalls easier to see at night or in adverse weather. However, those advantages may be lost due to staining or weathering. The appearance of the bridge may be maintained for longer periods by using darker-colored masonry coatings. Several new bridges in the Knoxville, Tennessee, area employ colorful masonry coatings that enhance general appearance. Those color schemes should be investigated for future use in Kentucky.

STAY-IN-PLACE FORMS

In February 1983, a survey on the use of stay-in-place forms was distributed. The survey was prepared from questions submitted by the Divisions of Construction and Materials. Questionnaires were sent to the 50 states and the District of Columbia. Forty-seven

replies as well as standard drawings were returned. The original replies and drawings have been turned over to the Division of Construction. A detailed summary of responses is contained in the Appendix.

Of the 47 respondents, 35 had used stay-in-place forms made of metal and 21 had used concrete stay-in-place forms (Table 1). The length of service for metal forms ranged from 40 years to 3 years. Service of concrete forms was much less, ranging from 19 years to only several months.

Use of the forms was usually restricted to certain types of bridges. Only four states permitted the use of either type form on all bridge decks. Thirty-one states employing stay-in-place forms restricted their use.

Only six states indicated maintenance problems were caused by the use of metal stay-in-place forms. Those problems were in three main areas: 1) corrosion of metal forms, 2) honeycombing in deck concrete, and 3) poor access to the slab for maintenance inspections. No problems were indicated for concrete forms.

Thirty-nine states required a constant-depth deck between beams; seven states did not. Thirty-three states adjusted elevations of the forms to match a computed grade. Six states placed the forms directly in the beams with the final grade made parallel to the beam camber. States that used the latter method felt that it provided good riding quality.

A variety of techniques were used to adjust form heights to match the computed grade. Among those methods were 1) shelf angles, 2) felt pads, 3) brick, 4) grout, and 5) fiber-based pads.

Thirteen states permitted variable depth decks. Of those, eight used variable-height reinforcing chains to maintain a constant concrete cover. Seven states permitted concrete cover to vary over the top reinforcing mat. Fifteen states used a pachometer to determine depth of concrete cover. Seven states checked concrete cover by coring. Eighteen states compensated for extra dead load due to variable thicknesses of concrete when designing beams.

Eleven states permitted reduction in slab thickness when using metal stay-in-place forms. Nineteen states permitted welding of metal stay-in-place forms.

Seven states checked bond between concrete stay-in-place forms and poured decks. Six of those found the bond to be good. Thirteen states required the reinforcing bars be cast into the concrete stay-in-place forms to tie the panel to the poured deck. Twenty-four states used prestressed strands in the concrete forms. Eight states used concrete forms as a design alternate when epoxy-coated reinforcement was used for the top and bottom mats.

Five states observed deck cracking resulting from the use of stay-in-place forms. Fourteen respondents did not encounter that problem.

Savings resulting from the use of stay-in-place forms in lieu of removable forms varied widely. For metal forms, savings ranged from \$0 to \$22.50 per square yard. Savings averaged about \$5 per square yard. For concrete forms, savings ranged from \$0.30 to \$9 per square yard.

