# Unique Methods Used in Constructing the Robert E. Lee Bridge

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The Robert E. Lee Bridge in Richmond, Virginia, consists of twin trapezoidal concrete box structures 3,400 ft long and consisting of constant-width and varying-width cross sections placed with form travelers. This bridge represents two firsts in North America: (a) the largest number of form travelers (eight) ever used concurrently, and (b) the first use of a strand stability system in segmental concrete balanced cantilever construction. The use of portable fabric cofferdams was a unique method for constructing the bridge piers in the James River. These fabric cofferdams provided a safe, environmentally sound structure within which to drill and blast for rock excavation and to place concrete for the pier footing and the first lift of the pier columns in the river.

The existing Robert E. Lee Bridge is a concrete spandrel arch structure constructed in 1935. Functionally obsolete, it had four traffic lanes and a load limit of 20 tons. Starting from the south, both bridges (new and old) span the Southern Railroad's main yard (20 tracks), the south channel of the James River, Belle Island, the north channel of the James River, C&O Railroad's track lines (two each), two city streets, and the historic Kanawha Canal. The upstream drainage area of the James River at Richmond exceeds 3,500 $mi^2$.

The new Robert E. Lee Bridge is a twin-trapezoidal box girder structure 3,400 ft long. The replacement structure provides two independent bridges (one northbound and one southbound). Each bridge has three 12-ft lanes, a 10-ft bicycle lane, and a 4-ft sidewalk. The bridge rises vertically 20.5 ft from its south to its north abutment. The new bridge's trapezoidal box girder cross section varies in width and depth.

## SUBSTRUCTURE

The majority of the bridge foundations are spread footings placed directly on the underlying granite rock. The concrete footings in the river were to be keyed into the granite river bottom. These river footings required drilling and blasting of the bedrock to excavate to the required plan depth. Weathered granite was found in some footings, and the plan depths were lowered to provide bearing on competent rock. Concrete for the footings was specified to be 3,000 psi.

### Fabric Cofferdams

The foundations for the river piers posed special problems. The granite river bottom was composed of an irregular solid granite surface with erosion-cut boulders, cobbles, and some sand. This configuration precluded the use of sheetpiling, because the overlying soil (sand, clay, and silt) was not thick enough to anchor sheetpile. A normal river stage provided water depths between 2 and 10 ft. Rigid environmental restrictions concerning erodable material in the river made any fill-type cofferdam unacceptable.

A common British concept for cofferdams, not yet applied in the United States, was used for the main channel river footings (Figure 1). A fabric cofferdam was installed in the river to provide a work area around each pair of footings, one northbound lane (NBL) and one southbound lane (SBL). The cofferdam consisted of steel frames and a rugged polyvinyl material draped from the frame to the river bottom. After the survey location was determined, divers installed the cofferdam in the following sequence of operations:

1. Cleaned the river bottom of cobbles along the frame lines.
2. Installed the steel frames, which interlocked and rigidly held the fabric. A hydraulic crane on the causeway helped rig the frames to the water. Typical spacing for the frames was 2 ft.
3. Installed the fabric. The fabric, in sheets 30 × 50 ft, was attached to the outside of the frame and rolled down the sloped outward leg to the river bottom and then along the river bottom. The divers weighted this fabric to the river bottom with a chain fixed to the end of the fabric. The fabric was overlapped and connected along the vertical fabric joints with velcro-type fasteners.

An 8-in. diameter discharge hydraulic pump was used for the initial dewatering of the cofferdam. After the fabric was in place and a pressure differential established, a minimal inflow leakage was sustained between the river bottom and the fabric. Leakage from the fabric was channeled by sandbags to sumps. Once water in the portadam was pumped down, a 6-in. hydraulic pump adequately handled cofferdam seepage. The three main pier cofferdams averaged 200 ft wide by 75 ft long.

The rock excavation required drilling and blasting operations. Water gel explosives and nonelectric blasting caps were used. The portadam was partially flooded before shooting the footing to help absorb the concussion from the blast (Figure 2).

