Design-Construction Feature

The Kentucky River Bridge

Variable Depth Precast Prestressed Segmental Concrete Structure

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The first precast segmental post-tensioned concrete box girder bridge in Kentucky was completed and the entire project was opened to traffic on December 6, 1979. The river crossing was the critical time factor on the construction schedule for the project.

This is a notable feat considering that design was started in April, 1976, and that bids for the structure over the Kentucky River were opened just 17 months later. A Design Report, an Environmental Assessment, and a U.S. Coast Guard Navigation Permit were required, in addition to the cost studies, preliminary plans, subsurface investigation, final plans, specifications, reviews, conferences and notice to contractors.

Bridge construction required just 27 months, despite a record flood at the site on December 10, 1978, a trucker's strike during the segmental erection stage, and several minor holdups and

Describes the preliminary studies, alternate structure schemes, superstructure design, post-tensioning layout, epoxy joint details, segment production and erection of the Kentucky River Bridge—a variable depth precast prestressed segmental structure, with a 323-ft (98.5 m) center span in which the segments were fabricated using the long-line casting method.

delays. Projects of this type commonly require from 5 to 10 years from beginning of design to completion of construction. Therefore, the 44-month schedule resulted in a considerable cost saving and made the highway available to the traveling public at a much earlier date.

Frankfort is the capital of the Commonwealth of Kentucky. The bridge over the Kentucky River is part of what is called the Frankfort East-West Connector (KY 676, a bypass road) which brings together several outlying areas and a number of state office complexes. The need for a new crossing was em-

phasized during the 1978 flood which closed the downtown area with its railroad and three highway bridges, leaving only the rural I-64 structures over the Kentucky River to connect the city's suburban areas.

Site Problems

Early studies showed that an interchange was not possible in the limited area available in the narrow valley approach to the west end of the bridge. Therefore, the intersection layout which was ultimately selected determined the alignment across the river.

The profile was set to provide 50 ft (15.25 m) of vertical clearance above the river pool stage, based on a steel two-girder system (with floor beams and stringers) with a center span to provide 300 ft (91.5 m) of horizontal clearance. These vertical and horizontal clearances were the minimum navigational requirements acceptable to the U.S. Coast Guard.

A structure over the railroad some 2000 ft (610 m) east of the river set the grade at + 1.56 percent.

The west end of the structure was determined by relocated Big Eddy Road, which passed under the bridge. The angle of this road crossing and the proximity of the intersection made it highly desirable to set the abutment normal to the centerline of structure.

The east end of the structure was established by the soil slope stability requirement. The abutment could be either normal or skewed to parallel the edge of water.

The hydrological study required the piers to be parallel to the stream flow. This requirement caused considerable problems in design and construction, but had the advantage of allowing a shorter center span while satisfying navigational requirements.

With the line and grade established, the roadway plans were completed and the west approach section was let to construction in September, 1976. Later the east approach contract was also awarded, so that roadway work (except final paving) was completed on both sides of the river before the bridge contract was awarded and construction started.

Alternate Structure Studies

Steel plate girder framing plans, including details for the connections at the skewed piers and abutments were prepared, analyzed, and a cost study was completed. (Composite load factor design was not allowed by the Department at that time.) Similar studies were made for the concrete box girder structure, with only the piers being skewed.

The first proposals for the concrete structure were based on a two-spline 12-ft (3.66 m) constant depth box section with sloping sidewalls. When these preliminary plans were presented to the Department of Transportation with cost studies which showed the concrete to be competitive with steel, the Department's interest and willingness to accept the segmental concrete box girder concept encouraged the design engineers to pursue this type of construction further.

The Consulting Engineering firm of BVN/STS had been established in Indianapolis, Indiana, with a nucleus of personnel and complete computer design programs brought in from the Dutch consulting firm of BVN. They were retained by the prime consultant and approved by the Kentucky Department of Transportation to prepare a preliminary report on the segmental superstructure.

It was at this point that the variable depth section was first proposed. Initial studies indicated the cost of constant depth and variable depth segments were approximately the same. Since appearance was considered important at this location, Charles G. Cook, Director, Division of Bridges for the

Kentucky Department of Transportation chose the variable depth (with its parabolic lines) primarily for aesthetic reasons.

With the line and spans set, it was possible for the Kentucky Department of Transportation, Division of Materials, Soils Section, to proceed with the subsurface investigation in October, 1976.

More complete and detailed cost studies were made at this time. They showed the initial cost of the steel to be considerably more than for the concrete structure. It was estimated that the segmental erection time and therefore the total construction time for the concrete bridge would be the shorter of the two schemes.

This, combined with the cost of painting the steel periodically and the undesirable framing connections of the steel plate girder caused the consultant to recommend the precast segmental concrete box girder structure. The Kentucky Department of Transportation concurred with this recommendation on December 16, 1976, and the bridge moved into the final design phase.

Construction Plans

This was prior to the time when it became common practice to prepare alternate plans for both steel and concrete structures. The contract plans were prepared only for the segmental concrete bridge.

It should be noted that the AASHTO specifications for segmental concrete box girders were not adopted until mid-1977. However, every effort was made to comply with the current tentative specifications.

In the course of preparing and reviewing the preliminary plans, the consultants and the Kentucky Department of Transportation had decided the way the bridge construction was to be handled. Since no Federal Highway Administration funds were involved in the project, it was possible for the Plans and Specifications to reflect these choices, without additional approvals.

Basic parameters were set for the fabrication. Segments were to be precast using the cross section specified, with the variable depth, as shown. A strand system was to be used, with a choice of post-tensioning anchorages allowed. The segment manufacturer was required to be experienced in this type of construction.

The architectural details had been worked out and agreed upon in the preliminary stage. BVN/STS prepared a complete superstructure design and detail, with American Engineering Company reviewing the load and geometric controls, coordinating the work and keeping the Kentucky Department of Transportation informed as decisions were required. American Engineering Company also prepared the substructure design, details, miscellaneous drawings and notes required for the Contract Plans.

Superstructure Design

There are many segmental bridges, either in service or being constructed. Much has been written about design concepts, production of segments, and erection. Rather than repeating generalities, this article intends to concentrate on some unique aspects of the structure, such as the variable depth, the treatment of the skew, the reduction of unbalanced moments during erection, and the reduction in number of segments. Future deck repairs and some structural details (such as post-tensioning layout, the shear keys, and the shape of the cross girders) will also be discussed.

