

Spliced Segmental Prestressed Concrete I-Beams for Shelby Creek Bridge



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The Shelby Creek Bridge is a five-span, high-level highway bridge nearly 1000 ft (305 m) long over a narrow valley in mountainous eastern Kentucky. Equal length prestressed concrete I-beams were spliced together in a unique design that eliminated costly falsework and provided full-length continuity over end spans of 162 ft 3 in. (49.5 m) and three equal interior spans of 218 ft 6 in. (66.6 m). Stay-in-place, precast, prestressed concrete deck panels eliminated most on-site forming and helped to speed up construction of the deck. The segmental prestressed concrete I-beam design was bid against two alternate steel designs and was \$417,000 less than the continuous composite welded plate girder alternate and \$2 million less than the originally designed steel delta frame girder bridge.

The relocated US Route 23-119 Bridge over Shelby Creek is located in the extreme eastern tip of Kentucky, about 5 miles (8 km) southeast of Pikeville in Pike County (see Fig. 1). The bridge is 985 ft 6 in. (300 m) long and 179 ft (55 m) above Shelby Creek.

The structure has five continuous spans, including three interior spans of 218 ft 6 in. (66.6 m) and two end spans of 162 ft 3 in. (49.5 m). Seven lines of post-tensioned concrete I-beams, spaced at 12 ft 6½ in. (3.8 m), support a deck 8½ in. (216 mm) thick and 85 ft 3½ in. (26.0 m) wide. Each line of beams is divided into nine equal length segments.



Fig. 1. Shelby Creek Bridge, Pike County, Kentucky.

ALTERNATE BIDS

A consultant, retained by the State of Kentucky Transportation Cabinet (KYTC), had initially designed a steel delta frame girder bridge for this site (see Fig. 2).

Several years passed before the project received funding for construction. About a year prior to advertising for bids, the KYTC's Director of Bridges asked American Engineering Company to study the Pike County site to determine if a segmental post-tensioned I-beam type of bridge was feasible and if it might yield a cost savings over the steel delta frame girder design.

Jointly, American Engineering Company and Janssen & Spaans Engineering, Inc., at no cost to the state, performed a preliminary design and cost estimate that indicated a pos-

sible \$2 million savings (see Fig. 3).

Rather than limit the bid to a specific type of structure, the KYTC decided to let the steel delta frame girder plans for bidding and allow contractors to lump sum bid alternate designs of other pre-approved types of bridges.

The Kentucky Division of Bridges approved five alternate conceptual design types of bridges proposed by four different consultants:

- A steel deck truss
- Two post-tensioned segmental concrete box girders
- A continuous composite welded steel plate girder using load factor design
- Segmental post-tensioned concrete I-beam

All of the alternate designs were required to be designed by the KYTC for AASHTO HS-25 live load. The

delta frame girder bridge had been designed for the lower AASHTO HS-20 live load.

Of the four bidders, two bid the plate girder design, one bid the steel delta girder design and one bid the segmental concrete I-beam design.

The bid on the segmental prestressed concrete I-beam bridge was \$7,373,475 (or \$87.72 per sq ft), which included the cost of the redesign, additional geotechnical work, checking shop drawings, limited construction inspection, personnel training, construction engineering and as-built plans. This bid was nearly \$2 million below the bid for the steel delta frame girder bridge, which did not include the cost of the consultant's design. It was also \$417,000 less than the second low bid for the continuous composite welded steel plate girder design furnished by a steel fabricator.

