Special Report

Construction of East Huntington Bridge

by

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The East Huntington Bridge is a cable stayed structure over the Ohio River connecting West Virginia and Ohio at East Huntington. The stayed girder has a main span of 900 ft (274 m) and a side span of 608 ft (186 m) with approach spans at both ends (Fig. 1). This is a two-lane bridge with a total deck width of 40 ft (12 m) at the cable stayed portion and 33 ft 6 in. (10.2 m) at the approaches. The actual curb to curb roadway is 30 ft (9.1 m) wide.

The cable stayed portion of the bridge girder has a concrete slab supported by steel floor beams spaced at 9 ft (2.7 m) on center. These floor beams are framed transversely into the two longitudinal concrete edge girders. The cables are anchored in the edge girders.

The 33 ft 6 in. (10.2 m) wide approach at the Ohio side is a single cell concrete box girder. A transition piece connects the box girder to the stayed girder.

The construction plans specified that the cable stayed portion of the girder shall be built in precast segments while the box sections are to be cast in place. The pylon is also cast in place.

Bidding the Project

Two alternative designs, one in concrete and one in steel, were developed for this project. The contractor had the option to bid either alternative.

The construction scheme suggested by the designer of the steel alternate features temporary supports in the river to support the steel girder. These temporary supports would be removed after cables have been installed and stressed.

The designer of the concrete alternative suggested building the cable stayed portion by balanced cantilever construction and the box girder approaches on falsework. After the box girder is cast and post-tensioned the falsework would be removed. An auxiliary pier would be required to be left in place to support the end of the cantilever so as to avoid excessive bending before the closure pour between the box girder and the cable stayed portion of the main bridge is cast.

Melbourne Brothers of Ohio was the successful low bidder at $23.5 million for the concrete alternate. They retained Contech Consultants, Inc. to provide the required construction engineering.

Synopsis

The author describes the various phases in the construction of the East Huntington Bridge — a 1966 ft (600 m) long prestressed concrete cable stayed structure. The cable stayed portion of the bridge girder was built using precast concrete segments. The 250 ton (227 t) segments were match cast off-site using the long line casting method. They were then barged to the site and lifted into position using a 600 ton (544 t) floating crane.

The South Approach was constructed by the cast-in-place cantilever method. Both the precast and cast-in-place portions of the structure utilized high strength concrete with a specified 28-day compressive strength of 8000 psi (55 MPa).

The bridge was completed successfully based on a well planned construction schedule.

Construction Engineering

As in most bridge projects the construction scheme described in the bidding documents is a suggested method to build the bridge. It lacks the required detailed information for actual construction. The contractor, through a consulting engineer, has to investigate all construction stages and come up with the missing information, such as cable jacking forces, camber curves, etc.

Generally speaking, the construction engineering consists of the following items:

1. Establish the construction scheme with the contractor.

2. Perform a stage by stage construction analysis to determine the required jacking forces of the cables and to ensure all stresses are within the allowable limitations during each construction operation.

3. Determine the casting curve for the precast segments if applicable.

4. Develop camber curve for each construction stage.

5. Determine any additional require-

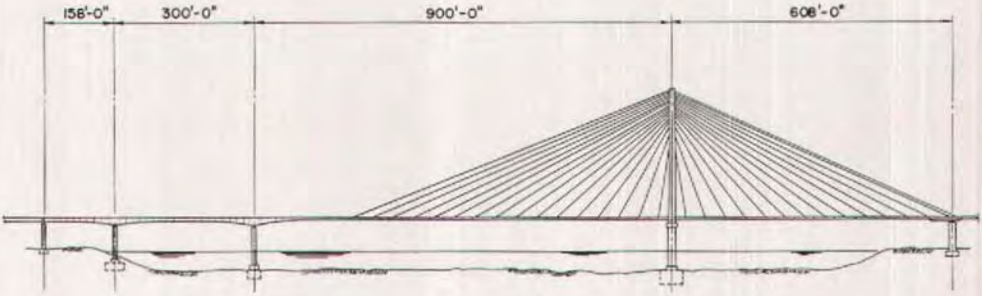

Fig. 1. Elevation of East Huntington Bridge.

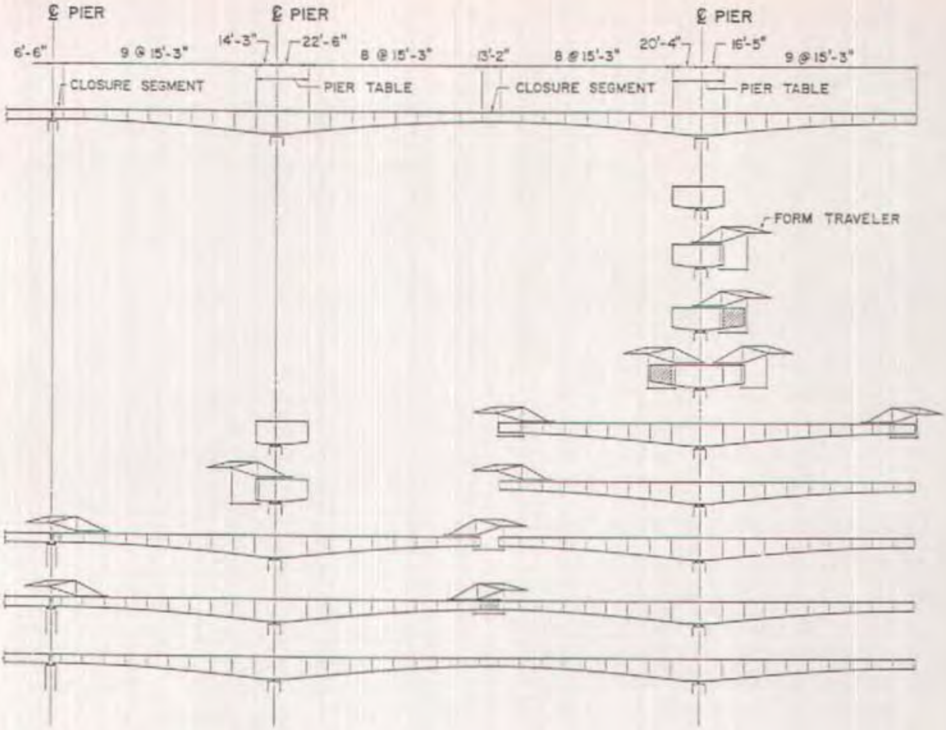

Fig. 2. Construction sequence (South Approach).