Constant depth box girders are usually more economical than variable depth girders up to span lengths of 300 to 350 ft (91.5 to 106.7 m). The main reason for this is the cost of forming. In the case of a variable depth, both inner and outer forms are more expensive be-

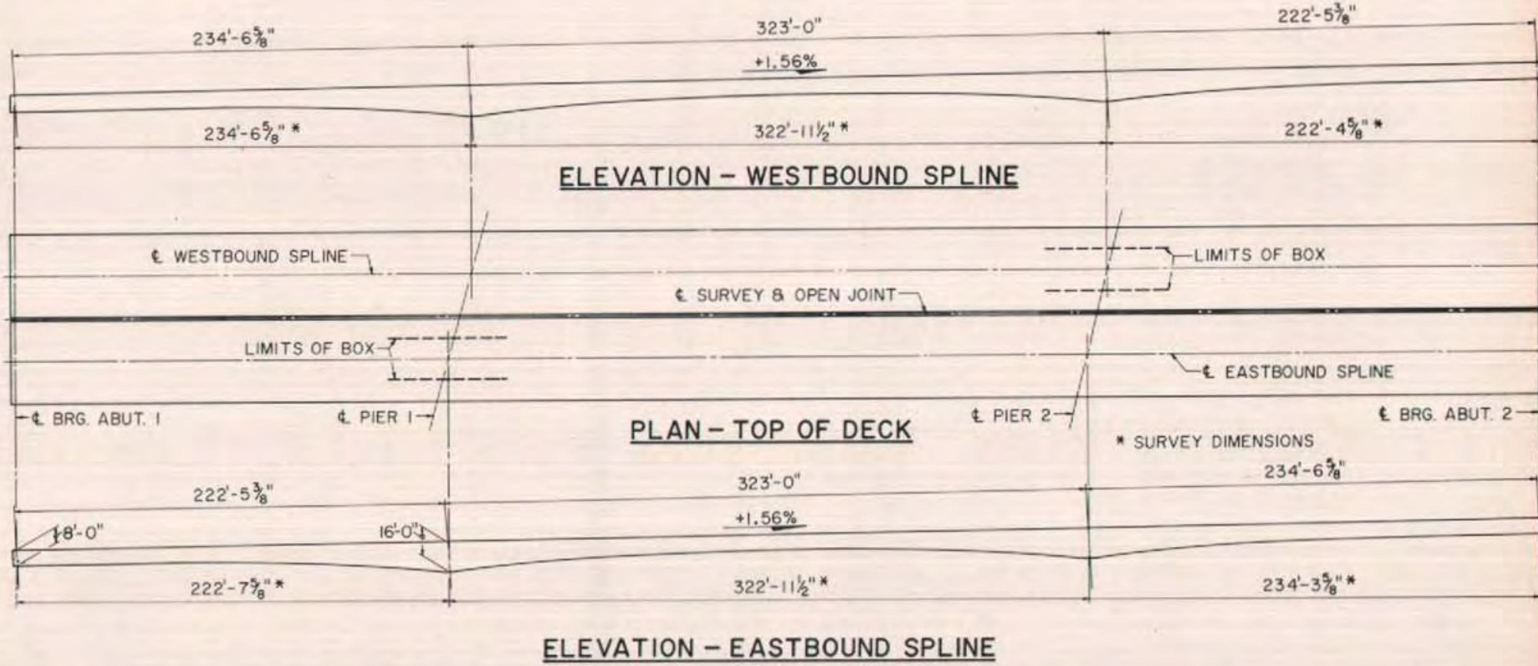

Fig. 1. Span arrangement to accommodate skewed piers. (Note: 1 ft = 0.305 m; 1 in. = 25.4 mm.)

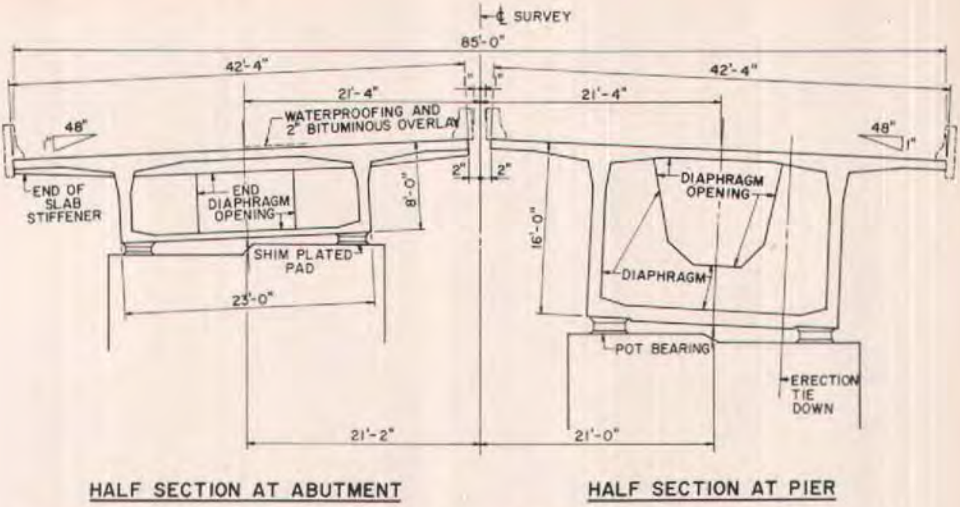

Fig. 2. Cross section of superstructure. (Note: 1 ft = 0.305 m; 1 in. = 25.4 mm.)

cause they must be capable of quickly adapting to different depths. Moreover, setup of the forms becomes more involved because of making the depth changes each casting day.

The cost of continuity post-tensioning may also be higher because the shallower depth does not allow placement of the tendons at as much eccentricity as would be possible with a constant depth. Consequently, the prestressing force will be higher, even after considering the sizable reduction in bending moments by superimposed loads (which concentrate at the high section over the piers) relieving the midspan sections.

Fabrication of the reinforcing bar cages is also more complicated because of the depth variation, although much can be done to simplify this. Reinforcing bars, for example, that must fit the depth of the section can be made in two parts with an oversize lap, thus providing ample tolerances.

Despite the skewed piers, this bridge is straight, that is, the abutments and all joints between precast segments are at right angles to the centerline of the structure. This aspect greatly simplifies design, detailing, manufacture of segments, erection of the superstructure and construction of the abutments.

In order to achieve this, the following measures were taken. First, the end spans of each of the adjacent structures are different (see Fig. 1). Secondly, the bearings are not placed on the centerline of the pier (see Fig. 3). Eccentric placement of the bearings on the piers as shown hardly changes the loading conditions, and only the effect of placing the bearings close to the edge of the pier requires further investigation. Additional skew could have been provided by also placing the bearings at a skew to the pier segment. This would cause additional bending in the superstructure which must be taken either in torsion of the cross girder or longitudinal bending of the box girder.

The segments were cast level and tilted to the cross slope of the roadway in the field (see Fig. 2). The superstructure length is measured along the roadway. This combination of length along the grade, variable depth on the grade, cross slope changing the

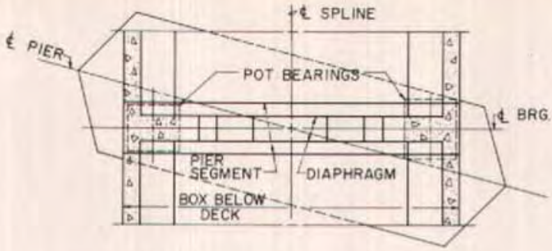

Fig. 3. Bearing arrangement on pier cap.

