$\Delta K$ = range of stress intensity factor = $f(a) S_r \sqrt{\pi a}$,
$S_r$ = constant amplitude stress range, and
$f(a)$ = correction factor for crack shape, stress gradient, and so forth (6).

Equation 1 may be rearranged and integrated to give an estimated life (N):

$$N = \int dN = \int_{a_i}^{a_f} \left(da / \{3.6 \times 10^{-10} [f(a) S_r \sqrt{\pi a}]^3\}\right) \quad (2)$$

For the hanger plate shown in Figure 7, an initial corner flaw of $a_i$ = 2.54 mm (0.1 in.) is assumed with a detectable final crack size of $a_f$ = 25.4 mm (1.0 in.). The constant amplitude stress range is estimated from the stress histogram by using Miner's hypothesis and is equal to 5.8 MPa x 3.2 = 18.5 MPa (2.7 ksi). By incorporating the stress gradient of Figure 7 into an expression for the connection factor f(a), the resulting estimate life is 506 x $10^6$ cycles. If 2,000 cycles per day are induced by trucks, it would take many years for the crack to grow. Thus, if the hanger plate is made of steel with adequate toughness against brittle fracture, there should be ample time for inspection if a crack would ever develop.

CONCLUSIONS

In conclusion, the following points are restated.

1. Hanger plates of suspended bridge girders are subjected to bending as well as axial forces.
2. In-plane bending results from friction at the pin and the relative rotations of the girders at the hanger plate.
3. Live-load stresses at the edge of pinholes are higher than those in the hanger plates.
4. Fatigue cracks could grow from pinhole edges.

For the case studied, there is ample time for inspection. Further studies are needed on the adequacy of current design assumptions and, particularly, on the behavior of plates in relation to girder geometry and bridge dimensions.

ACKNOWLEDGMENT

Field data and part of the results of this study were obtained under an FHWA research project. This sponsorship is acknowledged.

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*Publication of this paper sponsored by Committee on Steel Bridges.*

# Design of the Cable-Stayed Girder Weirton-Steubenville Bridge

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ABSTRACT

When completed, the Weirton-Steubenville Bridge will be the sixth cable-stayed girder bridge constructed in the United States. The design to be constructed at a $20 million cost was chosen in 1983 from three bridge designs presented for construction bids. Crossing the Ohio River between Weirton, West Virginia, and Steubenville, Ohio, the new bridge will be 1,965 ft from abutment to abutment and have a main span of 820 ft. A concrete, inverted Y-shaped tower, which rises 365 ft above the supporting pier, features above its apex a 140-ft-high pylon that supports a dual-plane cable system. Materials specified for the composite bridge were placed where their properties would provide the greatest advantages without sacrificing integrity and function. Fascia girders are I-girders with webs skewed at 10 degrees from the vertical, thus reducing cable-connection eccentricity, material quantities, and steel fabrication costs. The composite superstructure consists of longitudinal stringers, transverse floor beams, and a concrete deck--all treated as an orthotropic system. Further,

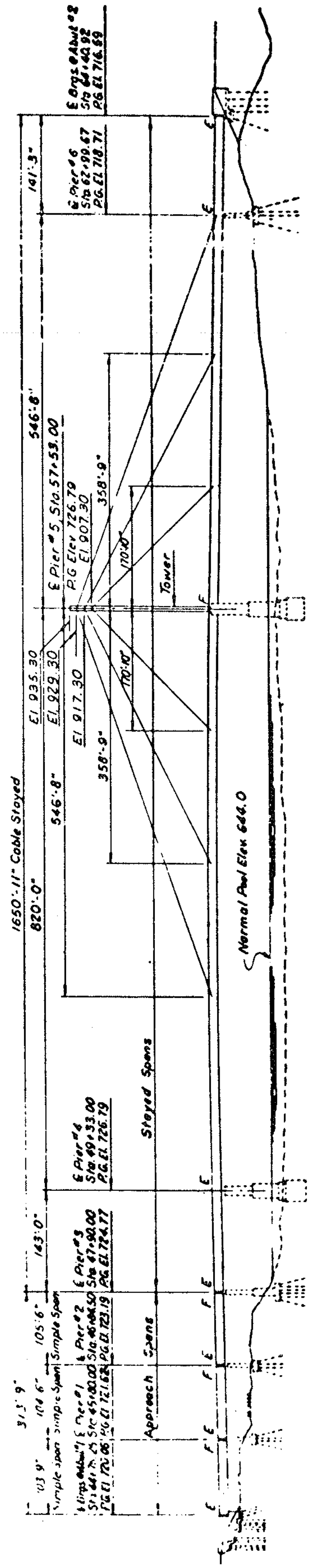


FIGURE 1 Steel alternate design 1.

horizontal trusses are placed at the bearings of the tower pier and at the outermost cable connections on the ends of the bridge to distribute axial load throughout the deck. The approach spans are continuous, composite, multigirder types. Load-factor design was used in the approach spans, substructure units, the tower, and the multigirder portions of the stayed spans.

In the early 1970s the West Virginia Department of Highways retained Michael Baker, Jr., Inc., to study and recommend a current state-of-the-art bridge to cross the Ohio River between the towns of Weirton, West Virginia, and Steubenville, Ohio. After studying the several bridge types proposed by Baker, the state of West Virginia requested that a cable-stayed girder bridge be designed for this river crossing.

STEEL ALTERNATE DESIGN 1

The initial bridge design consisted of a four-span, cable-stayed girder main river structure that had three simple-supported approach spans (see Figure 1). The approach spans were chosen so that future ramps could be added easily.

The cable-stayed girder portion of the bridge was a four-span continuous box girder that had spans of 143, 820, 547, and 141 ft. The superstructure consisted of a two-cell rectangular steel box girder with outriggers supporting the full six-lane bridge width. The deck was an orthotropic steel design that had an epoxy asphalt wearing surface. The tower was A-shaped with the deck passing through the steel box legs of the frame (see Figure 2). A single-plane, fan-shaped cable system was used that featured six cable lines. These cables connected to the tower and extended through the median, which was attached to the middle web of the two-cell box girder.