The joint venture of Bizzack Brothers

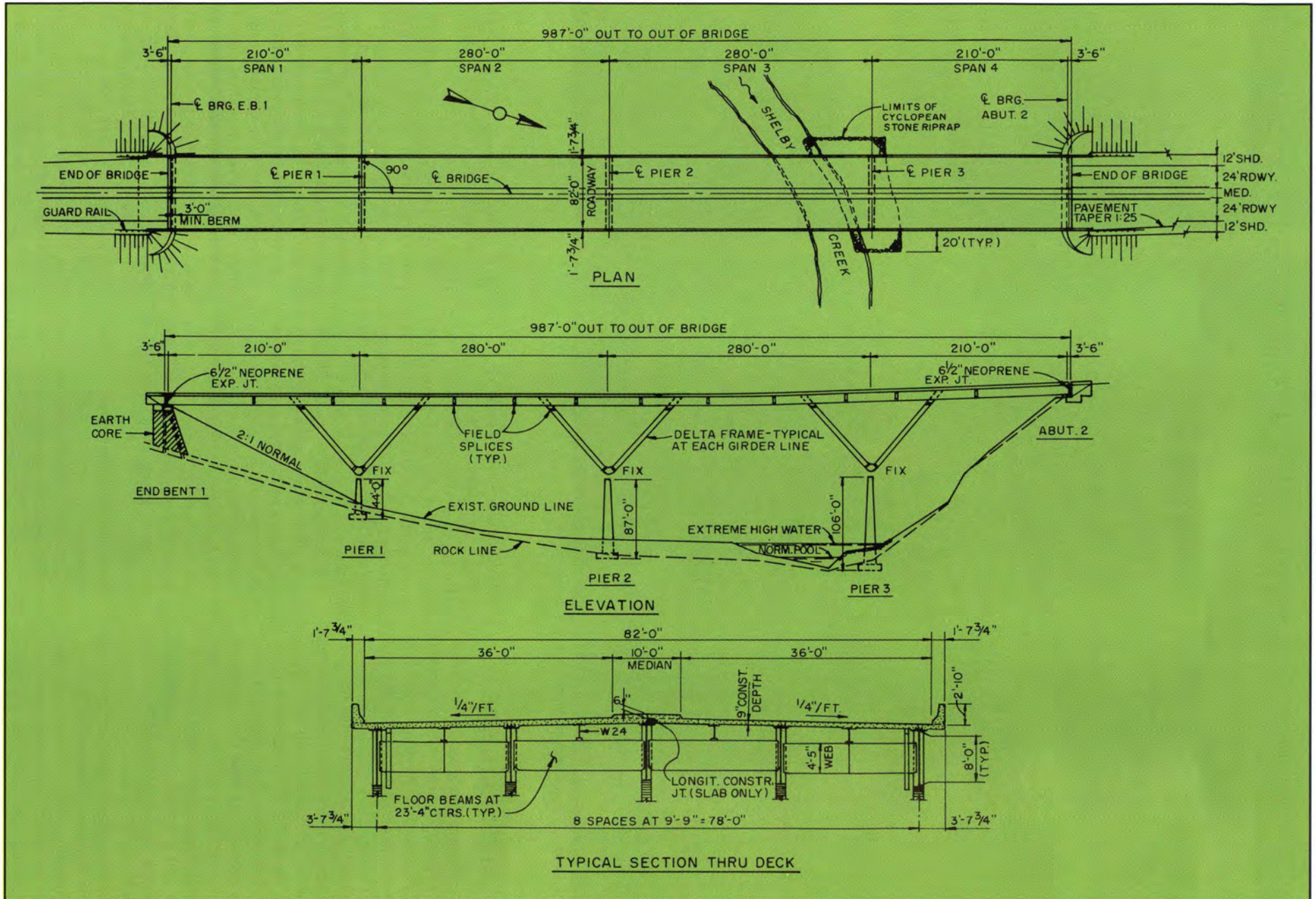


Fig. 2. Plan, elevation and section thru deck of steel delta frame girder bridge (original design).

