Design of the Cable-Stayed Mississippi River Bridge at Quincy, Illinois

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ABSTRACT

Two alternative two-lane, cable-stayed bridges have been designed to carry US-24 over the Mississippi River at Quincy, Illinois. The design and detailing information on both alternatives are presented. Both bridges include main spans of 900 ft and both use concrete towers and a concrete deck. The concrete alternative uses 6,000-psi concrete precast segments that consist of edge beams, floor beams, stringers, and a deck slab. The steel alternative uses welded I-shaped edge girders, steel stringers, and floor beams, and a deck composed of precast, full-width, full-thickness slab panels that are posttensioned longitudinally in erection lengths (comparable to the cable spacing) and are then made composite with the stringers and main girders.

The new westbound US-24 Mississippi River bridge at Quincy, Illinois, has been designed for two alternative methods of construction. In the first alternative the deck system is entirely of concrete. In the other alternative the deck system consists of steel members with a concrete deck. Inasmuch as both alternative cable-stayed bridges have been designed by one consultant, the same philosophy of design and detailing has been applied to both alternatives. This should result in two genuinely equal alternatives in terms of the designers' attention to both the immediate and long-term costs to the owner.

The entire project consists of several thousand feet of approach structure, some ground and approach work, and a cable-stayed main bridge. The prime consultant is Booker Associates, Inc., which designed the approaches to the cable-stayed bridge, including the flanking transition spans. Modjeski and Masters designed the cable-stayed bridges. The owner is the Illinois Department of Transportation. The discussion in this paper is limited to a description of the cable-stayed bridge.

The general elevation of the concrete alternative bridge is shown in Figure 1. In both alternatives the general span arrangement consists of (from left to right) a transition span of 200 ft, a 440-ft side span of the cable-stayed unit, the main span of the cable-stayed unit at 900 ft, another side span at 440 ft, and a transition span of 200 ft. The span arrangement was preset by a separate engineering contract for alignment studies. The two-lane bridge is relatively narrow, being only 32 ft, 0 in. curb to curb. In both alternatives the tower is of concrete. In the concrete alternative the deck system, which consists of edge beams, floor beams, stringers, and a deck slab, is composed entirely of concrete. There are 96 cables on the concrete alternative structure.

The steel alternative is shown in Figure 2. This alternative contains 56 cables. The deck system consists of longitudinal welded steel girders, steel floor beams, and steel stringers. The longitudinal members are composite with the concrete deck after

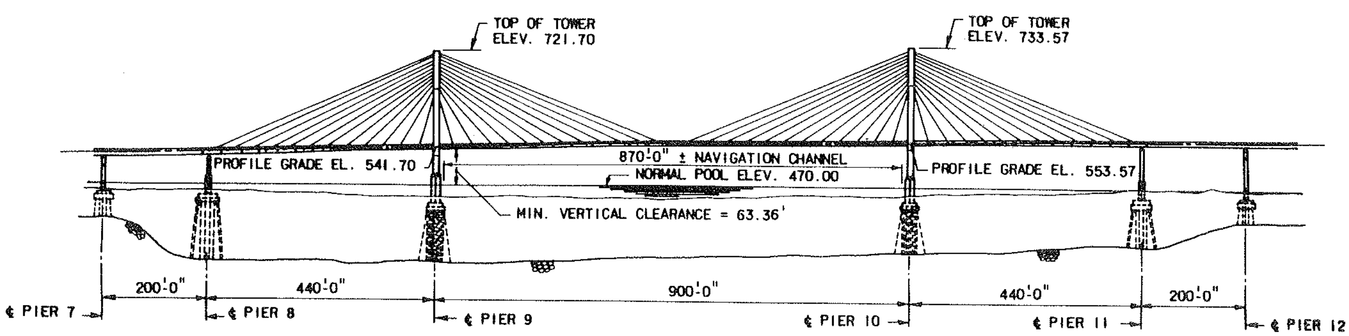

FIGURE 1 Elevation view of concrete alternative.