On the Ohio side the three 102-ft simple-sup-

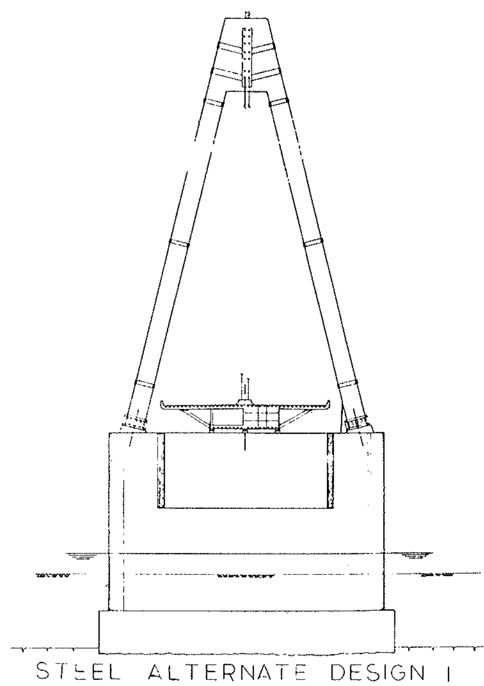


FIGURE 2 Typical cross section of cable-stayed bridge at pier 5 (design 1).

ported approach spans consisted of twin steel box girders with outriggers supporting an orthotropic steel deck that had an epoxy asphalt wearing surface.

Of the eight substructure units--six piers and two abutments--six were supported on steel H-piles driven to rock, whereas the two main river piers (piers 4 and 5) had footings founded on rock.

The abutments were standard U-shaped cantilevers with high back walls to accommodate the deep box girders.

All the piers were basic solid shaft design. Pier 6, as the anchor pier for the cable-stayed girder unit, was modified by a slot in the center to accommodate the tie-down assembly.

The configuration of pier 5 was dictated by the A-frame tower (see Figure 2).

CONCRETE ALTERNATE DESIGN

In 1978 the FHWA directed that an alternate concrete bridge be designed. Another engineering consultant was chosen to design the alternate by using the cable-stayed girder concept with concrete as the main material. This design used a concrete fascia girder, steel floor beam, and concrete deck and tower. This design was completed in 1983 and bid against the two steel alternates.

STEEL ALTERNATE DESIGN 2

The original steel alternate bridge was developed as a state-of-the-art bridge in 1974. Then nearly 7 years went by from the time that the original bridge design was initiated. This, along with the FHWA directive requiring a concrete alternate design, prompted the West Virginia Department of Highways in 1981 to request a redesign study to determine the economics of updating the original design or creating a new cable-stayed girder bridge design that reflected the current state of the art.

REDESIGN STUDY

In 1981 the West Virginia Department of Highways retained Baker to study whether or not a new cable-stayed girder design might have a lower construction cost than the original steel design. To arrive at a lower construction cost, two major cost areas were addressed: the first was the fabrication costs involved in different deck and tower cross sections; and the second was material quantities and types. These savings were to be made without sacrificing the serviceability or the structural integrity of the bridge. To accomplish these goals the study was divided into two phases. The first phase considered five superstructure cross sections of the bridge structure, as shown in Figures 3-7. The relative cost of the sections was estimated, and then each section was evaluated and ranked by using the criteria given in Table 1. On the basis of the results of the evaluation, Baker recommended cross-section type 2, which the West Virginia Department of Highways selected for use in the second phase of the study.

In the second phase of the study the bridge was investigated along its longitudinal axis. Pier and abutment locations already had been established in the original steel design. In addition, piers 4 and 5 had been constructed by this time. Consequently, the major items remaining to be determined were the cable arrangement and spacing as well as the tower height, cross section, and transverse configuration. Researching the literature, studying recent cable-stayed girder bridge designs, and performing preliminary calculations revealed that a dual plane of cables with connections spaced at about 60 ft was

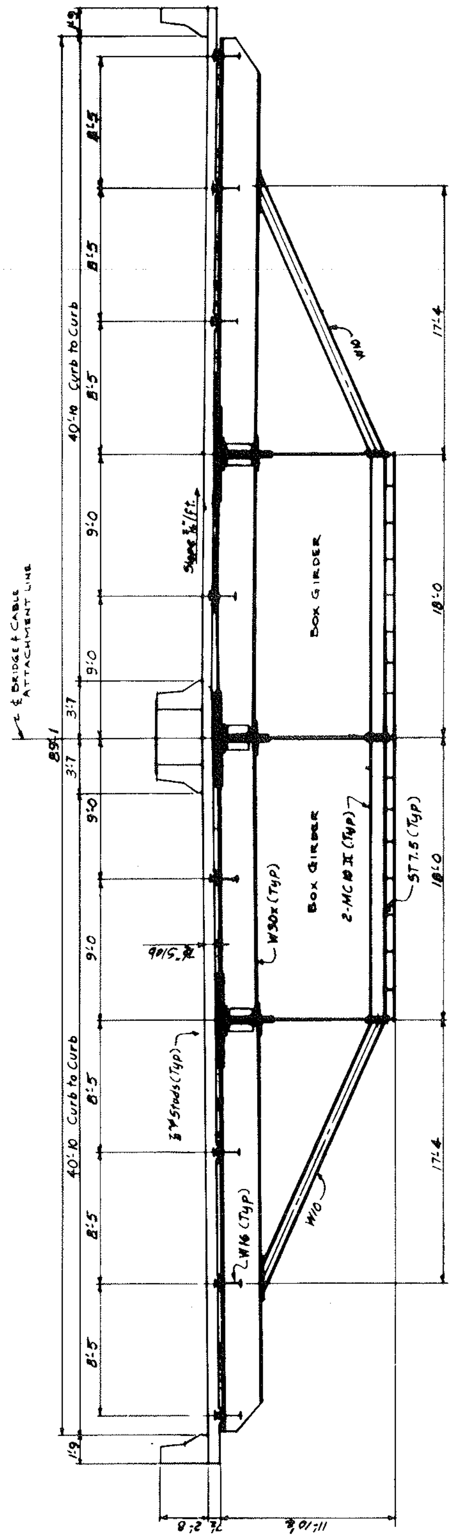

FIGURE 3 Redesign study: typical cross section, type 1.