ment of cable lengths and shim packs for the cables.

In the case of this bridge, design of erection travelers to erect the precast segments was also included in the construction engineering.

Basically, the construction design produces a construction scheme which will achieve a stress condition and camber which are prescribed by the designer of the bridge for a specific time after completion of construction. However, cable stayed bridges are highly indeterminate structures. Depending on the method of construction, the bridge may have different stress conditions under permanent loads after it is completed.

To avoid overstresses under service load condition, the stresses in the structure have to stay within a certain range at the time the structure is completed. Creep, shrinkage and relaxation effects must also be considered during and after the construction period.

The analysis was based on criteria specified by the engineer. During construction, the allowable stress in the cables is 57 percent of its ultimate tensile strength and the allowable tensile stress in the concrete is $6\sqrt{f'_c}$ except at the precast segment joints where no tensile stress is allowed. The engineer also provided the upper and lower bounds for the permanent stresses in the girder which would be acceptable for the final service condition of the structure.

Box Girder Approach Spans

For the actual construction of the box girder approach spans a scheme different than that suggested by the engineer was selected.

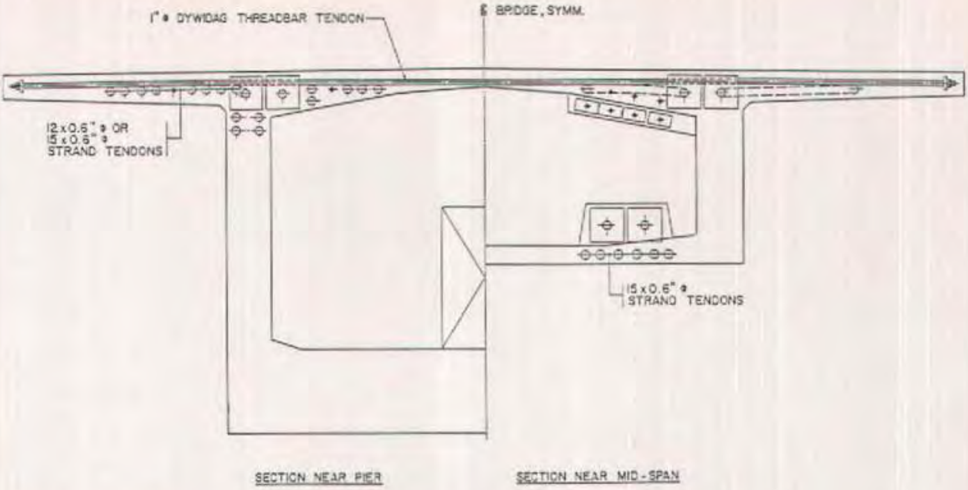

Fig. 3. Cross section and tendon arrangement (South Approach).

Instead of building this portion of the bridge on falsework, the actual construction used the balanced cantilever method employing formtravelers. The need of an auxiliary pier for the cantilever at the closure pour was eliminated by reducing the cantilever from the original 184 to 154 ft (56 to 47 m) as shown in Fig. 2.

This eliminated the danger of possible collision against the auxiliary pier by river traffic which could have serious consequences. The tendon layout in this portion of the girder was modified to accommodate segmental cantilever construction (Fig. 3). Despite the stringent quality requirement of 8000 psi (55 MPa) concrete specified for this portion of the bridge, construction proceeded successfully.

The layout of the segments is shown in Fig. 2. The pier tables are 36 ft 9 in. (11.2 m) long. They were constructed atop relatively small knee brackets. Formtravelers were then assembled on top of the pier table. The typical segments are 15 ft 3 in. (4.1 m) long.

The pier table was built off center eccentrically on the pier. This reduced the unbalanced load at each side of the cantilever to a half segment. The weight of the formtraveler was approximately 150 kips (667 kN).

The knee brackets were designed only to support the weight of the pier table. As the cantilevers grew longer the unbalanced moment also increased. Sand jacks and vertical tiedowns were used to support the superstructure at the piers to transfer the unbalanced bending moment from the superstructure to the substructure. This eliminated the heavy brackets which are common in most cantilever constructions.

The advantage of using sand jacks, aside from being less expensive, is that they are easy to install and dismantle. However, certain deformation characteristics of sand jacks during loading and unloading cycles must be taken into consideration to control the bridge camber during construction. This was done by plotting the as-built camber and comparing it to the theoretical camber to determine the rotation of the bridge due to the sand jack settlements so that the elevations of the new segment could be set properly.

The above procedure was successfully utilized on this bridge.

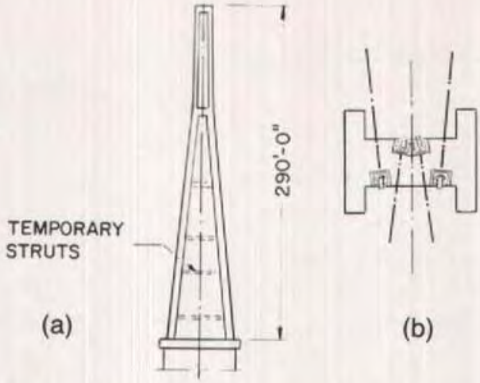

Fig. 4. Elevation and cross section of pylon.