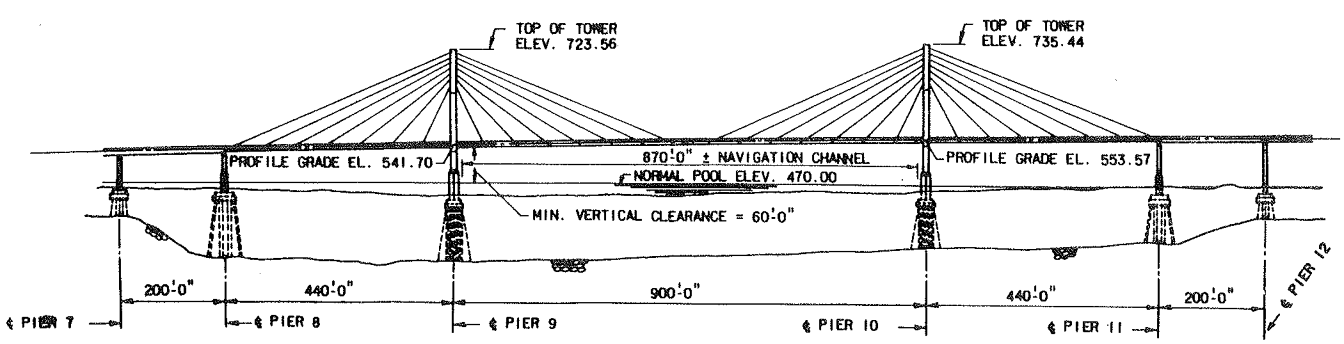

FIGURE 2 Elevation view of steel alternative.

posttensioning of the slab panels. The contract drawings contain the complete details for both alternatives. A third alternative is also being permitted wherein the main span may be of concrete construction and the transition spans may be of steel construction.

BASIC DESIGN PARAMETERS

Basic loads for both alternatives are given in Table 1, which is excerpted from the general note sheet of the contract documents. The basic design procedure used influence lines for all structural components. This imples a first-order analysis. The second-order effects were estimated from a limited number of iterative second-order analyses and were found to be relatively small. A percentage adjustment was made to in-plane and out-of-plane tower moments. The loading patterns that produced the largest second-order effects in the deck system, which were found to be quite small in absolute terms, were not the same as the loading patterns that produced the maximum first-order effect. Because these two effects offset each other, no adjustment in the deck moments was made. It is important to note that this conclusion is structure dependent and should not be generalized without further study.

The dead load of each of the bridges was balanced such that there was essentially no dead-load bending in the tower and only local (i.e., between-cable) bending in the deck structure. This was accomplished by selective cable adjustment and local ballasting. In order to allow for some tolerance in the actual as-constructed balance, an unbalanced dead load was applied, as explained in Table 1.

Dead-load moment and thrust curves, and live-load moment and thrust envelopes for the concrete and steel alternatives are shown in Figures 3 and 4, respectively. Both alternatives have been designed so that the concrete decks can be removed and replaced if necessary, and also so that any cable can be replaced without auxiliary support in the river.

Dynamic analyses and wind tunnel tests were performed for both alternative structures. The wind tunnel tests were performed at the low-speed aerodynamic test facility of the National Research Council, Ottawa, Ontario, Canada, under the direction of R.L. Wardlaw. Some alteration of both cross sections was required, as explained herein. A summary of the pertinent results is given in Table 2.

CONCRETE TOWERS

A front elevation, side elevation, and several sections through the tower are shown in Figure 5, as are the principal dimensions.

TABLE 1 Design Loads

DESIGN

EXCEPT AS NOTED HEREIN, THE BRIDGE IS DESIGNED IN ACCORDANCE WITH THE AASHTO STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, 12TH EDITION, DATED 1977 AND, THE AASHTO INTERIM SPECIFICATIONS THROUGH 1982.

LOAD FACTOR DESIGN METHODS HAVE BEEN USED TO PROPORTION THE TOWER, EDGEBEAMS, STRINGERS, BEARING PLATES, TRANSITION SPANS, AND THE CONCRETE ALTERNATE (CABLE-SUPPORTED SPAN) DECK SLAB. THE TRANSITION SPANS HAVE BEEN DESIGNED FOR AASHTO LOADING. THE FOLLOWING FACTORED LOADS WERE USED IN THE CABLE-SUPPORTED SPANS:

GROUP I:	1.4 (BD) + 2.17 (L + I)
GROUP II:	1.4 (BD + W)
GROUP III:	1.4 (BD + L + I + 0.3W + WL + LF)
GROUP IV:	1.4 (BD + L + I + S + T + R)
GROUP V:	1.35 (BD + W + S + T + R)
GROUP VI:	1.35 (BD + L + I + 0.3W + WL + LF + S + T + R)
	B = 1.0, B = 0.875

NOMENCLATURE IS THE SAME AS AASHTO.

SERVICE LOAD DESIGN METHODS HAVE BEEN USED TO PROPORTION THE CABLES, AND FLOORBEAMS IN CABLE-SUPPORTED STRUCTURE (CONCRETE ALTERNATIVE) AND HAVE BEEN USED TO DESIGN THE DECK SLAB AND CABLES IN THE CABLE-SUPPORTED STRUCTURE (STEEL ALTERNATIVE).

EDGEBEAMS AND STRINGERS ON THE CABLE-SUPPORTED STRUCTURE (CONCRETE ALTERNATIVE) HAVE BEEN DESIGNED TO HAVE NO TENSION UNDER GROUP I SERVICE LOADS.