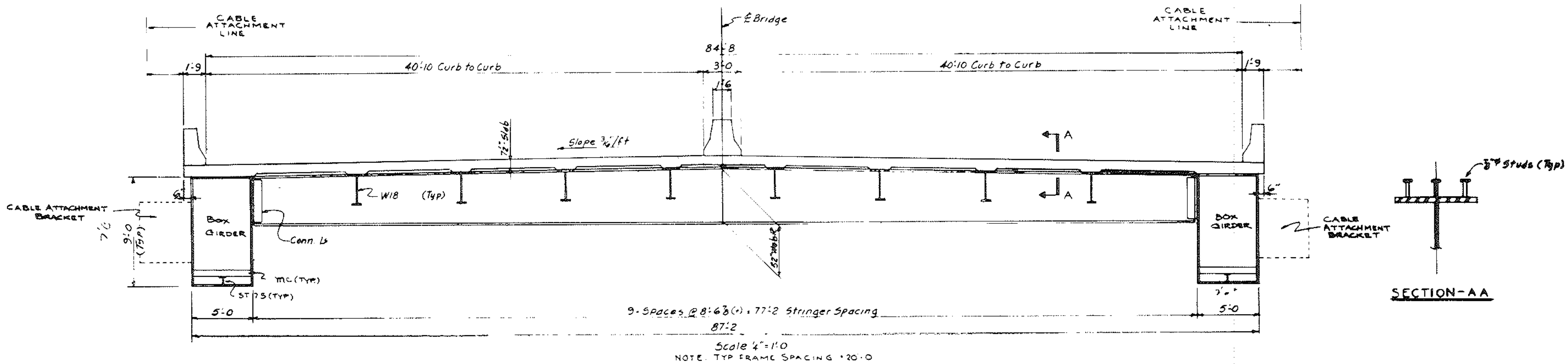


FIGURE 4 Redesign study: typical cross section, type 2.

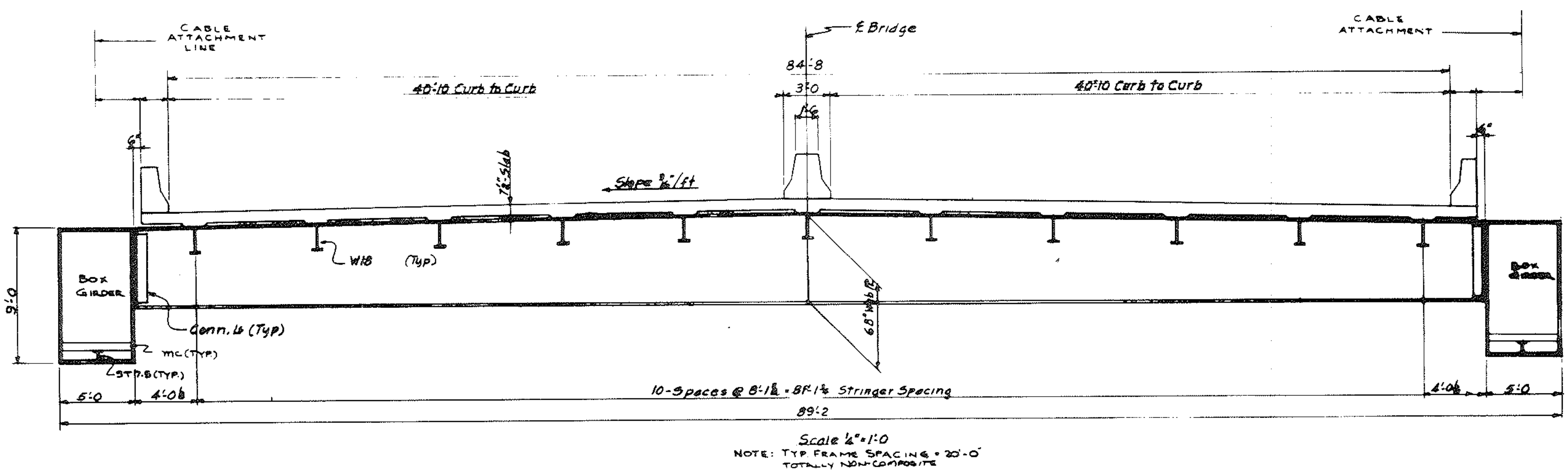


FIGURE 5 Redesign study: typical cross section, type 3.