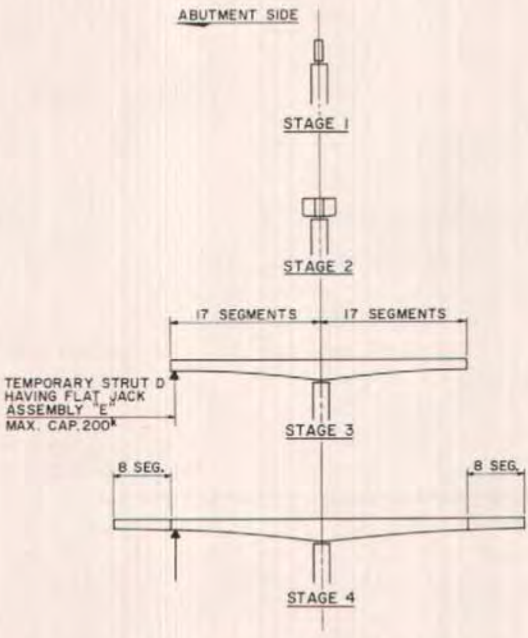

Fig. 4. Sequence of erection procedure and location of strut.

offset from survey line to center of box from top to bottom as the depth varied and the skewed pier line made the layout geometry quite complicated.

Due to the height of the piers and the length of the center span, expansion bearings were only needed at the abutments. The pier bearings were of the spherical pot type which accommodate rotations and small movements in the tranverse directions of the deck. Fig. 3 shows the typical bearing arrangement on the pier cap.

Transportation load limits made necessary the smaller lengths of the deeper segments. The shallower sections could be made longer therefore reducing the total number of segments to be manufactured and handled. Segments close to the piers, having a heavy

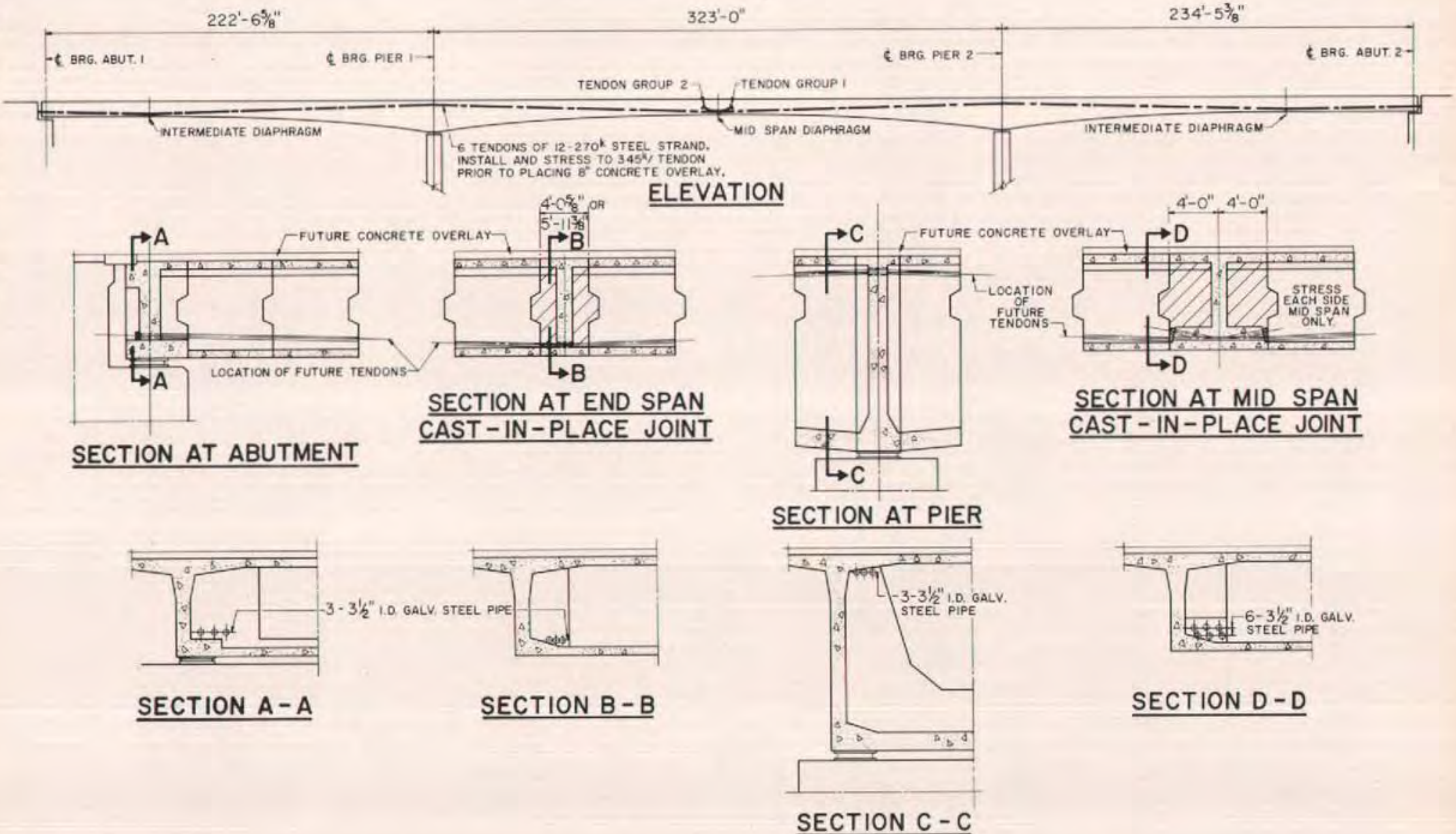

Fig. 5. Provisions for future tendon placement inside box at diaphragms. (Note: 1 ft = 0.305 m; 1 in. = 25.4 mm.)

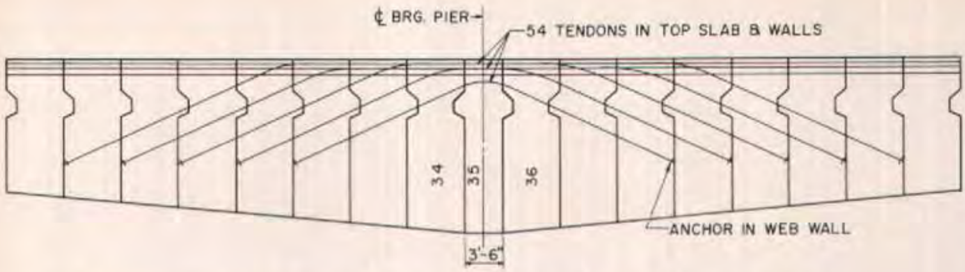

Fig. 6. Draped tendons in web walls. (Note: 1 ft = 0.305 m; 1 in. = 25.4 mm.)

bottom slab, and which are up to 16 ft (4.88 m) in height were made 5 ft 3 in. (1.6 m) long; whereas, segments closer to midspan are 7 ft (2.13m) long (see Figs. 6 and 7).

