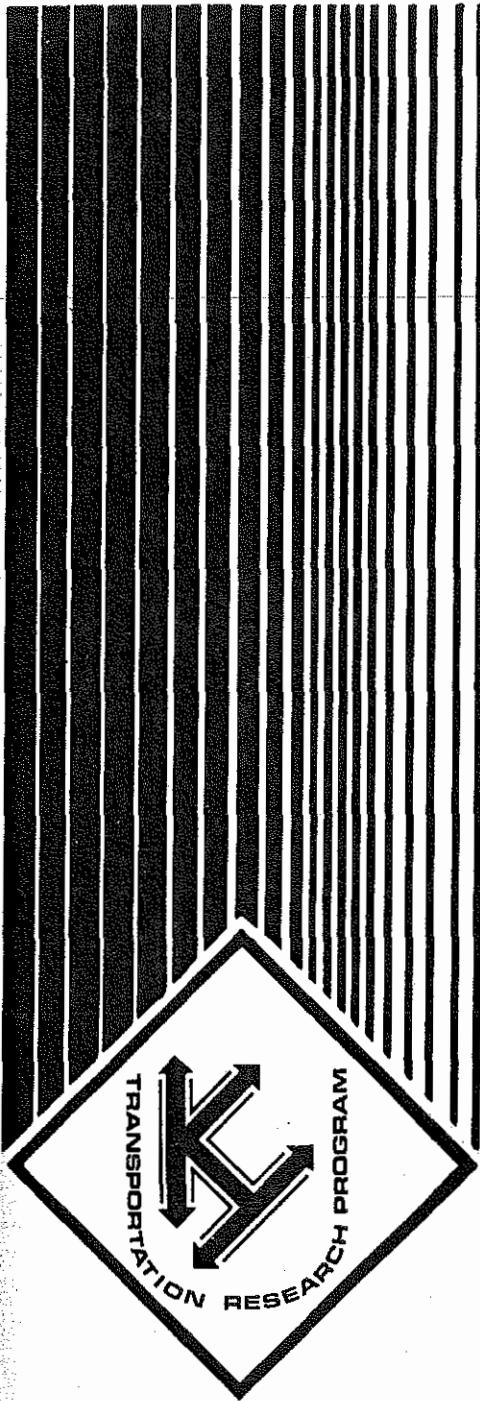SUMMARY OF EXPERIMENTAL BRIDGE FEATURES

by

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and

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Technician

March 1987

Kentucky Transportation Research Program

College of Engineering • University of Kentucky
Transportation Research Building
Lexington, Kentucky 40506-0043

January 20, 1988

Mr. Robert E. Johnson
Division Administrator
Federal Highway Administration
Frankfort, Kentucky 40602-0536

Dear Mr. Johnson:

SUBJECT: IMPLEMENTATION STATEMENT
          RESEARCH STUDY KYHPR 82-88, EVALUATION OF BRIDGE
          PERFORMANCE FOR CONSTRUCTION AND MAINTENANCE

The research reports produced for this study are: Research Report UKTRP 84-7, "Specially Constructed Bridges: Activities for Fiscal Year 1983"; UKTRP 87-1, "Bridge Decks and Overlays"; and Research Report UKTRP 87-5, "Summary of Experimental Bridge Features". The objectives of this study have been met in that the Kentucky Transportation Cabinet has been provided performance information on bridge features allowing appropriate action to be taken concerning their future employment. The Kentucky Transportation Cabinet either has or will take the following steps as a result of information gained during the course of the subject study.

The use of epoxy coating on reinforcing steel placed in the top mat for decks was assessed as being an effective deterrent against chloride induced spalling. Epoxy-coated reinforcing steel will continue to be used routinely for the top mat of conventional decks.

Latex concrete overlays and portland cement concrete overlays placed in accordance with prevailing construction requirements were observed as being a durable and suitable means for repair of deteriorated bridge decks. Those overlays will continue to be used for repair and restoration of deteriorated reinforced concrete bridge decks.

Integral abutment bridges were observed as being durable, low-maintenance structures. That type of bridge will continue to be employed. Longer span integral abutment brides have been recommended. That recommendation will be considered for future construction at suitable locations.

Some masonry coatings on bridges were deemed failures. Recommendations were made for improved application practices and more detailed inspections. Since 1986, contractors have been provided with a special note related to proper masonry-coating application requirements. Also, requirements for application and inspection of masonry coatings are discussed in annual meetings with district personnel.

Information gained from a nationwide survey of stay-in-place forms was furnished to the Division of Bridges for consideration in future use of those forms.

Precast segmental bridges were judged to be acceptable structures from a construction standpoint. While their durability has not been assessed, they will remain as construction alternates when applicable. A cast-in-place segmental bridge is presently scheduled for construction on the Ashland-Alexandria Highway over Twelve Mile Creek in Campbell County.

Retaining nuts on several aluminum handrail installations were observed as being badly corroded. A detailed investigation revealed that all corroded aluminum nuts were manufactured from alloys that did not meet specification requirements. Corroded retaining nuts are being replaced with nuts manufactured of the specified alloy.

A corrosion problem was observed on one weathering steel bridge. Based on that observation, no future bridges will be constructed using uncoated weathering steel.

Hot-dipped galvanized steel performed well; however, results to date do not justify the additional cost compared to painted steel. No further hot-dipped galvanized steel bridges are anticipated, pending long-term service results.

Sincerely,

R. K. Capito, P. E.
State Highway Engineer

Research Report
UKTRP-87-5

SUMMARY OF EXPERIMENTAL BRIDGE FEATURES

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Chief Research Engineer

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Kentucky Transportation Research Program
University of Kentucky

in cooperation with
Transportation Cabinet
Commonwealth of Kentucky

and

Federal Highway Administration
U.S. Department of Transportation

March 1987

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16. Abstract

Experimental bridge features and service problems investigated during this study are reviewed. Those were epoxy-coated reinforcing steel, stay-in-place forms, experimental deck features (rotary compaction and broomed deck finishing), precast segmental bridges, steel-corrosion control methods (weathering steel and hot-dipped galvanizing), microsilica concrete, failures of masonry coatings, and failures of aluminum guardrail-retaining nuts.

Several features including latex and low-slump overlays, epoxy-coated reinforcing steel, and broomed deck finishes (for state-built bridges), are now in common use. Most of the other experimental bridge features were performing satisfactorily. A corrosion problem was observed on one weathering steel bridge. Analyses of failures of masonry coatings and the aluminum nuts resulted in identification of the probable causes of the failures and remedial recommendations. The report also contains recommendations for further research.

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EXECUTIVE SUMMARY

Ten experimental features and service problems were examined. Those items included 1) overlays, 2) integral abutment bridges, 3) epoxy-coated reinforcing steel, 4) failures of masonry coatings, 5) stay-in-place forms, 6) experimental deck features (rotary compaction and broomed deck finishing), 7) segmental bridges, 8) steel corrosion control methods, 9) microsilica concrete, and 10) aluminum guardrail retaining-nut failures.

Two of those items, overlays and integral abutment bridges, were covered in final form in a recent interim report, UKTRP-87-1, "Bridge Decks and Overlays." The remaining features are reviewed herein.

In 1983, corrosion-potential tests (measuring the tendency towards corrosion of reinforcing steel) were conducted on four bridges. Two bridges contained epoxy-coated reinforcing steel, and the other two contained conventional uncoated reinforcing steel. One bridge of each type was located on a high ADT road and the other was on a low ADT road. Corrosion-potential tests revealed that the tendency toward corrosion had increased over earlier tests for all of the bridges. However, none of the bridges showed signs of active corrosion of reinforcing steel.

A number of bridges that had masonry coating failures were inspected between 1982-1984. The most recent inspections were in District 11 (Manchester). Coating failures were attributed to natural causes (i.e., plinth-wall cracking and pop-outs) and possibly to other reasons (incorrect coating application). A series of remedial steps to correct many of the potential causes of coating failures were recommended. Those steps included improved cleaning of the concrete prior to placing of the coatings and closer inspection during the coating application process.

In 1983, a nationwide survey was conducted on stay-in-place (SIP) forms using questions provided by the Transportation Cabinet. Of 47 respondents, 35 had used metal SIP forms, and 21 had used precast concrete forms. Only six of the metal-form users had experienced maintenance problems. None of the concrete forms had proven troublesome. Savings by using metal forms varied from $0 to $22.50 per square yard. Savings from concrete forms varied from $0.30 to $9.00 per

square yard.

In the 1970's, a number of experimental bridges were constructed on and over I 64 between Frankfort and Lexington. Experimental features included rotary compacted decks and broomed finishes. All decks (rotary compacted and conventional) showed signs of cracking; however, no decks showed extensive concrete spalling. Mainline I-64 bridges having broomed finishes were worn smooth in the wheel paths. Broomed finishes on the less-travelled I-64 overpass bridges had not worn significantly.

A segmental bridge, Ramp "B" over US 23 at Pikeville, was monitored during segment fabrication in 1985 and construction in 1986. The bridge had two sidespans of 93'-6" and one mainspan of 150'-0". Forty-eight concrete segments were precast using the cellular (fixed) form method. The bridge was constructed using the balanced cantilever method. The pier segments were placed first and other segments were attached alternatively on opposite sides of the pier segments.

Segments were glued and forced together by tightening threaded bars run through ducts in the segments. After placing two opposing segments on either side of a pier segment, the segment cluster was reinforced by post-tensioning steel strand tendons located in ducts cast in the segments.

As erection progressed, the superstructure was extended in cantilever form from the piers. When all the segments were placed, a key was cast connecting the cantilevered sections and completing the superstructure at the midpoint of the bridge.

A few problems were encountered. Initially, erection was slow due to the contractor's unfamiliarity with segmental construction. Also, a few segments had problems with oversized shear keys that spalled during placement. The cantilevered superstructure had to be rotated about the piers to achieve proper alignment prior to casting the connecting key. However, most of those problems were minor and were easily rectified.

Three weathering-steel (ASTM A 588) bridges and one hot-dipped galvanized bridge were inspected. One of the weathering-steel bridges (KY-1893 railroad overpass at Shawhan in Bourbon County) had extensive corrosion at the abutments. That problem was due to prolonged exposure of the steel to moisture leaking through the joints. It was recommended that the problem areas of the bridge be painted. The other bridges were

found to be in good condition.

In 1985, a microsilica overlay was placed on a bridge (KY 2097 near near Sebree in Henderson County). Some problems were encountered in finishing the concrete until a low-slump finishing machine was employed. Laboratory tests showed that microsilica concrete had acceptable properties. The microsilica portion of the deck had more cracks than the control portion (low-slump overlay).

In 1985, failures of aluminum nuts used to retain a plinth aluminum guardrail were investigated. Initial laboratory tests revealed that the nuts failed by several types of corrosion (exfolition and stress corrosion). Field inspections of three KY-4 bridges in Lexington indicated nut-failure rates from 0 to 40 percent. Kaiser Aluminum and Chemical Corporation analyzed the nuts and reported that those which failed were of the wrong alloy type (2024 series rather than 6061 series). Apparently the 2024 series alloy nuts were substituted accidentally for the proper type.

Further work on developing new overlays and on improving bridge-deck concretes and designs is recommended. Construction of more integral abutment bridges is also suggested. Remedial measures for masonry coatings should be adopted. Weathering steel bridges should be considered for specific applications where corrosion would not be a problem. Flame-sprayed zinc and diffusion-bonded epoxy are suggested for future experimental tests in place of hot-dipped gavanizing.

The four corrosion-potential comparison bridges and the segmental bridges should be monitored on a long-term basis.

INTRODUCTION

During the course of this study, a number of experimental features and service problems on bridges were investigated. Those included 1) overlays, 2) integral abutment bridges, 3) epoxy-coated reinforcing steel, 4) failures of masonry coatings, 5) stay-in-place forms, 6) experimental deck features, 7) segmental bridges, 8) special bridge steel corrosion control methods, 9) microsilica concrete, and 10) failures of aluminum guardrail retaining nuts. Three interim reports have been issued (1-3). Overlays and integral abutment bridges were reported in Report UKTRP-87-1, "Bridge Decks and Overlays". That report will serve as a final report for those items. The remaining experimental features will be covered in this final report.

The Summary and Conclusions section of this report contains suggestions for long-term monitoring of some features first investigated in this study. Also, some recommendations are presented for further research related to several of those features.

EPOXY-COATED REINFORCING STEEL

Epoxy coating for the top mat of deck reinforcing steel was first encouraged by the Federal Highway Administration in 1975. The first Kentucky bridge employing epoxy-coated reinforcing steel was completed in 1977. Additional corrosion protection steps instituted by the Kentucky Transportation Cabinet include Class AA concrete, added concrete cover (2 to 3 inches), and epoxy-coated reinforcing steel for both the upper and lower reinforcing mats on interstate bridge decks.

In 1983, corrosion-potential tests were conducted on four bridge decks. Two of those bridges, the KY-80 bridge over Buck Creek in Pulaski County (completed in 1978) and the CR-1381 bridge over Panther Creek in Daviess County (completed in 1977), contained epoxy-coated reinforcing steel. Two other control bridges, the I-64 bridge over Elkhorn Creek in Scott County (completed in 1971) and the Yarnallton Road bridge over I 64 (completed in 1972), used conventional reinforcing steel and a 2-inch concrete cover. The KY-80 and I-64 bridges were

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subjected to high ADT´s (including high truck volumes); the CR-1381 and Yarnallton Road bridges had very low traffic volumes (and very low truck volumes). Those bridges were selected specifically to provide performance comparisons between bridge decks containing epoxy-coated reinforcing steel and those containing uncoated reinforcing steel.

Corrosion-potential tests had been previously performed on the I-64 and Yarnallton Road bridges in 1973. The CR-1381 and KY-80 bridges had been tested in 1976 and 1978, respectively. None of those tests had revealed corrosion-potential values sufficiently high to indicate the onset of corrosion ($\approx$ 0.35 volts).

Follow-up corrosion-potential tests performed in late 1982 and early 1983 revealed no signs of corrosion on any of the bridges (Figures 1 through 4). At that time, the KY-80 bridge was 5 years old and the CR-1381 bridge was 6 years old. The I-64 (Elkhorn Creek) and the Yarnallton Road bridges were 12 and 11 years old, respectively, when inspected. However, corrosion-potential measurements for all of the bridges had increased over the values originally measured.

The relative differences between the corrosion-potential values of all the bridge decks was not significant at the time of the 1983 tests. Three of the bridges showed tendencies toward corrosion near the armored edges. Away from the ends of the bridges, the corrosion potentials for all four bridges measured in 1983 ranged between 0.05 to 0.15 volts, or approximately half the potential voltage required to indicate active corrosion on reinforcing steel. The Yarnallton bridge had the lowest corrosion potential. Although it is one of the older bridges (11 years) and contained plain reinforcing steel, it is probably the least used and salted of the four bridges. At the time of those tests, the data for all of the bridges was not meaningful beyond showing that the reinforcing steel in those bridges had not begun to corrode.