Pylon Construction

The portion of the main pier below the bridge girder was completed under a separate contract. Construction of the pylon started after the last lift of the pier shaft was cast.

The pylon has the shape of an inverted Y (Fig. 4). The inclined pylon legs were cast in lifts of approximately 18 ft (5.5 m). Because the legs were inclined it was necessary to place struts between them in order to reduce the bending moment due to the inclination. The struts were designed to be capable of applying a horizontal force to the tower legs to reduce the dead load bending moment.

The legs were also cast with an outward camber to compensate for the dead load deflection due to the inclination. The pylon stem, at the top, was cast in lifts of approximately 15 ft (4.6 m). This

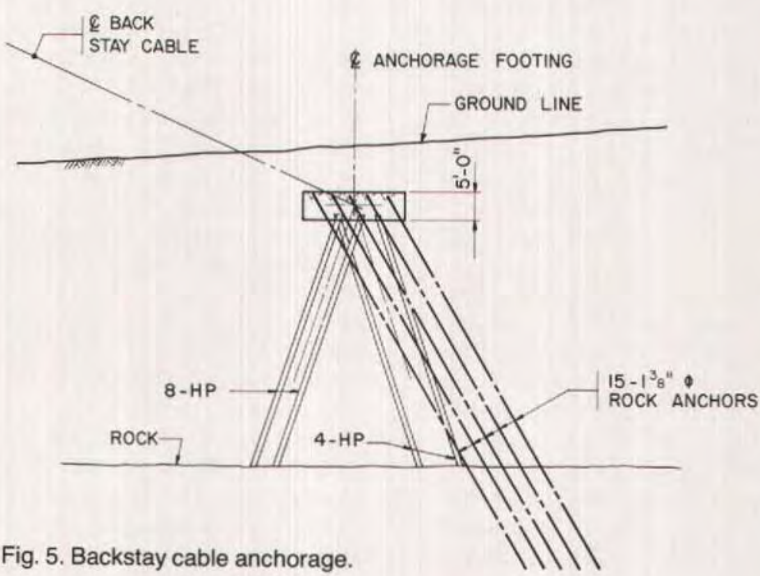

Fig. 5. Backstay cable anchorage.

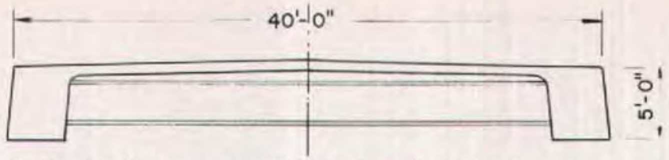

Fig. 6a. Typical cross section of precast segment.

stem has an H-shape cross section. All cables are anchored at the middle part of the H-section (Fig. 4b). In order to ease the process of accurately setting the cable anchorages at the tower head, the contractor used a prefabricated light steel frame to support the anchorages.

Fore and backstay cables were used to stabilize the tower during the cantilever construction of the main bridge. The forestay cable consists of two 55 ½ in. (13 mm) diameter strands bundled together. It was anchored at the lower end by looping around the approach pier foundation. The backstay cable utilized the final end cables by extending them to anchor at a concrete block which was tied down by piles and earth anchors (Fig. 5).

The specified concrete strength for the tower was 6000 psi (41 MPa) at 28 days. However, for part of the pylon the contractor elected to use the same 8000 psi (55 MPa) concrete mix which was used for the approach spans because he had since gained experience with this concrete mix.

A tower crane was used for material transport at the tower. This tower crane was supported by piles driven adjacent to the pier. The crane was also attached to one of the tower legs. It introduced to the tower legs additional forces and moments which were included in the stress analysis of the pylon.

Cable Stayed Girder

Except for a portion of the girder at the end of the side span, which has an enlarged cross section that was cast on falsework, the cable stayed portion of the girder consists of 40 ft (12 m) wide and 45 ft (13.7 m) long precast segments. This typical cross section consists of two 5 ft (1.5 m) deep and 3 ft 6 in. (1.2 m) wide edge girders (Fig. 6a). The deck slab is 8 in. (203 mm) thick supported by 33 in. (838 mm) deep steel floor beams spaced at 9 ft (2.74 m) centers. The steel beams are connected to the concrete slab and edge girders by shear studs.

The specified 28-day compressive strength of 8000 psi (55 MPa) for the girder elements was achieved without difficulty. The segments were match cast. Due to the relatively small number of segments (a total of 27), a long line casting method was chosen. It is easier to match the casting curve by using the long line method. It requires less sophisticated formwork and elevation controls than for the short line method. After two starter segments were cast, which were to be located at the pylon, the balance of segments were cast in two lines.

To ensure that no undesirable settlement occurred during casting, curing and storage, the forms and segments were supported by two lines of footing located underneath the edge girders. The concrete footing was cast on a compacted sub-base. The top surface of the footing was graded according to the casting curve of the precast concrete segments.

Soil between the footings was removed to facilitate easy removal of the segments by tractors (Fig. 6b). The slab form was supported by the steel floor beams and these steel beams were supported by timber piles during casting of the segments.

Fig. 6b. Casting bed arrangement of precast segment.

Fig. 7. Barge crane for segment erection.