Although it was not done here, other variations such as web thickness could have been made. The number of segment types should be limited to perhaps two or three and the variations should not require major changes of the form. Making short segments in a form designed to make longer segments did not cause any problems on this project.

It is quite common to design piers in such a way that they are capable of sustaining erection loads. The critical condition is the erection of the last unbalanced segment, combined with some working loads. In the erection of this structure, the controlling loading case would have necessitated a larger pier footing size with a considerable increase in the cost of the cofferdam, excavation, tremie seal, concrete, and other items.

Rather than increase the footing it was decided to use an auxiliary strut in the tail spans at approximately 100 ft (30.5 m) from the pier. This strut consisted of a single H pile, with a jacking arrangement (see Figs. 4 and 25), and was a very economical solution to this problem. A counterweight was placed on the deck above the strut in order to provide positive bearing forces at all times.

Future Deck Repairs

During one of the many meetings between the Kentucky Department of Transportation engineers and the consultants that took place in the design phase, concern was expressed for the durability of the deck and the possibility of its future repair or replacement. Decks of segmental bridges have considerable resistance against deterioration, since the air-entrained concrete is factory produced and therefore of superior quality and of strengths often in excess of the 5500 psi (37.9 MPa) required for this project.

In addition, the Kentucky River Bridge deck is also post-tensioned in both directions. An epoxy coating on the top mat of the reinforcement and 2 in. (51 mm) of concrete cover aid in preventing damage by chloride ions which may eventually penetrate into the deck.

Nevertheless, the possibility of deck replacement was considered, and a simple solution was developed. Removal of large portions of the deck is not possible, but the addition of a new deck overlay is quite easy. If required, the plans provide for the removal of the bituminous wearing surface and the deteriorated concrete down to the top reinforcing bar mat. This removal of weight is equivalent to about 6 in. (152 mm) of concrete. An 8 in. (203 mm) structural overlay could be added without exceeding the substructure capacity.

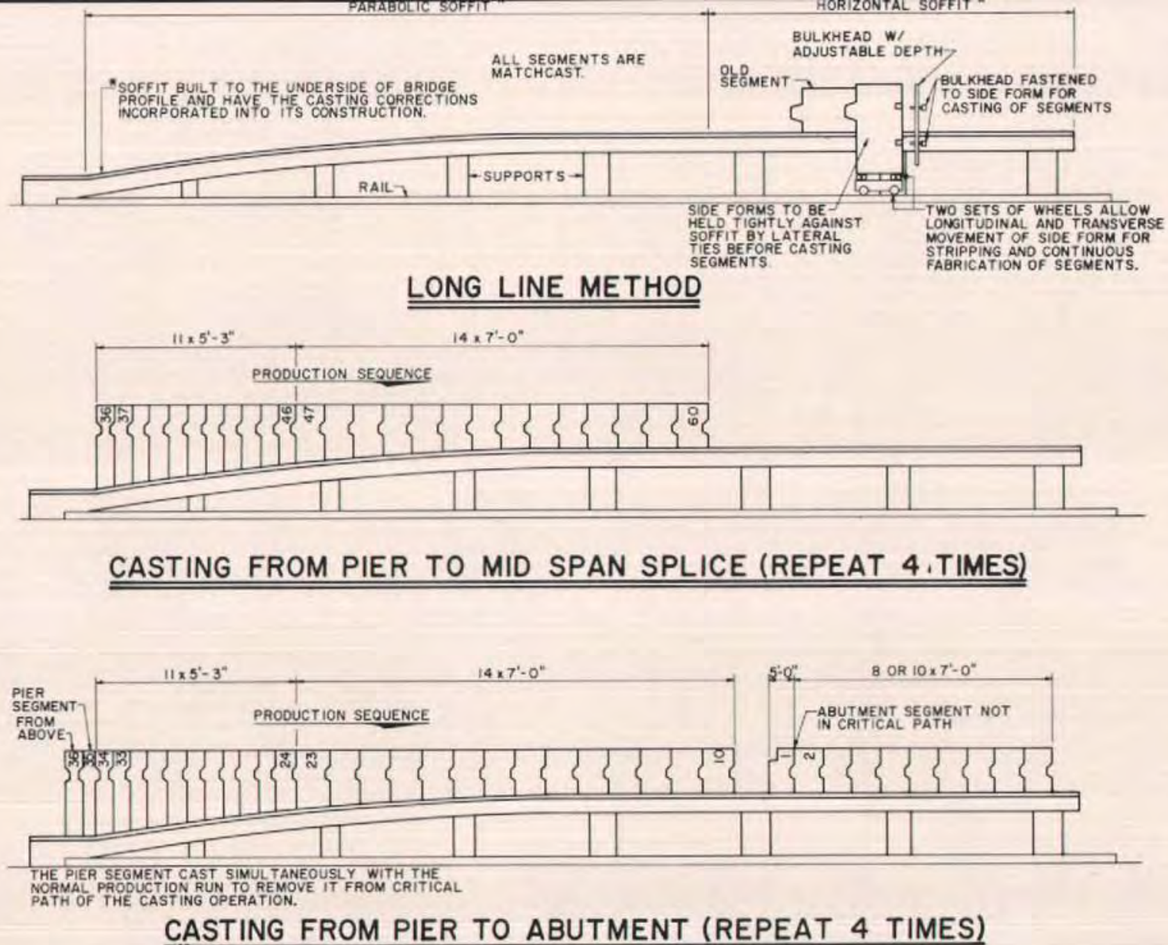

Fig. 7. Segment casting bed arrangement and production sequence. (Note: 1 ft = 0.305 m; 1 in. = 25.4 mm.)

Additional post-tensioning tendons to carry a full 8-in. (203 mm) overlay has been provided for in the design. This additional post-tensioning is to be placed in polyethelene pipes inside the box girders. The provisions for this have been included in this contract by designing and detailing for anchorage in the abutment, pier and midspan diaphragms. This post-tensioning could also be used to upgrade the load carrying capacity of the structure. Details of the additional post-tensioning are shown in Fig. 5.

Tendon Layout, Shear Keys and Epoxy Joints

The essentials of the post-tensioning layout are the use of draped tendons anchored in the webs as shown in Figs. 6 and 21. Although more complicated to install than straight tendons placed in the deck and the bottom slab, the structural advantages are sufficiently beneficial to warrant their use. The vertical component of inclined tendons (approximately 45 percent of the tendon force) reduces the external shear forces. A reduction of as much as 900 kips (4003 kN), or 50 percent of the total shear force, has been achieved in the high shear area of the structure by merely draping the tendons.