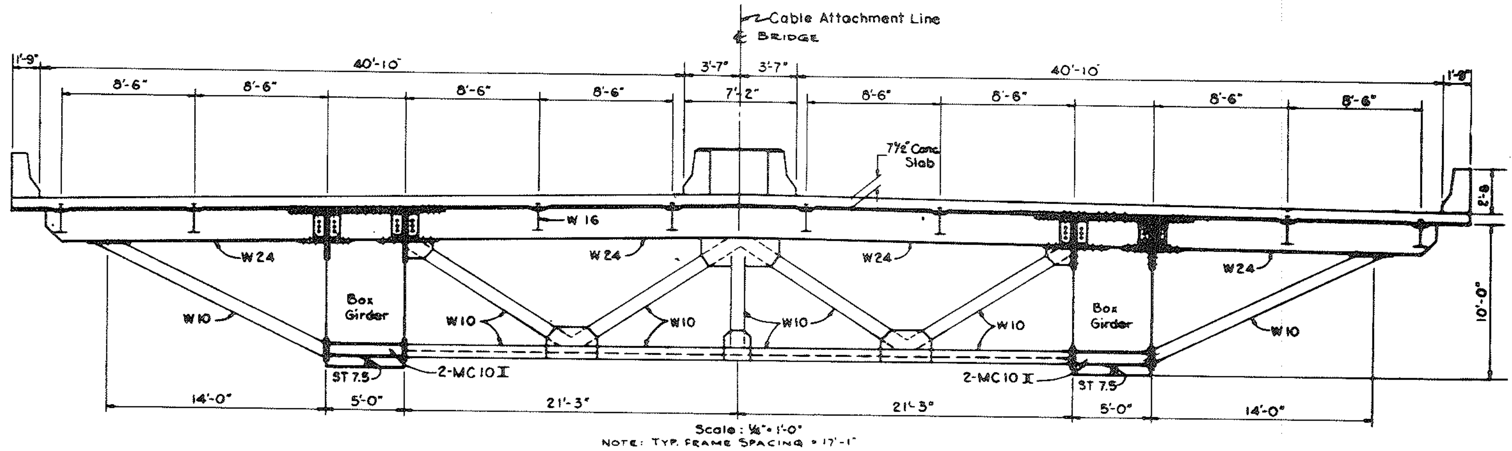


FIGURE 6 Redesign study: typical cross section, type 4.

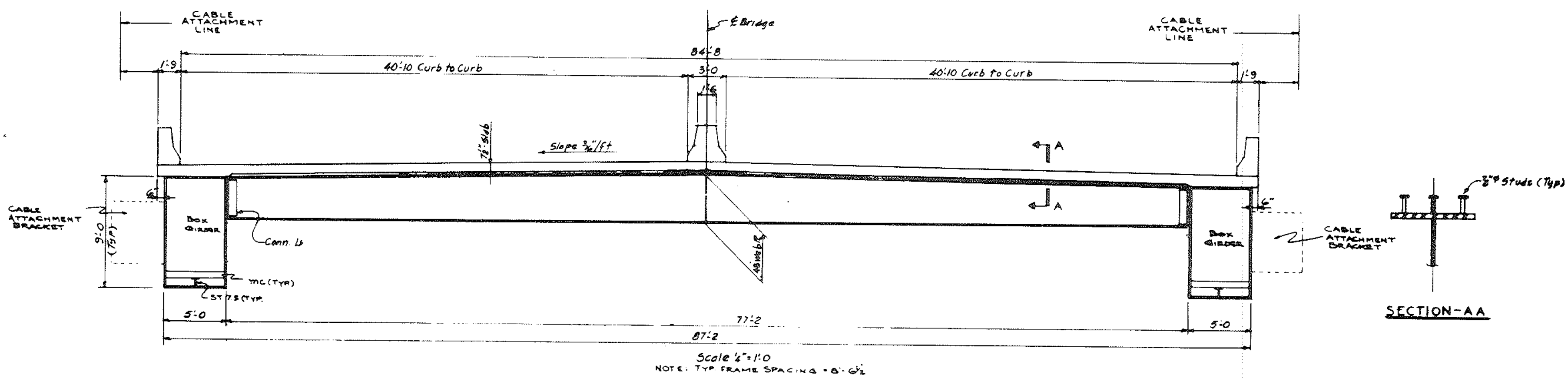


FIGURE 7 Redesign study: typical cross section, type 5.

TABLE 1 Redesign Study Type Comparisons

| | Type 1[a] | Type 2[b] | Type 3[c] | Type 4[d] | Type 5[e] |
|---|---|---|---|---|---|
| Steel quantities (cross-frame only) | 911#/lf bridge | 1047#/lf bridge | 2335#/lf bridge | 845/lf bridge (transverse diaphram not included) | 1843#/lf bridge |
| Concrete quantities | 2.64 cy/lf bridge | 2.5 cy/lf bridge | 2.5 cy/lf bridge | 2.64 cy/lf bridge | 2.5 cy/lf bridge |
| Weld quantities (1 = most, 5 = least) | 1 | 4 | 3 | 5 | 2 |
| Constructability | Fair to good | Good | Good to excellent | Fair | Excellent |
| Wind stability | Proven excellent | Good | Good | Good | Good |
| Live-load stability (torsional rigidity) | Good | Excellent | Excellent | Fair | Excellent |
| Adaptability to existing pier 4 | No modification required | Extensive modification required | Extensive modification required | Some modification required | Extensive modification required |
| Adaptability to precast deck units | Good | Good | NA | Fair | Excellent |
| Composite slab-girder efficiency | Good | Fair | NA | Good | Fair |
| Floor beam spacing adaptability | Poor aesthetically | Good aesthetically | Good aesthetically | Poor aesthetically | Good aesthetically |
| Slab removal for maintenance | | | | | |
| Degree of redundancy | Poor to good | Excellent | Excellent | Good | Excellent |
| Cable connection adaptability | Fair | Excellent | Good | Poor | Excellent |
| Adaptation to many cable connection points | Poor | Excellent | Good | Poor | Excellent |
| Fracture critical members | Poor | Good | Best; lower tension stress than 2 | Worst | Good |
| Adaptation for multigirder approach spans | Poor | Excellent | Good | Poor | Excellent |

Note: NA = not applicable.
[a]See Figure 3.
[b]See Figure 4.
[c]See Figure 5.
[d]See Figure 6.
[e]See Figure 7.

the most economical, stable configuration under the given conditions and with the chosen cross sections.

Supporting the cable system in this design is an inverted Y-shaped, reinforced-concrete tower that has a box section in the legs and an H-shaped section in the pylon. The approach spans were changed to a continuous composite steel, multigirder configuration. After sizing all the members and calculating the bridge quantities, $11 million was estimated as the savings in construction costs over the original steel design. Baker then recommended a total redesign of the bridge. Agreeing with Baker's recommendations, the West Virginia Department of Highways authorized the redesign of the bridge as shown in Figure 8.