THE DEAD LOAD ON THE CABLE-SUPPORTED SPANS CONSISTS OF THE GEOMETRIC DEAD LOAD, PLUS AN UNBALANCED DEAD LOAD EQUAL TO 3% OF THE DEAD LOAD TREATED IN THE SAME MANNER AS A MOVING LIVE LOAD, I. E., THE PORTION OF THE STRUCTURE CONTRIBUTING TO A GIVEN MAXIMUM FORCE OR MOMENT WILL BE CONSIDERED TO BE 3% OVERWEIGHT.

SEISMIC LOADS

AASHTO, ZONE 1

TEMPERATURE (CABLE-SUPPORTED SPANS)

ASSUMED ERECTION TEMPERATURE	50° F
ASSUMED MEAN TEMPERATURE	50° F
TEMPERATURE DIFFERENTIAL - CABLE TO CONCRETE	40° F
TEMPERATURE DIFFERENTIAL - CABLE TO STEEL	30° F
TEMPERATURE RANGE FOR EXPANSION DAMS	± 80° F
TEMPERATURE GRADIENT THROUGH DECK STRUCTURE	20° F
TEMPERATURE GRADIENT THROUGH TOWER	15° F
ALL DIMENSIONS GIVEN AT	50° F

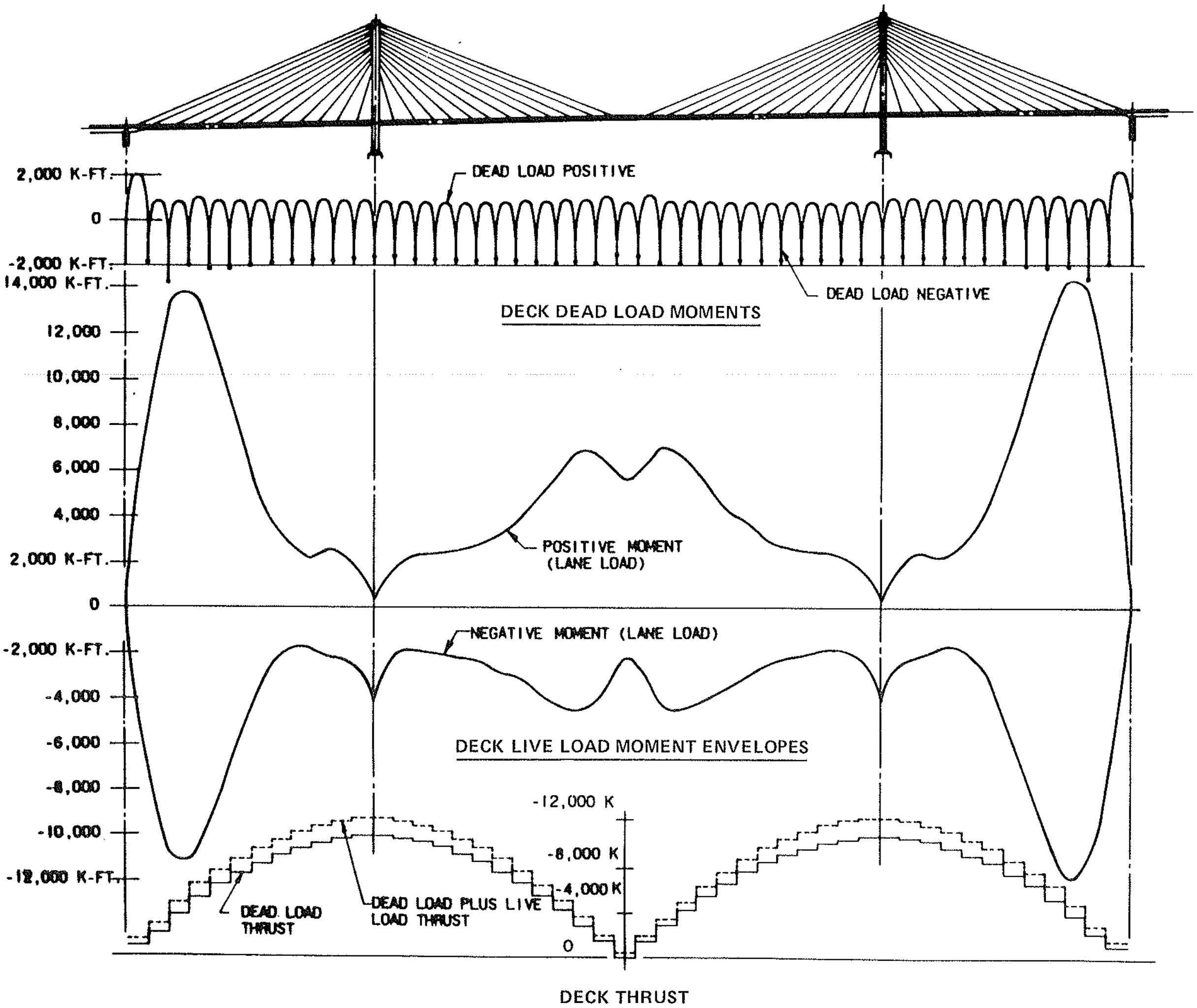

FIGURE 3 Moment and thrust envelopes for concrete alternative.