This reduction in shear force is beneficial, both for the vertical epoxied joint, which is relieved, as well as for high principle stress zones which will remain uncracked. Another advantage of the draped tendons is the reduction of shear forces acting on the keys during erection. Other methods to take shear, such as by reinforcing bars, are feasible but more costly, and less effective.

The Kentucky River Bridge segments are designed with large, single keys, the intent of which are to serve as erection aids only. These keys are simply designed as corbels, carrying a vertical force. This vertical force may be equal to the weight of up to four segments, or say, 200 tons (181 t) depending on the speed of erection and rate of hardening of the epoxy, which may be slow in low temperatures. Corbels for such high load carrying capacities must be carefully designed. By using draped tendons, however, the shear forces acting on the corbels during erection can be reduced to zero, or even become negative. This simplifies key design considerably.

The use of an epoxy resin in the joints is essential in the design of precast segmental bridges. Durability and structural adequacy are the most important reasons for this. The epoxy joint has been extensively tested and has been found to perform very well to the extent that no weakness was observed. Good quality control on product and application, however, is required for obtaining good joints in the field. Some of the measures that can be taken to insure quality are:

1. Testing of the epoxy products prior to selection, and also upon delivery of each shipment.

2. Color coding of resin and hardener to have a visual aide in mixing.

3. Training and supervision of the mixing and application of the product in accordance with manufacturers' instructions.

4. Daily verification of hardening.

5. Spot checks on cores drilled through joints of suspect quality.

The occurrence of inferior quality epoxied joints on this project was minor, and where present, could be easily and satisfactorily repaired. Quality control was improved after the publication of major problems on other projects. This improvement was not one of cost, but rather of efforts born out of greater awareness.

Those responsible for inspection of projects like this, should be thoroughly familiar with all aspects of the design and construction. This can be achieved best by participation of the designers of the structure in the inspection of the project on a regular schedule.

Construction Phase

The. bid opening, advertised for September 15, 1977, was preceeded by a Pre-Bid Conference on August 4, 1977. This allowed prospective bidders to ask questions and make suggestions which resulted in a number of minor changes to the Contract Plans.

There were four bids from general contractors on the project, with proposals furnished to the contractors from several segment manufacturers, post-tensioning suppliers, and segment erectors. All bids were over the engineer's estimate, with the major overage being on the erection. It appears that all contractors (several bought plans but did not bid) were concerned with the possibility of dropping a segment, its replacement cost and the time delays effect on the construction schedule. Other factors were the limited storage space at the site and the early completion date. The proposal was based on 300 working days, with erection to start early in the spring of 1979 (the minimum temperature on the epoxy controlling the schedule) and completion of the structure scheduled for December of 1979.

S. J. Groves and Sons Company of Minneapolis, Minnesota, submitted the low bid of \$6,596,000 (\$522,600 under the next lowest bid) from their Springfield, Illinois District Office. After award of the contract, a number of alternates were considered by the contractor, including placing their own casting bed near the site and subcontracting the erecting. They chose to buy the segments, post-tensioning materials and expertise from Construction Products Corporation of Henderson, Kentucky, and do the erecting with their own crews and equipment.

The contractor planned to erect both splines on the west side of the river first, with a barge-mounted crane in the

Fig. 8. Early construction of long-line casting bed in Henderson, Kentucky.

Fig. 9. First segments produced showing view of adjustable bulkhead.

center span and a crane on land for the end span. His first job was to site grade a work area between the relocated road and the edge of water and drive steel piles between the pier location and the top of the bank. Steel pontoons approximately 6 x 8 x 40 ft (1.83 x 2.44 x 12.20 m) were mounted on the piles, forming a work platform along the centerline of structure, from which the pier could be constructed and later the segments could be erected.

The pier footings are on extremely hard limestone approximately 36 ft (11 m) below the normal water surface. The contractor designed and fabricated a figure eight, two-level steel compression ring from welded plate girders salvaged from a highway bridge. The details were worked out so that the frame could be lifted up and floated from the west pier to the east pier. The frame was held in place with six round positioning piles.

Once it was in position and at the proper elevation, the sheet piles were driven, the cofferdam dewatered and the excavation completed. Before the pier footings were cast, and following the basic erection concepts shown on the Contract Plans, the contractor designed and detailed the temporary struts to rest on the pier footings and tie to the columns. These were placed before backfilling and flooding of the cofferdam. The struts would support the superstructure cantilever during the unbalanced phase of the segment erection.

Working with small crews, the contractor completed the west pier and most of the west abutment in the summer of 1978. The cofferdam was moved to the east pier where the layout was duplicated and construction continued on the substructure. Concurrently with this on-site work, the segment production was taking place. Figs. 9 and 10 show how the first segments were fabricated.

Long-Line Bed is Chosen

Prior to the bid, an analysis was made on the cost of a long-line forming system vs. the more conventional short-line system. On a construction cost only comparison, both systems seemed to be

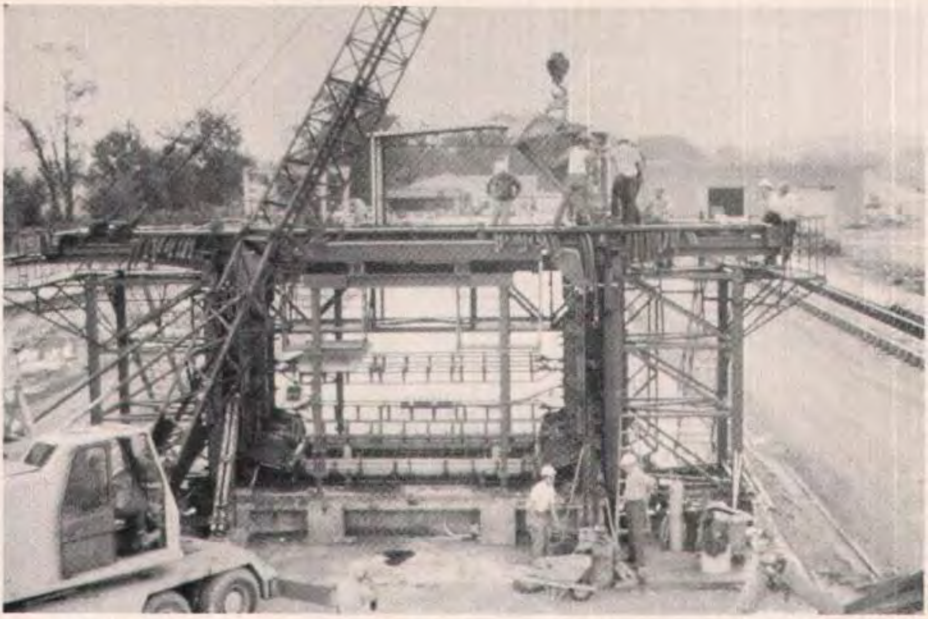

Fig. 10. First segments were produced by placing concrete with a bucket. Later the concrete was pumped into place.

competitive. Upon award of a contract to supply the segments, several other items entered into the decision to proceed with the long-line method.