The question exists as to when those bridges will begin to show corrosion-potential test values indicating reinforcing-steel corrosion. In 1986, the Pennsylvania Department of Transportation conducted corrosion-potential tests on a 13-year old bridge that had epoxy-coated steel (4). At the time, 39 percent of the bridge had corrosion-potential values exceeding 0.35 volts. A second bridge, that also contained epoxy-coated steel, had potential values exceeding the
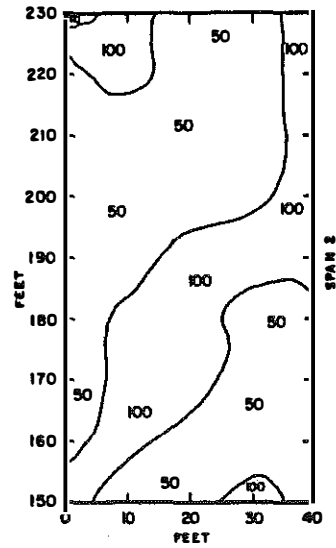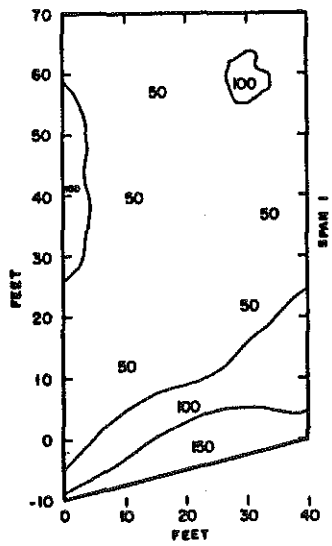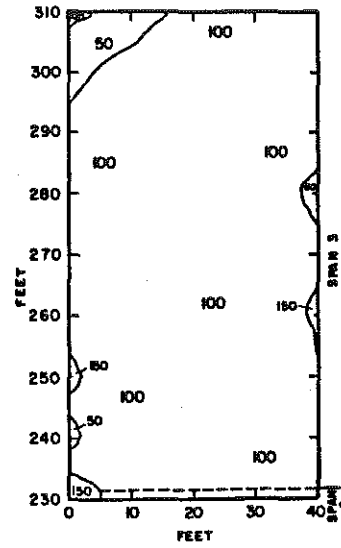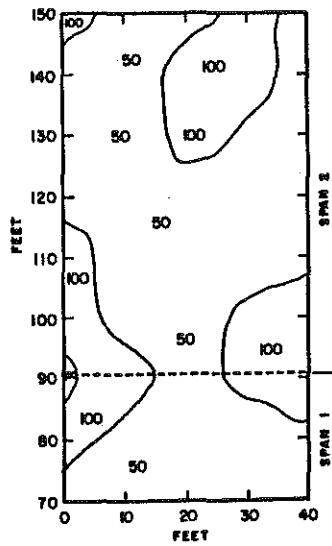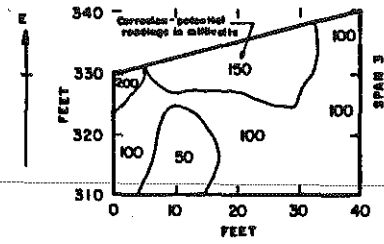
Figure 1. Corrosion-Potential Test of the KY-80 Bridge over Buck Creek, February 21, 1983 (1).
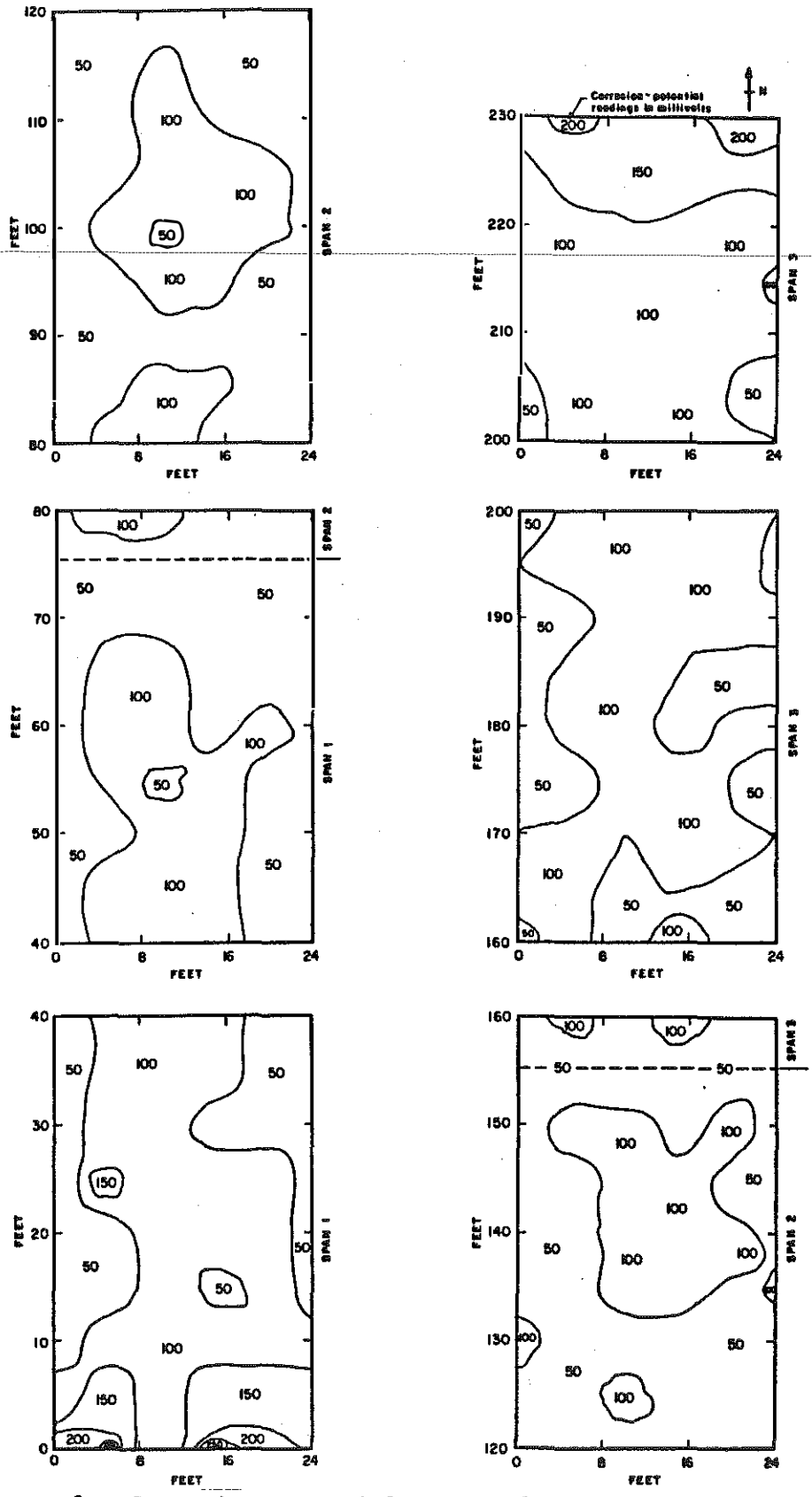
Figure 2. Corrosion-Potential Test of the CR-1381 Bridge over Panther Creek, October 13, 1982 (1).
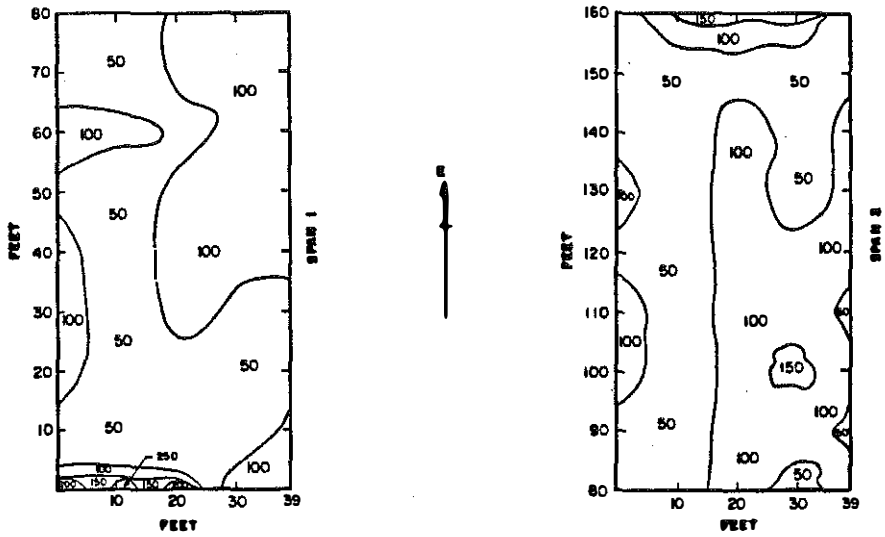
Figure 3. Corrosion-Potential Test of the I-64 Bridge over
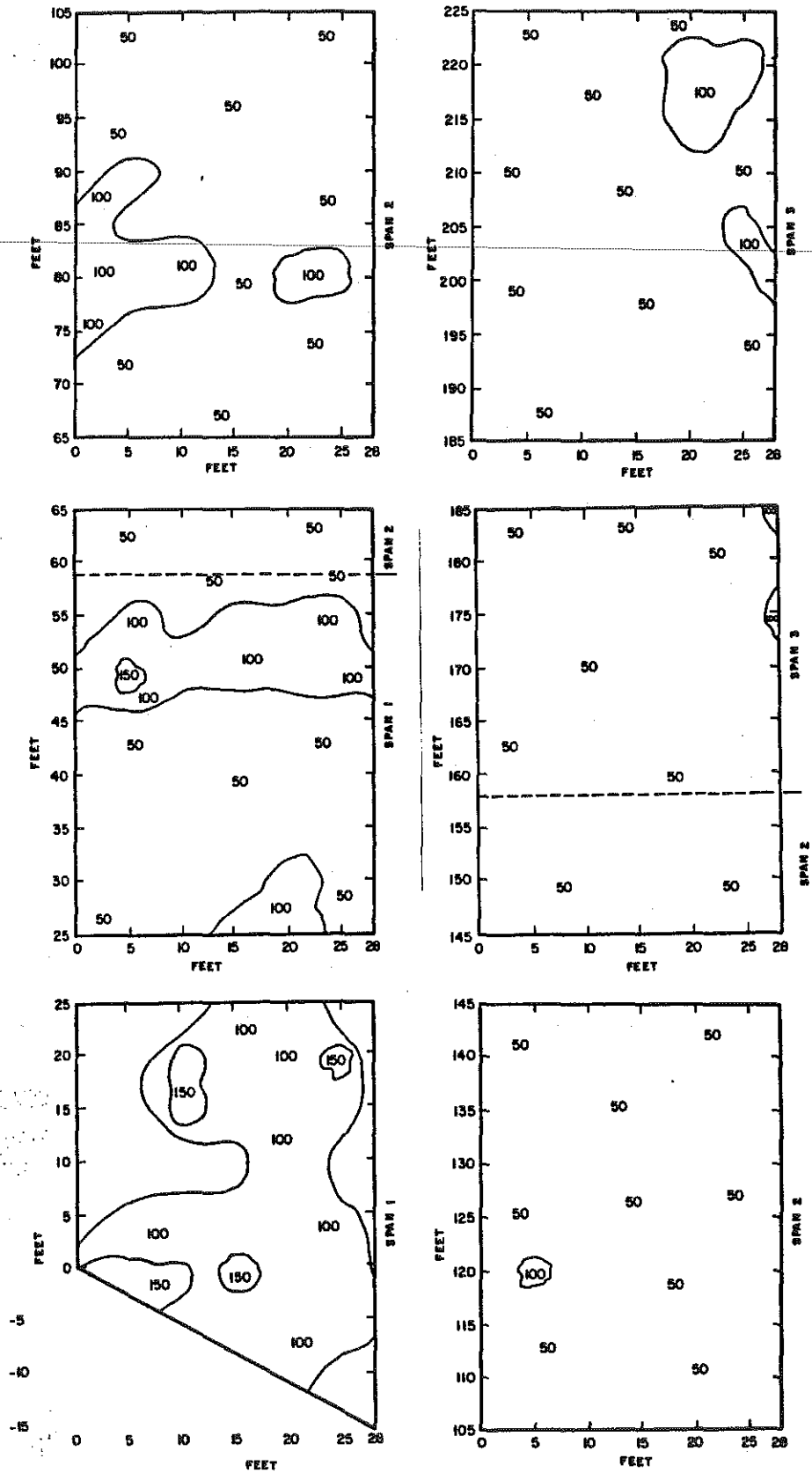Elkhorn Creek, February 20, 1983 (1).

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Figure 4. Corrosion-Potential Test of the Yarnallton Bridge
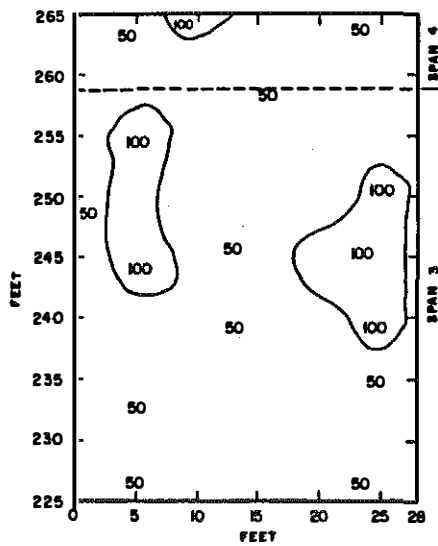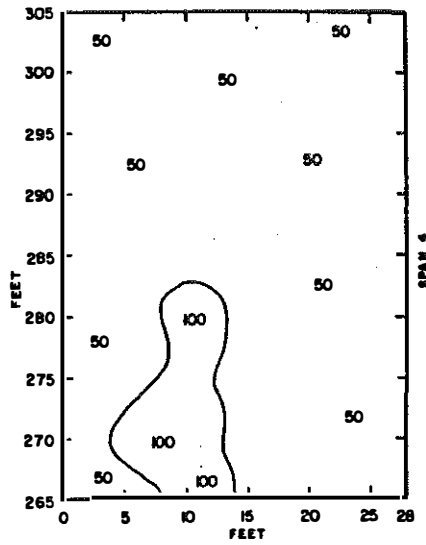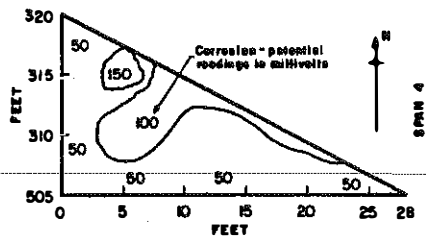over I 64, February 20, 1983 (1).

Figure 4. (continued)

corrosion threshold over 23 percent of its deck. The bridge was 11 years old when tested.

Based on those results, corrosion-level indications could be expected on any of the four test bridges in Kentucky within the next few years. However, it is likely that the Pennsylvania bridges were salted much more heavily, and the Kentucky bridges may not exhibit corrosion in the next 10 to 15 years.

FAILURES OF MASONRY COATINGS

In early 1984, Kentucky Department of Highways personnel in District 11 expressed concern about the number of failures of masonry coatings. Masonry coatings of several bridges had been previously inspected and reported in 1983, including the coating debonding problem on the I-471 bridge over the Ohio River at Newport, Kentucky (5). Possible causes of masonry coating failures were reported to be

1. vehicle impacts,
2. pop-outs of porous aggregates and shales,
3. cracking of concrete plinth walls,
4. pop-outs due to corrosion of reinforcing steel, and
5. points of geometric discontinuity such as corners and drain edges.

Coating failures may have been promoted by

1. freezing and thawing at initial debonding sites,
2. failure to adequately clean plinths prior to coating,
3. failure to adequately wet (or dry) plinths prior to coating,
4. application of the coating in extremely unsuitable weather, or
5. inadequacies in some batches of the masonry coatings.

In March 1984, four problem bridges in District 11 were inspected. Those were 1) the US-25E twin bridges over Big Richland Creek, 2) the KY-770 bridge over Laurel River Reservoir, and 3) the KY-312 bridge over Laurel River.

The US-25E twin bridges had a thin masonry coating that had begun to

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scale or peel from the outside walls of the plinths and also from the pier caps and abutments (Figures 5 and 6).

The KY-770 bridge is a continuous steel-girder bridge having a number of flexure cracks that extended up the plinth walls (Figure 7). Those cracks acted as nucleation sites for failures of the masonry coating (Figure 8). District 11 personnel were of the opinion that some steel bridges were prone to deck and plinth cracking. Similar observations have been made by research personnel in the last two years.

The KY-312 bridge was constructed by the U.S. Army Corps of Engineers. The concrete was covered with a thick masonry coating unlike those normally used in Kentucky. Failures were observed on both flat and vertical surfaces of the plinths (Figure 9). Failures were more random than those encountered on the US-25E bridges and were not associated with an obvious concrete problem as were the crack-related failures on the KY-770 bridge.

In April 1984, District 11 personnel furnished information concerning all of their bridges with masonry coatings. Ten bridges in District 11 used masonry coatings. Four of those bridges had experienced debonding problems. Several others exhibited surface cracking that had probably reflected through from the underlying plinth. As noted previously, such cracking acts as sites for future masonry coatings failures. Three different masonry coatings were used: Thoro-Seal Acryl-60, TCA Bridge Coat XL-70, and Tex-Cote. Tex-Coat was spray applied. The others were applied by rollers or brushing. The coatings were applied by five different contractors between 1975 to 1983. The two oldest bridges built in 1975 and 1976 used a spray-applied masonry coatings (Tex-Cote). Both of those bridges exhibited debonding failures. The single Thoro-Seal bridge in District 11 had not failed. Two of the seven bridges using the TCA Bridge-Coat exhibited debonding. The masonry coating failures did not appear to be entirely a function of age, as several of the failures were on bridges that were only 2 years old. The type of form used for placing the concrete did not appear to be a factor. Two of the failed coatings were on bridges made with metal forms and two were on bridges made with wood forms.

The high failure rate in that district (40 percent) prompted a canvass of district engineers in the other 11 highway districts with a

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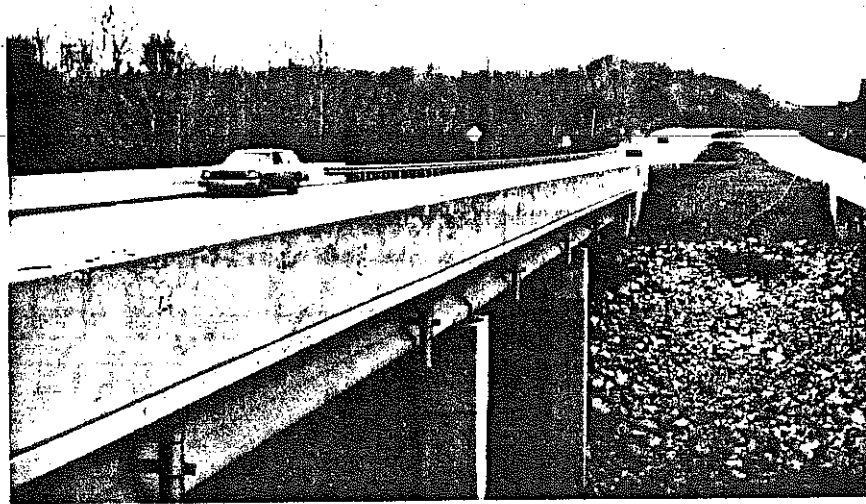
Figure 5.   Failure of Masonry Coating on US-25E Bridge over
             Big Richland Creek (Westbound) (1984).
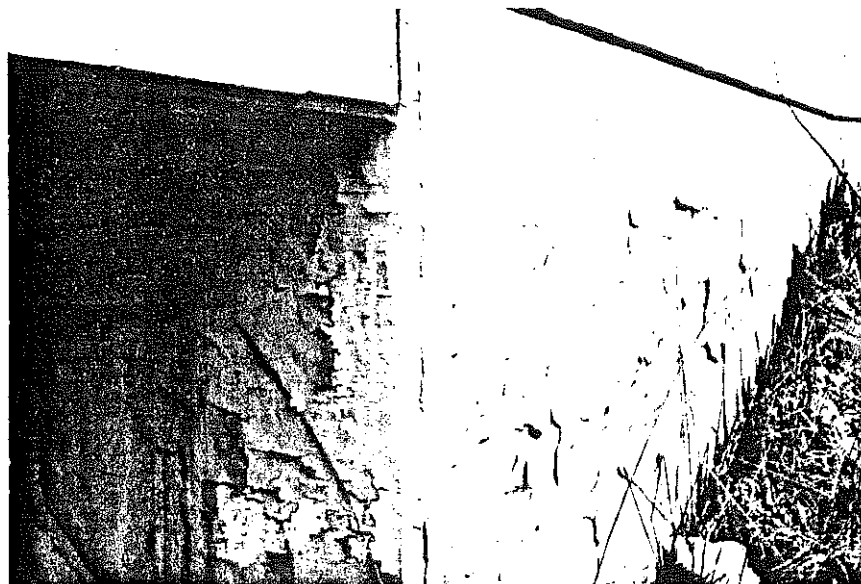
Figure 6.   Flaking-Type Failure of Masonry Coating on West
             Abutment of US-25E Bridge over Big Richland Creek
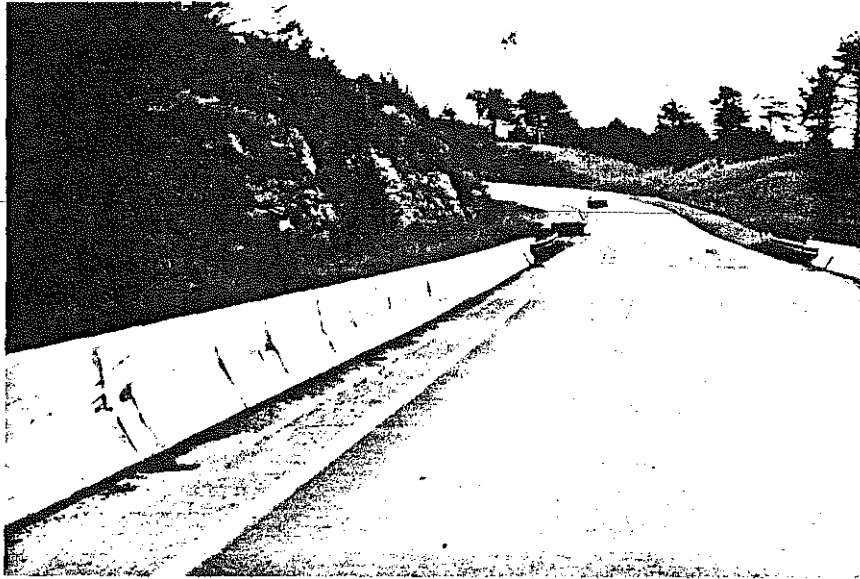   (1984).