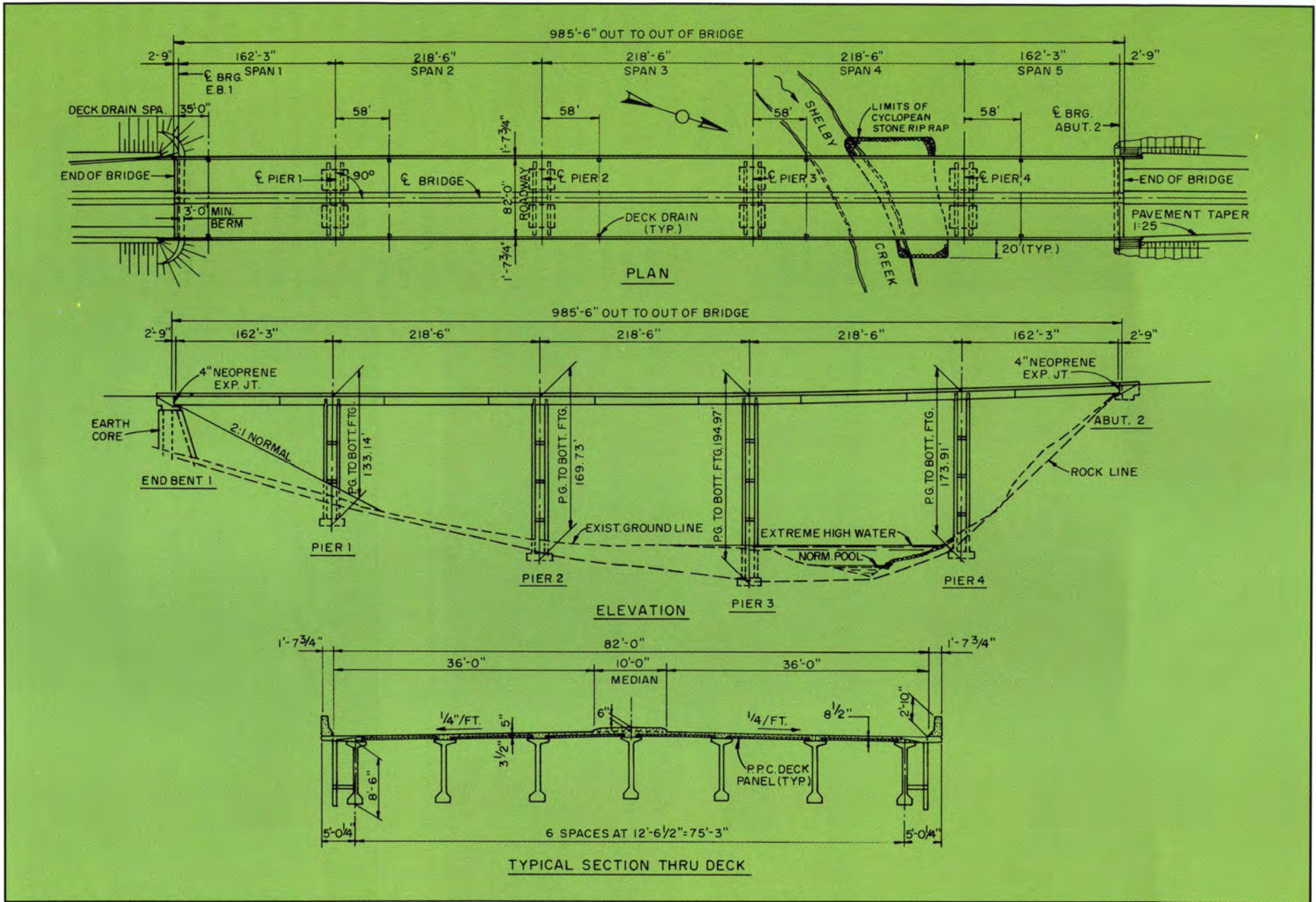


Fig. 3. Plan, elevation and section through deck of post-tensioned segmental concrete I-beam bridge (alternate design).



Fig. 4. Pier segments fit in pier cap slots.

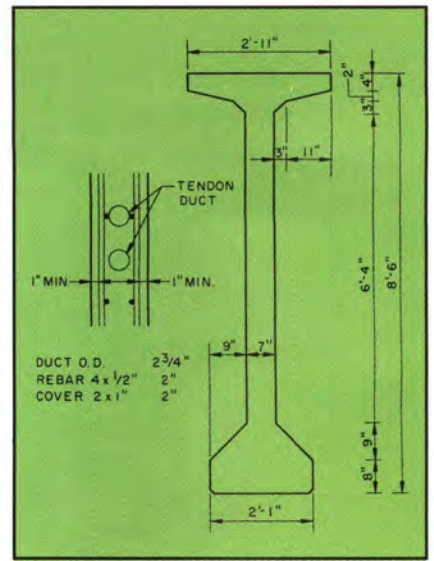


Fig. 5. I-beam cross section.



Fig. 6. Pier segments in place prior to receiving drop-in segments.

and Bush and Burchett, Inc., was low with its bid on the segmental prestressed concrete I-beam alternate and, thus, the contract was awarded in May 1989.

DESIGN CONCEPT

The bridge superstructure was designed to be erected without falsework. Piers consist of four slender columns with each side-by-side pair having a cap with deep slots to receive the 8 ft 6 in. (2.6 m) deep I-beams (see Fig. 4). The equal length, 108 ft 3 in. (33 m), I-beams are either pier segments or drop-in segments (see Fig. 5 for cross section dimensions).

With columns and caps spaced 15 ft (4.6 m) on centers longitudinally in each pier, and the pier segments grouted-in and post-tensioned transversely to the caps, a very stable set of cantilevers are ready to receive the drop-in segments (see Fig. 6). In addition, by placing the deck panels on the pier segments, a safe working surface was immediately available.

By using a modified Cazaly hanger, drop-in segments were quickly connected to and supported by the pier segments. The hangers maintained the drop-in beams position while precast concrete diaphragms and cast-in-place closures were placed and the post-tensioned tendons through girder end blocks and diaphragms were stressed. The remaining deck panels were set and concrete was pumped into place to

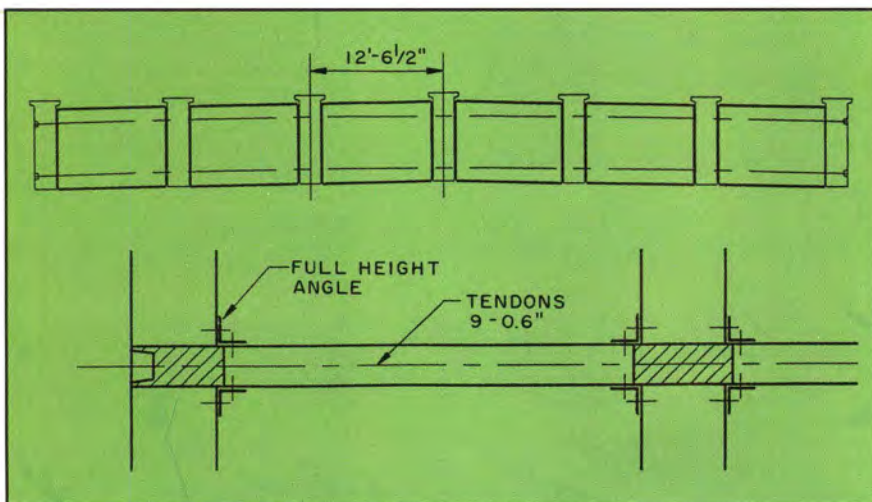


Fig. 7. Precast concrete diaphragms at beam splice points.

complete the deck.