The bow-legged arrangement was chosen in order to simplify the structural arrangement of the cables. On this bridge the cables all lie in a vertical plane. This greatly simplifies the detailing of the anchorage of the cables in the tower and in the deck section, in that the geometry at each cable anchorage point can be defined by a single angle, which the cable makes with the horizontal plane. The cables all frame into the pylon, and because the lower cable connections are located in a plane vertically beneath the cable connections in the pylon and the deck passes between the tower legs, it is necessary to sweep the legs outward to provide clearance for the deck section. Below the deck at the level of the bottom strut, the tower legs are turned inward to minimize the width of foundation required. The concrete in the tower is 5,000-psi cast-in-place concrete.

Typical reinforcing details in the pylons are shown in Figure 6. The location of the cables is shown in each of the three views, which illustrate the problems in detailing the reinforcing to clear the cable anchor pipes. The principal vertical reinforcing is mechanically coupled for continuity and is distributed around the outer face of the column and the inner face of the void. The horizontal reinforcing consists of stirrups and other reinforcement distributed around the exterior face of the column and the inner face of the void.

A partial elevation view and section that shows the distribution of posttensioning in the tower pylon is shown in Figure 7. As can be seen in section A-A, the cables are designed to be socketed on the inside tower walls in a manner that applies tension to the long walls. It was possible to use this arrangement because the relatively narrow bridge resulted in cable loads that were low enough that posttensioning to resist wall tension was practical. The cable sockets could, therefore, be relatively well protected by being within the tower leg voids.

The cable locations and bearing corbels are also shown in Figure 7. The detailed location of the posttensioning shows that the posttensioning must clear the cable anchor pipes. The tower pylon is posttensioned vertically and in both horizontal directions, thus providing a state of triaxial posttensioning. The vertical posttensioning is heaviest in the upper portion of the pylon; however, a reduced amount continues down to the bottom of the upper strut. The horizontal bars are the 14-ft, 6-in. posttensioning bars, whose primary function is to carry the horizontal pull from opposing cable pairs across the tower pylon. The posttensioning shown as dots are the 7-ft-long posttensioning bars, whose function is to control the bending that results from the cable bearing on the corbel that spans between the tower walls, and also to provide adequate shear friction for transfer of the cable load into the 14-ft, 6-in. posttensioning bars. The overall system of posttensioning is designed to keep the concrete in compression during all stages of erection and during the life of the structure; this will minimize crack formation in the pylon.

The pylon section has been subjected to a finite-element analysis, whereby it was found that the concrete is in compression at all service load stages during the life of the bridge, except for some minor tension from Poisson effects. It is important that these 7-ft posttensioning bars maintain the pylon concrete in compression. These bars are so short that a small amount of seating loss could seriously affect posttensioning. Additional requirements to assure that these bars will provide compression for the life of the bridge are included in the special provisions, which require restressing all of the horizontal bars in the tower pylon. The restressing shall take place no less than 30 days after the initial prestressing of each bar. The restressing will consist of two repetitions of jacking each bar to the desired tension and seating the anchor nuts on the anchorage. A test block was used to stress and restress a bar of 7-ft length as required on the contract drawings; based on that experience it is believed that the restressing operation as described will give satisfactory long-term performance of these short bars.

A cross section of a bottom tower strut is shown in Figure 8. The longitudinal and transverse reinforcing are shown as solid lines and small dots, and the longitudinal posttensioning is also shown. The longitudinal posttensioning extends out-to-out of the tower legs. The struts on these towers are structurally significant members that stabilize the tower legs and carry torsion, moment, shear, and thrust.

TABLE 2 Aerodynamic Characteristics

	Concrete Alternative	Steel Alternative
First bending frequency (Hz)	0.275	0.462
First torsional frequency (Hz)	0.631	0.706
Flutter		
Damping (% critical)	0.3	0.5
Speed (mph)	120	>130
Vortex: vertical		
Damping (% critical)	0.3	0.5
Speed (mph)	16	25
Displacement (in.)	0.5	0.5
Acceleration (% g)	0.4	1.1
Wind angle (degree)	0	0
Vortex: torsional	_[a]	_[a]

[a]None.