1. The long-line bed in this case was able to handle the entire structure due to the fact that the "corrections" or casting curves were similar for each free cantilever. This may not always be the case, however. With the corrections being similar, the casting of one soffit eliminated the very costly bottom carriages which would have had to be capable of adjusting to an 8-ft (2.44 m) differential segment depth.

2. Incorporated in the long-line bed was a means of casting the pier segment in between two adjacent segments (see Figs. 7, 11, 12, and 13). Thus, casting of the pier segments (with the heavy diaphragm) was done simultaneously with the casting of the normal cantilever segments, and in effect removed the very difficult pier (and also the abutment) segments from the critical

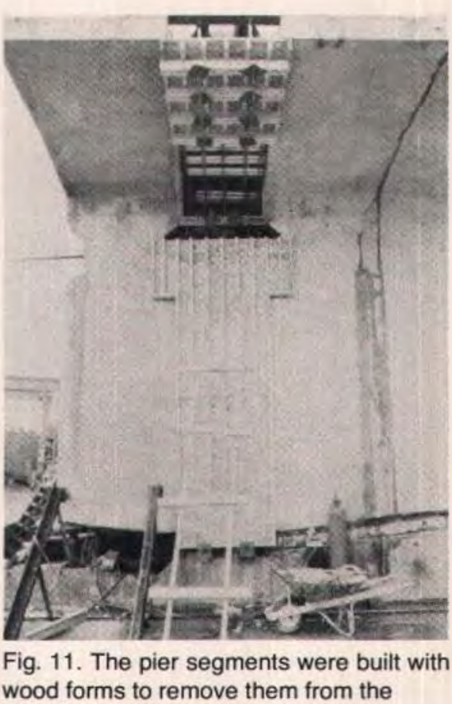

Fig. 11. The pier segments were built with wood forms to remove them from the critical production schedule.

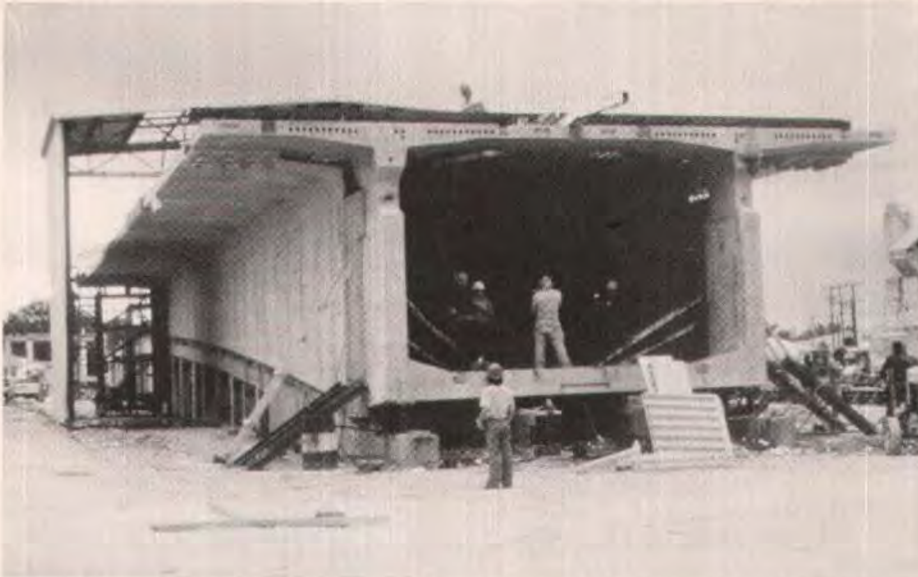

Fig. 12. A previously cast segment was brought back and positioned as in a short-line bed in order to cast the pier segment.

path of the production sequence. This was extremely important on this project due to the relatively tight construction schedule.

3. The manufacturer had already produced several bridges with the short-line setup and was well aware of the system. It was felt that the direct labor costs of segment production would be less using the long-line method. The long-line method required the positioning of the moveable steel bulkhead versus the exacting work required of positioning the "old" segment on a daily basis, which is required by the short-line system.

Fabrication of Segments

With the long-line method justified, a foundation was designed and installed in a short period of time (see Fig. 8), using elements regularly produced in the manufacturing plant. The basic elements included prestressed concrete piling driven approximately 60 ft (18.3 m) to support 40 tons (36 t) each. Prestressed panels 4 ft (1.2 m) wide and 21 ft (6.4 m) long spanned between each beam (column) line and provided positive reinforcement and a bottom form. These were topped with 2 in. (51 mm) of structural concrete that became the "soffit" form of the superstructure.

While the foundation was being built, the segment steel forms were ordered and the details incorporated into the foundation construction. Also during this period, the shop drawings were completed and reviewed by the consultant.

Fig. 14 shows a section of the casting bed where segments could be produced during inclement weather. Fig. 15 shows a segment being placed in the storage yard.

Production began (in June, 1978) with casting of the segment adjacent to the pier and proceeding towards the river splice. When this run was completed a second run was begun with the segment adjacent to the pier and pro-

Fig. 13. The heavy pier diaphragm was also formed with wood.

ceeding towards the tailspan splice. While the second run was being cast, the first segment cast was brought back to the bed and positioned so that the pier segment could be cast between its two adjacent segments. This procedure eliminated the pier segments from the critical path of the casting operations.

The segment casting operation consisted of four basic operations:

1. Place reinforcing cage and related work
2. Place steel bulkhead
3. Place inner mandrel and close side forms
4. Cast concrete

The reinforcing bar cage was prefabricated in a near-by building. It was then carted to the casting bed and placed on the soffit with a crane. The steel bulkhead was then placed at a predetermined position on the soffit. The bulkhead had interchangeable panels to account for varying segment depth. The inner mandrel was mounted on a frame that straddled the soffit and utilized the same rail as the side form for longitudinal movement.

The sides of the inner mandrel had a "slip" adjustment to vary with both a depth change from joint to joint and a varying bottom slab slope. The side forms required only occasional adjustment and had such a capability. The side forms traveled on a set of rails on each side of the soffit.

A concrete pump was used to cast the segments and this worked very efficiently. Casting time of a typical segment was approximately one hour. As the segment casting preceeded along the soffit, the top slab was transversely post-tensioned in a separate operation. Grouting of the transverse tendon ducts was done in the storage yard.

Transportation of the very tall segments from the precast plant to the job site posed some special considerations. Due to restrictive overpass clearances by land transport, 36 segments had to be transported by barge on the Ohio and Kentucky Rivers to the job site (see

Fig. 14. A work shed was placed over the bed so that production continued in bad weather. Average production was over four segments per week until all 244 were made.

Fig. 15. Once the length of the bed was produced the segments were stored in the yard to await delivery.