## FINAL DESIGN

In the final design of the bridge three goals guided the research and design approach to devise a cost-effective bridge for bidding. The load-factor design method, as described in the AASHTO Standard Specifications for Highway Bridges (1), was used wherever practical as opposed to the service load design method as described in the same publication. In addition, different materials (i.e., concrete or steel) were used in the most effective manner. Finally, the most cost-effective cross sections were used in appropriate areas. These goals were to be obtained without sacrificing member and section safety or function.

## APPROACH SPANS

In the redesign the approach spans took on a new configuration. The superstructure changed from simple-supported, dual steel box girders over three spans to a 315-ft, three-span continuous, composite steel multigirder with a concrete deck (Figure 9). The greatest economy identified here was in steel fabrication costs. To accommodate this wider structure, the piers were changed from a straight concrete shaft to an open frame concrete bent, which resulted in lower concrete quantities.

## STAYED SPANS

To use the most economical superstructure configurations in applicable areas, a continuous, composite steel multigirder with a reinforced-concrete deck again was used in the non-cable-supported portion of the cable-stayed girder spans (Figure 10). This, then, transitioned into a cross section by using stringers, floor beams, and fascia girders with a composite concrete deck. The arrangements are shown in Figures 11 and 12. This configuration is most effective because it permits combining the major deck support elements with the cable reaction points.

In the final design stage the superstructure took on a slightly different configuration than that used in the redesign study. A significant item that changed was the fascia girder. In the redesign study a box girder was thought necessary for handling the torque induced into the fascia girders by the cable connections. After further investigations of state-of-the-art designs and extensive studies of shear centers and centers of gravity, a modified I-type fascia girder cross section was found to be more feasible and economical than the box girder because of reduced material quantities and fabrication costs. The web of the fascia girder was skewed at 10 degrees from the vertical, which closely approximates the shallowest cable inclination transverse to the bridge. This then enables the web to follow the cable from the critical clearance areas to their point of connection, thus drastically reducing the eccentricity in the cable connections (about 40 in. with the box section to 10 in. with the weldment). Reducing the eccentricity drastically minimized the structural requirements of the cable connection. In addition, the floor beam was connected at the cable work points to aid in the stability of the fascia girder. The fascia girder was optimized further by using stringers in conjunction with the concrete

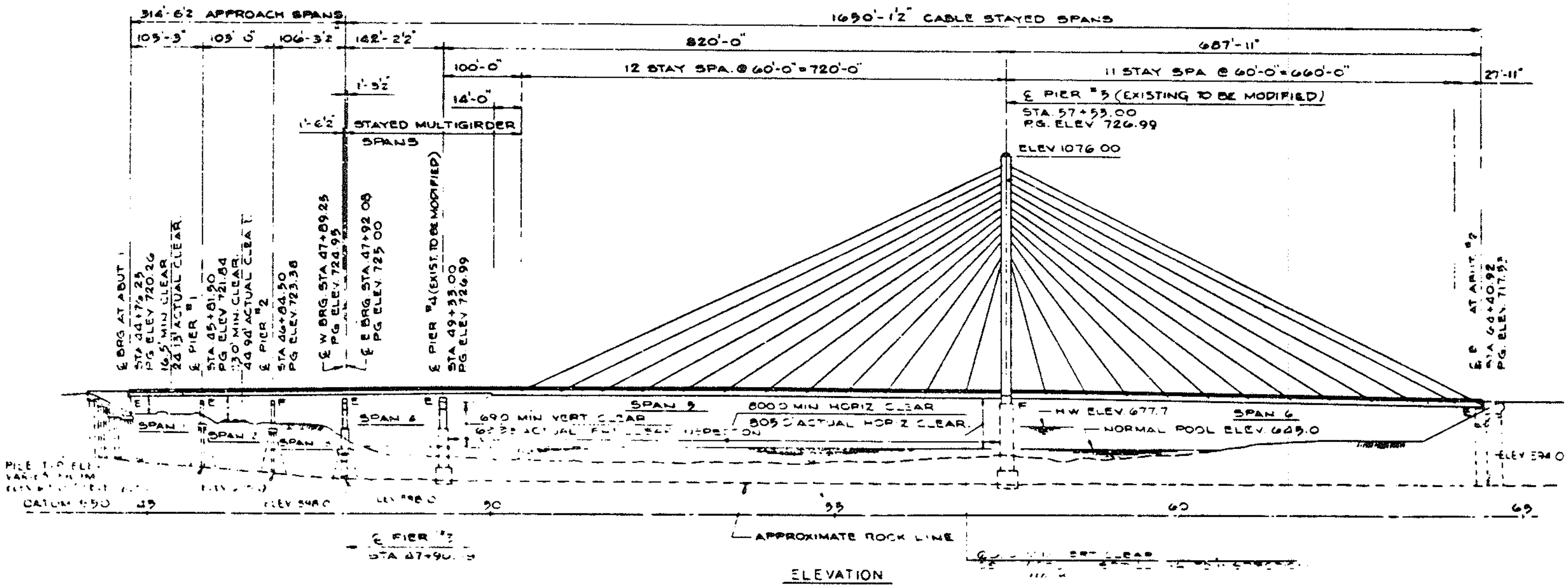


FIGURE 8 Redesigned bridge.