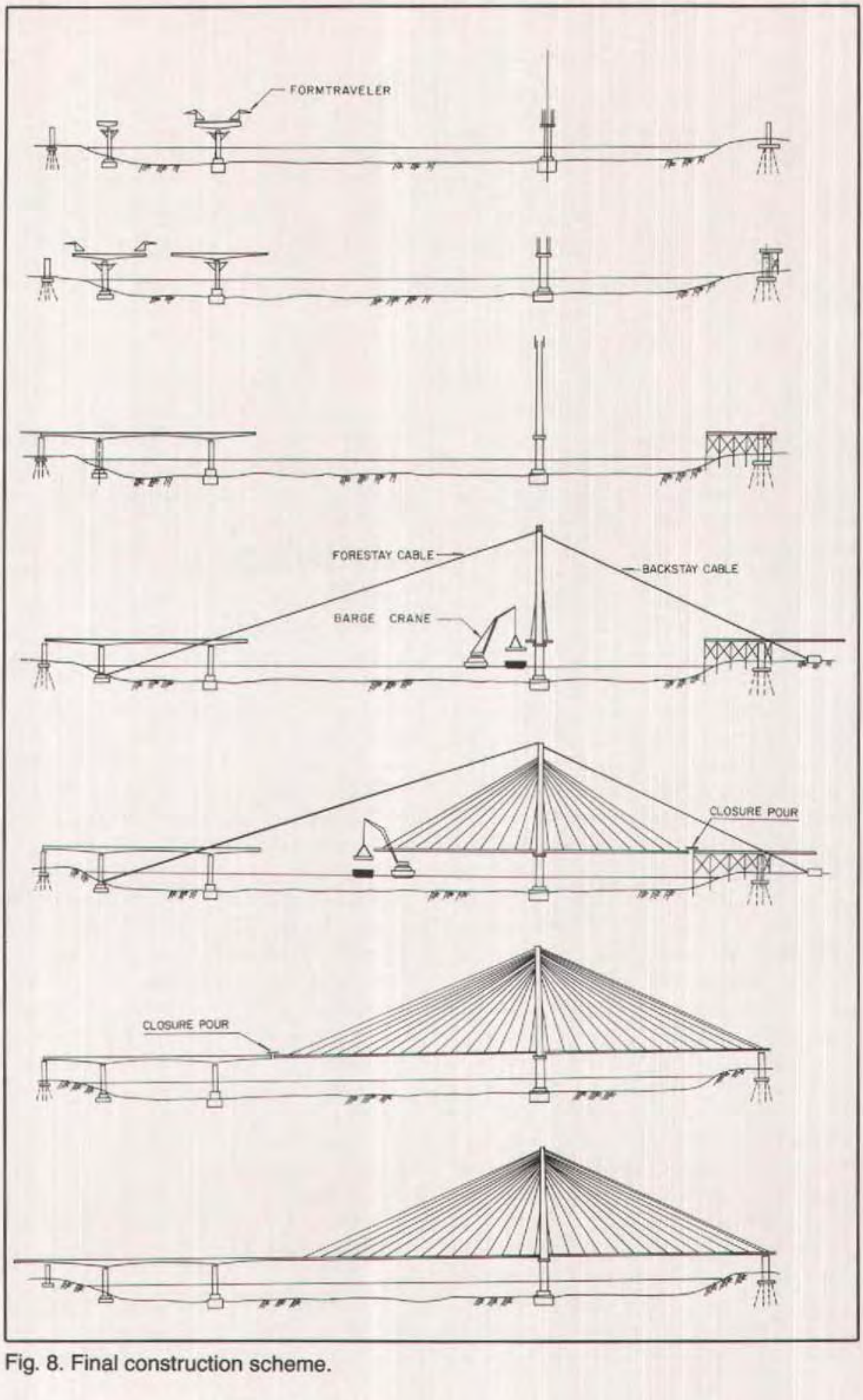

Fig. 8. Final construction scheme.

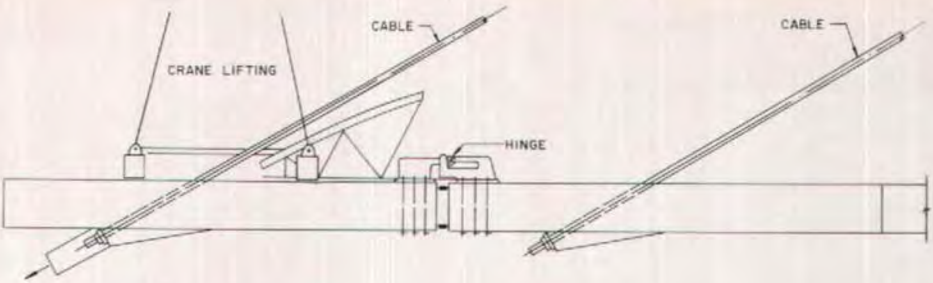

Fig. 9. Arrangement of erection equipment.

A casting curve was provided by the engineer in the bid documents based on the suggested construction method. But the actual construction loads and construction schedule are seldom the same as assumed by the design consultant. This changes the creep and shrinkage deformation of the structure. A new casting curve had to be calculated. The bridge girder for this structure is so flexible that deviations in the construction sequence, in the construction schedule, or in the weight of the form-traveler would cause a noticeable change in the casting curve.

A new casting curve was obtained based on the selected construction sequence. Unfortunately, it was decided that the method of segment erection should be modified because of economy after several segments had been cast according to the first casting curve. The revised construction scheme yielded a new casting curve which was different from the original scheme. It became necessary to manipulate the construction procedure to produce a new casting curve which matched closely the one used for the already cast elements. This was quite a tedious task due to the limitations of allowable stresses in the cables and in the very flexible bridge girder.

Fortunately, a very sophisticated computer program was available to perform this task. Numerous casting curves were generated by the computer based on manipulation of cable forces and closure pour procedures. A final casting curve was obtained which adequately matched the casting curve used for the previously precast segments.