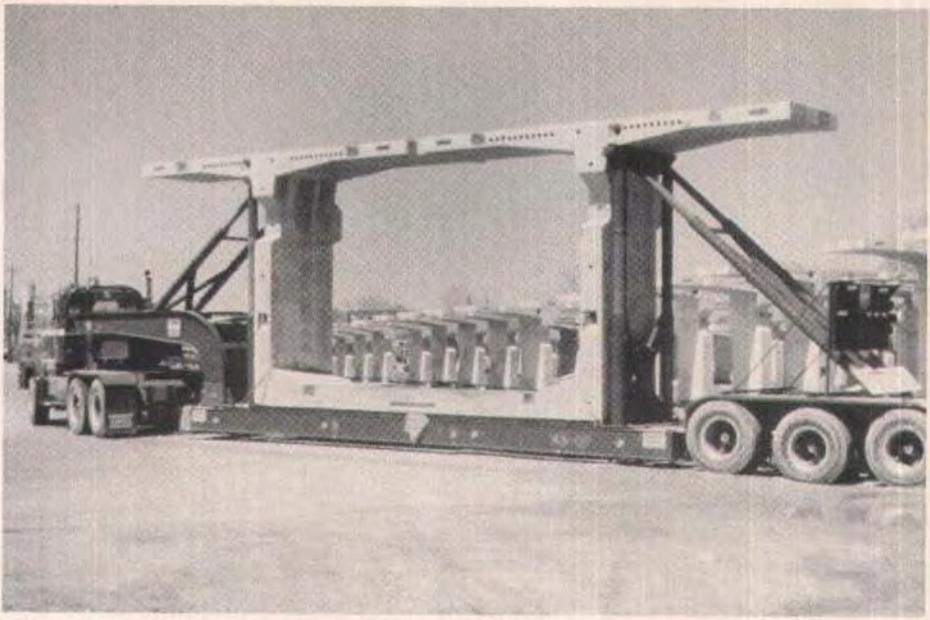

Fig. 16. A special trailer was built to transport the deeper segments.

Fig. 17. The deeper segments were loaded on barges.

Fig. 18. The pier segments arrived in Frankfort in April, 1979.

Figs. 17 and 18). Nine segments could be accommodated safely on the barge and the trip took about 3 days depending upon the river traffic.

Those segments that ranged from 12 to 14 ft (3.66 to 4.27 m) tall were transported on a special trailer designed for this project (see Fig. 16).

The special trailer supported the segments under the cantilever wing adjacent to the web, allowing a segment to be hauled with just 6 in. (152 mm) of ground clearance. Limited storage areas on the job site necessitated close coordination between the contractor and the precaster.

Fig. 19. The first segment was placed May 1, 1979. Note the vertical tie down rod on the side away from the second segment. The concrete risers (with the flat jacks) support the structure until the load is transferred to the pot bearings which are already in place.

Fig. 20. Erection continued from the pier in each direction always keeping within one segment of balanced cantilevers.

Segment Erection

Erection of the superstructure segments was begun May 1, 1979, with the first of a pair of three-segment pier units (see Figs. 19 and 20). Each segment was joined to the previous segment by means of an epoxy resin on the match cast joint face and six temporary post-tensioning bars (see Fig. 22). The temporary bars provided a uniform compression on the epoxied joint face.

Due to the variable depth, and thus variable section properties at each joint, the forces on the temporary post-tensioning bars were varied to keep the uniform compression of 50 psi (0.034 MPa) minimum on the joint face. Permanent post-tensioning consisted of twelve ½-in. (13 mm) diameter 270K strand and Freyssinet anchors. Most of the tendons were placed as individual strands by a machine that pushed the strand through the conduit.

Flat jacks were used to set the structure to elevation and to grade (see Fig. 23). Pot bearings were grouted in place after the balanced cantilevers were lined up (see Fig. 24).

The unbalanced moments encountered in erecting the free cantilever over 100 ft (30.5 m) from the piers were controlled by a strut assembly which was placed on the abutment side (see Fig. 25). Segments were then hung on the abutment side of the cantilever first, alternating sides until the closure splices were reached. Several segments in the tailspans were erected on falsework, from the abutment to the closure splice to the abutment.

Both splines were erected concurrently (for minimum movement of materials and equipment) with the west side (Pier 1) being placed first. While the land crane placed the segments on falsework (these went very quickly), the river crane erected the first segments at Pier 2.

Erection on the east side proceeded much faster, with the crews trained and

Fig. 21. The hydraulic jack, shown here pulling draped tendons in the web wall, tensioned twelve ½-in. (13 mm) diameter 270K strands and had a piston to seat the anchorage cone.

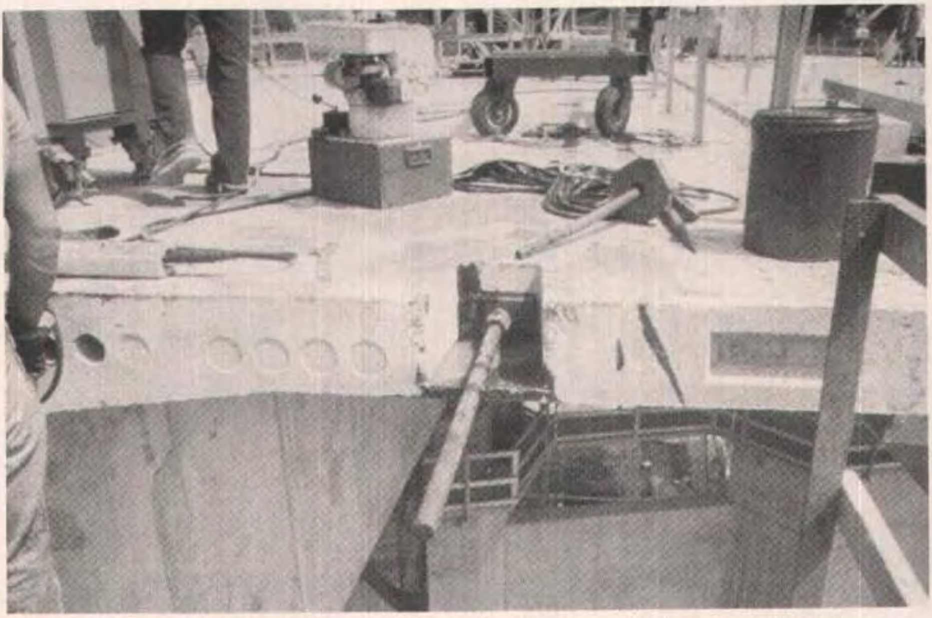

Fig. 22. High strength steel rods were used to hold segments in place until tendons were placed and stressed.

Fig. 23. Flat jacks were used to set the structure to elevation and to grade.

Fig. 24. The pot bearings were grouted in place after the balanced cantilevers were lined up.

Fig. 25. An erection strut was placed in each end span to stabilize the cantilevers.