On completion of one pair of footings, the cofferdam was quickly and easily removed and reinstalled. Typical installation time was 4 weeks—3 weeks of single shift work and 1 week for dismantling. The fabric cofferdam was environmentally advantageous in that it eliminated the use of steel boxes

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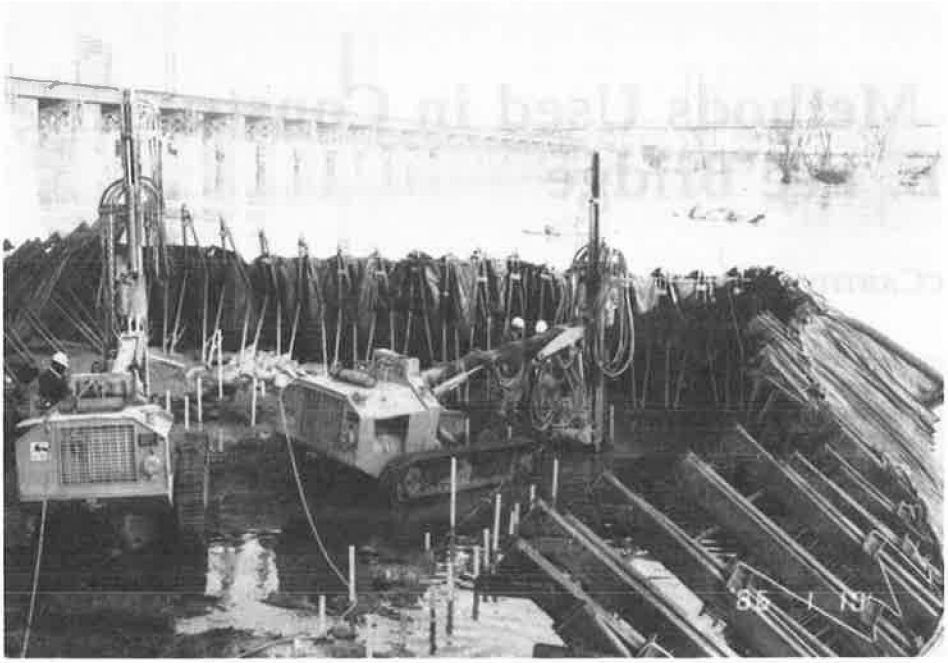

**FIGURE 1 Fabric cofferdam drilling of rock excavation.**

**FIGURE 2 Fabric cofferdam on steel frames after rock blasting.**

with tremie concrete, timber cribs, impervious fill, and other such materials (Figure 3).

The working limitation of the fabric cofferdam is a 9-ft head of water. The portadam was overtopped on two occasions during construction. The worst flood in 25 years caused irreparable damage to the frames and fabric.

### Piers

The pier columns were octagonal in shape and ranged in height to 100 ft. Concrete for the piers was 4,500 psi and was placed either by pump or crane. Steel-plate girder concrete forms were used to form the piers.

**FIGURE 3 Placing of pier concrete in footing with protection of cofferdam.**

### Bearings

Pot bearings were specified for the three ramps and main bridge to allow free rotation at the piers. Guided pot bearings were specified at four locations to allow for expansion and contraction in the longitudinal direction only.

### Pier Tables

The pier tables (the superstructure section cast at each pier to permit erection of the form travelers) were constructed by using falsework supported by steel brackets posttensioned to the piers with threadbars (Figure 4). Wood forms were used to build the pier tables. Concrete was placed in three stages—the bottom slab, the walls, and then the top slab. A typical two-web pier table required 243 $yd^3$ of concrete; a three-web pier table required 436 $yd^3$. Superstructure concrete strength was specified to be 5,000 psi.

## SUPERSTRUCTURE

The main bridge consists of two 15-span structures separated by a gap ranging from 1 in. to 9 ft. The original superstructure construction method called for Span 1 to be constructed on falsework, Spans 2 through 8 to be constructed by the cast-in-place balanced-cantilever method, and Spans 9 through 15 to be constructed on falsework. The three on-and-off ramps were designed to be built on falsework.

After studying the terrain and construction obstacles under Spans 9 through 15, a decision was made to use balanced-cantilever construction methods. The major concern was that falsework under Spans 9 through 12 would be positioned in the highly volatile James River. The consequences of damage to the falsework during flooding was unacceptable. A balanced-cantilever system would transmit all construction loads through the permanent piers, eliminating the need for intermediate supports between piers.

The configuration and design of the mainlane bridge superstructure presented challenges to construction by the balanced-cantilever method. The first order of work was to design a stability system for the slender 9-ft-thick pier stems that would be capable of carrying out-of-balance moments created by cantilevered construction.

### Stability System

Conventional methods used to resist out-of-balance moments during cantilever construction are either brackets fixed to the top of the pier or props from the pier footing (or another foundation) to the box girder soffit.