Segment Erection

The bid documents suggested that the erection of segments be done with a gantry supported by erection cables. Stress analysis, casting curve and camber curves based on this scheme were developed. An erection traveler was designed to perform this task. Because no tensile stress is allowed at the precast segment joints, the bridge girder has a very small capacity to resist bending.

Several steps of cable adjustments would be required during segment erection and launching of the erection traveler to meet this criterion. Cable adjustment is a time consuming task which should be minimized in any way possible.

Due to the subsequent availability of a large barge mounted crane (Fig. 7), it was found that an alternative method of erection using the crane would be more economical, faster and safer. A new analysis was carried out based on an erection scheme using the barge crane (Fig. 8).

The weight of a precast segment including strong backs for lifting is about 250 tons (227 t). The barge crane is capable of lifting this segment, with all attendant loads, and placing it in its final position for attachment to the structure. The segment was supported by the crane until the stay cables were attached and stressed (Fig. 9).

Because the crane was mounted on a barge and subjected to wave action of the water, movement of the crane during segment erection was investigated to ensure that the bridge would not be adversely affected during this sensitive operation. A hinge device was developed to facilitate segment attachment to the girder while maintaining full alignment. The hinge mechanism has several hydraulic jacks for fine adjustments in all three directions. The hinge elements were attached to the segments by Dywidag tendons before erection. It was designed as a connecting device to easily receive the subsequently delivered segment.

The first piece of the bridge deck was cast in place on top of a knee bracing together with a short starter segment at each end. These same starter segments had been used as the first segment in the match casting. The far end of the side span had a different cross section than the regular segments because it was used as a ballast to counterbalance the uplift forces created by the longer main span. A stronger girder was also needed to resist the higher bending moments at this region. This portion of the bridge girder was cast in place on formwork supported by falsework before the cantilever segments were erected (Fig. 10).

When a segment was ready, it was placed on a barge and floated to the site. The barge crane picked up the segment and raised it to the appropriate elevation where the hinge mechanism at the ends of the segments engaged each other (Fig. 11). In order to ensure perfect matching, the segments were pulled together for a trial match. This was done at the beginning for several segments until all concerned were confident that the segments would match properly. The horizontal jack then pushed the new segment about 18 in. (457 mm) away from the old segment and epoxy was applied to both faces of the matching joint.

In the meantime, the cables were erected and were tensioned while the segment was still suspended from the barge crane. A synchronized step by step operation of cable stressing, retreating of the horizontal jack as well as

Fig. 10. Falsework support for end segments.

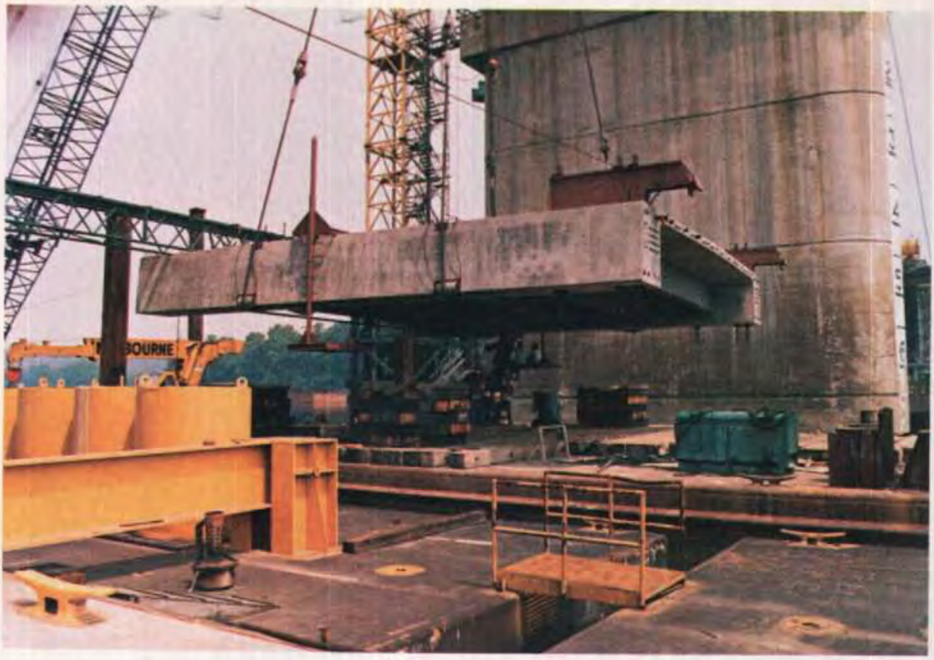

Fig. 11 (top and bottom). Views of segment erection.

stressing of the longitudinal tendons had been worked out on paper before the actual erection.

The axial force applied a predetermined pressure across the matched joint. Safety shim plates were inserted or removed during all operations so that the gap between the segments was kept to about 2 in. (51 mm) at several locations (Fig. 12). This was a safety precaution to eliminate the very improbable event that if the hinge mechanism skidded, the impact caused by the collision of the segments would be limited to an acceptable level. The actual erection went very well without any incident.

End Span Closure

The end span closure pour was located between the sixth and seventh cables at the north end of the structure. The bridge girder north of the closure segment was cast in place several months before the closure took place. This allowed the concrete to age so that the effect of creep and shrinkage was minimized. This portion of the bridge girder had a heavy solid cross section. It was cast on forms supported by local falsework and was cast in three horizontal pours.