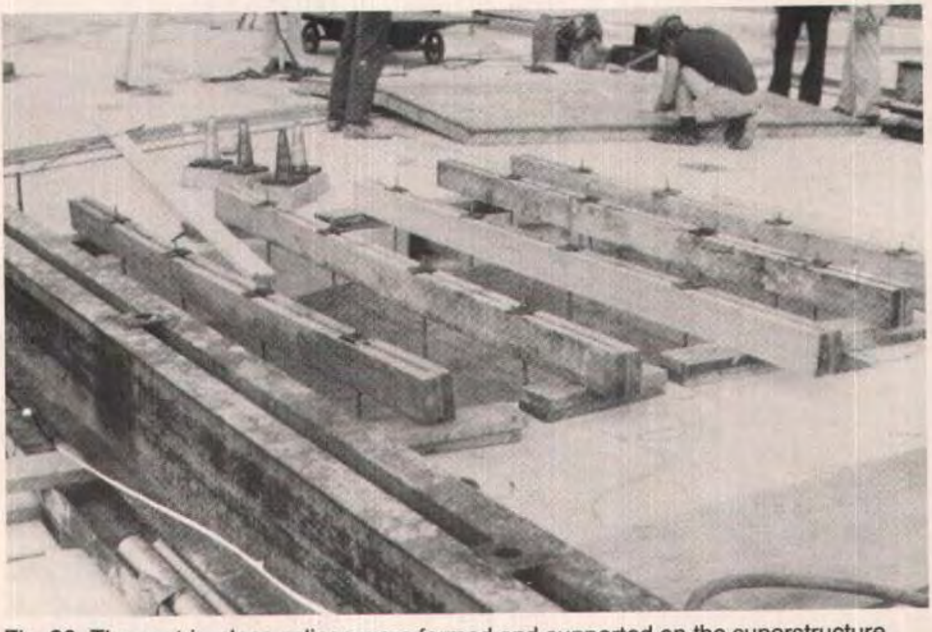

Fig. 26. The cast-in-place splices were formed and supported on the superstructure.

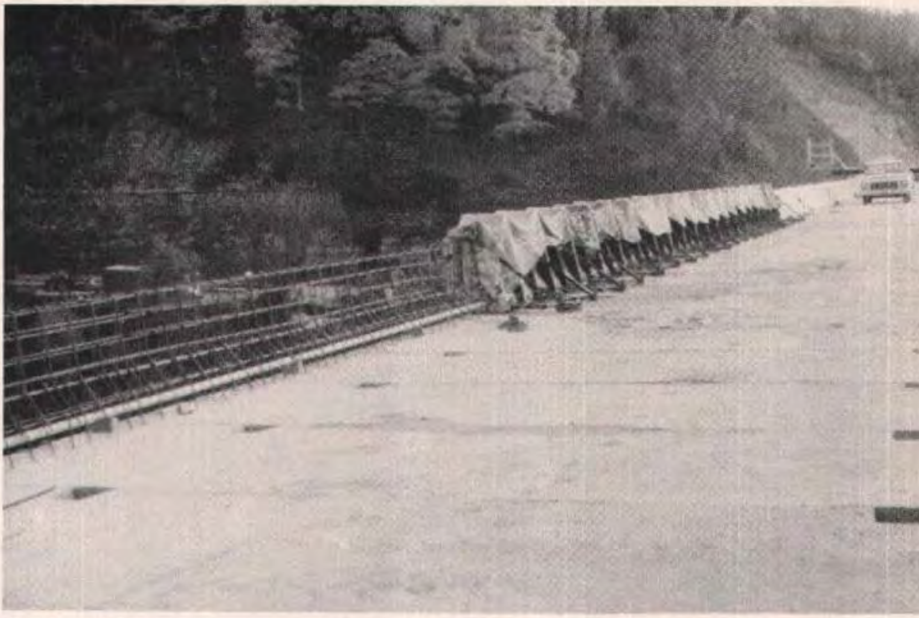

Fig. 27. The barrier walls were formed and cast in place in the late fall of 1979.

Fig. 28. The final cleanup just prior to final completion of the structure.

working a regular routine. Four segments (two each spline) were the normal schedule. On several days six segments were erected and eight would have been possible except for the uncertain delivery schedule caused by the truckers' strike.

With erection crews on the east side, carpenter crews formed the closure splice in the west end span (see Fig. 26). These were cast in place and the positive moment tendons were placed and tensioned. Then the east end span splices and last the midspan splices were cast in place.

Barrier walls were formed and cast in place in the late fall of 1979 (see Figs. 27 and 28).

CLOSING REMARKS

All those who had a part in the design and construction of the Kentucky River Bridge are very pleased with the final results. Fig. 29 shows an overall shot of the finished structure.

Through the entire project, there was a spirt of cooperation as if everyone realized that it was only by working together that the structure could be completed on schedule.

The Kentucky Department of Transportation, Division of Construction, was responsible for all plant and on-site inspection, with all decisions made by them. However, the Consultant Agreement included design, checking shop and erection drawings and a per diem arrangement for superstructure inspection.

Thus for a small cost, they could have the advice of experts who were familiar with the design and construction procedures. As erection got into full swing these inspections were scheduled on a regular basis.

The consultants were on top of each crisis as it developed and were able to work with the Kentucky Department of Transportation, the segment supplier, and the contractor in finding a satisfactory solution to the problem.

It is believed that four "first time in the United States" features are included in this project. The 323-ft (98.5 m) center span is the longest span to date to be built of precast segments erected in free cantilever, although several longer spans are under construction or have been designed. The skewed pier arrangement is unique to this structure. This is the first precast variable depth structure, although numerous cast-in-place structures have this feature. The long-line bed has not been used before.

Certainly, all of these features were proved to be feasible on this project. The longer spans and variable depth have already been included in other projects under construction. The long-line bed proved to be a fast, efficient and economical method of production which will become common practice in the future. The skewed piers and cast-in-place splices are expensive, and we will eliminate them wherever possible on future jobs.

Access to the inside of the abutments and box girders was provided for inspection. In fact, a lighting system was installed for this purpose. The Kentucky Department of Transportation plans to watch the structure closely but no provisions were made for a check of the stresses on the bridge.

The structure appears to be functioning properly. In the last 20 months there has been no maintenance required on the project. The increasing use of this highway indicates that the project was much needed and the timely completion was justified.

Fig. 29. The finished structure with the State Capitol buildings in the background.

CREDITS

Consulting Engineer: American Engineering Company, Consulting Engineers, Lexington, Kentucky.

Segmental Consultant: BVN-STS Consulting Engineers, Indianapolis, Indiana.

General Contractor: S. J. Groves and Sons Company, Springfield, Illinois.

Precast Concrete Manufacturer: Construction Products Corporation, Prestressed Concrete Division, Henderson, Kentucky.

Owner: Bureau of Highways, Division of Bridges, Kentucky Department of Transportation, Frankfort, Kentucky.