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Figure 7. Failure of Masonry Coating on a Plinth Wall of
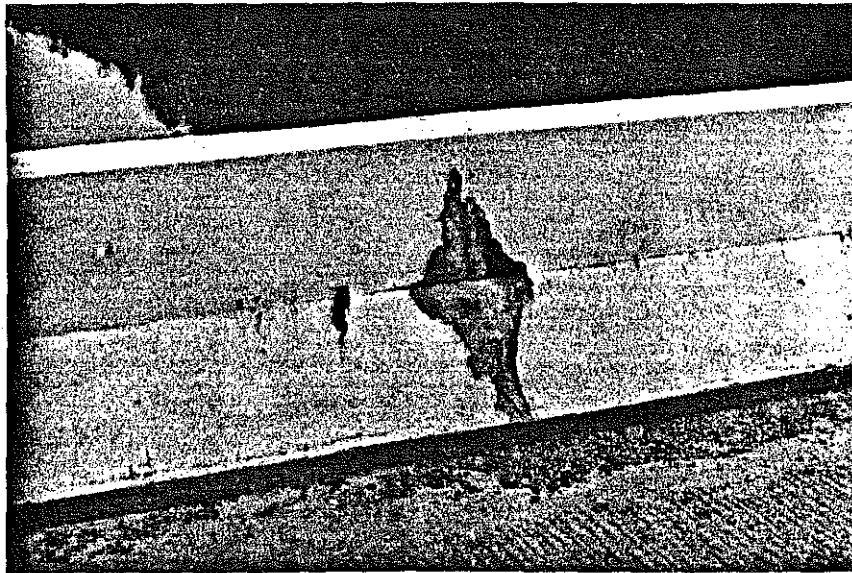the KY-770 Bridge over the Laurel River Reservoir
(1984).

Figure 8. Failure of Masonry Coating Spreading from
Vertical Crack in the KY-770 Bridge (1984).

Figure 9.   Failure of Masonry Coating on a Plinth of the KY-312 Bridge over Laurel River (1984).

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survey questionnaire to determine the extent and severity of masonry coating failures. Since the number of bridges in District 11 having masonry coatings was relatively small, no valid correlations or conclusions were apparent. The survey of the districts was necessary to achieve a more representative range of responses. Insight into the causes and possible cures for the debonding failures also were desired from the survey. Responses from the 11 other districts are summarized in the Appendix.

Responses to the questionnaire revealed a wide variance of field experience and service performance of masonry coatings. At the time of the survey, the districts' field experience with the coatings ranged from 4 to 12 years. Six of the other districts reporting exact figures had more bridges with masonry coatings than District 11. The number of bridges reported using masonry coatings ranged from 22 to 100. The five remaining districts did not furnish exact figures, but indicated widespread use of masonry coatings.

Five districts had few problems with masonry coatings (0 to 6 bridges). Six districts provided data indicating a higher frequency of masonry coating failures. Three districts reported 12 to 20 failures. One district reported typical masonry coating failure rates of 5-10 percent. Another district engineer stated that problems with masonry coating could be detected on every bridge, but noted that some problems were more severe than others.

Several districts furnished lists of bridge failures, including the location, contractor, brand of masonry coating, method of application and construction date. Review of that limited data revealed no definitive correlations for those variables. However, that information tended to indicate that many of the reported failures were construction-related. Many of the bridges that had masonry coating failures were only 2-5 years old at the time of this survey. A number of bridges reported to be in good condition had masonry coatings that were about 10 years old. One district reported 10 construction failures (which were repaired) but no subsequent service-related problems. Another district reported seven construction failures.

Survey responses indicated that wood was a more widely used form material than metal for concrete work. The data obtained contradicts

the opinion that forms built with wood yield a better adhering surface for masonry coatings than those built with metal. A possible correlation existed between masonry-coating failures and form-release compound. The three districts that reported using plain oil or kerosene for form-release lubricants experienced 5 to 20 failures. Two of those districts had failures that occurred predominantly during construction. Form oils are not retained by the concrete surface for an extended time period. Common oil or kerosene retained on the surface of the concrete are suspected sources of masonry-coating failures. Their presence on a concrete surface would prevent proper bonding between the concrete and a masonry coating.

Eight of the districts had no preference in the brand of masonry coating.

Eight respondents believed that inspections of masonry coating operations could be improved. Personnel from the three districts that said the present inspections were sufficient stressed the same inspection items as those who favored improved inspections. Those were for the inspector to assure that 1) the concrete surface was clean and properly prepared, 2) the ambient conditions were checked to determine if they were satisfactory, and 3) sufficient information from the manufacturer on properly applying the specific coating be provided to the contractor and inspector.

Nine respondents thought the contractor should better clean concrete prior to applying masonry coatings. Several respondents were unsure of the need for better cleaning. Two respondents favored washing the concrete prior to applying the coating, while several others favored rubbing.

District personnel felt that the worst masonry coating spalling problems were associated with plinth walls. Several failures also were noted on pier caps and other concrete members. Most spalls occurred on vertical surfaces. Some masonry coating failures were associated with horizontal surfaces, corners, cracks, and expansive aggregate pop-outs.

Seven respondents felt that there was an on-going problem with masonry coating failures. One felt the problem was related to metal forms. Another respondent believed certain brands of masonry coatings were prone to problems. A third respondent felt a problem existed, but

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that it was not a major one.

Eight districts had experienced problems with the contractors applying the masonry coatings. Those problems were related to 1) the contractor failing to properly clean the concrete surfaces, 2) the contractor not filling concrete voids prior to application of the coating (a commonly noted deficiency), and 3) the contractor failing to follow the manufacturer's coating instructions.

Eight respondents thought that masonry coatings were of benefit (i.e. low cost and good appearance). They felt debonding problems could be remedied or avoided with care. Two district personnel still favored rubbing.

Eight respondents favored making spot repairs and eventually reapplying new coatings over deteriorated ones. Several district personnel thought spot repairs would be unattractive and/or impractical but favored eventual recoating of spalled coatings. One respondent recommended lowering the allowable percentage of chert in the exposed concrete to reduce damage from pop-outs.

The survey did not pinpoint a single cause of masonry-coating failures, but did reinforce the opinion that prior cleaning of the concrete was an important factor. Also, the survey indicated the masonry-coating spalling problem was widespread and warranted further attention. Apparently, that problem was sufficiently common, nationwide, to be the subject of an article in the July 1984 issue of the Concrete International magazine (6).

Recommendations for resolving the problem of premature masonry coating failures were offered to the Transportation Cabinet (7). Those were based on three assumptions (lacking a comprehensive review of the problem): 1) masonry-coating materials supplied to the contractors were usually of good quality, 2) most failures of masonry coating were attributable to poor application practices, and 3) failures could have been prevented by proper and timely construction inspection.

The principal physical causes of failures were believed to be 1) failure to properly prepare concrete surfaces receiving the coatings and 2) failure to follow manufacturers' recommendations regarding ambient weather conditions during application.

Recommendations of an administrative and educational nature were

presented to increase the awareness of and the need for an understanding of the masonry-coating process. Additional suggestions related to the application process also were provided: 1) manufacturers should furnish necessary information for acceptable application procedures, 2) waste lubricants should not be used as a form-release oil, 3) the water-bead test should be conducted prior to applying the masonry coating, 4) all areas failing that test should be cleaned prior to application of the coating, and 5) temperature tests should be performed on the concrete surface prior to the application process to determine whether the masonry coating could be applied.

Presently, a special note is placed in invitations to bid regarding requirements for cleaning the concrete. Provisions in the note are based on the article on coatings from the Concrete International magazine. Also, the masonry-coating problem is stressed in annual meetings with district personnel. Apparently, some success has been achieved in reducing the frequency of failures of masonry coating.

STAY-IN-PLACE FORMS

In February 1983, a nationwide survey on the use of stay-in-place forms was conducted. 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 and standard drawings were returned, reviewed (see Table 1), and subsequently submitted to the Division of Construction.

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

Use of the forms usually was restricted to particular bridge types. 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 had resulted from 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,

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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	4
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 camber?	6	24	17
8a. If you permit the final grade to parallel the beam camber, 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 corregationa 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 reinforcement for the top and bottom 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 removeable forms?			
Metal (per square yard)	0-$22.50		
Concrete (per square yard)	0.30-$9.00		

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and 3) poor access to the slab for maintenance inspections. No problems were indicated for concrete forms. Problems with the metal forms in Kentucky have been corrosion and fall-outs.

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 on the beams with the final grade parallel to the beam camber. Personnel in states using the latter method felt 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 decks of variable depths. Of those, 8 used variable-height reinforcing chairs 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 to the beams.

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 that 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 any cracking problems.

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 $9.00 per square yard

and averaged about $6.50.

EXPERIMENTAL DECK FEATURES

In the early 1970's, a number of experimental bridges were constructed on or over I 64 between Frankfort and Lexington. Construction features were reported previously (8). Included were three bridge decks consolidated with a rotary disc, six bridge decks with broomed finishes, and six conventional (at the time) control bridges. One of the decks consolidated with a rotary disc also featured bi-layer construction.

The experimental features were inspected in May 1982. At the time of that inspection, the decks consolidated with the rotary disc (Kelly) compactor did not show superior performance compared to the control decks. In part, that may be due the absence of corrosive attack on the upper reinforcing mat of both types of decks. Rotary compacted decks are expected to be more resistant (due to superior compaction (i.e., decreased permeability to chlorides)) to deterioration than conventional (control) decks. The 2-inch minimum cover and Class AA concrete employed on the I-64 decks are effective deterrents to chloride-induced deck deterioration. Spalling of these decks in the near future is not likely because of these features. The bi-layered bridge (I 64 eastbound over US 60) had some 70 visible transverse cracks in the deck at the time of inspection. The westbound (control) bridge deck had only 25 visible cracks.

All of the I-64 mainline bridges have cracks in their decks. Many of those cracks were open and could be penetrated by a pocket knife blade. Those bridges will eventually be good indicators of the susceptibility of cracked bridge decks to chloride-induced spalling.

The I-64 bridges that had broom-finished decks exhibited wear in the wheel tracks about 1/16-inch deep in the outer (traffic) lane. That wear was sufficient to obliterate the broomed striations. The wheel tracks in the inner lane were worn, but the striations were still visible. The broomed finishes on the overpass roads were in good condition. Striations in the wheel paths of several of the bridges

showed some wear, but striations were still intact.

Initially, the skid resistance of the broomed decks was higher than the conventional drag-finished decks. Wear of the broomed finish in the wheel tracks of the outer lane of the mainline bridges resulted in a decrease in skid resistance. In 1979, after 5 years of service, the skid resistance of broomed-finished mainline decks, in the outer lanes, was equal to that of drag-finished decks. Broomed-finished overpass bridges did not show a significant decrease in skid resistance. That corresponded to the lower amount of wear observed in the wheel tracks of those bridges. Oddly, the skid resistance of the drag-finished overpass bridges increased with time until they compared favorably with the broom-finished bridges. That may be due to a slight wear or roughening of those decks during service. At the time of the inspection, it was recommended that less frequent but deeper and broader striations be placed in bridge decks by rolling or tining. Based on more recent observations of broomed and tined finishes, it is recommended saw-cut grooves be considered.

SEGMENTAL BRIDGES

The construction of a precast segmental bridge, Ramp "B" over US 23 in Pike County, was monitored. The bridge was a three-span continuous bridge with vertical and horizontal curvature and superelevation. The bridge had two 93´-6" side spans and a 185´-0" main span.

The bridge superstructure consisted of precast concrete segments held together by epoxy and post-tensioned tendons (bars and strands). The segments were single-cell trapezoidal box sections (Figure 10). The boxes were of constant depth along the bridge. The upper flanges of the boxes were 9 1/2-inches thick and extended outward 7´-5 1/4" on both sides of the box. The upper flange served as the deck. The lower flanges were 8 inches deep. The boxes had slopping webs 8´-3" deep. The segment webs were 1´-1" wide. Corners of the boxes were provided substantial chamfers to house the post-tensioning cables and strengthen the box. Ducts for placing the post-tensioning tendons were cast into the chamfered corners and into the webs and flanges. Both webs and the
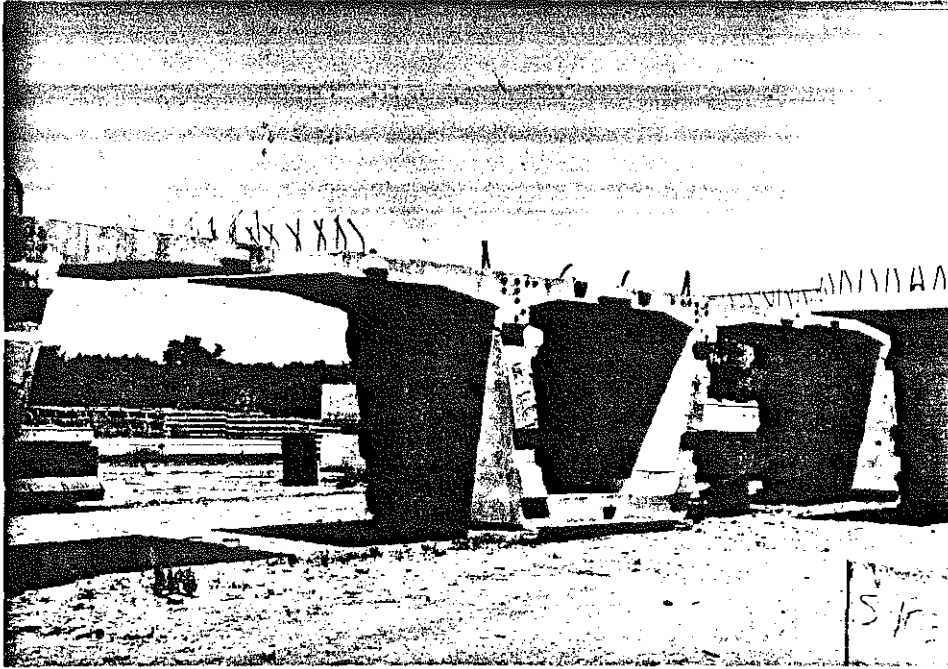
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Figure 10. Typical Box Segment.

upper flange contained three shear keys, each cast in the end faces of the segments.

The bridge used 48 precast segments and a cast-in-place key. The normal segments were 7´-10" long. The two pier segments were 8´-0" long and the two abutment-end segments were 5´-5" long. The cast-in-place key was 3´-11 3/4" long. The pier and abutment segments contained transverse diaphrams. The abutments and their adjacent segments were filled with concrete to act as gravity anchorages.

In May 1984, fabrication of the precast concrete segments were observed. Segments were cast at the Construction Products Corporation (CPC) Shop in West Lafayette, Indiana. CPC had the form necessary to cast the small segments used in the approach ramp. That form was previously used to precast segments for three small bridges built in Indiana: 1) the Muscatatuck River bridge US 50 near North Vernon (1975); Sugar Creek bridge, State Route 1620 (1976); and the Turkey Run State Park bridge (1977).

The segments were fabricated according to "Special Notes for Precast Concrete Structural Members, Ramp "B" over US 23, ACAPP23-1(43), Drawing 20373, Pike County." Specifications required use of Class D concrete. The maximum size of aggregate was 3/4 inch. The mix design specified a maximum of 6.5 bags of Type I or III cement per cubic yard of concrete. The 28-day compressive stress was required to be 5,500 psi. The concrete had to achieve 2,600 psi prior to form removal. The concrete was to have 5.5 $\pm$ 1.5 percent entrained air and a maximum slump of 5 inches (when using a water-reducing admixture). CPC used 2,800 psi as the minimum compressive strength for form removal. This usually was obtained in about 18 hours. A segment was not lifted from the casting site until the concrete had obtained a minimum strength of 3,500 psi, which was achieved normally after about two days of curing. Originally, a super water-reducing agent was included to achieve the necessary concrete strength; however, the concrete achieved the required strength without the admixture. CPC elected to not employ the super water-reducing agent. W. R. Grace air-entraining agent, Daravair (approximately 2 1/2 oz per cubic yard), and a set-retarding agent, Daratard 17 (18.3 oz per cubic yard), were used. The cement was supplied by Lone Star Cement of Indianapolis, Indiana.

Reinforcing steel was specified to be Grade 60. Reinforcing steel, including tie wires, chairs, and supports, in the top slab of the segments was epoxy-coated (Figure 11). Welding was not permitted on any of the top-slab reinforcing steel. Prestressing steel strands were specified to be ASTM A 416, Grade 270. The prestressing steel bars were specified to be ASTM A 722, Grade 150.

Box segments were cast using the precast-cell method (9). Forms for a single segment were supported by two small cars on tracks in front of a foreman's shed. The shed contained two transits for adjusting the casting form and for placing the adjacent segment. CPC performed a dimensional inspection on the completed segments as soon as they were moved into place for casting the adjacent segment. CPC consulted with segmental bridge experts to determine any necessary dimensional adjustments. The casting process was a continuous operation wherein each successive segment was match-cast in the order of its placement in the structure from the lowest to highest location. Match-casting required the end faces of completed segments to serve as forms for succeeding units. Also, each completed segment and the matching form of the adjacent uncast segment had to be positioned to accommodate the curvatures and superelevation of the ramp.

As shown in Figure 12, the car holding a completed segment had been cleared from the side forms; and a reinforcing cage had been placed on a car located at the casting site. Then, the completed segment was repositioned adjacent to the form to serve as a match-casting for the uncompleted segment (Figures 13 and 14). The completed segment and form were adjusted using hydraulic jacks mounted on the cars. Surveying instruments were used to precisely position the two in relation to each other. The reinforcing steel and post-tensioning ducts were placed prior to concrete casting (Figures 15 and 16).

The interior cavity of the box segment was created by a central form inserted through the reinforcing cage by another car on which that form was mounted (Figure 17). The sidewall (outer face) forms were stationary in respect to the casting site, but could be moved in and out or up and down.