A bracket system was studied, but the cross section of the piers did not provide a practical means of anchoring the brackets to the piers. The slender cross section of the piers did not provide enough room to lock the connection between the pier and the bottom soffit of the box. In addition, steel was congested in the top of the pier from rebars, pot-bearing anchor bolts, and surrounding spiral reinforcement.

A prop-type stability system was ruled out for two reasons. First, the props would have to support the cantilever at a height of 100 ft if they were to be reused throughout the structure. This required prop sections to be added and subtracted to accommodate the varying girder heights. Secondly, the props could not be used at the 10 piers in the river.

**FIGURE 4 Pier table (two-web) with brackets awaiting traveler erection.**

Exposing the props to forces from the James River during flooding was an unacceptable risk.

The selected stability system incorporated brackets designed to act as pier-table falsework and yet resist the out-of-balance moment resulting from the first segment placement. The pier-table moments were much smaller than the maximum moments produced by the full cantilever. Before the second segment was cast, brackets were replaced by routing the piers through a duct to its anchor plate (Figure 5). The upper anchorage was made in a concrete wall blister that was added and post-tensioned to the web, inside the box at a distance of 28 ft from the centerline of the pier. With completion of the last cantilever, using multistrand stays, the system worked without failure, despite some obstacles.

The stability tie system was costly in time and money. Similar to a prop setup, two separate systems were fabricated and erected. Special tapered concrete shoes were fixed to the faces of the pier-table stability brackets, which mount on a tapered face of the pier. Concrete anchor blocks were required at both the live and dead ends of the ties. The strands and wedge plates could not be reused. Casting the anchor blocks and replacing the brackets with ties took 5 days on a two-web cantilever section and 9 days on a three-web section (Figure 6).

With the stability tie system, the cantilever is not fixed against rotation, as it is with more conventional systems. Provisions were made to compensate for deflections of up to 24 in. at the cantilever tips caused by elongation of the ties that would allow the entire cantilever to pivot at the pier. These rotations were calculated before casting the first segment to ensure that, when the last segment was cast, the tips of the cantilever would be aligned for the closure segment. The rotation of the system added another variable to the camber analysis and increased the risk of actual tip location's not matching the theoretical position. Although the tips were easily adjusted by counterweighting at either end of the cantilever, this fix