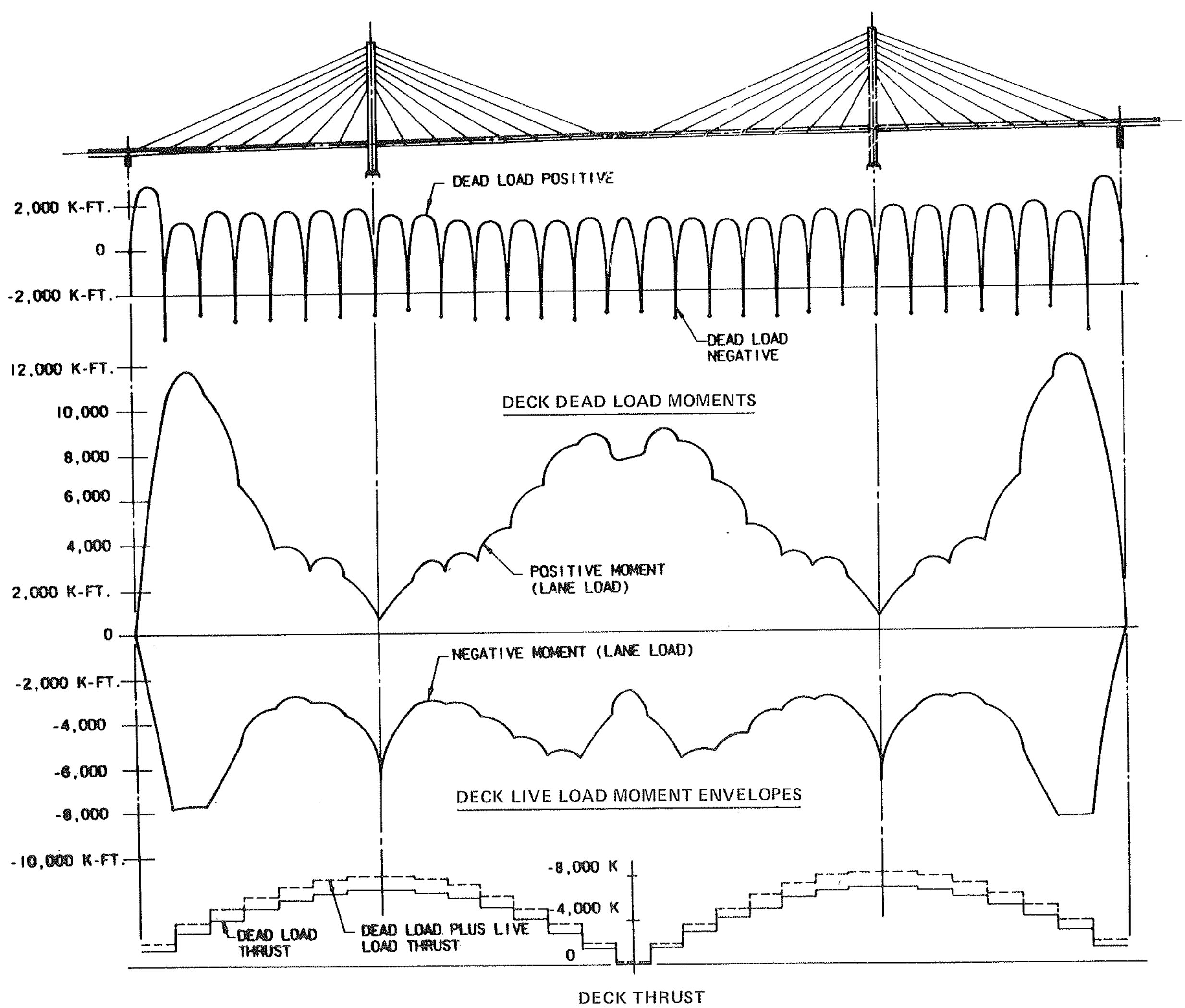

FIGURE 4 Moment and thrust envelopes for steel alternative.