The face of the completed segment that was to serve as a mold was covered with pipe-gasket compound to prevent bonding with the new
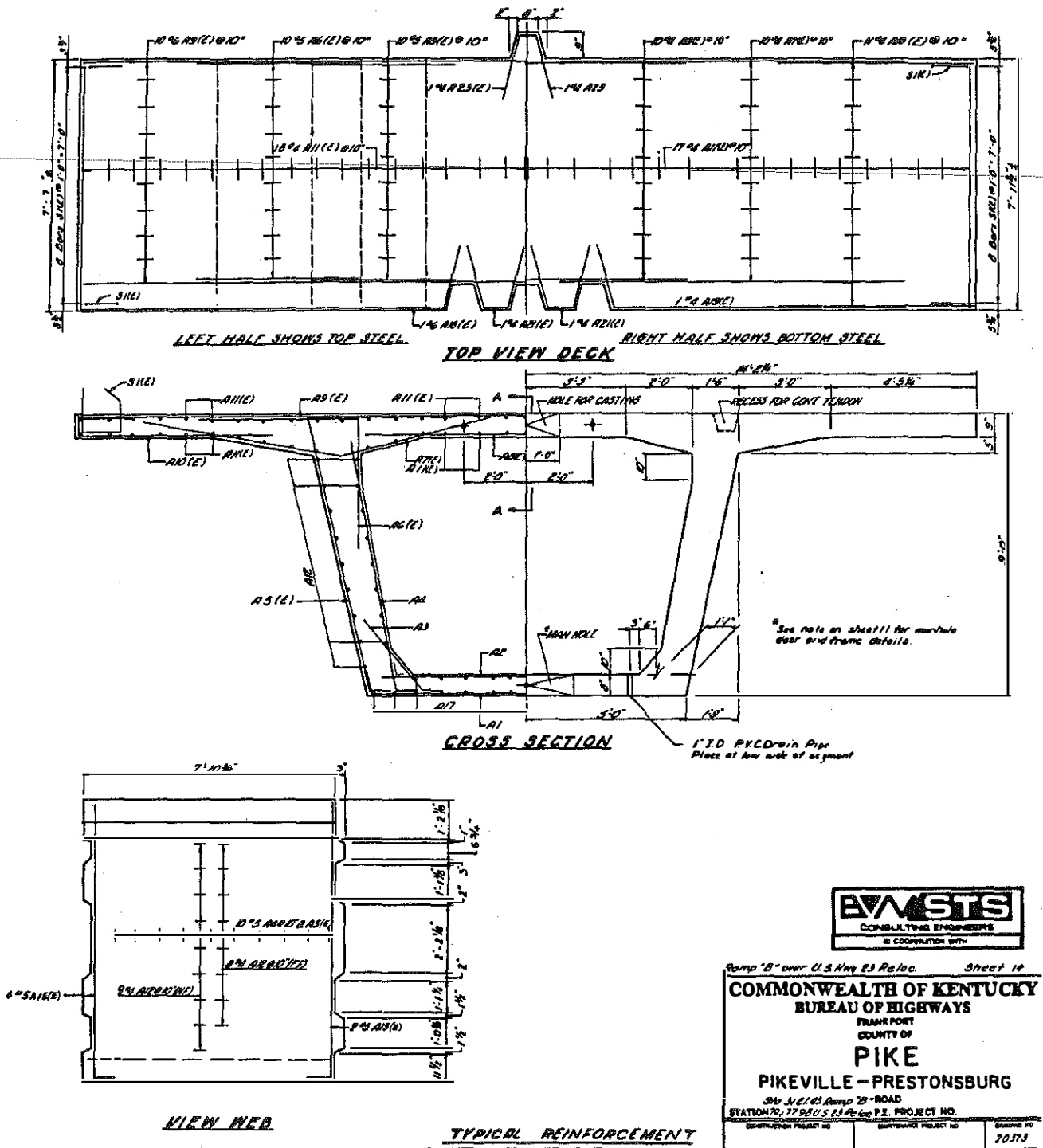
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Figure 11. Typical Bridge Segment Showing Reinforcing Steel
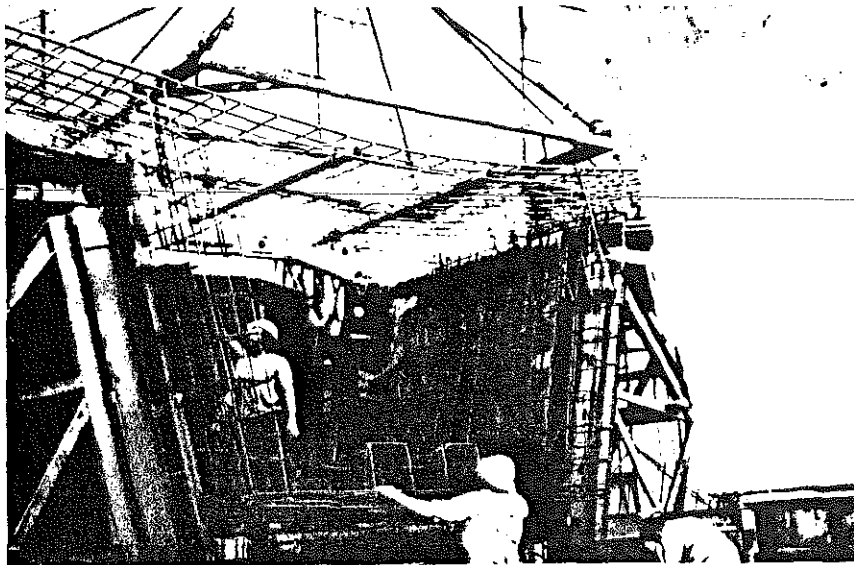   Placement.

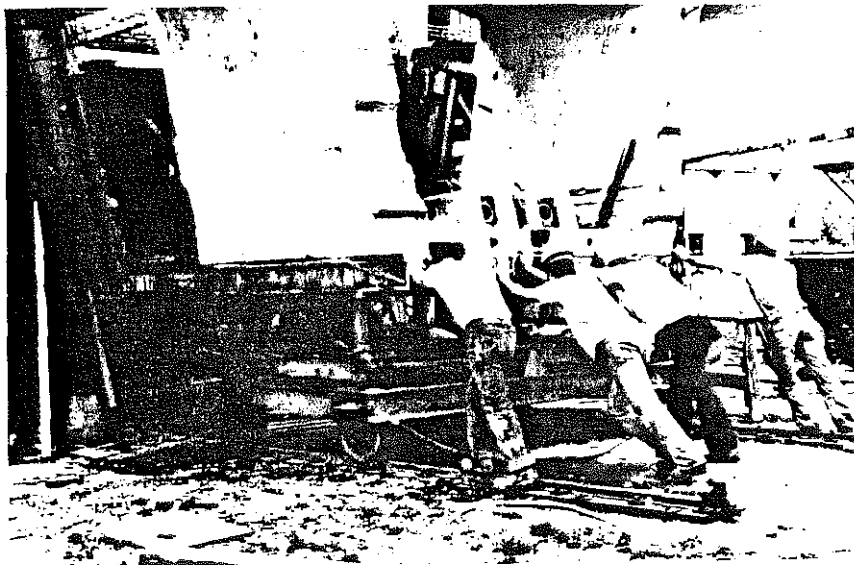Figure 12. Placing Reinforcing Steel in Segment Casting Form.

Figure 13. Repositioning the Completed Segment Prior to the Casting Operation.
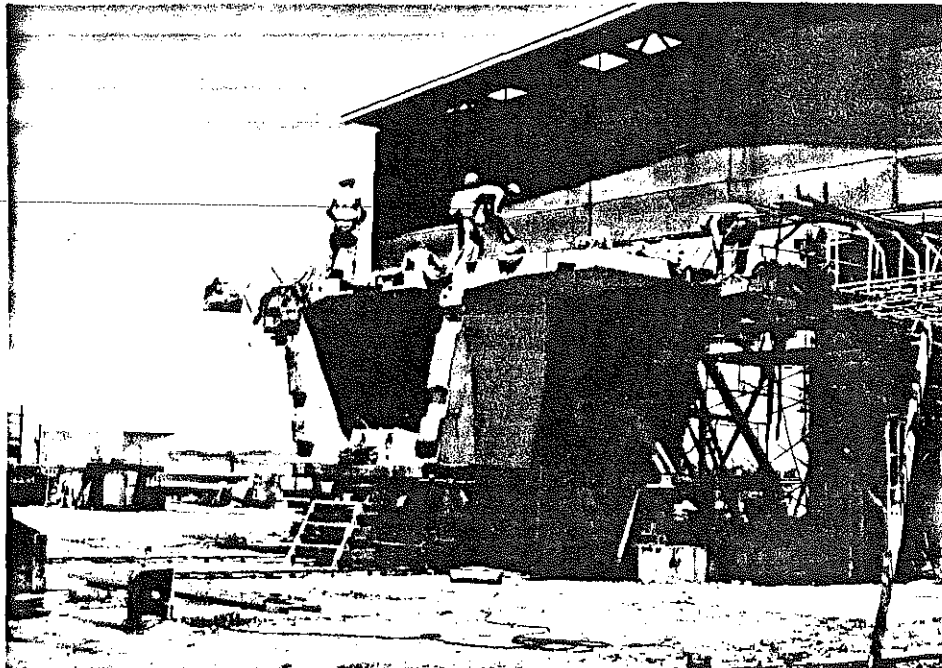
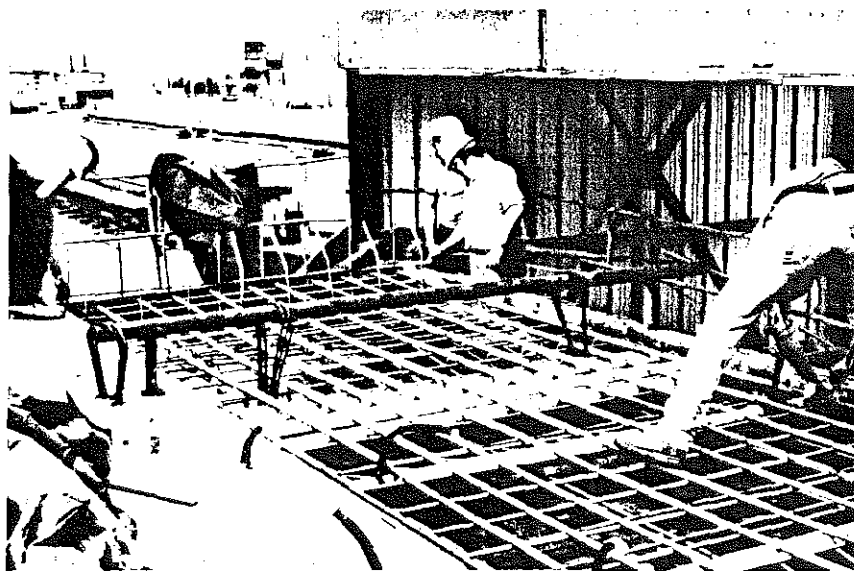Figure 14. Segment Abutted against Casting Form.

Figure 15. Adjusting Reinforcing Steel and Post-Tensioning
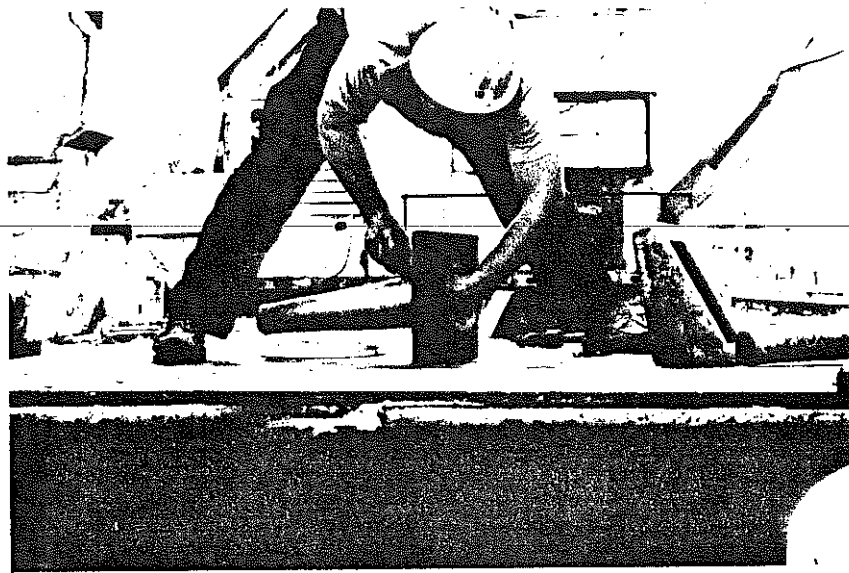Ducts Prior to Casting.

Figure 16. Preparation of Strand Wedging Cones Prior to Placement in the Form.
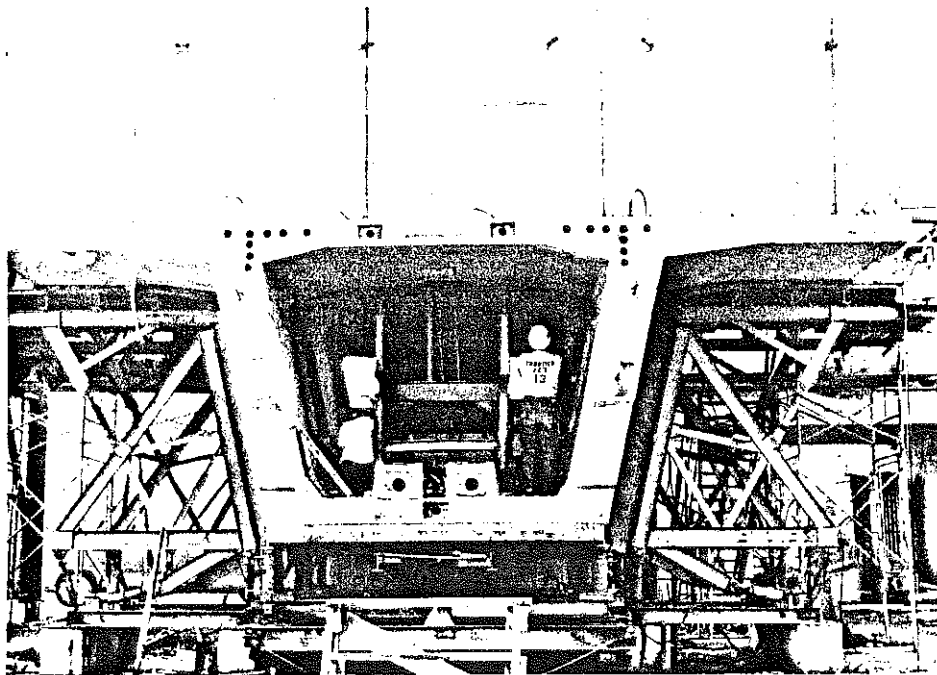

Figure 17. Inner Form for Segment Box in Place Prior to Casting.

concrete. Form faces were covered with a concrete mold-release compound called Slippit manufactured by the ParkChem Company. The compound was applied as a liquid using mineral spirits as the vehicle. It was allowed to dry to a powder before the casting process was started.

Segments were cast by overhead pouring from a 1-cubic yard crane-mounted bucket (Figure 18). Each standard segment required 14.2 yards of concrete.

The concrete flowed from the open top of the form down the sidewalls (webs) to the open-topped lower flange. Concrete flow was facilitated by internal vibrators and bin-type vibrators mounted on the sidewall forms. Concrete on the top surfaces of the upper and lower flanges was manually screeded and floated (Figure 19).

As soon as the required setting strength was achieved (based on cylinder strengths from samples taken at the time of pour), the matching segment was rolled away from the newly cast piece (Figure 20). Usually, by that time, that segment possessed sufficient strength to be lifted from the car and taken to a storage area prior to shipment (Figure 21). Side forms were pulled from the newly cast segment (Figure 22). Then, the car-mounted interior cavity form was pulled backward to clear the segment. The first car was removed from the tracks and the second car containing the newly cast segment was towed to its place. The first car was then placed on the tracks in the casting location, and the process was repeated.

Due to curing-strength requirements, the casting rate was less than one per day. The CPC work began in late October 1983 and ended in mid-September 1984. The 48 segments were scheduled to be shipped in September. Each segment cost $8,608, for a total delivered cost of $413,184. Section weights varied from 35 to 40 tons. Pier segments weighed 42 tons.

The average form-release concrete compressive strength was 3,987 psi. The average 28-day compressive strength of the concrete was 6,134 psi. Air contents varied from 5 1/2 to 6 1/2 percent.

The construction contractor was the Melco-Greer Construction Company. Field work began on September 28, 1984. The two ramp abutment pedestals and two piers were cast between September and November 1984. Segments were delivered to the jobsite in May 1985 (Figure 23). The
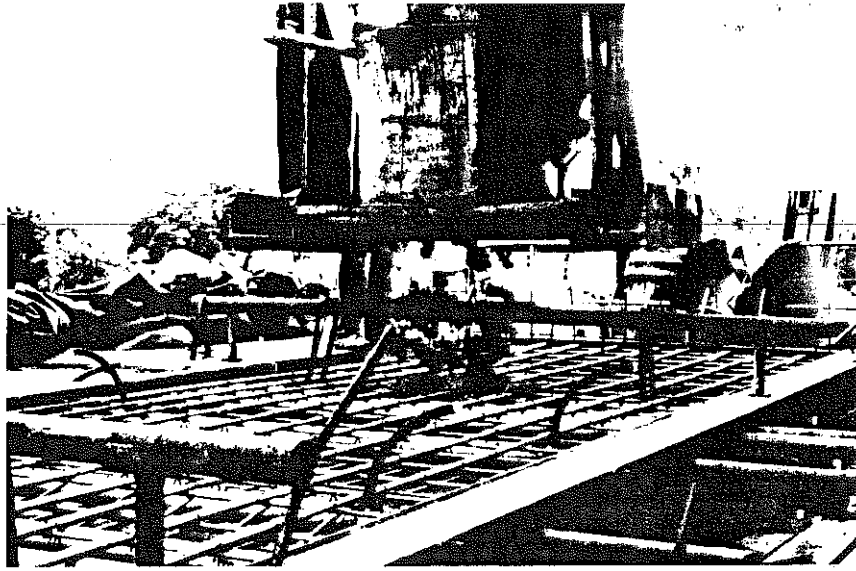
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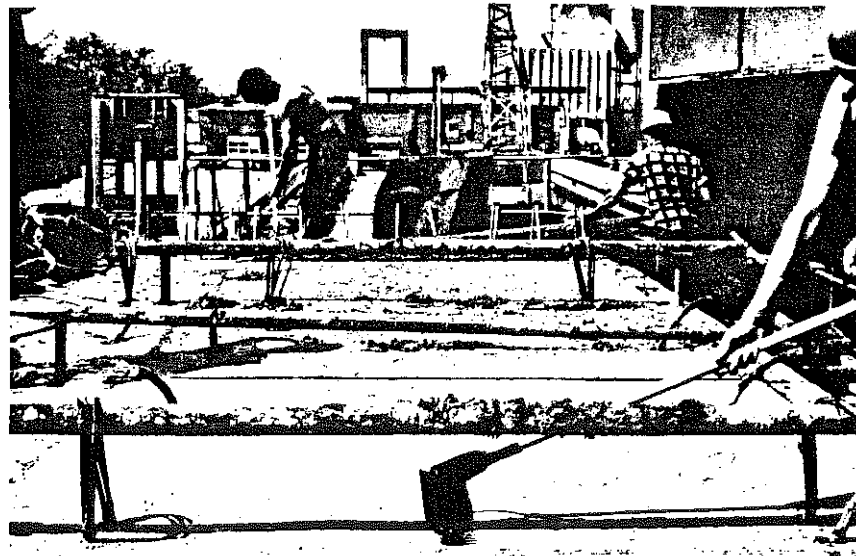Figure 18. Casting a New Segment from a Crane-Mounted Bucket.