TABLE 1. SUMMARY OF STAY-IN-PLACE FORM SURVEY

QUESTION	NUMBER OF RESPONSES		
	YES	NO	NO REPLY
1a. Does your organization use metal stay-in-place forms?	35	12	0
1b. Does your organization use precast concrete panel stay-in-place forms?	21	25	1
2a. How long has your agency used metal stay-in-place forms?	3-40 years		
2b. How long has your agency used concrete panel stay-in-place forms?	1-19 years		
3a. Does your agency permit either type of forms on all bridge decks?	4	39	6
3b. If not, what criteria are used to determine when they may be used?	See Appendix		
4a. Has the use of stay-in-place metal forms caused any maintenance problems?	6	30	11
4b. Has the use of stay-in-place concrete forms caused any maintenance problems?	0	17	30
5. Describe those maintenance problems.	10	0	37
6. Do you require constant depth or thickness bridge decks between the beams?	38	7	2
7a. If a constant-thickness slab is required, are stay-in-place forms adjusted in elevation to match a computed grade?	33	3	11
7b. Or, are the forms placed directly on the beams with the final grade made parallel to beam center?	6	24	17
8a. If you permit the final grade to parallel the beam center, is the riding quality good?	8	1	38
8b. If the stay-in-place concrete forms are adjusted in height to match a computed grade and maintain a constant-thickness slab, what method of supporting panels has proven best? i.e., grout pads, felt pads, wood strips, other?	See Appendix		
9. Do you permit variable depth or thickness bridge decks over the stay-in-place forms?	13	28	6
10. When variable-depth slabs are permitted, do you require variable-height reinforcement chairs to maintain a constant concrete cover?	8	7	32
11a. Do you permit the concrete cover to vary over the top mat of reinforcement?	7	37	3
11b. If you do, what is the minimum concrete cover, and what is the maximum concrete cover?	1-7/8" minimum 1-7/8" No maximum		
11c. Do you do any after-the-fact checking of concrete			
With pachometer?	15	24	8
With cores?	7	26	14
12. Do the designers compensate for the extra dead load due to the variable thickness of concrete when designing the beams?	18	14	15
13. Do you permit the reduction of slab thickness with metal stay-in-place forms, when the spacing of corrugations match the transverse rebar spacing?	11	28	8
14. Do you permit welding components of metal stay-in-place forms to steel girder or stringer flanges?	19	21	7
15a. Have you performed any checks to determine if bond was obtained between the concrete stay-in-place form and the poured-in-place deck?	7	28	12
15b. If yes, was good bond obtained?	6	0	41
16. Do you require reinforcement bars to be cast into the concrete stay-in-place forms to tie the panel to the poured-in-place deck?	13	10	24
17. Do you require the strands in the concrete stay-in-place forms to be pretensioned?	24	1	22
18. Do you permit the concrete stay-in-place forms with uncoated strands as an alternate when the design required epoxy-coated reinforcements?	8	7	32
19. Have you encountered any additional cracking of the bridge decks, attributable to stay-in-place forms?	5	14	28
20. How much savings do you estimate your agency realized from the use of stay-in-place forms in lieu of removable forms?			
Metal (per square yard)	0-\$22.50		
Concrete (per square yard)	0.30-\$9.00		

MODULAR EXPANSION JOINTS

Attempts to determine the costs of modular expansion joints were unsuccessful. Cost comparisons between different types of joints may be difficult without undertaking an extensive review of design drawings. Many joints are made entirely or almost entirely of steel, which is usually bid lump-sum with all steel incorporated into the structure.

Since many joints described in an earlier report (10) are no longer manufactured, and since many bridges incorporating those units were constructed more than three years ago, it may be impossible to obtain costs of those items. A more important question is whether those units should be employed on future bridges.

Costs of expansion dams are included in Tables 2 and 3.

SUMMARY

Four experimental bridges have been targeted for long-term corrosion-potential tests. Two of those bridges have epoxy-coated reinforcing steel in the decks. The other two bridges have uncoated reinforcing steel in the decks. The bridges were tested in fiscal year 1983. To date, no active corrosion has been detected on the reinforcing steel. Long-term monitoring is anticipated.

Three weathering-steel bridges were inspected in fiscal year 1983. Abnormal corrosive activity was detected on the girder ends and bearings of the KY-1893 bridge at Shawhan. That problem should be remedied by cleaning and painting the affected areas. The other bridges were weathering satisfactorily.

The zinc-coated bridge (I 24 over KY 93) was inspected and observed to be in satisfactory condition. The condition of the painted companion bridge appeared to be slightly better at the time of inspection. Long-term monitoring of those bridges will be required to determine relative performance of the two coatings.

Masonry coatings were inspected on two bridges and one culvert. The masonry coating on the I-471 Dan Beard bridges had failed on many portions of masonry sidewalls. Several possible causes of those failures were identified, but the major construction-related cause could only be surmised. Masonry coating on the KY-35 bridge at Sparta was performing satisfactorily. Several changes in types of masonry coatings were suggested.

A questionnaire was distributed to transportation agencies in all 50 states and the District of Columbia to assess the extent of usage of stay-in-place forms. Forty-seven agencies replied. In general, stay-in-place forms appeared to be favorably appraised, based on replies from states that have employed them.