Three types of precast concrete members were used in the bridge:

- 63 prestressed, post-tensioned I-beam segments, 108 ft (33 m) long, 8 ft 6 in. (2.6 m) deep.
- 60 precast, post-tensioned diaphragms, 12 in. (305 mm) thick, 8 ft (2.4 m) deep and 9 ft 5½ in. to 10 ft 1½ in. (2.9 to 3.1 m) long.
- 744 precast, prestressed deck panels, 10 ft 5½ in. (3.2 m) long, 3½ in. (89 mm) thick and 2 ft 9 in. to 8 ft (0.84 to 2.4 m) wide.

COMMUNICATION AND COOPERATION

From the start, the designers included the contractor and the precasting fabricator in planning the bridge details for the most cost-effective use of personnel, equipment and materials. An example of this was the consultants' initial suggestion of a variable depth beam section over the piers. The fabricator could anticipate problems in forming, lifting and hauling this variable depth section and the contractor could foresee erection difficulties. Therefore, a constant depth beam was designed.

The consultants had used precast, prestressed concrete deck panels as stay-in-place forms for the deck slab on many previous bridge projects. Since the fabricator had capabilities to produce the panels and the contractor had previous experience with deck panels and wanted to eliminate form placement and removal on this high bridge, it was a unanimous decision to use deck panels.

Actually, the designers were able to take advantage of the deck panel weight by placing the panels on the pier segments after the pier cap closures were poured and the caps were post-tensioned. This reduced compression stresses in the top flanges over the piers and allowed the last four tendons to be post-tensioned in each beam before placing the drop-in segments. The contractor also used the panels as a work platform to great advantage during the erection. Another mutual decision was to use precast concrete diaphragms at the splice points (see Fig. 7).

While the piers were being con-

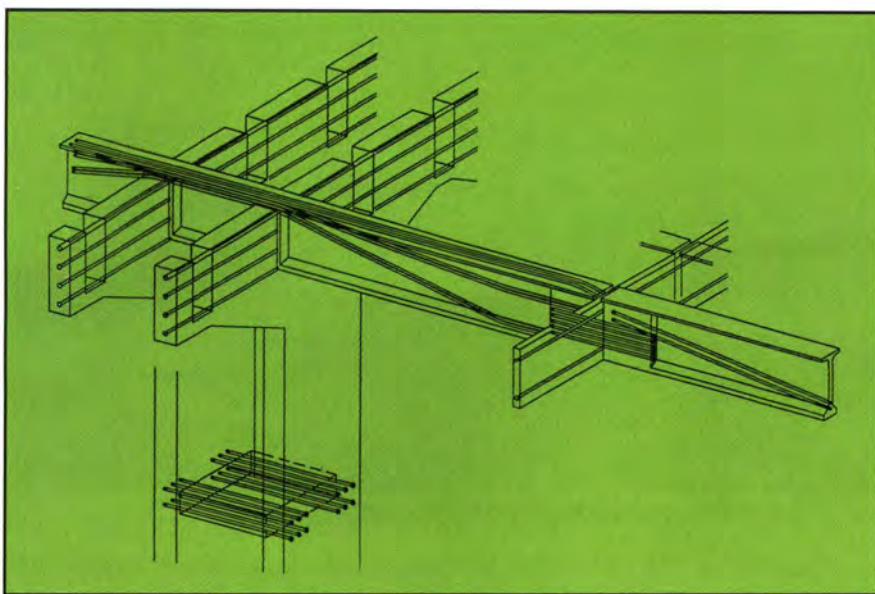


Fig. 8. Layout of post-tensioning tendons for pier struts, pier caps, beam segments and beam splices.



Fig. 9. Tall piers for high-level bridge.



Fig. 10. Pier struts are post-tensioned to columns.

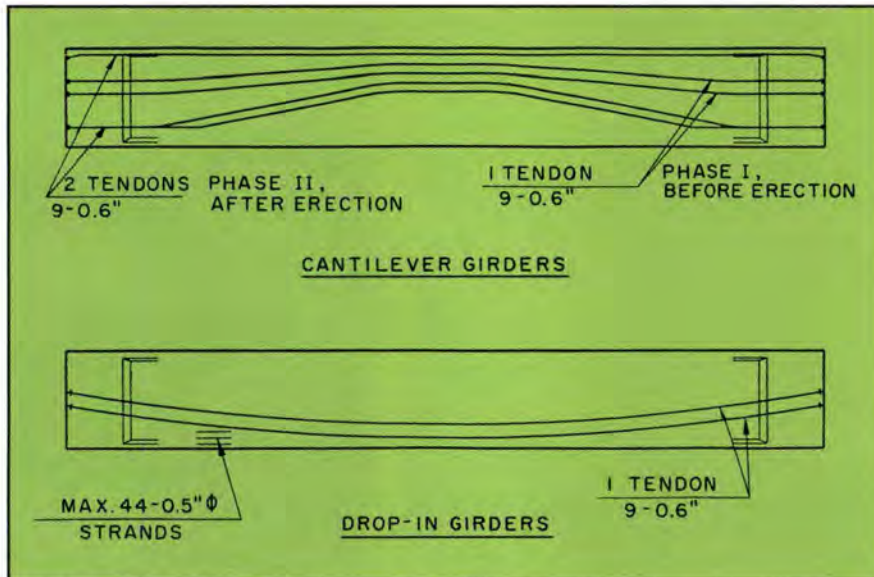


Fig. 11. Tendon layout for pier and drop-in beam segments.