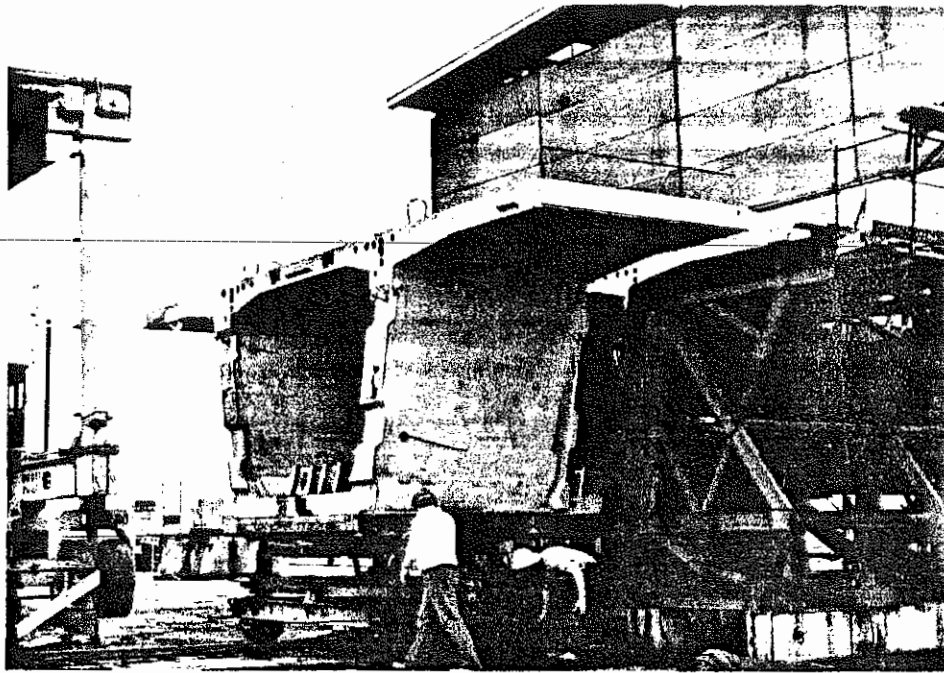Figure 19. Finishing Top Surface of Segment Upper Flange.

Figure 20. Removing Previously Cast Segment from the Newly Cast Piece.

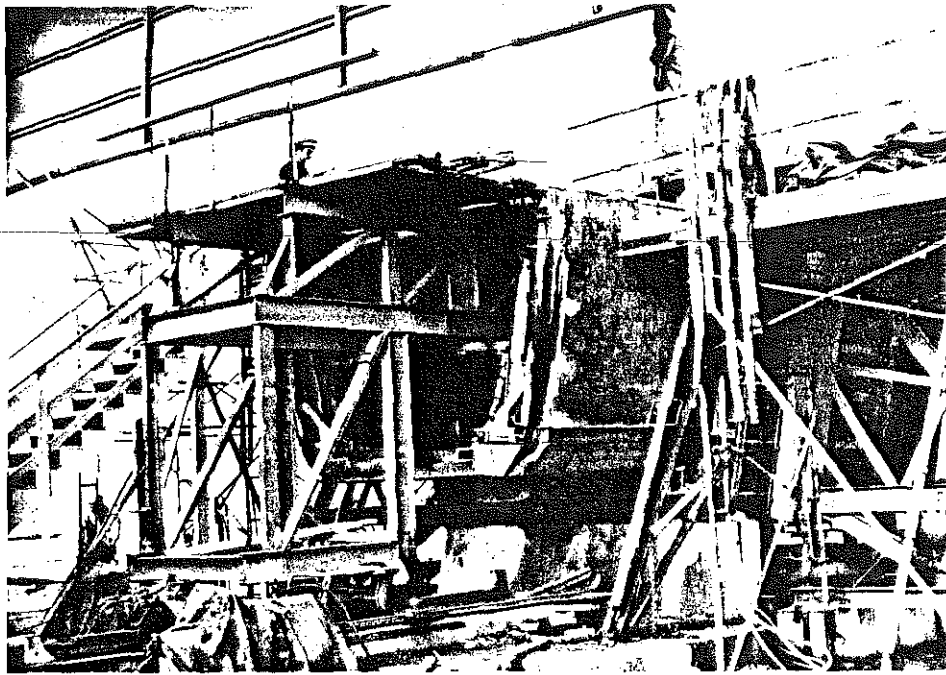Figure 21. Lifting the Previously Cast Segment from the Car.

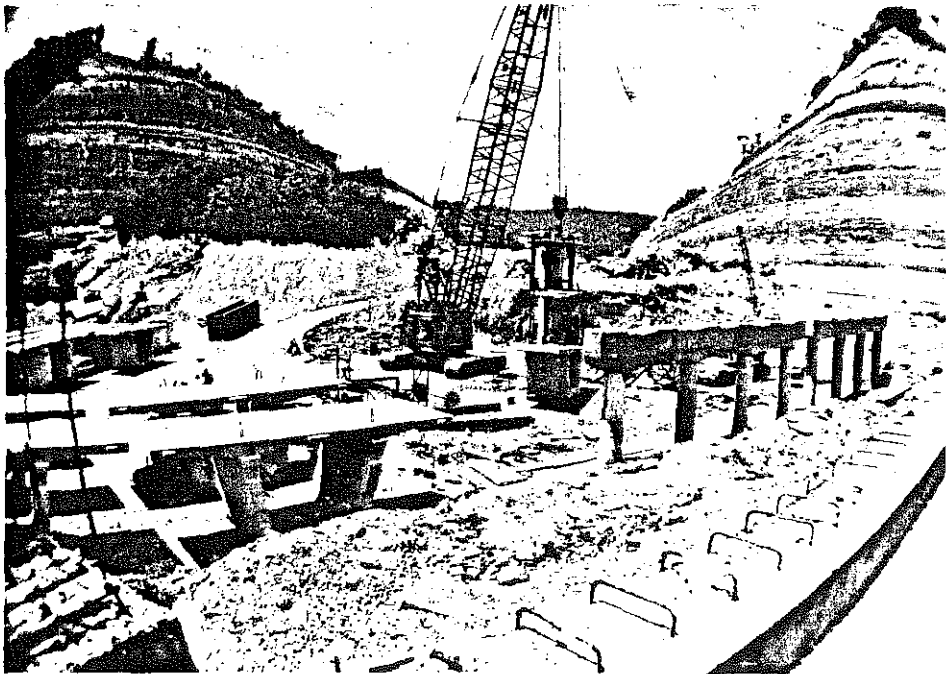Figure 22. Newly Cast Segment after Removal of Side Forms.

Figure 23. Construction Site at Pikeville Prior to Segment
Installation.

first segment over the west pier was placed in mid-May 1985 (Figure 24).

Segments were hoisted into place using a special four-point lifting rig designed to keep the lifted segments relatively level (Figure 25). The rig was attached to cables cast into the segments. Those were later torched flush to the top of the slab prior to placement of the deck overlayment. A 150-ton capacity Manitowoc 4000M crane was used to position the segments.

The balanced cantilever method of construction required balanced placement of segments on both sides of the pier segment (Figure 26). The platform shown on the left segment contained reels intended for paying out the post-tensioning strand. The reels apparently were never used. Pier segments were temporarily supported and attached to the ground by four prestressing bars cast into the pier footing. Actually, the cantilever construction is not balanced as an extra segment must be attached on one side of the pier at times during construction. The additional moment was compensated for by the four prestressing bars and the four-legged temporary support erected about the piers. Pier segments were carefully placed as they would greatly affect the alignment of the cantilevered beams as construction progressed.

Prior to joining the segments, their mating faces were manually coated with a two-component epoxy (Figure 27). Mating segments were aligned with three 1 1/4-inch diameter post-tensioning thread bars (Dywidag). Two of the thread bars were located in the upper flange, the third in the lower flange. Then, the hoisted segment was pulled into the cantilevered segments by tightening nuts on the post-tensioning bars (Figure 28). The bars were post tensioned to 131.3 kips. For bars 7´-10" to 8´-0" long that corresponded with an 11/32 inch bar extension and a 7/32 inch bar extension for 5´-0" long bars. Excess epoxy was squeezed from the joint. The epoxy between the segments acted both as a bond and as a sealant between segments (Figure 29). Once the epoxy had set, the nut was removed from the end of the exposed Dywidag bar and a coupler was screwed on the end. That was used to splice on a new bar added with each segment. The threaded bars were stressed usng portable electric-powered hydraulic jacks. The jack fitted over a pull rod designed to thread over the thread bar protruding from the anchor nut. The jack contained a socket wrench and rachet that allowed the nut to be
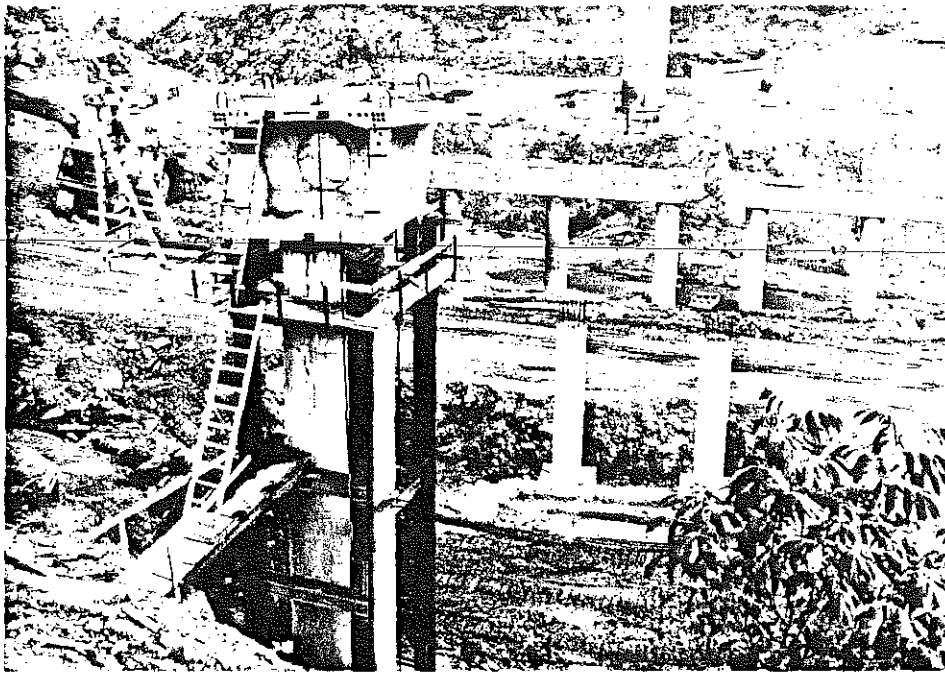
Figure 24. Segment Installation on West Pier (June 1985).
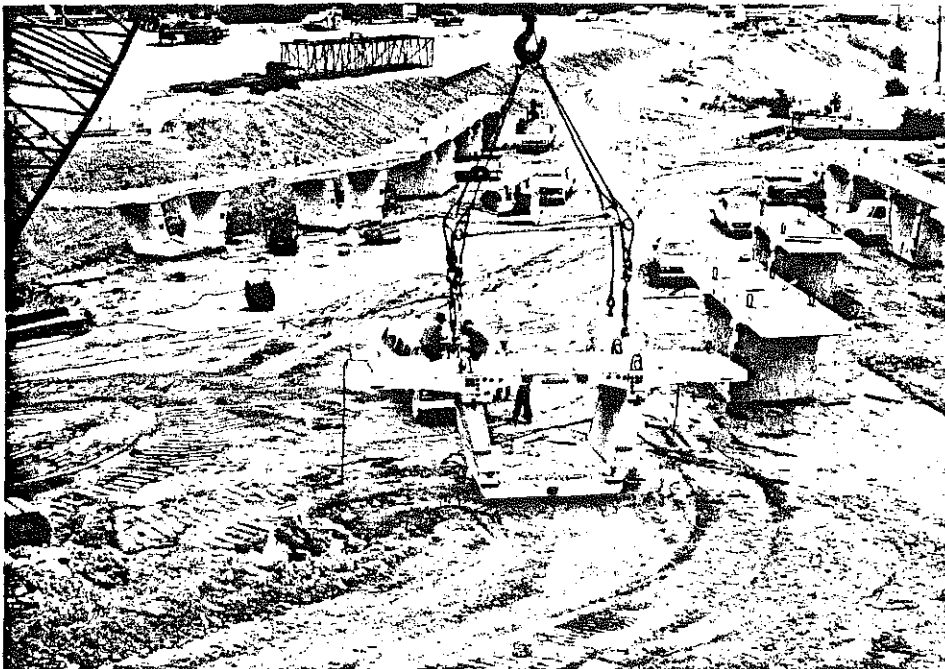
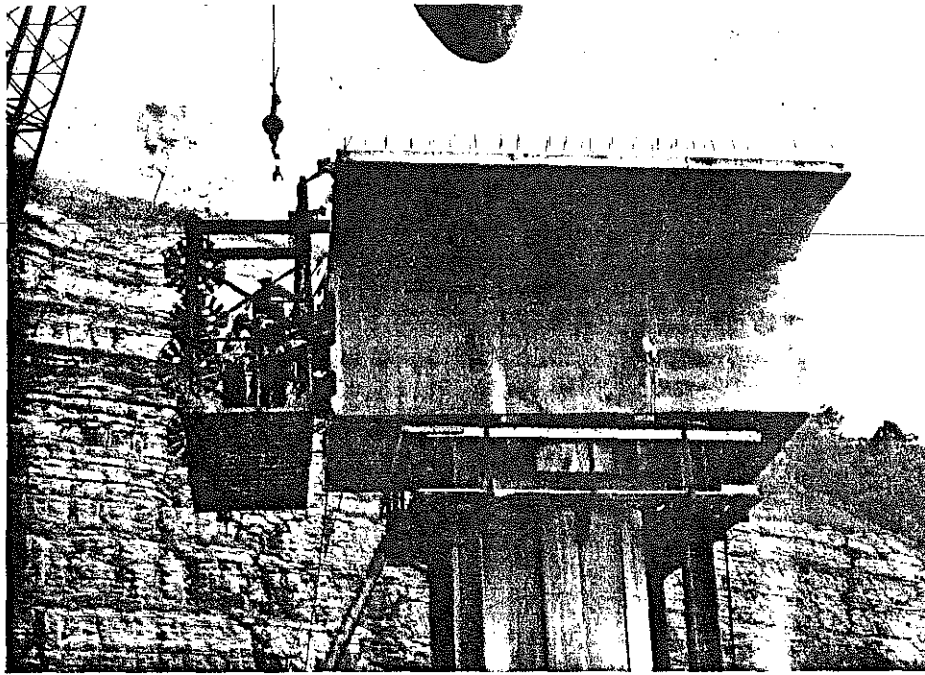

Figure 25. Lifting Rig Used to Hoist Segments.

Figure 26. Placement of Cantilever Segment over the West Pier (June 1985).

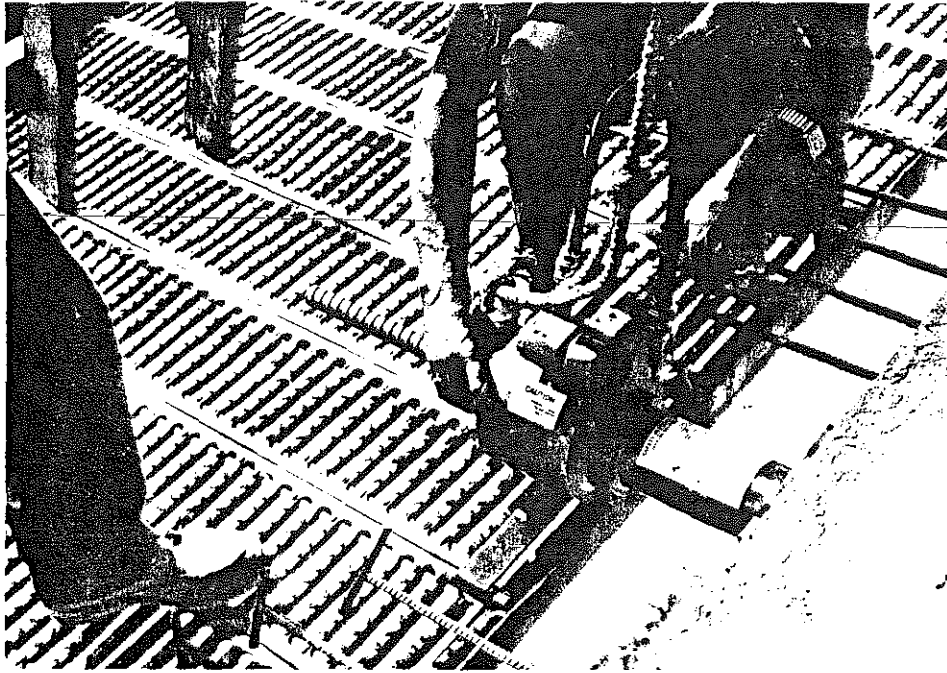Figure 27. Manually Applying Epoxy to Mating Faces of the Segments.