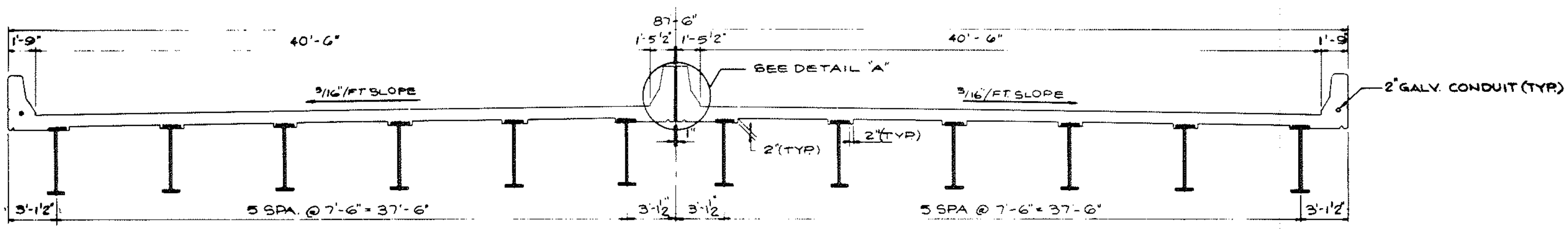


FIGURE 9 Approach spans cross section.

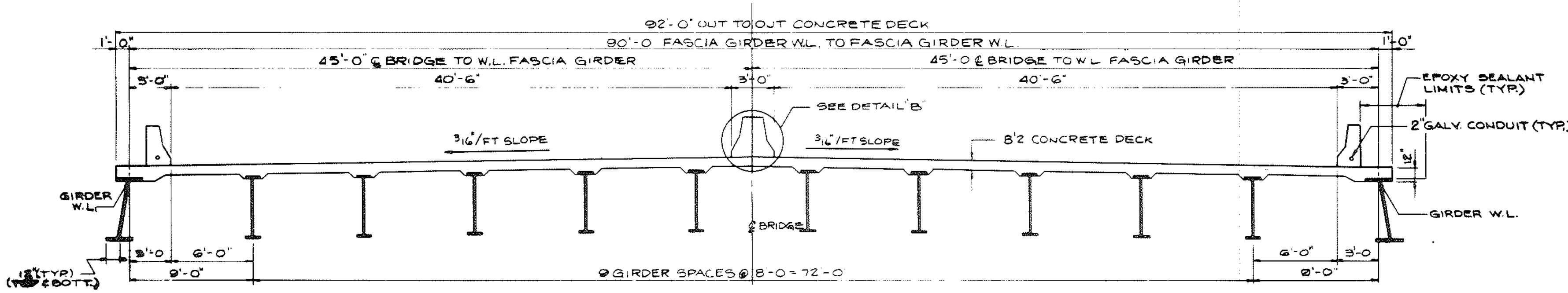


FIGURE 10 Stayed span multigirder cross section.

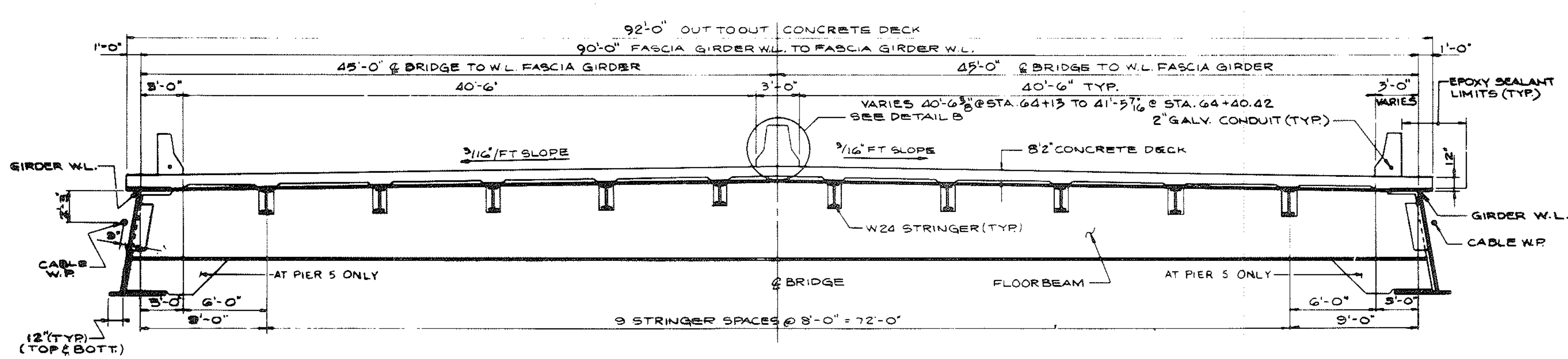


FIGURE 11 Cable-stayed girder span cross section at floor beam FB2-FB7 and FB9.

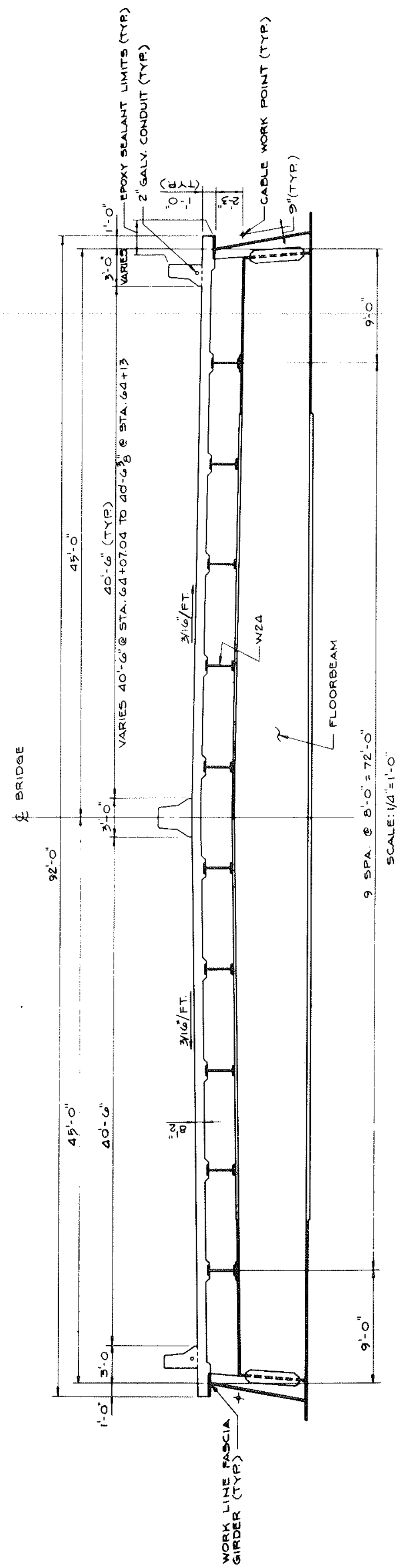


FIGURE 12 Cross section of cable-stayed bridge at floor beams FB1 and FB8.