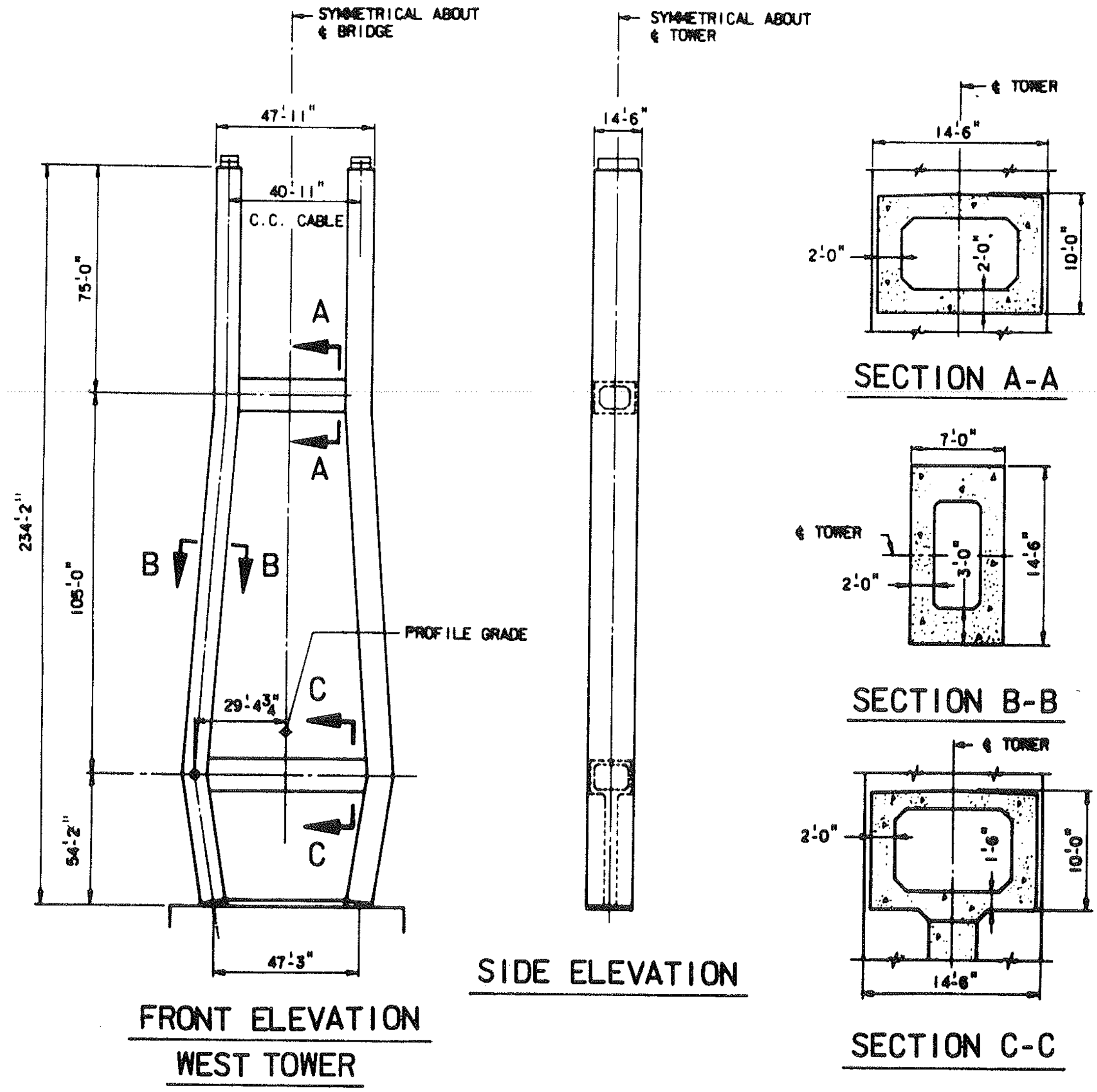

FIGURE 5 Concrete tower: elevations and sections.

CABLE SYSTEM

The cable system proposed for the Quincy bridge shown on the contract drawings consists of cables made of bundles of parallel wires of 0.25-in.-diameter, ASTM A421, grade BA that have 240-ksi tensile strength. In the socket the wires splay out and are buttonheaded and anchored by a locking plate that is attached to the socket. The splayed area in the socket is filled with a special epoxy casting material that further anchors the wires within the socket. Emerging from the socket, the wires are wrapped with a spirally wound wire strand and protected by a polyethylene pipe. In addition, near the socket a short length of steel sleeve is attached to the socket and surrounds the polyethylene pipe. These details are shown in Figure 9. The number of wires per cable in the two alternative structures varies from 81 to 283 wires per cable. The stress in each cable has been limited to 103 ksi. In some cases the cable design is based on a temporary design condition in which one cable is assumed to be removed and the adjacent cables must carry not only their load but also part of the load that would normally be carried by the cable that is removed.

As a matter of interest, this design calls for about 943 miles of wire for the concrete alternative and about 630 miles of wire for the steel alternative.

Tower attachment details for the proposed cable are shown in Figure 10. This is the dead end of the cable. It bears on a concrete corbel that is an integral part of the tower wall. The cable anchor pipe is welded onto the bearing plate, and the entire assembly is cast into the tower pylon. The cable and socket are threaded upward through the cable anchor pipe and anchored with split washers. An elastomeric washer is positioned at the lower end of the cable anchor pipe, along with a protective neoprene boot that is clamped to the polyethylene tube and to the cable anchor pipe.

The cable hardware at the deck connection is shown in Figure 11. First, there is a bearing plate that has an opening large enough to pass the socket. Next is the split bearing washer followed by the adjusting split shims. On top of that is a socket alignment shim, and finally, the cable socket. Nominal shim thickness of 6 in. is anticipated. The special provisions require that if the thickness of shims needed to adjust the cable to its required tension exceeds 12 in., the engineer must be con-