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Figure 28. Tightening Post-Tensioning Bars to Join Two Segments.
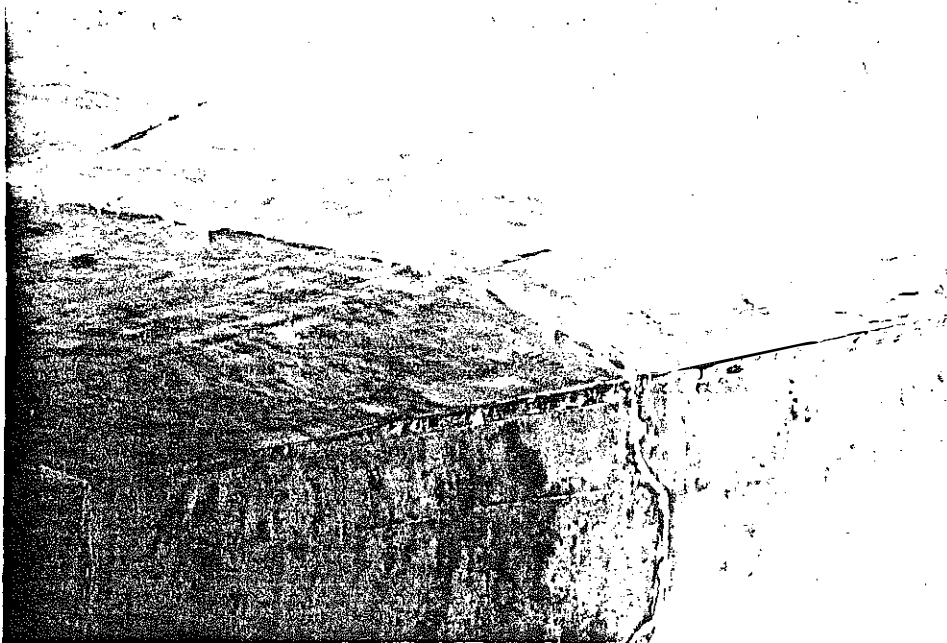

Figure 29. Excess Epoxy Extruded from a Joint after Post-Tensioning.

tightened, elongating the thread bar. The prestress force was read from a hydraulic pressure gage or by measuring thread-bar elongation. Also, a counter mounted on the jack recorded the number of revolutions of the anchor nut, which was a measure of thread-bar elongation. The three post-tension bars were left in place as permanent reinforcement.

During the post-tensioning process, it was determined that shear keys on the webs of several segments were too large due to shop repairs of casting defects. Those keys spalled when the segments were being pulled together (Figure 30). Later, those areas were ground and the spalls were repaired by epoxy patching. Also, cracks were detected in the bottoms of two segments. Those were repaired by epoxy injection prior to post-tensioning.

Once the necessary number of segments were placed, the 12-wire, 0.6-inch diameter prestressing strands were manually pushed through the strand ducts. Retaining plates were located at the ends of the strand ducts and hydraulic jacks were used to post tension the strands (Figure 31). The strands were tensioned using center-pull rams. Alternating-end stressing was used for the cantilever tendons and one- or two-end stressing was used for other segment locations.

The bridge contained 68 six- or nine-strand tendons. The tendons were mounted in pairs, one on each side of the superstructure. Twenty-four cantilever tendons were anchored in the end faces of the webs at approximately mid-height and were ducted upward through the web-to-upper flange chamfered corners at each pier. From there, they were ducted back down into the webs of mirroring segments on the opposite side of the pier segment. The tendons radiated out from the pier segment with one or two tendon pairs anchoring each matching segment pair or cluster. Ten mainspan continuity tendons located in the mainspan were ducted along the lower flange at the lower chamfered corner of the trapezoidal box. They were routed along the mainspan until they radiated inward and anchored in bosses cast into the upper surface of the lower flange. Two other centerspan continuity tendons were ducted along the upper flanges of the box near the webs. Those terminated in the upper flanges of the pier segments on the sidespan faces. Four sidespan continuity tendons were located in the lower flange of each sidespan connecting the abutment segments with several segments near the piers. Those were
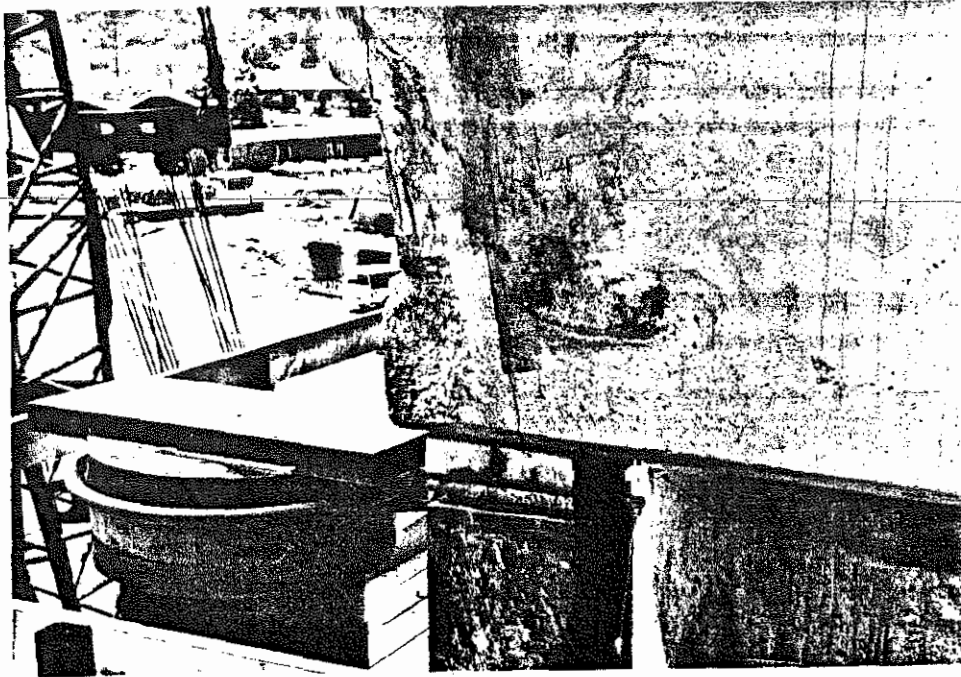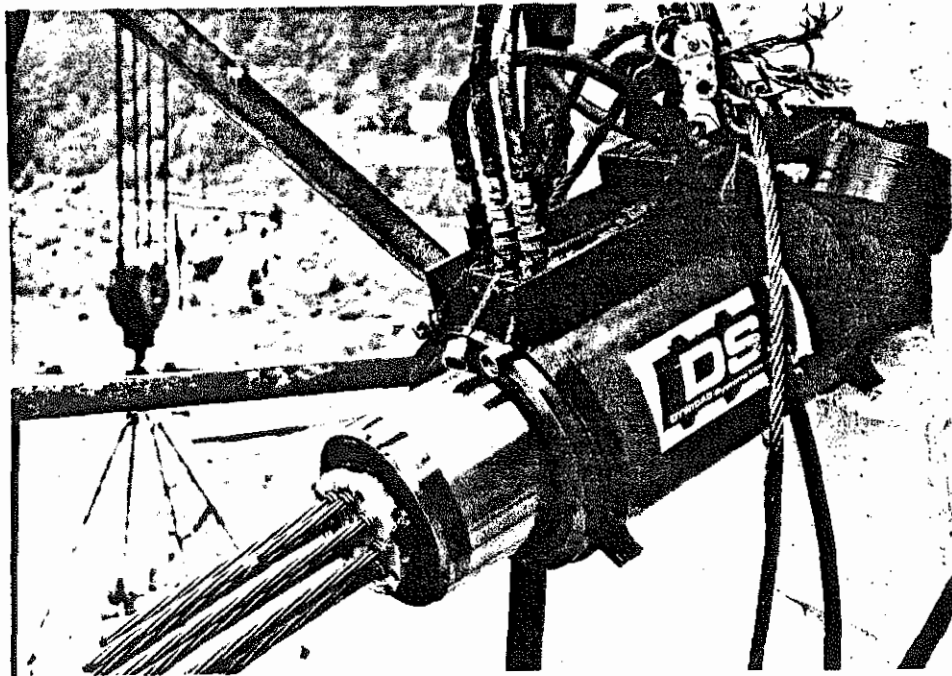
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Figure 30. Damaged Shear Key.

Figure 31. Post-Tensioning of the Strand.

anchored in the diaphrams of the end segments and in bosses cast into the lower flanges of the segments at the opposite ends of the tendons.

The 24 cantilever tendons transversing each pier segment were post-tensioned first. The post-tensioning started with the segments closest to the piers and progressed with construction to tendons terminating at segments farther from the piers. The four sidespan continuity tendons were tensioned next. The two upper-flange tendons in the mainspan were post-tensioned next. Those were post-tensioned when the concrete in the center cast-in-place key reached a minimum concrete strength of 1,000 psi. The remaining ten mainspan tendons were post-tensioned last when the key strength reached 4,500 psi.

The six-strand tendons had ultimate strengths of 351.5 kips. Those were post-tensioned to either 261.5 or 281.5 kips. The nine-strand tendons had ultimate strengths of 527.3 kips. Those were post-tensioned to either 392.2 to 421.8 kips, depending on location.

Dywidag jacks were placed over the strand extensions and were in bearing on the wedge plate (Figures 32 and 33). During the stressing, the tendon strands are secured by a gripping device at the rear of the jacks. The tensioning force was measured by hydraulic pressure and tendon elongation. The Dywidag jacks power-seated all strand wedges after the desired initial post-tension force was applied. After stressing, the jack was removed and the excess strand elongation cut off.

The cantilever construction proceeded from both sides of the west pier until the end segment was attached over the west abutment (Figure 34). Placement of the west pier segment was completed in July 1985. Cantilever construction from the east pier was begun in September 1985 and completed in November.

To achieve the required alignment between the cantilevered sections, they had to be rotated about the piers prior to being set on the permanent elastomeric bearing pads. That was done by pushing on the cantilevered ends near the abutments with a bulldozer. The west section was rotated in August 1985, and the east section was rotated in November 1985.

The completed sections were 3′-11 3/4" apart once the sections were rotated and placed on the bearings (Figure 35). The midspan key was

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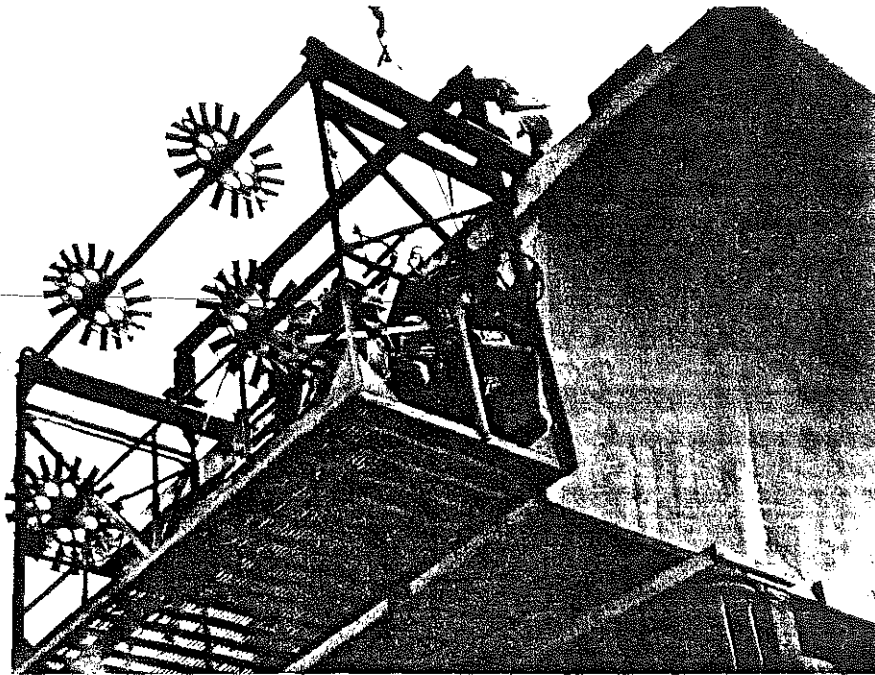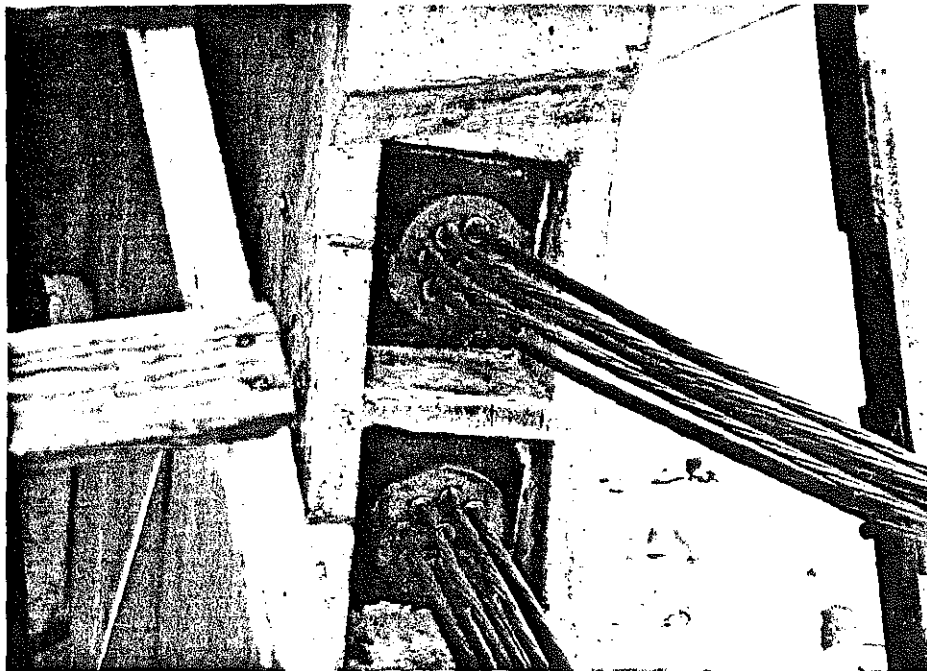Figure 32. Post-Tensioning of a Cantilever Tendon.

Figure 33. Post-Tensioned Nine-Strand Tendons. Note the Backing Plate and Split Wedges Securing the Tendon Strands.
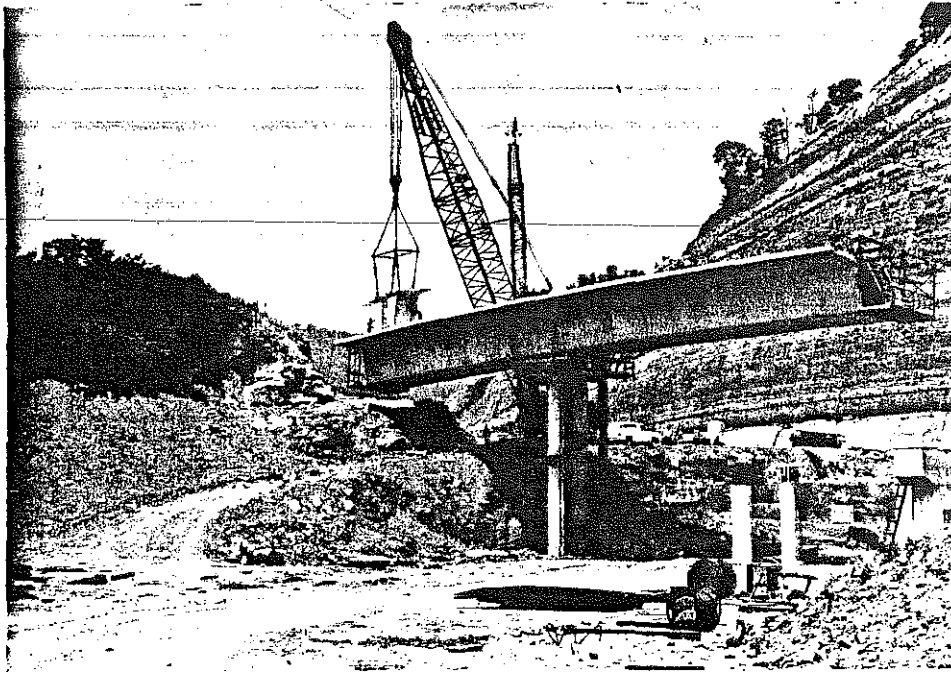
Figure 34. Cantilevered Construction at the West Pier. The
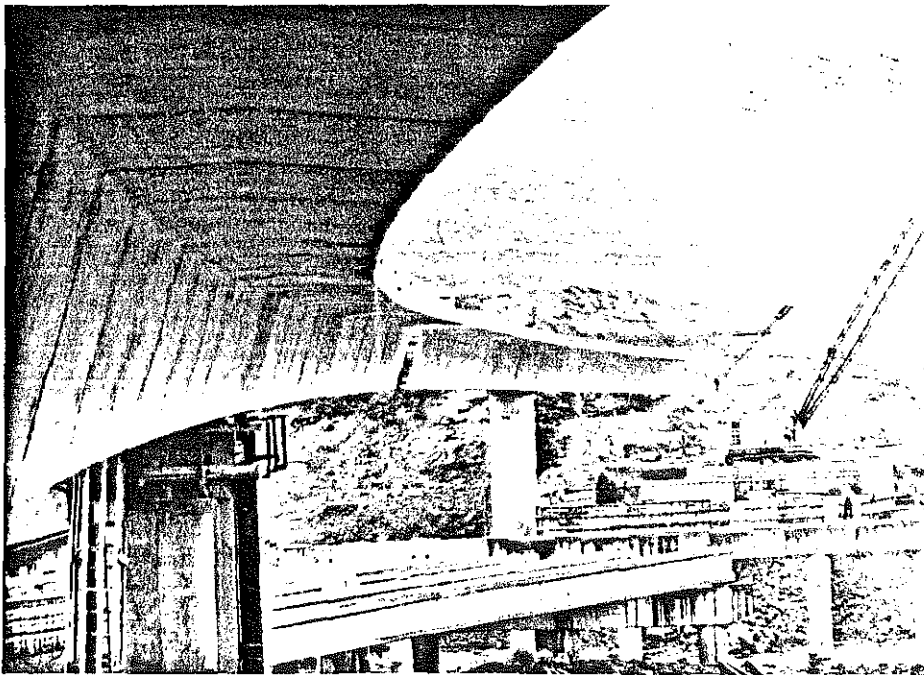Segment over the Abutment Is Being Hoisted into
Place (July 1985).

Figure 35. Gap between the East and West Cantilever Sections
Prior to Placement of the Midspan Splice.

made by forming the gap between the sections and field casting the splice (Figure 36). Ducts were placed in the key to provide necessary tendon ways for post-tensioning. The key was cast in March 1986. The cast-in-place key used concrete that had a specified 28-day compressive strength of 5,500 psi. Final post-tensioning was completed a week later. All tendon ducts were filled with grout within 30 days of the post-tensioning operation (Figure 37).