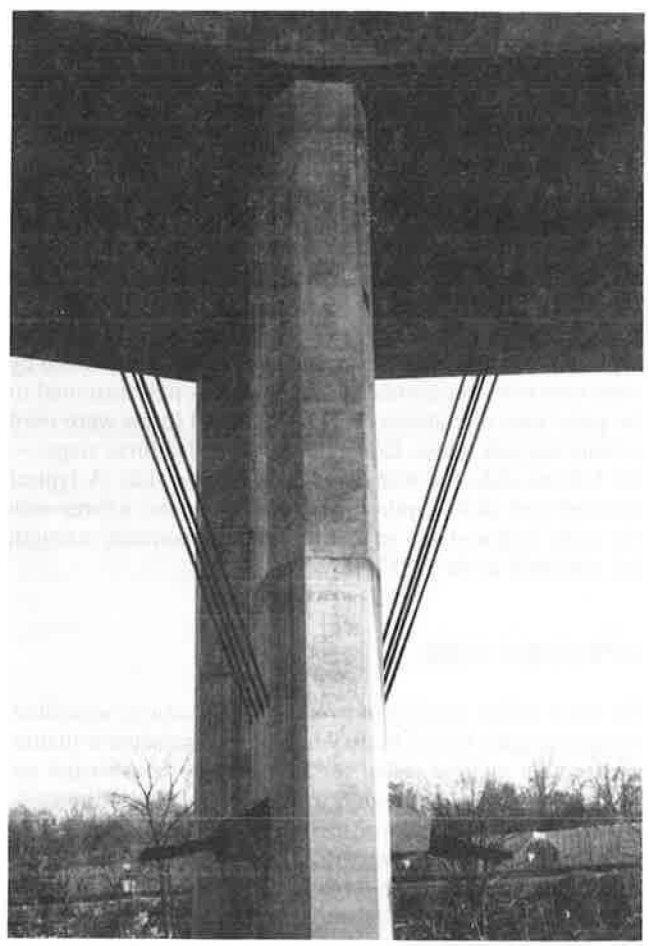

**FIGURE 5 Stability strand ties restraining a three-web cantilever.**

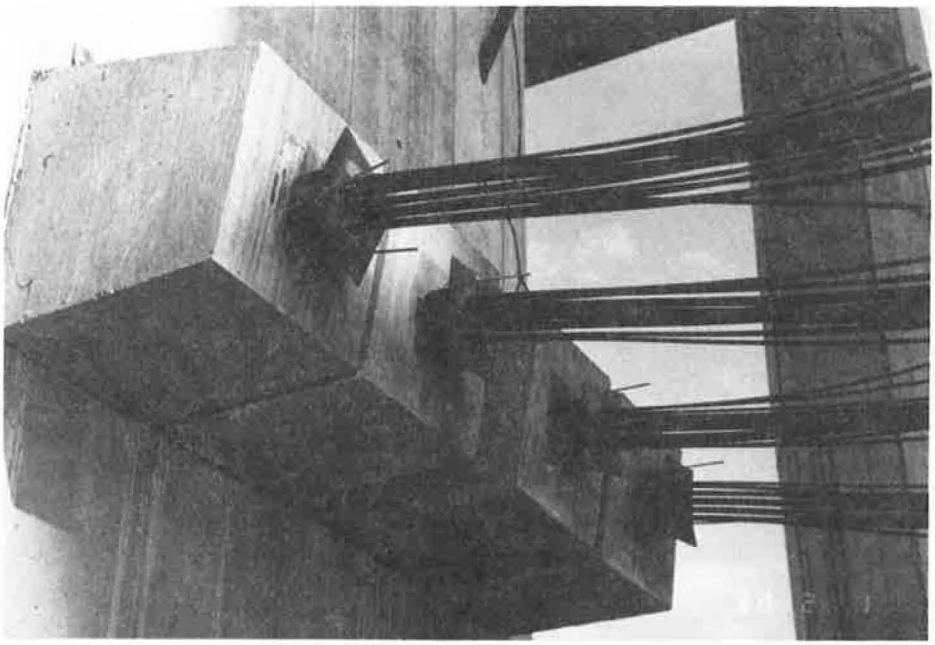

**FIGURE 6 Lower anchorage of strand stability system.**

was limited by the amount of reserve capacity in the ties. If rotations became excessive, the only answer was to discover the rotation problem in the early cantilever segments and replace the stability ties one side at a time to correct the rotation. This type of rotation fix was required on one cantilever.

The wedge-type anchoring system is not designed for repeated load cycles, so the risk of the failure because of strands slipping through the wedges was of constant concern. Care was taken to ensure that the strand, wedges, and wedge plates were free of any dirt and rust before installation. The ties were continually monitored for any signs of slippage. They were designed to have a minimum of 2.50 kips per strand at all times to keep the wedges seated in the anchor plate. The allowable load on the ties was set at 50 percent of their ulti-