deck to withstand the axial load induced by the cables into the superstructure. In doing this the deck with the wide flange stringers was analyzed as an orthotropic system. A finite-element computer analysis, which also checked the shear lag effect, proved this innovative approach to be reasonable.

The fascia girder also was optimized by requiring that (a) the construction of the superstructure be in 60-ft segments and (b) the fascia girder be shored until the cast-in-place concrete deck reached a specified strength. Therefore, the composite section supports the dead and live loads. As with all cable-stayed girder bridges, the dead-load moments were adjusted by selecting the proper cable tensions to optimize the superstructure cross section.

Horizontal trusses were then added at pier 5 (the tower pier) and at the last cable connections on the ends of the bridge. The end trusses ensured that the axial load induced by the end cable was distributed directly throughout the entire deck cross section, specifically in the stringers. The pier truss transferred the unbalanced stringer axial forces to the fascia girder bearings at pier 5. These trusses ensured that there was no shear lag effect at any of these locations.

These trusses also provide flexibility for bridge deck replacement while the bridge is still able to carry two lanes of traffic.

After a review by the American Institute of Steel Construction Bridge Committee of preliminary bridge design plans, the stringers were made continuous and the floor beams were lowered except at the truss locations. Doing so significantly cut the number and complexity of connections in the design and also lowered construction costs, despite an increase in steel weight.

## TOWER

In this design the tower configuration changed from a 225-ft-high steel A-shape to a 365-ft-high concrete inverted Y-shape, as shown in Figure 13. As anticipated, with reduced fabrication and because the tower is basically a compression member, concrete was the most economic material to use.

After researching the literature on cable-stayed girder design and calculating preliminary estimates, the most economic cable-to-horizontal angle in the longitudinal plane was determined to be between 25 and 65 degrees. Cable 1A was close to 25 degrees; however, cables near the tower had to exceed 65 degrees to maintain proper overhead clearances in the transverse direction, according to West Virginia requirements.

By using these design criteria, the tower height was raised to 365 ft above pier 5 and 430 ft above normal pool elevation. To accommodate the expanded number of cables and to obtain optimum tower height, an H-shaped, concrete, single-leg pylon was used to anchor all the cables. Supporting this pylon are two concrete box-shaped legs.

The H-shaped pylon cross section was selected for its simplicity in designing and constructing the cable connections. A concrete fascia panel was used to protect the cable connections from the weather and to conceal inspection platforms and access ladders.

The posttensioning was placed in the pylon's web to counteract the tension induced by the cables, thus simplifying design and construction.

The height of the apex of the inclined legs that were accommodated required transverse clearances between the tower leg and traffic and between the cable and traffic. This then gave the pylon a total height of 140 ft above the apex.