The abutments were completed in June 1986 and a 1 1/2-inch thick latex concrete overlay was applied in July (Figure 38). Figure 39 shows the completed bridge prior to masonry coating.

In addition to early shear key problems with a few segments, several of the post-tensioned strands loosened and needed to be reset. The segment erection rate was slow at the onset of construction. Initially, placement rate on the west section was about one segment every two days. The anticipated rate of two segments per day was achieved by the time the contractor was finishing segment erection on the east section.

A cost comparison was made between the two Kentucky segmental bridges, the Ramp "B" bridge over US 23 in Pike County and the East-West Connector KY 676 twin bridges over the Kentucky River in Frankfort. The contrasting structures were two steel plate-girder bridges: the Hulen-Alva Road bridge over KY 52, the Seaboard R.R., and the Cumberland River in Bell County and the KY-461 bridge over Buck Creek in Pulaski County. Two spans for the Ramp "B" bridge were 93'-6" and one span was 185 feet. The bridge had a useful width of 16'-2" and a deck area of 9,970 square feet. The KY-676 connector bridges had two spans of 228 feet and one of 323 feet. Each structure had a useful width of 41'-6" and a deck area of 34,845 square feet. The Hulen-Alva bridge had two spans of 135 feet and one of 210 feet. The bridge had a useful width of 19'-2" and a deck area of 13,490 square feet. The KY-461 bridge had two spans of 145 feet and one of 220 feet. The bridge had a useful width of 45 feet and a total deck area of 23,270 square feet.

The final bid cost of the Ramp "B" superstructure was $74.65 per square foot of deck (1983). The final bid cost of the KY-676 twin bridges was $66.03 per square foot (1978). The final bid cost of the Hulen-Alva bridge was $49.60 per foot (1986) and of the KY-461 bridge was $55.80 per square foot (1985). The bid costs and final bid dates

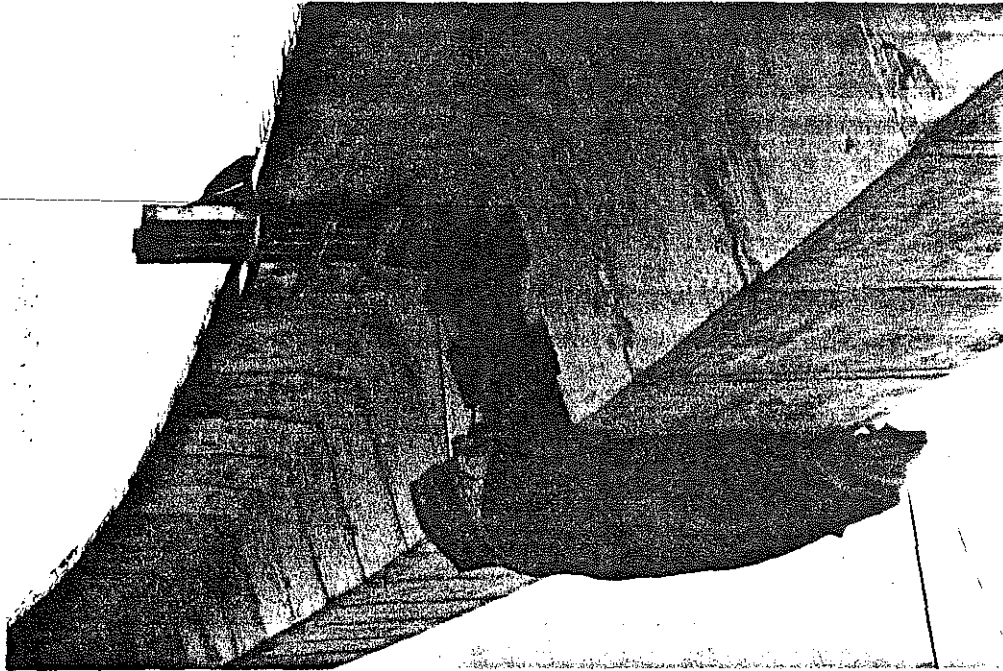41

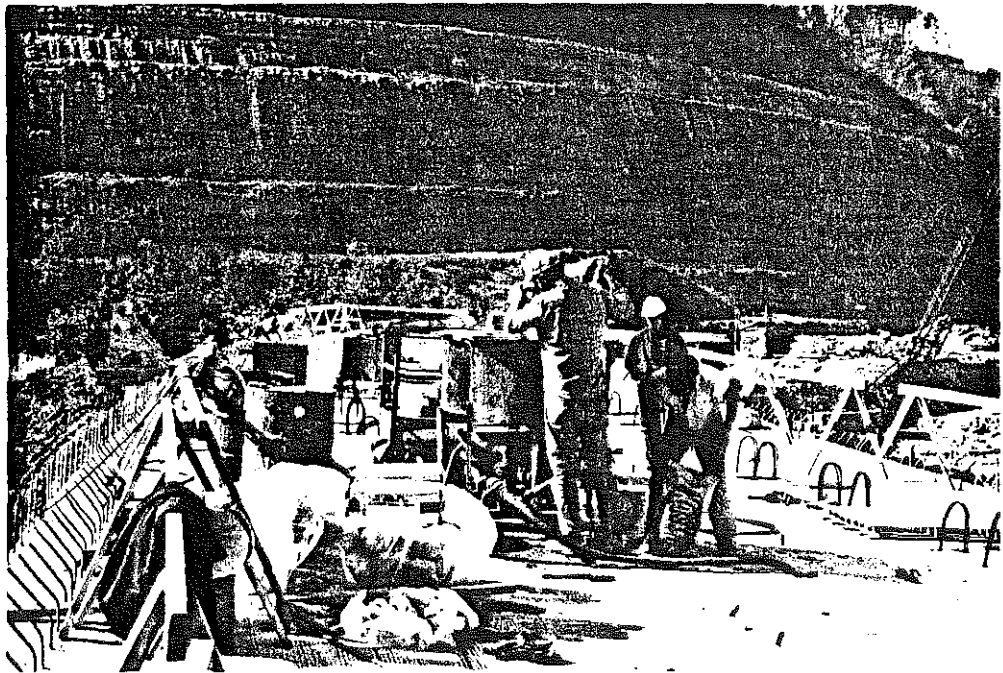Figure 36. Form for Placing the Midspan Splice.

Figure 37. Grouting the Strand Ways.

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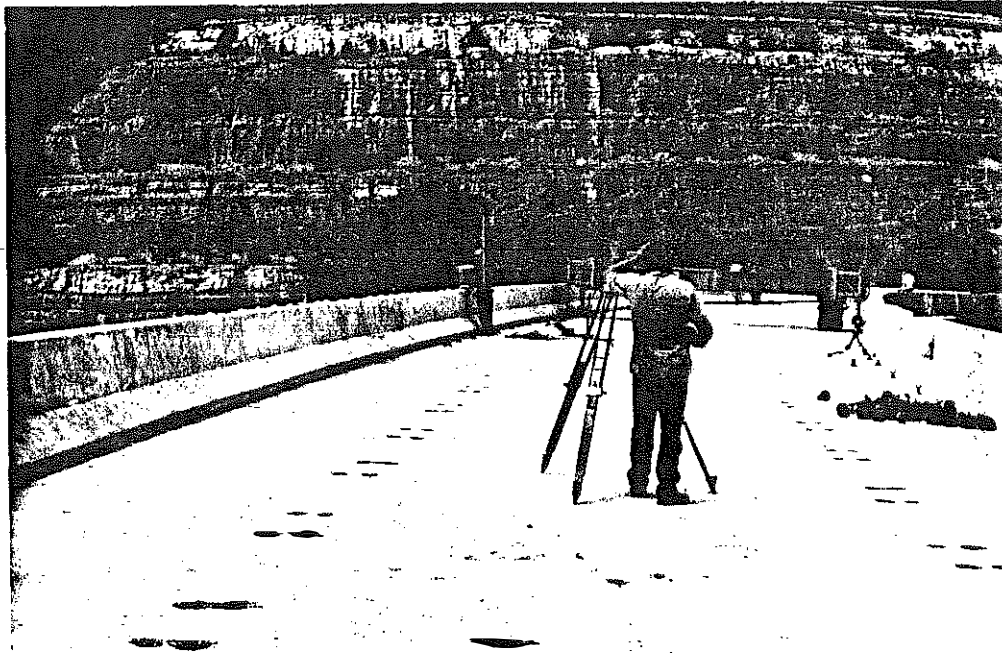
Figure 38. Completed Bridge Deck Prior to Placement of the Overlay.
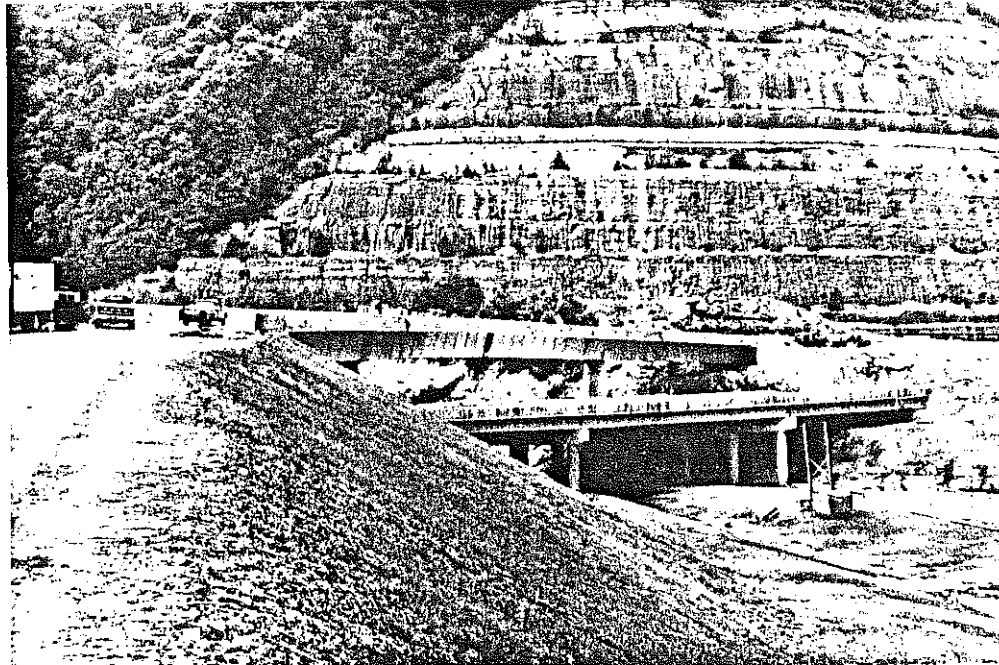
Figure 39. Completed Segmental Bridge Prior to Masonry Coating.

(in parenthesis) were taken from the Division of Bridges Summary of Bridge Design Data Sheets.

STEEL CORROSION-CONTROL METHODS

Several experimental corrosion-control methods were inspected during the course of this study. Included were three weathering (ASTM A 588) steel bridges: the CR-5418 bridge 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 at Shawhan in Bourbon County (constructed in 1977). One bridge, I 24 westbound over KY 93 near Paducah (constructed in 1977), utilized a hot-dipped zinc coating for corrosion protection.

The weathering steel bridges were inspected in 1982. The CR-5418 and equestrian bridges were in good condition. The KY-1893 bridge had unhindered corrosion at the girder ends at the abutments. Normally, weathering steels will corrode at a very slow rate. When weathering steel remains wet for long periods, the corrosion rate is much higher. That was the apparent problem on the KY-1893 bridge. The ends of that structure were screened from wind that would normally evaporate ponding water. It was recommended that the ends of that bridge be painted to prevent further corrosion. Also, it was suggested that the use of weathering steel be restricted to bridges where frequent wetting and water retention would not be a problem.

The zinc-coated westbound I-24 bridge was inspected in 1982. The zinc coating showed some signs of initial corrosion. The superficial appearance of the zinc coating was satisfactory, but not exceptional. Also, at the time of inspection, the zinc-coated beams showed some signs of dirt adhesion from initial construction. The bridge was cleaned by maintenance personnel shortly after the inspection. The bridge showed no signs of rusting, indicating the zinc coating was functioning properly. The eastbound I-24 bridge was painted with two coats of lead-based primer and two coats of aluminum finish paint and was intended to be a control bridge. The quality of the paint job was excellent.

A comparison of those two bridges (10) indicated the galvanizing of

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the westbound bridge cost $17,000 for 117.64 tons of steel. Painting the eastbound bridge cost $6,500 for 132.31 tons of steel. The hot-dip galvanizing cost $95.35 more per ton of steel than the paint. However, the actual difference is somewhat less due to need for field inspection of the painting on the eastbound bridge.

MICROSILICA CONCRETE

Microsilica is an ultrafine superpozzolanic material that densifies concrete, improves its strength, and decreases its permeability. In May 1985, an experimental microsilica concrete bridge deck overlay was placed on the KY-2097 bridge over the East Fork of Graves Creek near Sebree in Henderson County. The microsilica additive used was Emsac, produced by Elkem Chemicals Inc. The initial mix design was as follows:

cement.	94.0 lbs
Emsac	33.5 lbs
water	14.0 lbs
fine aggregates	200.0 lbs
coarse aggregates	200.0 lbs

The material was mixed at the jobsite using a mobile mixer. The slump varied from 7.0 to 8.5 inches and the content air varied from 7.8 to 12.0 percent (11).

The concrete was placed with a spinning-drum paving machine. When two-thirds of the overlay was in place, the operation was stopped due to the inability of the mix to produce an adequate amount of paste at the overlay surface. As a result, an excessive amount of water was sprayed on the concrete to get proper finish.

The following day, the overlay was completed using a Bidwell vibrating-screed (low-slump) paver. The mix design was altered by increasing the fine aggregate to 220 lbs and decreasing the coarse aggregate to 180 lbs. The concrete had a slump of 8 1/4 inches and an air content of 6.5 percent. The overlay was completed using the new paver and concrete mix without further problems.

In June, ten microsilica prisms were provided by the Division of Materials for freeze-thaw testing. Five specimens were prepared using laboratory mixes. The other five were made from materials taken at the jobsite during placement of the initial overlay mix. Testing was performed in accordance with ASTM C-666 "Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing." One freeze-thaw cycle was approximately three hours in duration with a temperature variation of $0^\circ \pm 3^\circ F$ to $40^\circ \pm 3^\circ F$. Periodic measurements were taken of the prism weight and sonic modulus during the test.

All specimens passed the durability test. Performance is given by the Durability Factor (DF) equation (12):

$$DF = P_c N/M \qquad (1)$$

in which DF = durability factor of the test specimen,

P_c = relative dynamic modulus of elasticity at N cycles, percent,

$$= P_c = (n_1^2/n^2) \times 100 \qquad (2)$$

N = number of cycles at which P reaches the specified minimum value for discontinuing the test or the specified number of cycles at which the exposure is to be terminated, whichever is less,

M = specified number of cycles at which the exposure is to be terminated.

n = fundamental transverse frequency at 0 cycles of freezing and thawing, and

n_1 = fundamental transverse frequency after c cycles of freezing and thawing.

The average Durability Factors of the five laboratory samples was 98.0 after 419 rapid freeze-thaw cycles. The average Durability Factor of the five field samples was 67.0 after 352 freeze-thaw cycles. An additional four laboratory specimens made with field materials used on the KY-2097 bridge were subjected to freeze-thaw testing to 350 cycles.

The average Durability Factor of those specimens was 91.6.

Air-void contents were determined for a field specimen and a laboratory specimen by Phillip E. Cady P.E. of Pennsylvania State University for Elkem Chemicals Inc. (13, 14). Those tests were performed in accordance with ASTM C 457 "Microscopical Determination of Air-Void Content and Parameters of the Air-Void System in Hardened Concrete." The specimens had satisfactory air contents; however, the void spacing of the field secimen indicated its air bubbles were too large. Mr. Cady felt the freeze-thaw performance of the laboratory specimen would be superior to the field specimen, which was supported by the freeze-thaw tests.

The Division of Materials performed 90-day saltwater ponding tests on 18 microsilica prisms. The tests were performed in accordance with AASHTO T-295 and T-260. Test results indicated that salt-penetration characteristics of microsilica concrete are similar to latex concrete used for bridge overlays.

Corrosion-potential tests were performed on the bridge deck in August 1985. Tests were run on both the control (east) and the micro-silica (west) sides of the deck. Most test values were high (over 0.35 volts), indicating the deck had been tested too soon after placement of the overlay. The final construction report noted that an inspection of the bridge in October 1985 had revealed some cracks in the microsilica side of the deck, but no corresponding cracks on the control side. The deck finish on the microsilica side of the deck was inferior to that on the control side, due in part to placement problems experienced with the rotating-drum paving machine.

No further microsilica applications have been made by the Transportation Cabinet.

FAILURES OF ALUMINUM NUTS

In 1985, an investigation of failures of aluminum nuts that retained guardrails on bridge plinths was undertaken. That type of failure had been observed previously (1974-5) in the Lexington area on New Circle Road (KY 4).

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A recently cracked aluminum guardrail nut from an unspecified bridge (furnished by the Division of Bridges), along with several other corroded nuts taken during the previous examination of the KY-4 bridges, were examined at the University of Kentucky Metallurgical and Materials Science Laboratory. Scanning electron microscope (SEM) analyses revealed the fractures were characteristic of stress corrosion cracking (Figure 40). Previous visual inspection of the nuts had revealed exfoliation (leafing) corrosion typical of preferential corrosion of one phase of a two-phase alloy. SEM analysis also revealed that the fracture surfaces showed traces of chlorine.

In August 1985, three New Circle Road bridges were inspected. Those were constructed between 1965 and 1969 and included the Clays Mill Road, Lansdowne Drive, and Richmond Road overpass bridges. Varying degrees of nut failures were observed. The Lansdowne Drive bridge had no failures; the Clays Mill Road and Richmond Road bridges had 20 and 40 percent failures, respectively. Both bridges exhibited stress-corrosion cracking and exfoliation corrosion type failures (Figures 41 and 42). Oddly, the nuts that had not corroded were in very good condition.

Subsequently, sample nuts from the New Circle Road (failed specimens from the Clays Mill overpass and good specimens from the Lansdowne Drive overpass) and the failed aluminum nut furnished by the Division of Bridges were forwarded to Kaiser Aluminum and Chemical Corporation for elemental analysis (Figure 43). Tests revealed the corroded nuts were made from a 2024 series alloy rather than the 6061-T6 alloy specified (15). The uncorroded nuts met the 6061-T6 specifications. The 2024 aluminum alloys have two-phase microstructures susceptible to both exfoliation and stress corrosion. The active corrodant was probably deicing salt normally found around bridges. It would provide the chlorine detected by the SEM analyses.