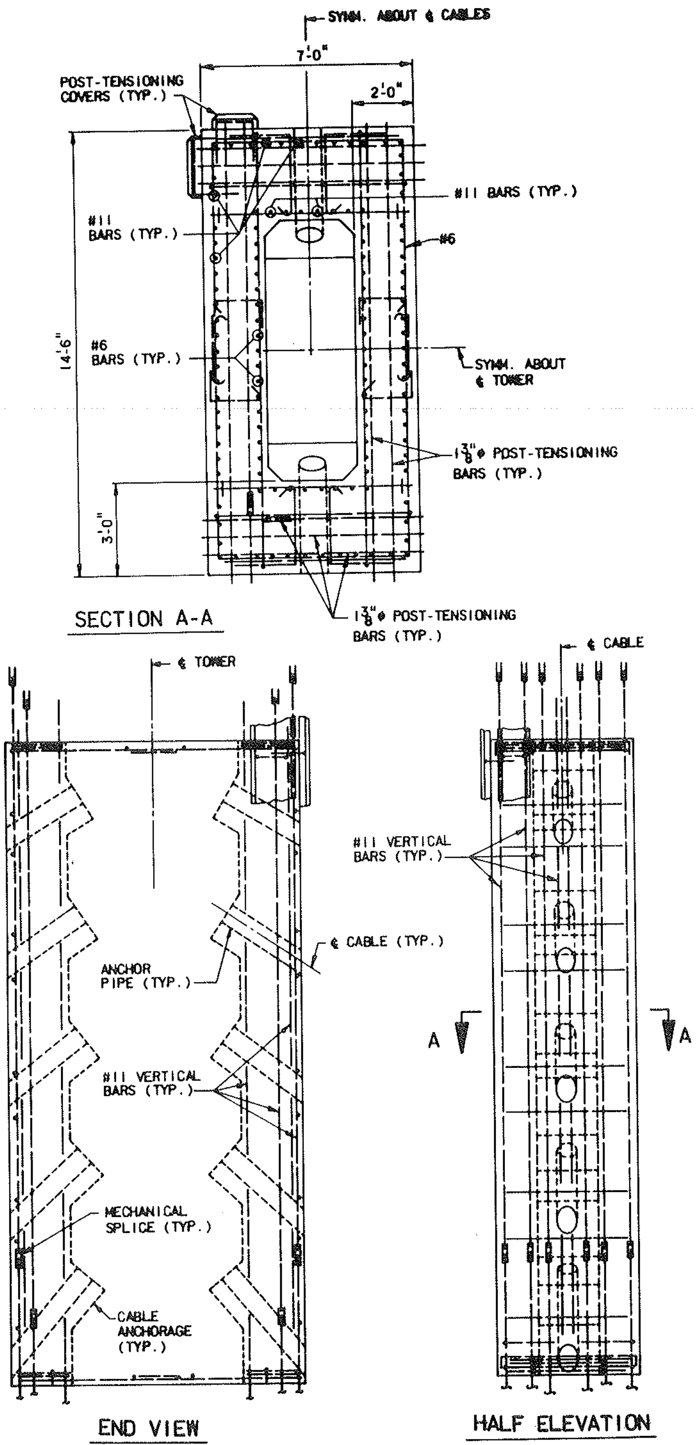

FIGURE 6 Concrete tower pylon: reinforcing details.

sulted. More than 12 in. of shims would be outside of the anticipated normal range; thus it would then be necessary to consider the reason for the discrepancy.

The special provisions provide for an alternative cable system that uses a 0.6-in.-diameter, low relaxation ASTM A416, grade 270, seven-wire strand in a parallel configuration. Such a system may be proposed by the contractor as summarized herein. The parallel wire cables, as shown on the contract drawings, have previously demonstrated acceptable fatigue characteristics. It is mandatory that any alternative cable system proposed shall demonstrate equivalent fatigue resistance. The total number of stays and the location of the cable work points may not be changed. If a socket with an external thread is proposed, the special provisions contain requirements for the amount of thread required. Any alternative system proposed shall provide cables of essentially the same stiffness and weight as those shown on the contract drawings; that is, the product of cable area times the effective modulus elasticity shall be the same as those shown on the plans. Note that some of the cables on the contract drawings