Before casting the closure, the north cast-in-place girder was pushed away from the closure to compensate for future movements at the end bearings. A predetermined shear force was also introduced into the closure segment. Steel beams were used to clamp the bridge girder at both sides of the closure pour in place. The closure segment was then formed and cast. Post-tensioning was applied after the closure segment attained a concrete strength of 3500 psi (24 MPa).

Once the end span closure was completed, the bridge became a highly indeterminate structure because all falsework columns became elastic supports of the bridge deck. They must be considered in the structural analysis of the bridge in the subsequent construction stages.

Fig. 12. Safety shims.

Main Span Closure and Transition Segment

The main span closure connects the cable stayed precast girder to the cast-in-place box girder. Because the cable stayed girder is 40 ft (12 m) wide and the cantilevered box girder is 33 ft 6 in. (11.2 m) wide, a special section is required to facilitate this transition (Fig. 13). Various construction schemes were studied for this transition segment. Finally, the contractor decided to cast this transition segment in three stages. The last pour, which was 3 ft 6 in. (1.2 m) long, was the closure segment.

Because no temporary supports were used, the cantilevered box girder was not capable of supporting the transition segment by itself. Steel girders were

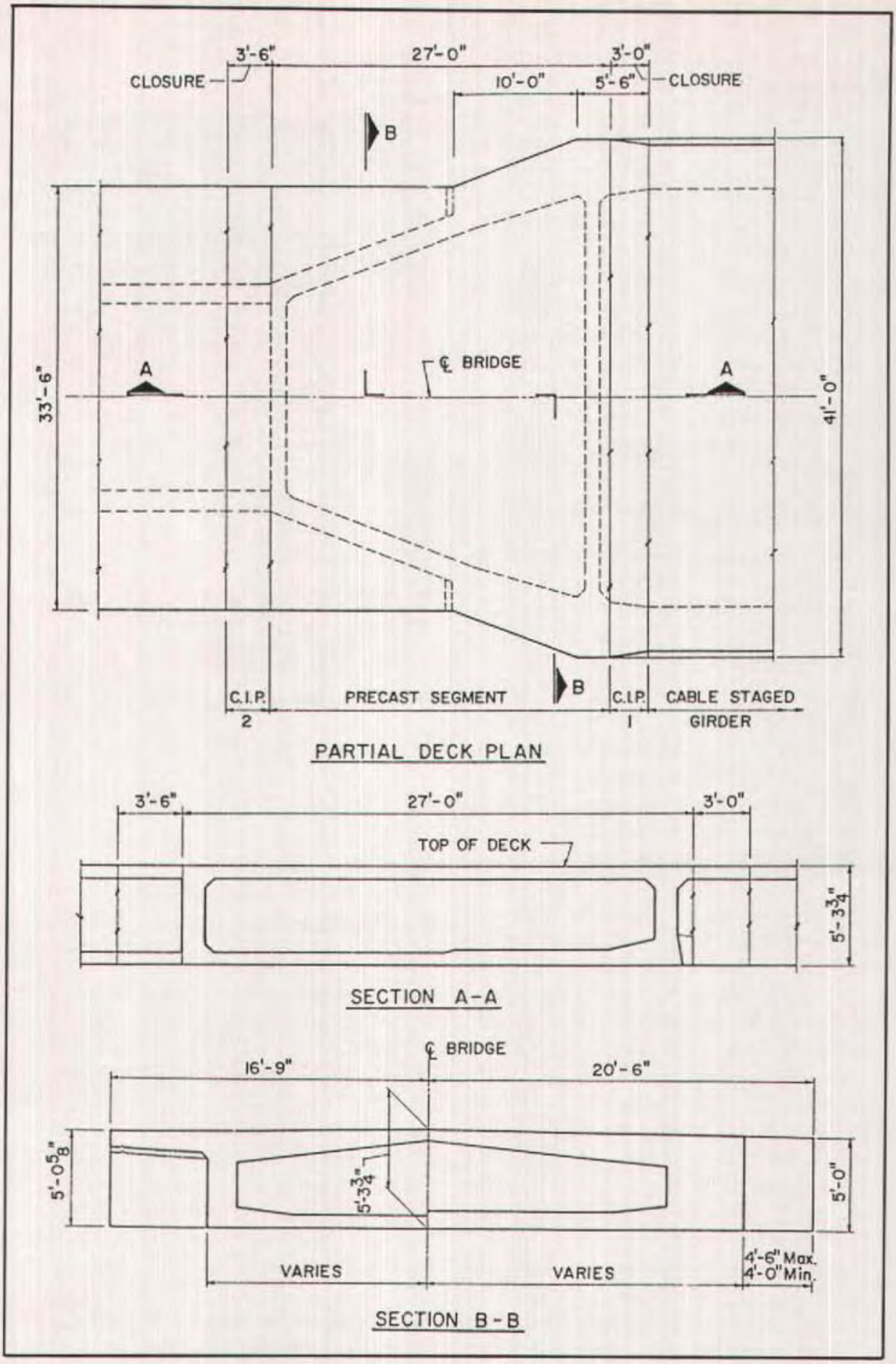

Fig. 13. Transition segments.