Apparently, some party unaware of the consequences mixed wrong nuts with batches of correct 6061 alloy nuts. That would account for random nut failures experienced on Kentucky bridges having the currently obsolete aluminum guardrails.

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Figure 40. Scanning Electron Microscope Picture of Nut Fracture Surface (X300).

Figure 41. Nut on Guardrail Exhibiting Exfoliation Corrosion -- New Circle Road (KY 4) over Clays Mill Road In Lexington, Kentucky.
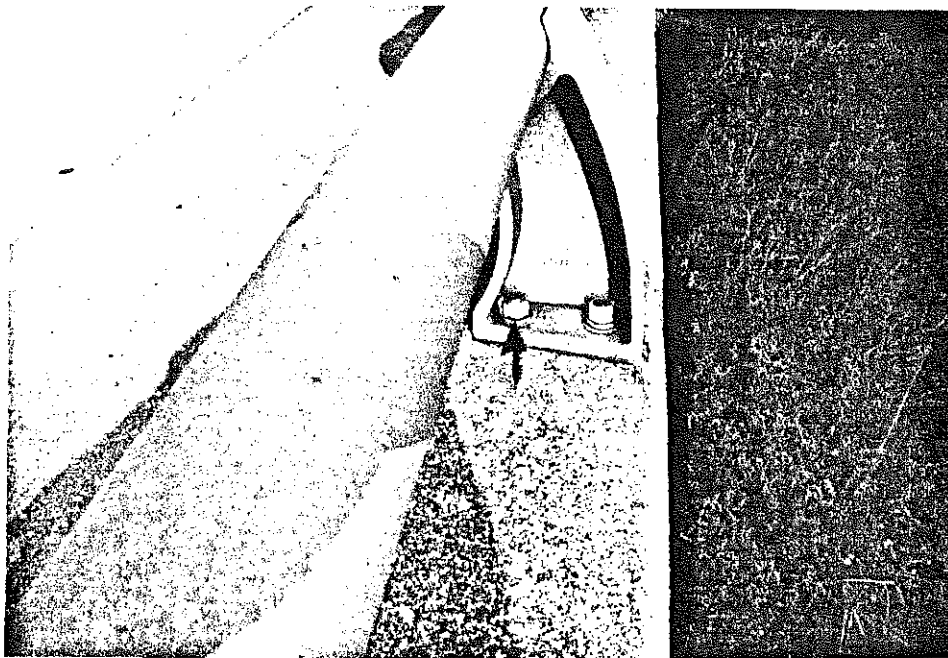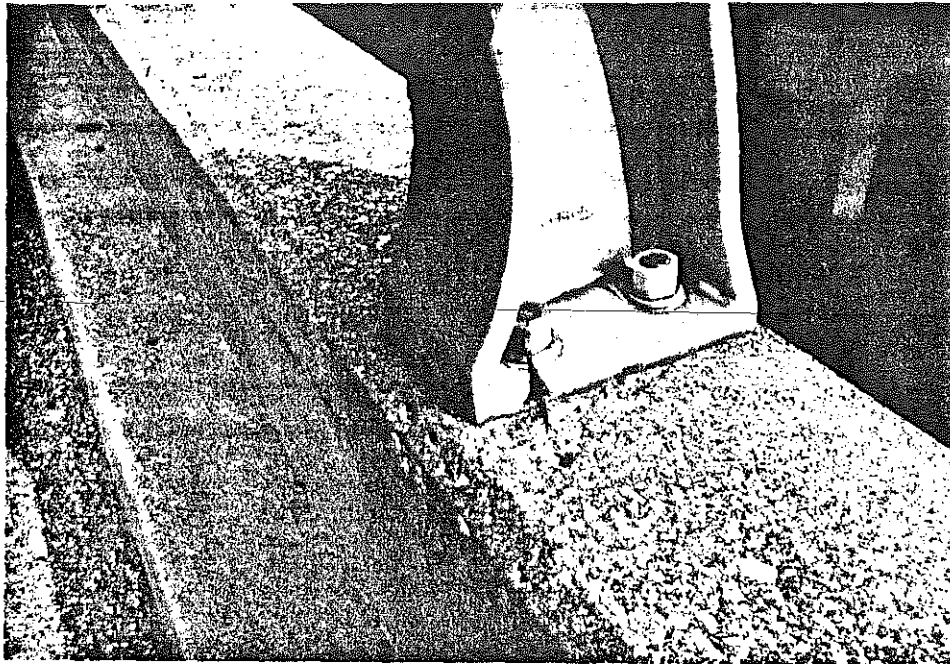
Figure 42. Nut on Guardrail Exhibiting Stress Corrosion Cracking -- New Circle Road (KY 4) over Clays Mill Road in Lexington.
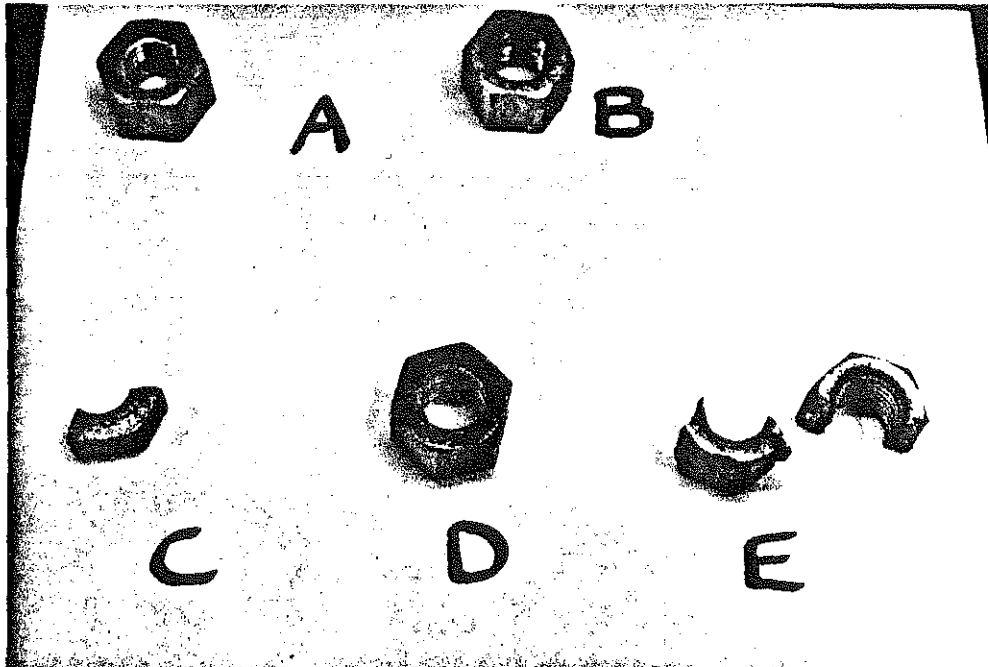

Figure 43. Nuts Sent to Kaiser Aluminum and Chemical Corporation for Element Analysis and Alloy Identification. Note that Nuts A and B Are in Good Condition. Nuts C, D, and E Have Cracked.

SUMMARY AND CONCLUSIONS

Most of the experimental bridge features analyzed in this study have performed well. Some, including latex and low-slump overlays, epoxy-coated reinforcing steel, and broomed deck finishes (on state-built bridges), are now in common use. Others, such as microsilica concrete and segmental bridges, await further applications and added service experience before widespread use. Features such as the experimental I-64 bridges and the zinc-coated steel on the I-24 bridge, while not widely recognized as successes, have performed satisfactorily.

The following conclusions and recommendations are offered.

OVERLAYS

Bridge-deck overlays are performing well and are extending the services of many bridge decks. The effect of deck cracking on overlay durability has yet to be determined. An effort should be made to revise overlay specifications and/or procedures to minimize cracking. A problem exists similar to that encountered with masonry coating in how to specify proprietary materials to insure successful applications. Tyning of overlays needs to be reviewed. Sawcut grooving should be considered as an alternate form of deck texturing. Further experimental overlays using microsilica and calcium-nitrite concrete need to be considered.

EPOXY-COATED REINFORCING STEEL

Epoxy-coated reinforcing steel has been an effective deterrent against chloride-induced deck deterioration. The beneficial effect of this feature has been masked by the use of thick concrete cover and the AA concrete. Corrosion of all types of deck reinforcing steel is inevitable. But, it has not been determined when significant corrosion will occur in decks using epoxy-coated reinforcing steel compared to those benefitting solely from thicken cover and AA concrete. It is recommended that the four bridges corrosion-tested in 1983 be considered for long-term monitoring.

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BRIDGE DECKS

Besides use of epoxy-coated reinforcement, other effective steps to improve bridge-deck durability involve making the deck concrete more impermeable to salt penetration and more resistant to freeze-thaw deterioration. Some improvements in the wear quality of deck concrete also may be possible. Further experimental bridge decks featuring combinations of concrete additives (possibly including microsilicas and super water-reducers) should be constructed. Other deck concrete improvements may be possible by other methods, including the use of higher cement contents, the addition of fly ash, and the use of high-purity aggregates (i.e., low expansive aggregate and shale contents). Bridge-deck design improvements that minimize bridge-deck cracking may be possible.

INTEGRAL-ABUTMENT BRIDGES

Field inspections have shown that bridge-deck cracking depends to a great extent on bridge type. Integral abutment bridges using prestressed concrete girders are generally free of deck cracks. More of those bridges should be considered. Some integral abutment bridges employing longer spans should be considered.

MASONRY COATINGS

Past recommendations for eliminating failures of masonry coatings were based on the assumption that most of the previous failures could have been prevented by proper application and inspection techniques. In part, those recommendations reflected the Transportation Cabinet's inability to evaluate proprietary products furnished by coating manufacturers. Similar failures have been observed in other states. Obviously, the problem is not limited to Kentucky bridges. It would be desirable to implement the formal masonry coating inspection program that has been recommended. If the program was not successful, that might indicate the need for acceptance-testing of coatings and the possible development of improved coatings.

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SEGMENTAL BRIDGES

Presently, there are two segmental bridges in Kentucky. Although the construction of the KY-676 twin bridges at Frankfort were monitored by research personnel, no follow-up performance inspections have been made. At this time, the Pikeville segmental bridge is not opened to traffic. Most contractors are not experienced in constructing segmental bridges. To show the greatest economic benefit, a number of those structures must be built over a reasonably short time period to allow contractors to gain experience. Otherwise, very few of them will be attempted in the future.

WEATHERING AND GALVANIZED STEEL BRIDGES

Since the recent weathering-steel corrosion problems nationwide and on the KY-1893 bridge, no bare weathering steel bridge has been constructed by the Transportation Cabinet. There are some applications where weathering steel would be advantageous. It should not be excluded from future consideration based on previous problems. Further construction of hot-dipped galvanized bridges may not be economically justifiable. However, flame-sprayed galvanized coatings are being used experimentally in several states. Those coatings have been applied on both newly constructed and existing bridges. Another coating method, thermally-diffused epoxy coating (shop applied), is presently being tested in some regions. That coating should eventually be considered for use by the Transportation Cabinet.

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13. Laboratory Concrete Air-Void Determination Report to Elkem Chemicals, Inc., from P. D. Cady, Pennsylvania State University, July 12, 1985.

14. Laboratory Concrete Air-Void Determination Report to Elkem Chemicals, Inc., from P. D. Cady, Pennsylvania State University, September 7, 1985.

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APPENDIX

SUMMARY OF MASONRY-COATING SURVEY, APRIL 1984

MASONRY COATING
SURVEY

1. How long has your district used masonry coatings on bridges?

Years	No. of Respondents
4	1
6	1
7	1
9	1
10	4
12	2
19	1

2. On how many bridges in your district are masonry coatings used?

No. of Bridges	No. of Respondents
22	1
30	1
36	1
52*	1
100	2
Unknown**	5

COMMENTS:
 * Last five years, District 4
** Typical answer - Many or all new construction

3. How many bridges in your district have significant problems associated with masonry coatings spalling from concrete?

No. of Bridges	No. of Respondents
0	1
3	2
5	1
6	1
12	1
20	1
Unknown*, **, ***, ****	4

    *   There is a problem with every bridge, some more severe
           than others.
    **   All problems that occurred before final inspection were
           corrected before final inspection.
    ***   Many bridges have some scaling.
    ****   Approximately 5 - 10 percent estimate.

4. List all bridge masonry coating failures by bridge, route, location and construction date. If possible, include the brand of masonry coating used, method of application, and the contractor who applied it. See individual questionnaires and attached letter from District 11.

5a. What type of forms (metal or wood) were used for concrete sidewalls?

Type	No. of Respondents
Metal	0
Wood	3
Both*	6
Unknown	2

COMMENTS:

* Typically most questionnaires say wood was used for a majority of the forms.

5b. What kind of form-release compound was used?

Type	No. of Respondents
Form Oil	5
Petroleum Oil	3
Unknown	3

6. Do you believe that one type of masonry coating is superior to others in your district? If so why?

Answer	No. of Respondents
Yes *, **, ****	3
No ***	8

COMMENTS:

* According to Construction (personnel), there is a good coating, but expensive. Contractors are using the less expensive.
** Thoroseal -- Good color and durability, also consistency can be varied as conditions require.
*** Some contractors prefer one type over another, but differences are not noted.
**** Bridge-Cote is the least desirable.

7. Do you believe that inspection of masonry-coating bridge applications could be improved? If so why?

Answer	No. of Respondents
Yes *, **, ***, ****, (*), (**), (***), (****)	8
No	3

COMMENTS:

* Insure that concrete is clean prior to application.
** Yes, be sure that concrete surface is thoroughly cleaned before application. Be sure that coating is properly mixed. Be sure that air temperature and concrete surface temperature are satisfactory for application.
*** Contractor's personnel need to do better job on ordinary surface finish prior to placing masonry

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coating.

**** Yes, by using instructions on containers at all times.
After structure has been prepared according to specs.

(*) More detail to surface preparation.

(**) Need more information from manufacturer concerning application procedures and restrictions.

(***) I talked with our former construction manager and he thinks that using spraying application rather than brushing or rolling would increase durability. Provided the surface is free of foreign material.

(****) Better cleaning procedure enforced.

8. Do you believe that more care and effort should be taken to insure that the concrete surfaces are suitably cleaned before application of masonry coatings?

Answer	No. of Respondents
Yes *, **, ***, ****,	9
No	0
Unknown	2

COMMENTS:

* In my opinion, most failures occurred because coating was applied over dusty or dirty surface. It might be wise to require hosing down with water before application.

** The surface should be rubbed to insure that form oil does not remain on surface of concrete.

*** The concrete should be rubbed. Masonry coatings do not replace the rubbing of concrete.

**** Specifications might be changed to require concrete be cleaned or washed to remove any residual oil.

9. If you have masonry-coating spalling failures, are they located on: _____ interior side walls, _____ exterior side walls, _____ pier caps, _____ other concrete members.

Location	No. of Respondents
Interior Side Walls	10
Exterior Side Walls	8
Pier Caps	3
Other Concrete Members	4
No Spalling	1

COMMENTS:

Some problems may be from winter salt applications.

10. Are spalls associated with _____ horizontal surfaces, _____ vertical surfaces, _____ corners or changes in concrete profiles, _____ cracks in the underlying concrete, _____ expansive aggregate failures (popouts or spalls).

Failure Location	No. of Respondents
Horizontal Surfaces	6
Vertical Surfaces	10

Corners	4
Cracks	5
Expansive Aggregate	4
No Spalling	1

11. Do you feel that there is an on-going problem involving masonry coating failures?

Answer	No. of Respondents
Yes *, **, ***	7
No	4

COMMENTS:

* It seems that use of metal forms causes problems with some coatings.
** Only with certain brands.
*** Yes, but not a major problem. Some work needs to be done to improve performance.

12. Have you encountered problems with contractors applying those coatings?

Answer	No. of Respondents
Yes *, **, ***, ****, (*), (**), (***)	8
No	2
No Response	1

COMMENTS:

* Unclean surfaces not prepared before coating applied, surface and temperature too cold and surface too wet.
** Some contractors do not want to prepare surface. When forms are removed, patch cavities, depressions, air voids etc. They try to fill cavities and depressions with masonry coating. Contractors do not want to give walls etc. an ordinary surface finish prior to applying masonry coating.
*** Improperly preparing surfaces and also with application procedure.
**** Contractor caught using gasoline as thinner.
(*) Premixed coatings cannot be thinned and are vary hard to apply, especially TCA roll-on.
(**) Contractors expect the coatings to "hide" imperfections in concrete surface -- it does not.
(***) Problems in obtaining uniform coating.

13. What are your opinions about masonry coatings?

COMMENTS:

- They offer sealant as well as cosmetics. Rubbed concrete is preferred.
- It is protection to the concrete. When properly applied to a prepared surface, it greatly improves the appearance of the

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structure. I believe that it is good construction to use the material.

- Masonry coating does excellent job if wall etc. have been given a good ordinary surface finish before applying masonry compound.

- Probably serves no more purpose than a rubbed finish, but should cost less than rubbed surface.

- It presents a pleasing appearance. I believe it has protective qualities when properly applied.

- When applied properly, you get a good-looking finished product. I feel manufacturers know more about their product than what is indicated in their brochures. If results from their testing were made available, we could get a better final product.

- I believe the coatings are being used in place of rubbing the concrete and that is one of the main reasons for failure.

- Thoroseal brush-on is much better than Tex Coat brush-on. I am not very familiar with the spray-on products.

- When the coating sticks, it appears to add protection and beauty to a bridge.

- If properly applied, I think that the minute problem of failures can be minimized and this product has replaced the rubbing of various portions of concrete structures, resulting in time and labor savings.

- Prefer rubbing and sacking.

14. Should provisions be made for spot repairs and eventual recoating?

Answer	No. of Respondents
Yes *, **, ***, ****, (*)	8
No	3

COMMENTS:

*   I doubt spot repairs would lend itself to an attractive appearance and it would be difficult to make spot repairs. Probably recoating would give the best results and appearance.

**   Yes, also more care should be taken during winter snow removal operations. Keeping the bridges clean would eliminate chipping from pieces of aggregate being thrown against the walls by traffic. Also, lowering the allowable percentages of chert in exposed concrete surface could cut down on the damage from expansive aggregates.

***   All bridges with failures should be recoated.

****   Spot repairs are probably not feasible, eventual recoating might be needed.

(*)   I think spot repairs would be sufficient for correction.