No costs could be determined for modular expansion joints. However, fiscal year 1982 costs were determined for expansion dams.

TABLE 2. 1982 COSTS OF EXPANSION DAMS

JOINT SIZE	AMOUNT INSTALLED (FEET)	AVERAGE COST (PER LINEAR FOOT)
2 inches	3,885	\$ 72.41
4 inches	602	\$170.41
6 inches	108	\$222.89

TABLE 3. 1982 RANGE OF BIDS (LOW-HIGH)
FOR EXPANSION DAMS

JOINT SIZE	RANGE (ALL BIDS)	RANGE (SUCCESSFUL BIDS)
2 inches	\$20 - 200	\$ 50 - 120
4 inches	\$45 - 250	\$150 - 191
5 inches	\$80 - 350	\$200 - 260*

*Only two successful bids

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8. "Coating Concrete Structures Prevents Deterioration," Better Roads, Better Roads Pub., Park Ridge, Illinois, May 1983, pp 25.
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APPENDIX

STAY-IN-PLACE FORMS
FOR
CONSTRUCTION OF CONCRETE BRIDGE DECKS

SURVEY RESULTS

1. a. Does your organization use metal stay-in-place forms?

YES	35
NO	12

b. Does your organization use precast concrete panel stay-in-place forms?

YES	21
NO	25
NA or left blank	1

COMMENTS:

- Yes, but only on special conditions.
- Will be allowed as an option after July 1983.
- Metal stay-in-place forms are used at hazardous crossings only: over electrified railway lines, over major rivers, or over a highway carrying heavy traffic.
- Have allowed the contractor the option of using stay-in-place metal forms on most projects. However, they have only been selected for use on steel girder bridges. Permitted the contractor the option of using concrete panel stay-in-place forms on one project, but conventional forming methods were selected. Presently, do not provide a stay-in-place concrete panel alternate.
- Have used mild reinforced concrete panels, occasionally.
- Precast concrete panel stay-in-place forms: one time - experimental.
- Precast concrete panel stay-in-place forms: only in two special cases.
- Quoted from letter: "The _____ Department of Transportation does not use stay-in-place forms as a standard procedure. Metal forms have been used for a few projects with no adverse maintenance effects, although most of these projects are fairly recent. It is our judgment that the use of such forms should be avoided unless a definite cost advantage exists..."

2. a. How long has your agency used metal stay-in-place forms?

Years	Number of Respondents
3	1
4	1
5	3
6	1
7	1
9	1
10	6
11	1
13	1
14	2
15	3
16	1
19	1
20	7
23	1
38	1
40	1

Blank, NA or zero years listed by
11 respondents

Several years (1 respondent)

COMMENTS:

- Many respondents added that, although the forms had been used for years, they were used on a very selected basis or on a few selected structures. Many respondents added the word "approximately" to the number of years shown, or a plus/minus sign.
- First projects will be constructed within the next two years and are welded plate-girder bridges with camber cut into the web for dead load deflection and grade.
- Do not permit -- were used some in 1960's.
- 20 (plus/minus) years -- used about 25 percent of time at contractor's option.

2. b. How long has your agency used concrete panel stay-in-place forms?

Years	Number of Respondents
0.5 or less	2
1	1
2	1
3	4
4	2
5	4
6	1
7	1
9	1

10	1
11	1
19	1

COMMENTS:

- A few respondents added that the forms were used on an experimental and/or selective basis. One respondent indicated the forms were used for three years, but only on two experimental projects, and that the forms were recently allowed on all projects.
- Experimental bridge -- 1973. General practice since 1981.
- Just getting to design stage.

3. a. Does your agency permit either type form on all bridge decks?

YES	4
NO	39
NA or left blank	4

b. If not, what criteria are used to determine when they may be used?