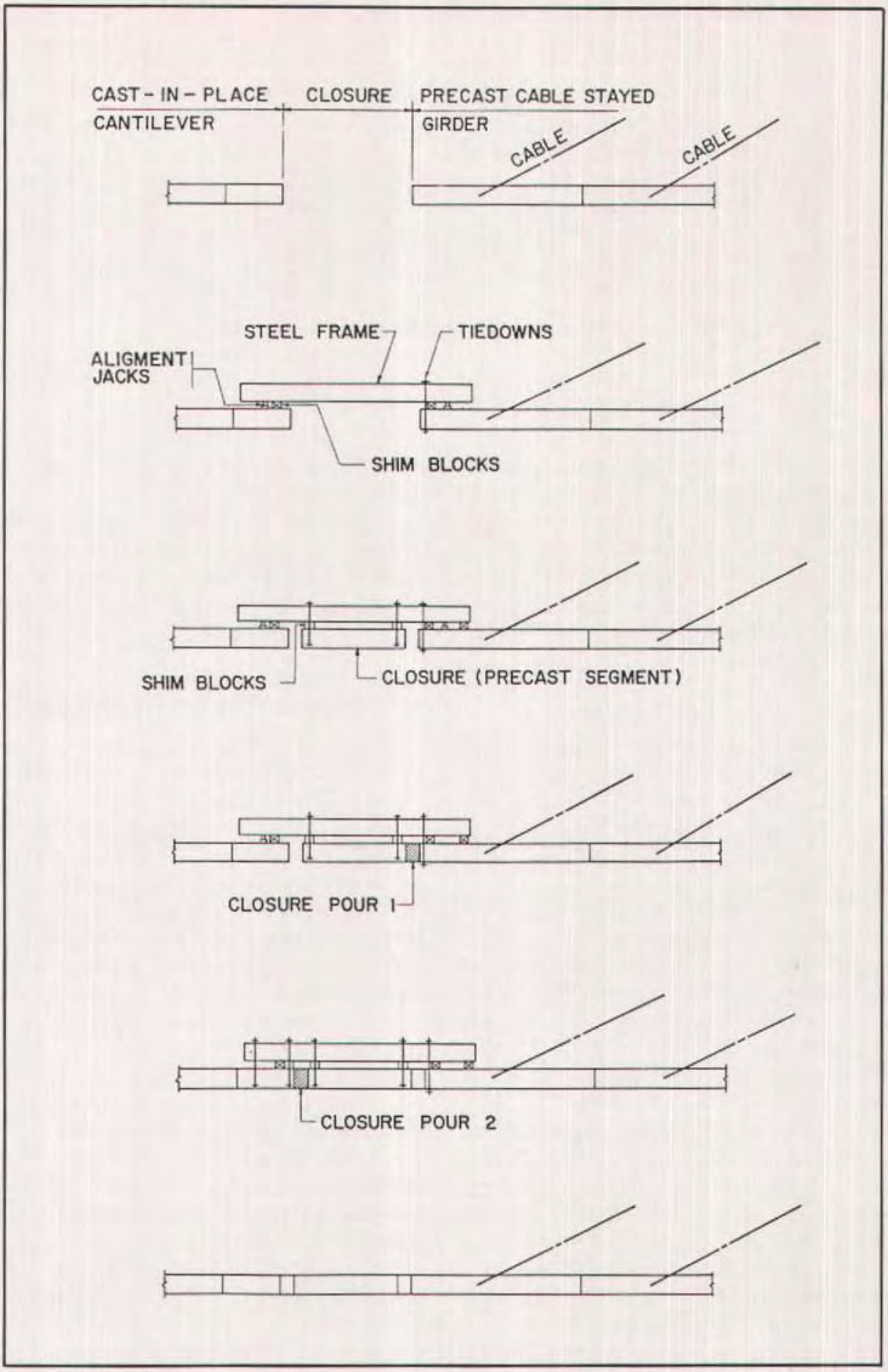

Fig. 14. Closure procedure.

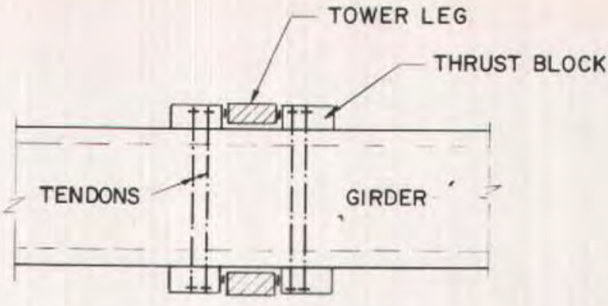

Fig. 15. Horizontal restraining device.

placed across the gap which supported the formwork and distributed part of the segment weight to the cable stayed girder (Fig. 14).

Before the closure pour was cast, a series of jacking operations introduced to the girders a set of predetermined bending moments and shear force which were locked in after the closure was cast. This operation was necessary to achieve the given permanent load condition as prescribed by the design engineer. In addition to this operation, adjustment of several cables before and after the closure was necessary.

Cables

Permanent cables consist of 0.25 in. (7 mm) diameter wires with high amplitude (Hi-Am) sockets. They were preassembled and delivered to the site in coils.

To erect the cable, the coil was placed near the tip of the cantilever. The cable was pulled out of the coil and laid on the bridge deck. The anchorages were then pulled through the anchorage pipes at the tower and at the edge girders by winches. Stressing of the cables took place at the edge girder. The cables were stressed in pairs at each segment to avoid twisting of the deck.

The cables were grouted with cement grout after completion of all required adjustments. The grouting was done in lifts to avoid excessive hydrostatic pressure which could cause cracking of the polyethylene (PE) pipe.

Tedlar tape was wrapped on top of the PE pipe after grouting was completed. This wrapping provides a temperature shield and an additional layer of protection for the cables.

Horizontal Restraint at Tower

The final bridge has a pair of horizontal bearings to restrain the relative movements between the girder and tower. But they are designed for relatively small forces which are much lower than the possible horizontal loads during construction. Therefore, temporary restraints had to be installed to stabilize the bridge deck against unbalanced loadings acting on the girder.

There were two groups of loads that required the installation of restrainers at the tower. The first was caused by unbalanced vertical loads between the cantilevered girders. Although the segments are arranged symmetrically, the segment at one cantilever must be erected before the corresponding segment at the other cantilever can be erected, so that an unbalanced moment cannot be avoided. An additional unbalanced live load of 5 psf (0.24 kPa) of bridge deck was also assumed for design purposes.