structed, the fabricator and the post-tensioning material supplier developed their shop drawings. Prior to segment fabrication, the design consultants held special seminars at the prestressed concrete fabricating plant and the KYTC District Office to familiarize the KYTC inspectors and the fabricator's and contractor's employees with the beams and piers and how they would fit together. The seminars also provided time for questions concerning prefabrication and placement of reinforcing steel, tendon ducts, Cazaly hangers, anchorages, beam erection, tendon stressing and other operations.

The fabricator made two mockups of the beam end blocks to work out placement of anchorages, tendon

ducts, mild steel reinforcement, Dywidag bars and Cazaly hangers prior to the start of beam casting. By fabricating large sections of the beam reinforcing steel away from the casting bed, it was possible to set and adjust the reinforcing steel cages in the bed and reduce set-up time.

The consultants worked with the fabricator and contractor throughout the construction of the bridge. The post-tensioning was supervised directly by the consultants' personnel until the stressing crews and inspectors were thoroughly familiar with each specific stressing procedure, at which time they were certified for that particular operation.

There were four types of tendons for which crews were certified: pier struts,

pier caps, beam segments and beam splices (see Fig. 8). The contractor's registered engineer on the job was also certified and was required to be present during all post-tensioning operations. He made use of a laptop computer and a spreadsheet program to significantly shorten elongation calculations and improve accuracy.

PIERS

Each pier has four slender columns, each 3 ft x 12 ft (0.9 m x 3.7 m), connected with struts and with two columns supporting each of the two parallel caps (see Fig. 9). The pier caps, struts and columns were cast-in-place using normal weight concrete with a design strength of 5000 psi (35 MPa). This strength was usually attained in 3 days or less through the use of super-plastizers and low water-cement ratio, allowing faster form turnover.

Pier columns and struts have shear tie spacing to comply with earthquake requirements of the *AASHTO Guide Specifications for Seismic Performance — Category B Earthquake Design*. These ties were so closely spaced at the joints between columns and struts that it would have been extremely difficult to put the required mild steel connecting reinforcement in place. Instead, the struts were post-tensioned to the columns with either 10 or 12 — 5 x 0.6 in. (15 mm) strand tendons (see Fig. 10).

Post-tensioning these straight, short tendons afforded both the contractor and the KYTC inspection crews a good introduction to the stressing procedure, note-keeping and elongation calculations. This early introduction to post-tensioning on simpler tendons allowed the crews to work out procedures that aided them on the more complex tendon arrangements to come later in the erection process.

The alignment of these tall piers was held to close tolerance. The designers had provided a tolerance of ± 3 in. (± 76 mm) in the hanger system, pier segment setting slots, and the beam web blockouts where the top three pier cap tendons passed through; yet, it was still necessary to hold as closely to design dimensions as possible. After the piers had been constructed, mea-

measurements indicated that the alignment of the tops of the piers were within $\frac{3}{4}$ in. (19 mm) or less, even though these piers range in height from 133 ft to 195 ft (41 to 59 m).

BEAM FABRICATION AND DELIVERY

Delivery of these large beams to the job site, 140 miles (225 km) from the fabrication plant, presented major problems. To reduce weight and facilitate handling, semi-lightweight concrete weighing 125 to 130 lb per cu ft (2000 to 2080 kg/m³) was used. A design strength of 7000 psi (48 MPa) was specified; actual strength varied from 6000 to 7000 (41 to 48 MPa) in 14 hours, with a required strength at release of 4500 psi (31 MPa). The 28-day concrete strength of the beams approached 8000 psi (55 MPa). Beam weights varied between 135,000 and 145,000 lb (61.2 and 65.8 t).

Drop-in beam segments were designed with either 32 or 44 – 0.5 in. (13 mm) diameter pretensioning strands, augmented with two 9 x 0.6 in. (15 mm) strand tendons to be post-tensioned on site. The pier segments had 10 temporary pretensioning strands for transportation and handling only. Negative moment design requirements were achieved with six 9 x 0.6 in. (15 mm) strand tendons, also post-tensioned on site. Fig. 11 shows the tendon layout for both types of segments. Fig. 12 shows the reinforcing cage and tendon layout of a typical post-tensioned concrete member.

The deep I-beams have a high center of gravity above the roadway, making transportation difficult. Therefore, the fabricator had special trailers designed and built to haul these beams (see Figs. 13 and 14). They have steerable rear wheels and can be leveled pneumatically from the cab of the tractor while moving along the road. Because of tensile stresses in the top flanges, the segments must be supported near their ends for hauling. This makes lateral instability or buckling during transit a real possibility.

Early in the project, the fabricator drove the route to the job site and measured all the maximum highway cross slopes with an electronic inclinometer.