COMMENTS:

- Permit the use of permanent and metal or the use of removable forming on all jobs except where construction is over traffic, in which case specify permanent steel forming.
- Do not use either type as a general policy, except for value engineering changes by the contractor when approved.
- Most projects allow metal stay-in-place forms. Concrete-only, when specified.
- Concrete option will be allowed on selected contracts. Steel option will be allowed on selected contracts.
- Not used on cast-in-place concrete box girders or T-beams.
- Do not regard precast stay-in-place forms as being equal to metal stay-in-place forms.
- On certain AASHTO prestressed concrete (PSC) beams, near the upper limit of stress, forms are not allowed. PSC forms are not allowed on short radius curved steel beams.
- Precast concrete panel forms are allowed on composite steel and precast prestressed concrete I-beam bridges.
- Metal forms on steel beam/girder only, contractor's option to use. Concrete on concrete beam only, contractor opted to use on all but one.
- Do not have criteria -- have only used precast panels on one job.
- Steel forms are permitted as alternate to conventional forming. Prestress panels are designed as part of the deck slab.
- On main bridges where both top and bottom mat of steel requires epoxy coating, concrete stay-in-place forms are not allowed. Steel forms must be quite expensive because requests for use are minimal.
- Metal stay-in-place forms are permitted on most projects.

- To date, only allow steel forms.
- Only use steel.
- By authorization if over traffic or very high (40 feet plus) underclearance.
- Metal forms have been allowed in special cases only.
- No stay-in-place forms for grade separations.
- Concrete panels are not used on steel structures.
- It is not permitted at any time.
- Economic consideration.
- Metal stay-in-place forms an acceptable alternate for all but unusual superstructure configuration.
- Metal forms may be used on all decks. At present, concrete panel forms are only allowed on bridges with prestressed concrete girders.
- Have only used concrete stay-in-place forms on a trial basis and steel stay-in-place for two steel-box girder bridges.
- Allow them for concrete decks over prestressed I-girders only.
- When a significant benefit might result.
- Usually if there are crowded utility bays.
- Metal stay-in-place forms may not be used over salt or brackish waters.
- Do not recommend metal forms for coastal areas and not applicable for wide girder spacings (over 10 feet). Do not allow concrete panels on prestressed beams continuous for live load nor on steel stringer spans.
- Not permitted.
- Some restrictions on the use of metal forms in the coastal area because of potential corrosion from salt spray.
- When conventional forming is difficult or costly.
- Concrete panels on an experimental basis. Metal on high major structures.
- Not used.
- Use metal stay-in-place forms only when construction and stripping of conventional forms would be very difficult.
- Steel forms may be used anytime.

4. a. Has the use of stay-in-place metal forms caused any maintenance problems?

YES	6
NO	30
NA or left blank	11

b. Has the use of stay-in-place concrete forms caused any maintenance problems?

YES	0
NO	17
NA or left blank	30

COMMENTS:

- Not in use long enough to tell (2 respondents).
- None known to date.

- Potential problem (note added beside Question 4b).
- Too soon to say.

5. Describe those maintenance problems:

NA or left blank 37

COMMENTS:

- Advanced corrosion requiring removal of stay-in-place forms.
- Honeycomb and voids in bottom of deck, corrosion around drains and accessories.
- Corrosion at longitudinal construction joints. Now require removal of metal forms at those joints.
- Concrete has only been in place for about three years and no problems have been identified. However, steel (galvanized) forms have been in place long enough to develop some trapped moisture problems.
- Cannot inspect bottom of slab. Metal forms trap and hold saltwater solution.
- Forms trap moisture and accelerate corrosion of stay-in-place forms. Hinder inspection of deck.
- Both have been in use too short a time to determine problems.
- Not used long enough to make realistic evaluation.
- Infrequent cracking of deck over transverse joints between the concrete panels.
- Can not see bottom to inspect.

6. Do you require constant depth or thickness bridge decks between the beams?

YES 38
 NO 7
 NA or left blank 2

COMMENTS:

- One response was yes for metal; no for concrete. (These answers are counted in above numbers)
- One "no answer" commented that it usually depends on design and detailing.
- One "yes answer" commented for steel stay-in-place forms.
- With precast planks, thickness may vary to compensate for camber, deflection, etc. or top flange thickness of prestressed I-girder may be varied to hold slab thickness constant.
- One "no answer" commented -- especially not to compensate for crown. However, a minimum thickness is required.
- One "yes answer" commented -- within reason. Will allow corrugated metal forms.

7. a. If a constant thickness slab is required, are stay-in-place forms adjusted in elevation to match a computed grade?