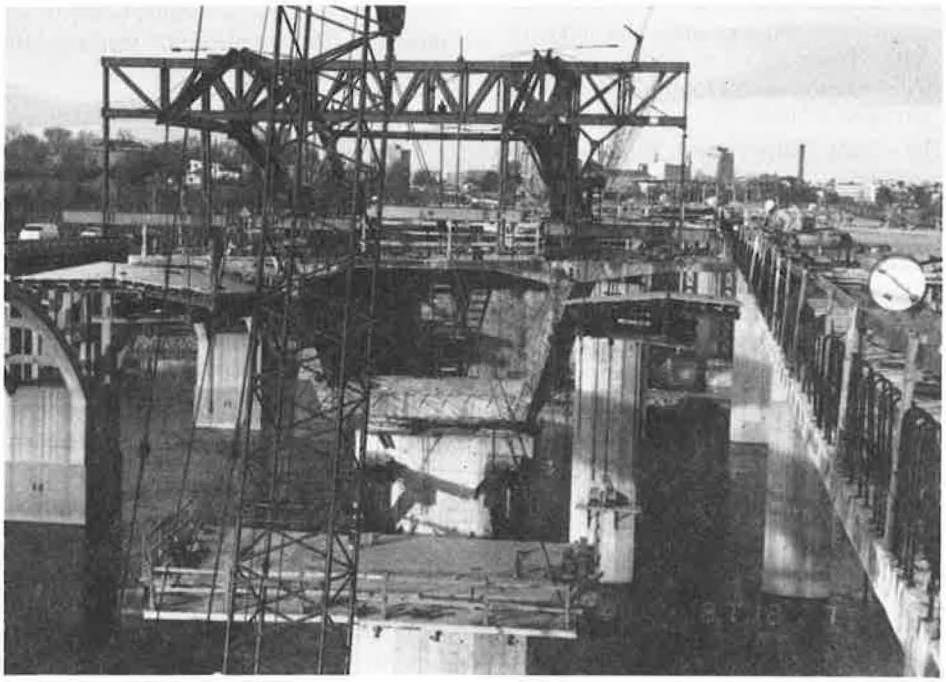

**FIGURE 7 Single-cell (two-web) box—bottom drive deck erection of A side.**

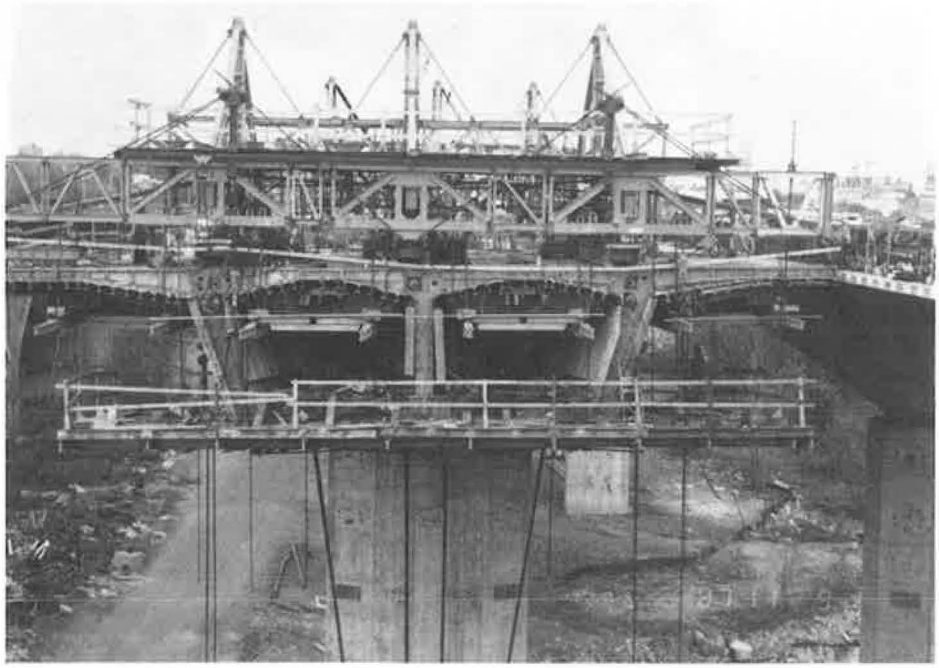

**FIGURE 8 Double-cell (three-web) box construction.**

mate strength. Some strand slippage did occur, and in an isolated single strand, slippage was corrected by restressing the strand with a monostrand ram. In two instances, the entire 19-strand stability ties were replaced because of multiple-strand slippage.

### Travelers

The maximum out-of-balance moment was 44,861 ft-kips at Pier 10 NBL and the maximum rotation under this moment was 0.428° at Pier 2 NBL. Two pairs of constant-width form travelers and two pairs of variable-width form travelers were used to construct the balanced-cantilever portions of the bridge (Figures 7 and 8). The weight characteristics of these are as follows:

| *Type* | *Traveler Weight (kips)* | *Segment Concrete Weight (kips, maximum)* |
|---|---|---|
| Constant width (two-web) | 186.5 | 348 |
| Variable-width (three-web) | 279.75 | 488 |

Segment concrete was placed monolithically using a concrete mix with a specified strength of 5,000 psi. The mix design was modified by the contractor to use superplasticizer. Wood forms were chosen to provide the maximum flexibility in forming the varying soffit width and decreasing wall height of the segments.

A typical segment construction cycle duration was 1 week for the two-web travelers and 1½ weeks for the three-web travelers. Concurrent work was performed on both the A and B travelers on each pier.

## CONCLUSIONS

1. The fabric cofferdams provided a safe, environmentally sound structure within which to excavate and place concrete for pier construction.
2. The inverted-stay stability system was chosen for the Robert E. Lee bridge because conventional systems were not practical. The stability ties worked without failure on 22 cantilevers.

## ACKNOWLEDGMENTS

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## REFERENCE

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