Fig. 12. Reinforcing cage and tendon layout of typical post-tensioned concrete member.

The greatest cross slope observed was 11 percent. The consultants took this information and designed additional pretensioning tendons for the top flange and a strong-back truss to stiffen the 35 in. (889 mm) wide top flange during transportation (see Fig. 15).

Once the trucks arrived at the north end of the bridge site, they had to descend a haul road having a 10 to 14 percent grade. As an extra precaution, a D-7 bulldozer was hooked to the back of the trailer to help control the descent.



Fig. 13. Transportation of 108 ft (33 m) long bridge beam.



Fig. 14. Special trailer and stabilizing frame used to haul 108 ft (33 m) bridge beams.

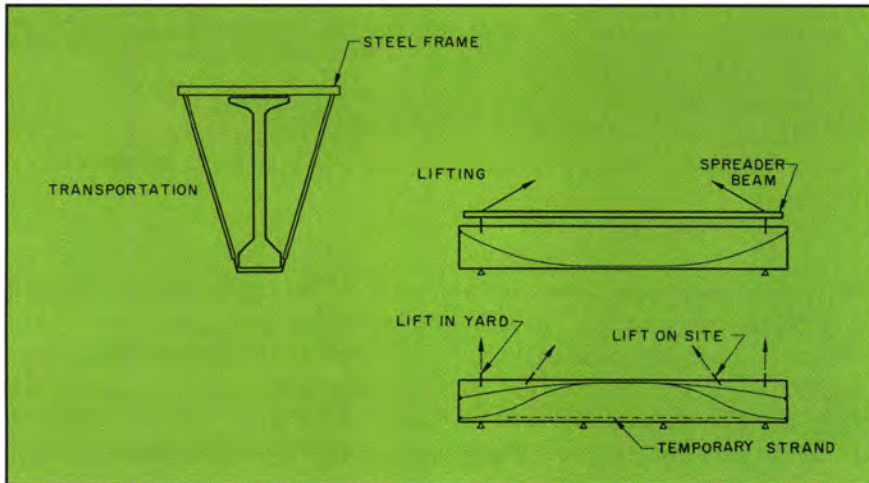


Fig. 15. Design considerations for handling beam segments.



Fig. 16. Hoisting of 108 ft (33 m) bridge beam.

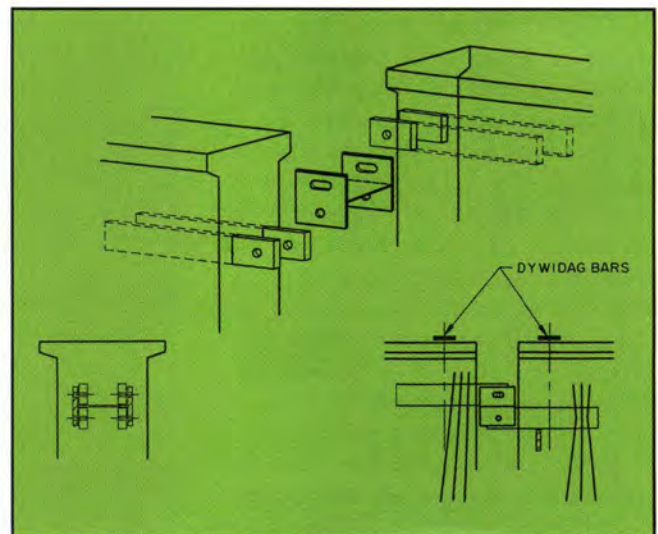


Fig. 17. Cazaly hanger system.

To obtain access to the south end of the bridge, it was necessary to drive down the haul road at the north end of the bridge, drive across Shelby Creek on a low water crossing, and then drive up a haul road with a similar grade. For the ascent of the south haul road, one bulldozer was hooked to the front for pulling and one to the rear for pushing.

ERECTION

Because of the height of the lift and the weight of the beams, the designers had studied the use of a single crane to erect the beams. After considering a number of different possible cranes, the contractor elected to use a Manitowoc 4100 Ringer with a capacity of 300 tons (270 t) (see Fig. 16).

Although the crane required many locations for the erection, it generally took little time to move and set up. Using a 280 ft (85 m) boom, the ringer had a horizontal reach of 125 ft (38 m) with a load of 72½ tons (66 t). With two cranes, as were used for the end span drop-in segments, the ringer required 320 ft (98 m) of boom and had a lifting radius of 205 ft (62 m).

Prior to erecting the pier segments, two of the six post-tensioning tendons were stressed. Next, the pier segments were placed in the pier cap slots on steel shims, and then aligned and blocked into the slots with timbers. Temporary steel cross frames were installed between the beams near the ends to stabilize the beams for the



Fig. 18. Cazaly hangers in pier segment end blocks are ready to support drop-in segments.

drop-in segments. Jacks were placed in pockets cast into the pier cap under the beams and used to position the segments to the correct vertical alignment.