YES 33
 NO 3

NA or left blank 11

7. b. Or, are the forms placed directly on the beams with the final grade made parallel to the beam camber?

YES 6
NO 24
NA or left blank 17

COMMENTS:

- All steel girders are cambered for dead load deflection. Therefore, only minor adjustments are generally required.
- Adjustment for difference in elevation due to girder camber and finished grade is accomplished by varying the depth of the cast-in-place concrete. Slab steel is placed parallel to the panel top. Concrete cover is varied from a minimum of 2-1/2 inches to a maximum of 3-1/2 inches to accommodate girder camber up to 1 inch. For cambers larger than 1 inch, panel supports are thickened to compensate for camber minus 1 inch.
- Answers apply for precast concrete forms. (a. Was a "yes answer"; b. Was a "no answer").
- Riding surface must match the computed grade and crown shown on the plans. Forms on the underside should match as close as practical by varying the thickness of the fiberboard bearing material, but the bottom of the slab is straight across from beam to beam even when the two beams adjacent to the center line are straddle the center line with a 8-foot parabolic crown on top for the finished surface.
- Metal closure angles.
- Adjustable shelf angles.
- Grout, felt pads.
- 5,000-psi brick fiber board.
- Only grout permitted.
- Concrete forms are set on grout to give them even bearing, but this does not adjust form to match grade or camber.
- Combination of vertical wood strips held in place by a drilled-in Jay-hook.
- Fiberboard bearing strips.
- Not in use.
- Now allow variable thickness of top flange of prestressed I-beam girders and use 1/2-inch joint filler between flange and panel.
- Inverted angles attached to stringers.
- Use felt pads.
- Grout.
- Adjustment for difference in elevation due to girder camber and finished grade is accomplished by varying the depth of the cast-in-place concrete. Slab steel is placed parallel to the panel top. Concrete cover is varied from a minimum of 2-1/2 inches to a maximum of 3-1/2 inches to accommodate girder camber up to 1 inch. For cambers larger than 1 inch, panel supports are thickened to compensate for camber minus 1 inch.

- Adjusting screws on flange.
- Grout pads.
- Not enough installations to evaluate.

8. a. If you permit the final grade to parallel the beam camber, is the riding quality good?

YES 8
 NO 1
 NA or left blank 38

b. If the stay-in-place concrete forms are adjusted in height to match a computed grade and maintain a constant thickness slab, what method of supporting panels has proven best? i.e., grout pads, felt pads, wood strips, other?

NA or left blank 29

COMMENTS

- Metal closure angles.
- Adjustable shelf angles.
- Grout, felt pads.
- 5,000-psi brick fiberboard.
- Only grout permitted.
- Concrete forms are set on grout to give them even bearing, but this does not adjust form to match grade or camber.
- Combination of vertical wood strips held in place by a drilled-in Jay-hook.
- Fiberboard bearing strips.
- Not in use.
- Now allow variable thickness of top flange of prestressed I-beam girders and use 1/2-inch joint filler between flange and panel.
- Inverted angles attached to stringers.
- Use felt pads.
- Grout.
- Adjustment for difference in elevation due to girder camber and finished grade is accomplished by varying the depth of the cast-in-place concrete. Slab steel is placed parallel to the panel top. Concrete cover is varied from a minimum of 2-1/2 inches to a maximum of 3-1/2 inches to accommodate girder camber up to 1 inch. For cambers larger than 1 inch, panel supports are thickened to compensate for camber minus 1 inch.
- Adjusting screws on flange.
- Grout pads.
- Not enough installations to evaluate.

9. Do you permit variable depth or thickness bridge decks over the stay-in-place forms?

YES 13
 NO 28
 NA or left blank 6

COMMENTS:

- Variable depth longitudinal camber strip over beams only.
- Yes, but only over concrete panels.
- No, not in general; however, if conditions warrant, it is allowed.
- Yes, precast panels only.

10. When variable depth slabs are permitted, do you require variable height reinforcement chairs to maintain a constant concrete cover?

YES	8.
NO	7
NA or left blank	32

11. a. Do you permit the concrete cover to vary on the top mat of reinforcement?