The dual, inclined tower legs, each 225 ft high,

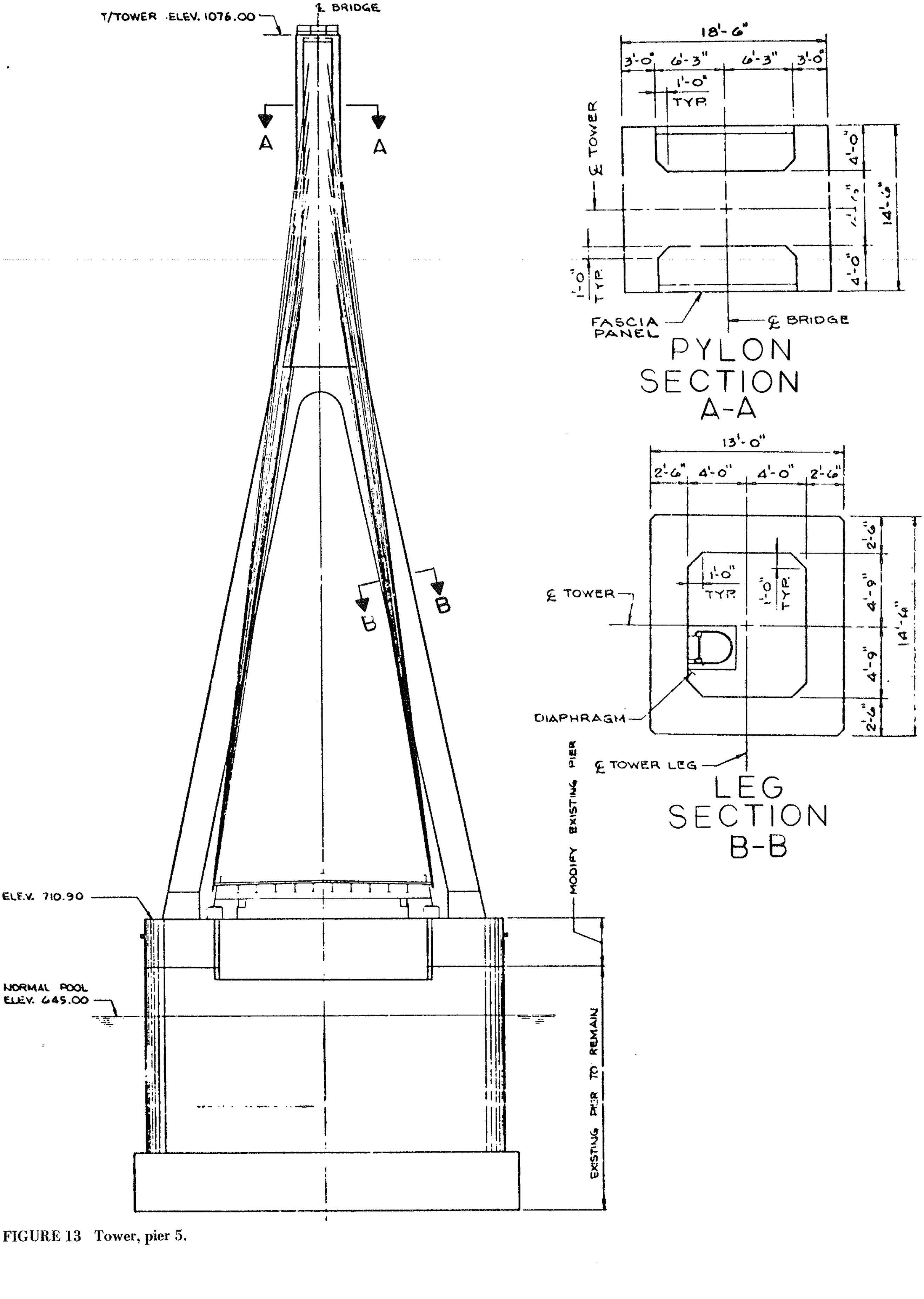


FIGURE 13 Tower, pier 5.

maintained minimum deck widths and fully used the conditions dictated by the already constructed pier 5. These legs will set on a concrete tower pedestal on a yet-to-be-constructed portion of pier 5. Dual access for cable connection inspections is provided by an elevator in one leg and a ladder in the other.

## SUBSTRUCTURE

To arrange the cables symmetrically in the first steel design, pier 6 was placed between pier 5 and abutment 2. With the last cable anchored into it, pier 6 served as a tieback for the entire structure. In the second steel alternate design, the increased number of cables made abutment 2 the symmetrical tieback location. As a result, pier 6 was eliminated in the second steel design, thus leaving four substructure units in the stayed spans.

These units had to be designed to accept a much wider bridge--92 ft as opposed to 36 ft for the bottom flange of the box girder--than the original steel alternate. Pier 4 was modified from a straight pier to a hammerhead, and pier 3 was designed as a hammerhead; for this reason, and because the piers are in the river's flood plain, pier 5 was changed slightly to accommodate embedment of the bearing pedestal reinforcing steel.

Eliminating pier 6 meant that abutment 2 would tie back the entire structure. Under certain load cases, the uplift on the abutment became substantial.

The tiedown is a large mass of concrete placed beneath abutment 2. This mass is posttensioned to a weldment that attaches to the tiedown bars from the last floor beam and is supported on piles. The maximum pile load occurs in the construction phase before tensioning the tieback cable. The majority of the load on the piles is relieved when the superstructure tieback cable is tensioned. The abutment back wall and wing walls are cantilevered on pile footings.

## DESIGN CONSIDERATIONS

As with all cable-stayed girder bridges, the dead-load moments in the stayed areas can be adjusted by changing the tension in the cable stays. This assisted in material optimization of the fascia girder. When the live-load moment envelope was obtained, the maximum negative moments at the interior hard points (piers 4 and 5) were large compared with the positive moment.

To reduce the effect of the large negative moments on the girder, the dead-load moment was made positive by adjusting cable tensions. The live-load moment envelope was investigated along the entire stayed span length, and the dead-load moment was then adjusted to optimize the superstructure sections.

Temperature and wind loads did not govern the deck stress design, but they did affect the tower design in some areas. Wind loadings were applied as recommended by A.G. Davenport of the University of Western Ontario in Canada, who also reviewed the wind stability studies and confirmed the stability of the bridge. Differential temperatures were applied between the concrete tower, steel cables, and the mixed concrete and steel deck. The loads that were induced affected the tower design only.

The method by which this bridge will be erected has an effect on the final stresses in the deck. Several erection schemes were investigated to optimize material use.

The first erection scheme specified that the bridge be built on falsework from abutment 2 to pier 5. It then used cantilever construction to erect the main span, tensioning the forestay and the backstay as the main span progressed. This scheme was abandoned in favor of balanced-cantilever construction because of the high cost of the falsework. The initial balanced-cantilever construction contemplated that all the steel be erected in the superstructure, and then the concrete deck poured. The vast majority of the dead-load stresses went into the steel by using this scheme. Because this type of construction required a great deal of steel, a panelized, shored-erection scheme working away from pier 5 in a balanced-cantilever manner was chosen. This scheme optimizes the superstructure cross section without penalizing the erection process significantly.

The tower, multigirder cross sections, and all substructure units were economized further by using the load-factor design method in these areas. The cable-supported areas of the bridge were designed by using the service load design method because the AASHTO code criteria do not address composite beam column design and design schedule constraints precluded developing such criteria.

## CONCLUDING REMARKS

On September 9, 1983, construction bids were opened for the three alternate bridges. The lowest bid was made on the second steel alternate design, and the construction contract was awarded to S.J. Groves and Sons Company for its $20 million bid.

There were nine bids on the second steel alternate, one at $32 million on the first steel alternate, and none on the concrete alternate. Several contractors who bid the second steel alternate said that they priced the concrete alternate and estimated it to be about $4 million more than the second steel alternate.

All three alternates were satisfactory designs, with economic conditions and preexisting constraints creating advantages and disadvantages for all three. However, common practice states that concrete exerts its best qualities in compression and that steel exerts its best qualities in tension and bending. Exploiting the materials' best qualities and reducing the quantity and complexity of steel fabrication appeared to allow the second steel alternate design to gain final advantage over the other alternatives.

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*Publication of this paper sponsored by Committee on Steel Bridges.*