High strength, non-shrink grout was flowed under the beams and allowed to reach a compressive strength of 4000 psi (28 MPa). The ducts through the cap and beams were spliced and the 6 in. (152 mm) spaces between the beams and the sides of the slots were formed and poured. Once the grout and side closures had reached the specified strength, three pier cap tendons were installed and stressed. Precast concrete deck panels were placed on the pier segments to reduce top flange compression stresses, thus allowing the four remaining tendons in the pier beam segments to be stressed and grouted.

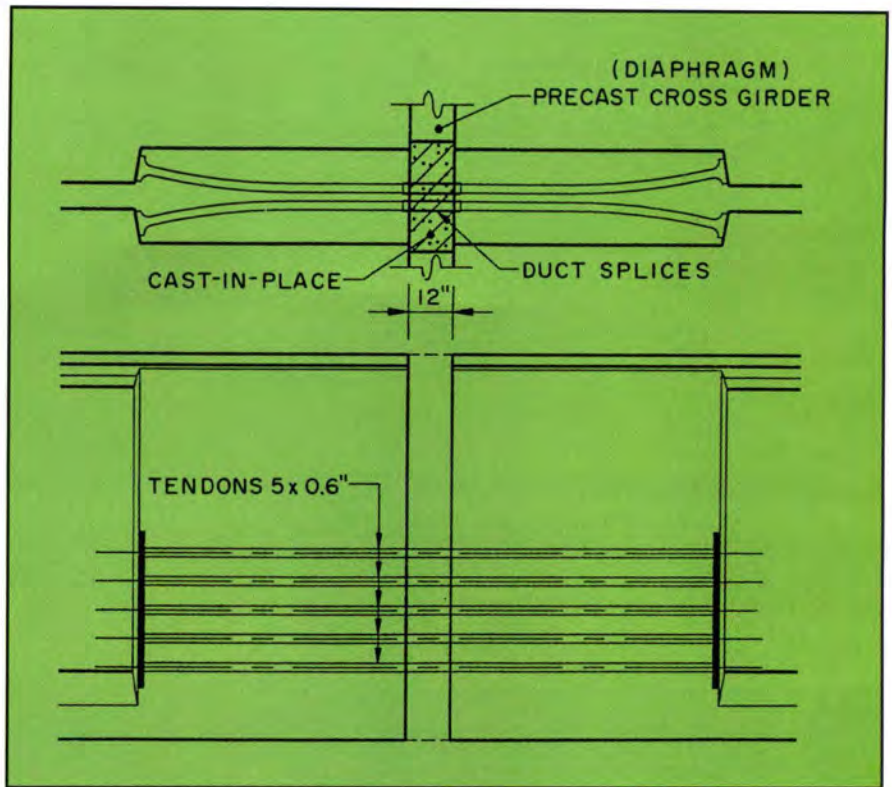


Fig. 19. Arrangement for longitudinal beam splices.

The drop-in segments were supported by modified Cazaly hangers, consisting of heavy steel bars cast into the ends of the beam end blocks (see Fig. 17). These hangers greatly simplified the erection of the drop-in segments; no temporary steel towers or other erection devices were needed. Dywidag bars were stressed vertically through the end blocks very close to the Cazaly hangers to prevent cracking. The pier segment hanger bars

temporarily supported the drop-in segments until the beam splices were completed.

Just prior to lifting the drop-in segments, the two 9 x 0.6 in. (15 mm) strand tendons were stressed and then the segments were lifted and set on the Cazaly hangers (see Fig. 18). When the segments were lifted with one crane, a spreader beam was required to prevent buckling the beams. After the beam segments were properly posi-

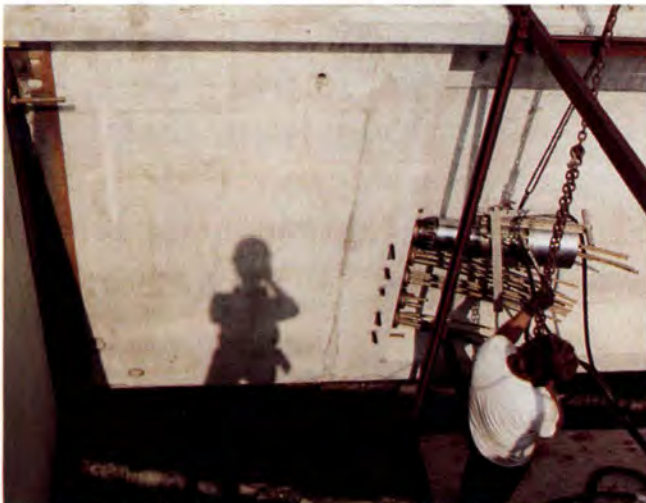


Fig. 20. Post-tensioning the splice tendons through beam end blocks.



Fig. 21. High density foam strips used to support stay-in-place deck panels.