YES	7
NO	37
NA or left blank	3

COMMENTS

- No, only by tolerance.
- No, set minimum cover.

b. If you do, what is the minimum concrete cover? _____, and what is the maximum concrete cover? _____.

MINIMUM	MAXIMUM
2-1/4"	probably 3-14"
2-1/4"	2-1/2"
2"	only by tolerance
2"	2-1/2"
1-7/8"	2-1/4"
3-3/8"	3-3/8"
3"	not specified
2-1/2"	NA
2-1/2"	3-1/2"
2"	no maximum
2-1/2"	2-1/2"
2-1/2"	2-1/2"
2-1/2"	Do not specify a maximum

NA or left blank 34 surveys

c. Do you do any after-the-fact checking of concrete cover with pachometer? Yes _____ No _____, or with cores? Yes _____ No _____.

PACHOMETER		CORES	
YES	15	YES	7
NO	24	NO	26
NA or left blank	8	NA or left blank	14

COMMENTS:

- Depth check while concrete still plastic.
- No to both answers; not after immediate construction.
- Cores, only in special cases.
- No to both answers; check plastic concrete with rulers.
- Cores, except when cover quantity is questionable.
- Pachometer on selected projects only.
- Pachometer, not very often.

12. Do the designers compensate for the extra dead load due to the variable thickness concrete when designing the beams?

YES	18
NO	14
NA or left blank	15

COMMENTS:

- No, not directly, but use reduced section properties for design, which is compensating factor.
- Yes, would if such a system were under design.
- Yes, due to form weight and extra concrete.
- No, contractor is responsible.
- Yes, if application warrants.
- No, design is checked when the contractor elects the variable-depth option.
- Yes, if it is significant.

13. Do you permit the reduction of slab thickness with metal stay-in-place forms, when the spacing of corrugations match the transverse rebar spacing?

YES	11
NO	28
NA or left blank	8

COMMENTS:

- Yes, however, final product would have to be structurally equivalent to the conventional deck.
- No, require corrugations to match rebar spacing.
- Must accommodate longitudinal bars.

14. Do you permit welding components of metal stay-in-place forms to steel girder or stringer flanges?

YES	19
NO	21
NA or left blank	7

COMMENTS:

- Nine "yes" responses, but only in compression areas (or compression zones, or compression flanges).
- Allowed only in positive moment areas for continuous girders.
- Yes, but not in top flange tensile zones.
- Yes, only in areas of positive bending.

- Yes, where flange is in compression or properly designed for fatigue.
- No welding of any kind.
- No welding, clips used in tension areas.

15. a. Have you performed any checks to determine if bond was obtained between the concrete stay-in-place form and the poured-in-place deck?

YES	7
NO	28
NA or left blank	12

b. If yes, was a good bond obtained?

YES	6
NO	0
NA or left blank	41

16. Do you require reinforcement bars to be cast into the concrete stay-in-place forms to tie the panel to the poured-in-place deck?

YES	13
NO	10
NA or left blank	24

COMMENTS:

- Yes, are reviewing this, probably is unnecessary.
- Yes, limited number used mainly for handling.
- No experience.
- Yes, a nominal amount.

17. Do you require the strands in the concrete stay-in-place forms to be pretensioned?

YES	24
NO	1
NA or left blank	22

COMMENTS:

- Yes, prior to November 1982, all uses have been mild reinforced.

18. Do you permit the concrete stay-in-place forms with uncoated strands as an alternate when the design required epoxy-coated reinforcement for the top and bottom reinforcement mats?

YES	8
NO	7
NA or left blank	32

COMMENTS:

- Require only the top reinforcement mat to be epoxy coated.
- Yes, not certain if this situation has occurred -- but would probably allow it. Denser mix in panel should resist

chloride penetration better than poured deck slab, strands are not connected to top mat steel.

- No epoxy used.
- Yes, do not use epoxy-coated bars in bottom mat.
- Do not use bottom coated bars.
- Designs do not require epoxy-coated reinforcement.
- Do not require epoxy-coated reinforcement in the bottom mat.
- Only top mat is epoxy-coated.