The second group of loadings is

caused by unbalanced horizontal wind load acting laterally to the bridge deck. This wind load was taken as 50 percent of the balanced wind loads based on the assumption that it was very unlikely that wind will act on one side of the cantilevers only. These two groups of loadings are to be added to each other, thus, resulting in a maximum restraining force of 800 kips (3636 kN) at one side of the deck.

Four large steel brackets were tied to the edge girder of the deck against each side of the two tower legs (Fig. 15) by Dywidag Threadbar tendons to provide the restraining capacity.

Stage Analysis and Camber Control

All structural analysis was performed using a computer program, called CABB, developed by the author. Creep and shrinkage factors were assumed as 2.0 and 0.000020, respectively.

The computer program is capable of performing both linear and nonlinear analyses. However, a study of several critical stages indicated that for the selected construction scheme, linear analysis was sufficiently accurate so that the complete analysis was carried out according to the linear theory. The cable stiffness was modified to consider the sag of the cables.

The computer program checks all stresses at every joint and compares them with the allowable stresses. It also compares the ultimate bending moment and shear of the girder and tower in each construction stage with the ultimate capacities at the corresponding joints. Any overstress or under capacity will be printed out so that cable forces can be adjusted to alter the stress condition of the structure until all stresses are within the allowable ranges.

As a result of this construction stage analysis, a set of camber curves and corresponding cable forces were given to the contractor for use at the site. The contractor surveyed the bridge to obtain the actual geometry of the bridge at all critical stages and compared his surveying results with the calculated camber values. The survey must be done with minimum effect of temperature. Therefore, it was done during early mornings before sunrise.

All survey data were transmitted to the engineering office for immediate evaluation. Excessive deformations were immediately investigated.

Throughout the construction, the actual surveyed geometry of the bridge matched the predicted camber very well. Generally, the deviations in deflection were less than 2 in. (51 mm) except for very few situations where a deviation of up to 4 in. (102 mm) was detected. Subsequent investigations found that the excess deviations were all explainable and mostly due to differences between the actual and the assumed loading conditions.

Aerodynamic Investigations

Wind tunnel tests of the section model of the bridge were carried out by the University of Colorado for the design engineer before the bidding. A copy of the test report was made available to the contractor and his consultant after the bidding.

Although the original testing did not address construction stage behavior of the bridge, test results of the section model had been sufficient to predict the aerodynamic behavior of the bridge during various construction stages after some modifications. No additional tests were carried out for construction purposes.

Calculations for flexural vibrations, torsional vibrations and flutter found that no unacceptable wind induced vibrations were to be expected during construction. Therefore, no additional measures were necessary to stabilize the bridge against the predicted wind speed.

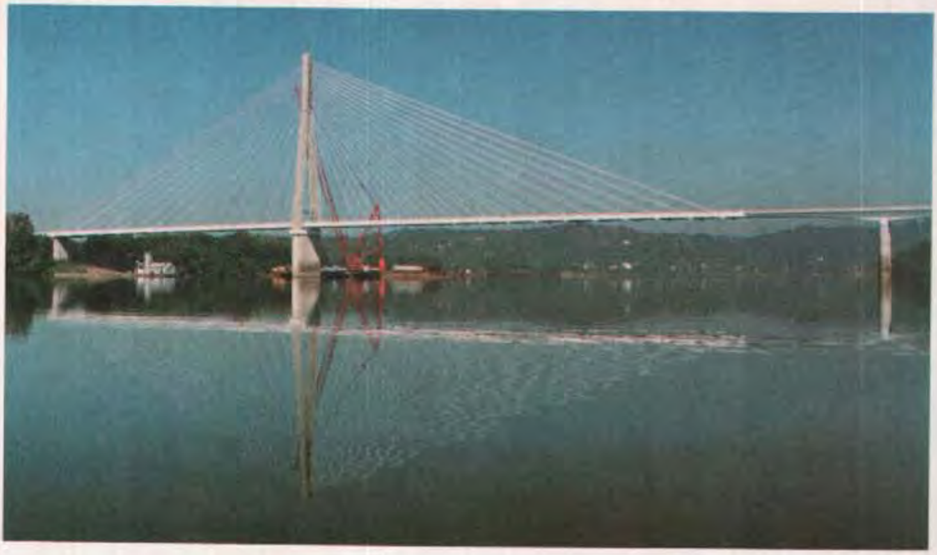

Fig. 16. East Huntington Bridge nearing completion. Photo courtesy: Arvid Grant and Associates.

Closing Remarks

The East Huntington Bridge was completed successfully based on a well planned construction scheme. The bridge behaved basically as predicted by the calculations. It is a very attractive structure that blends well with the surrounding landscape (see Fig. 16). The bridge won an award in the 1985 PCI Professional Design Awards Program.

Acknowledgment

The owners of the bridge are the State of West Virginia and State of Ohio, with William Domico, director, Structures Division of West Virginia Department of Highways, as manager of the design project. Luther Wickline and Stan Meadows under the direction of Earl Scyoc, chief engineer, Construction and Materials, were responsible for construction supervision.

The engineer for the main bridge is Arvid Grant and Associates in collaboration with Leonhardt und Andra Partners, and project engineers were Conrad Bridges and David Goodyear.

The contractor was Melbourne Brothers. Their project manager was Brian Danaher and project engineers were John Macrae and Doc Woodard.

Construction engineering and equipment design were done by Contech Consultants, Inc. Project engineers were Joseph Tse and Nien-Sheng Chung under the supervision of Man-Chung Tang.

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