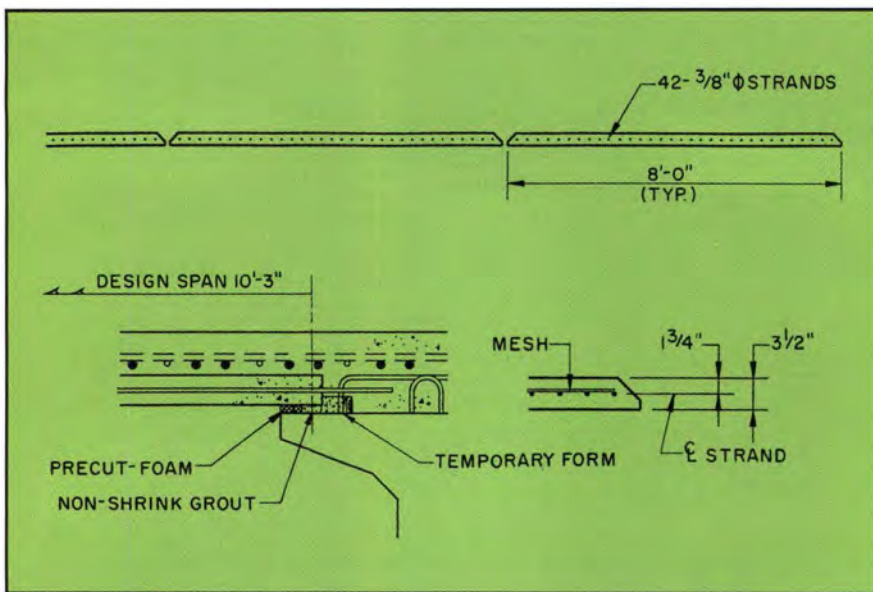


Fig. 22. Design details for precast, prestressed concrete deck panels.



Fig. 23. Deck panels provide working surface until composite deck concrete is placed.



Fig. 24. Completed five-span highway bridge over Shelby Creek, Pike County, Kentucky.

tioned, the Cazaly hangers were bolted and welded before the crane was released.

When the drop-in segments were in place on each side of a pier, the temporary strands in the pier segments were released, by slowly heating the strand through an access hole in the bottom beam flange, until they yielded and broke. The precast concrete diaphragms were placed and temporarily supported at the beam ends with steel angles in each corner. A form was placed under the beam ends and at the outside face of the fascia beam. The beam splice was now ready for casting. Once the splice closure concrete reached 4000 psi (28 MPa), the transverse diaphragm tendons were stressed.

There are no continuity tendons going the length of the bridge. Instead, beam segments were individually stressed and then spliced to one another with 10 – 5 x 0.6 in. (15 mm) strand tendons through the end blocks (see Figs. 19 and 20). There are 1098 post-tensioning tendons in the piers and beams.

DECK PANELS

After all the segments were in place and spliced, elevations were taken on the tops of the beams. From this information, the consultants calculated cutting dimensions for the high density foam strips that support the precast,



Fig. 25. Overall view of Shelby Creek Bridge, Pike County, Kentucky.

Credits

Owner: Commonwealth of Kentucky
Transportation Cabinet, Frankfort,
Kentucky

Design Consultants:

- American Engineering Company,
Lexington, Kentucky
- Janssen & Spaans Engineering, Inc.,
Indianapolis, Indiana

Prime Contractor: Bizzack Brothers
and Bush and Burchett, Inc., Allen,
Kentucky

Bridge Subcontractor: Bush and
Burchett, Inc., Allen, Kentucky

Precast Prestressed Concrete Manu-
facturer: Prestress Services of Ken-
tucky, Inc., Lexington, Kentucky

Post-Tensioning Hardware: Dywidag
Systems International, Chicago,
Illinois

prestressed stay-in-place deck panel forms to achieve a constant slab depth (see Fig. 21).

Because there is a sag vertical curve at the north end of the bridge, some of the high density foam panel supports exceeded 7 in. (178 mm) in height. After the deck panels were adjusted to the proper height by the foam supports, they were grouted with a non-shrink flowable grout, making sure grout flowed under the panel edge. Deck reinforcement was placed and the deck was poured in a conventional manner using a concrete pump.

The stay-in-place precast, prestressed deck panels are 3½ in. (89 mm) thick with a design span of 10 ft 3 in. (3.1 m) (see Figs. 22 and 23). Design concrete strength was 6000 psi (41 MPa). The 8 ft (2.4 m) wide panels were designed with 42 - 3/8 in. (10 mm) diameter strands. The cast-in-place deck slab was normal weight concrete with a design strength of 5000 psi (35 MPa).

The project proceeded very smoothly with few problems and produced a very economical, aesthetically pleasing

structure (see Figs. 24 and 25). The bridge was opened to traffic on December 20, 1991. During the past year, the bridge structure has been operating with total satisfaction.

CONCLUDING REMARKS

This type of continuous post-tensioned/pretensioned concrete segmental I-beam bridge should compete with steel bridges in the span range from 130 to 250 ft (40 to 76 m). This closes a gap in cost-effective concrete bridges between the 140 ft (43 m) pretensioned I-beam bridges, that are continuous for live load, and the over 300 ft (91 m) post-tensioned concrete segmental box beam bridges. The method allows precasters to produce bridge beams for spans they were not able to readily achieve previously, and opens up a whole new span range of bridge structures for design in concrete.

The Shelby Creek Bridge project won an award in the 1992 PCI Design Awards Program for bridges with spans greater than 135 ft (41 m).