19. Have you encountered any additional cracking of the bridge decks, attributable to stay-in-place forms?

YES	5
NO	14
NA or left blank	28

COMMENTS:

- Fear of the concentration of cracking at the joints is one reason have not used this type of forming.
- No such forms used on state highway system to date.
- No, have observed this in other states. This is one of the reasons limit their use.
- Yes, in other states. That is why not used.
- Yes, hairline cracking longitudinal to the bridge along the ends of panels have occurred in most installations; however, this has not posed any problems. Since all structures are continuous, transverse cracking at panel joints also occur in negative moment regions; however, the panels promote controlled cracking in lieu of random cracking that occurs with monolithic slabs.
- Yes, not exactly additional. Maybe even less cracking, but over the panel parts.
- Inadequate experience for evaluation.
- Do not know -- only have one job just completed and another just let.
- Negligible.

20. How much savings do you estimate your agency realized from the use of stay-in-place forms in lieu of removable forms?

Concrete (per square yard)
Metal (per square yard)

-Concrete: --

-Metal: \$6.00 Average

-Concrete: --

-Metal: \$2.25 to \$4.50

-Concrete: --

-Metal: \$2.00+

-Concrete: \$0.30

-Metal: \$0.25

-Concrete: --

-Metal: \$2.25

-Concrete: Not known. Selected by contractor in all but one of 10 (plus or minus) projects where offered.

-Metal: \$0.25 to \$0.40 Selected by contractor about 25 percent of time when offered.

-Concrete: --

-Metal: \$0

-Concrete: --

-Metal: \$8.00

-Concrete: Not enough projects at this time for realistic evaluation.

-Metal: \$22.50

-Concrete: \$9.00 (Better than metal)

-Metal: Undetermined

-Concrete: \$9.00

-Metal: Do not know

-Concrete: \$8.00

-Metal: \$8.00

COMMENTS:

- No definitive way of estimating any such savings.
- No experience with the use of stay-in-place forms.
- Have no way of determining dollar savings.
- Removable forms used 16 years ago. Figures not available.
- Difficult to arrive at a monetary sum. Contractors bid stay-in-place forms up front and it boils down to offsetting less labor and time against more material costs.
- It is not possible to determine how much money is saved because contractors do not opt to use panels on all projects, indicating the savings are little, if any.
- None, about the same.
- Not enough history to evaluate. Contractors now bid precast panel alternate on most prestressed I-girder bridges. Practically no use of stay-in-place metal forms on routine structures.
- Contractor has option of substitution. Not a bid item.
- Have used limited number, so cannot determine.
- Since alternate bids are not solicited, have no figures. It is felt the options for permanent as well as removable forms gives the contractor the opportunity to submit best bid, depending on particular operation.
- None, contractor's option when used.
- Not used.

Comments from states that did not enclose drawings:

- No standard drawings available.
- Offer concrete forms as an option to metal; have no standard drawings.
- Use FHWA guidelines.
- Will send standard drawings when they are completed.

Additional comments:

- Have allowed a steel stay-in-place form option on selected structures since 1969. Contractors have selected this option on a few major structures, and achieved satisfactory results.
- Suggest you contact Mr. Daniel P. Jenny, Prestressed Concrete Institute, 210 N. Wells Street, Suite 1410, Chicago, Ill. for more information. They conducted a similar survey in July-August 1982.
- Questions 6 through 10: A uniform thickness slab is used, with camber variations being adjusted in a haunch. Experience with stay-in-place forms is very limited, so most of the questions are not applicable.
- Very limited use (two structures) of stay-in-place forms. Both instances were at the contractor's request. In both cases, the contractor made a minor credit to the project.
- Most questions answered reflect attitude toward potential use in state.
- Would appreciate a copy of survey results as are considering the use of precast concrete stay-in-place forms.
- Have allowed the contractor the option of using stay-in-place metal forms on most projects. However, they have only been selected for use on steel-girder bridges. Permitted the contractor the option of using concrete panel stay-in-place forms on one project, and he selected conventional forming methods. Presently, do not provide a stay-in-place concrete panel alternate.
- Do not use either metal or precast concrete stay-in-place forms on the state highway system as a standard practice. Response to the above questions are therefore limited to opinions and anticipated policy, should such forms be used on a routine basis.