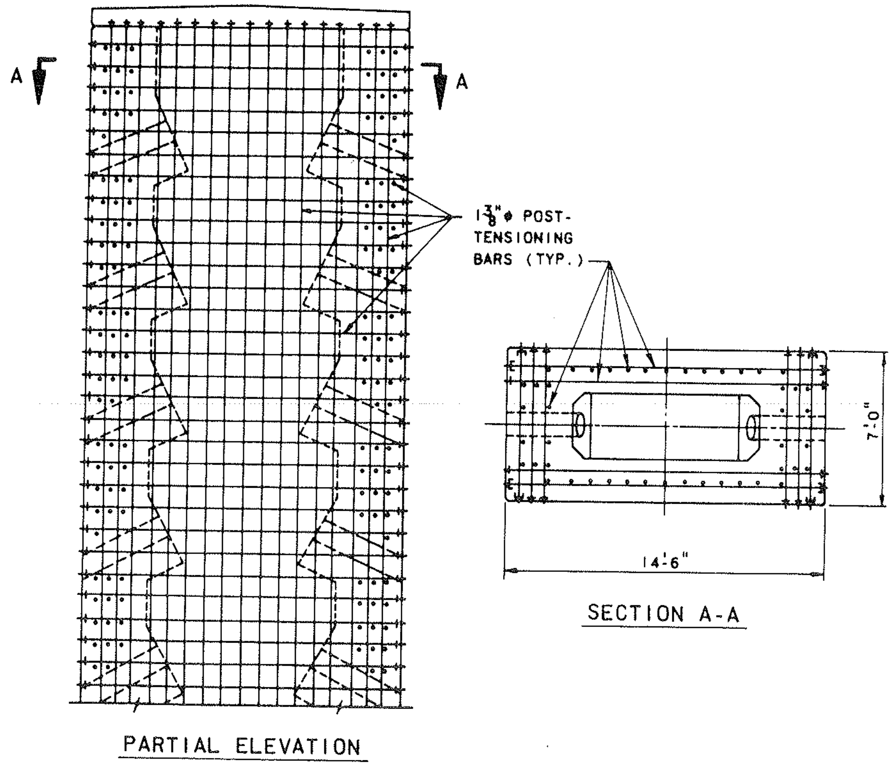

FIGURE 7 Concrete tower: posttensioning arrangement.

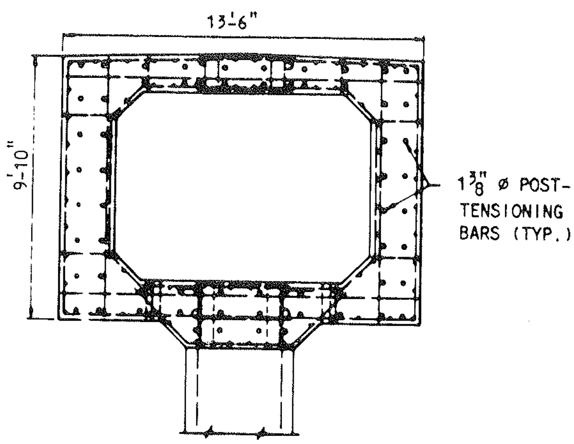

FIGURE 8 Concrete tower: section through lower strut.

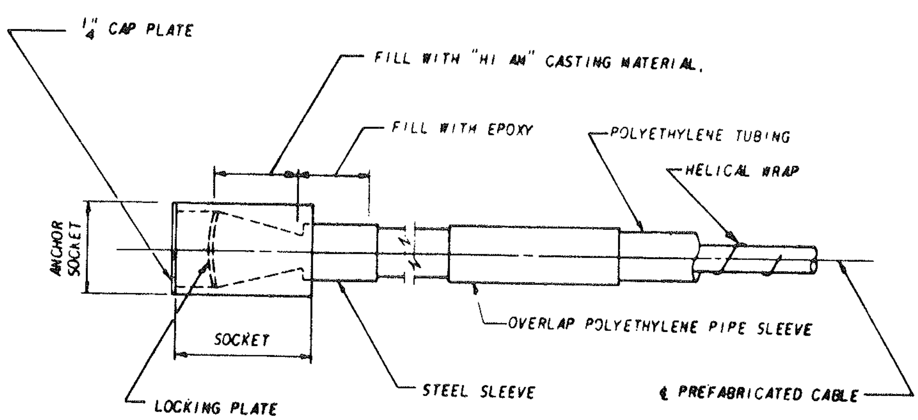

FIGURE 9 Cables: schematic of parallel wire cable.

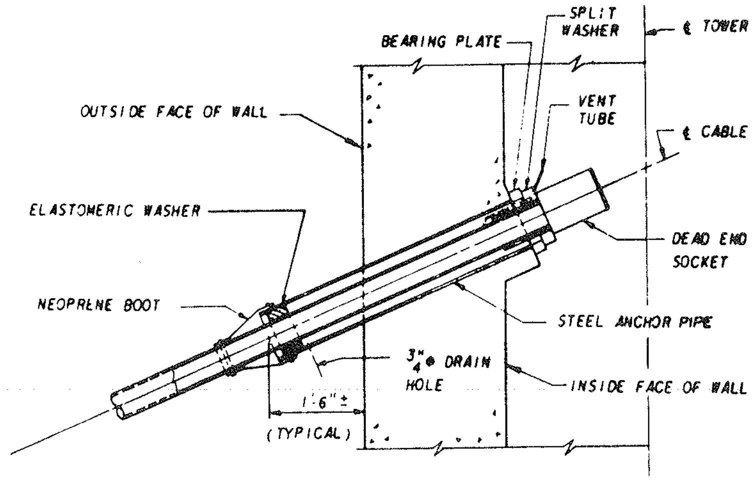

FIGURE 10 Cables: details of tower attachment.

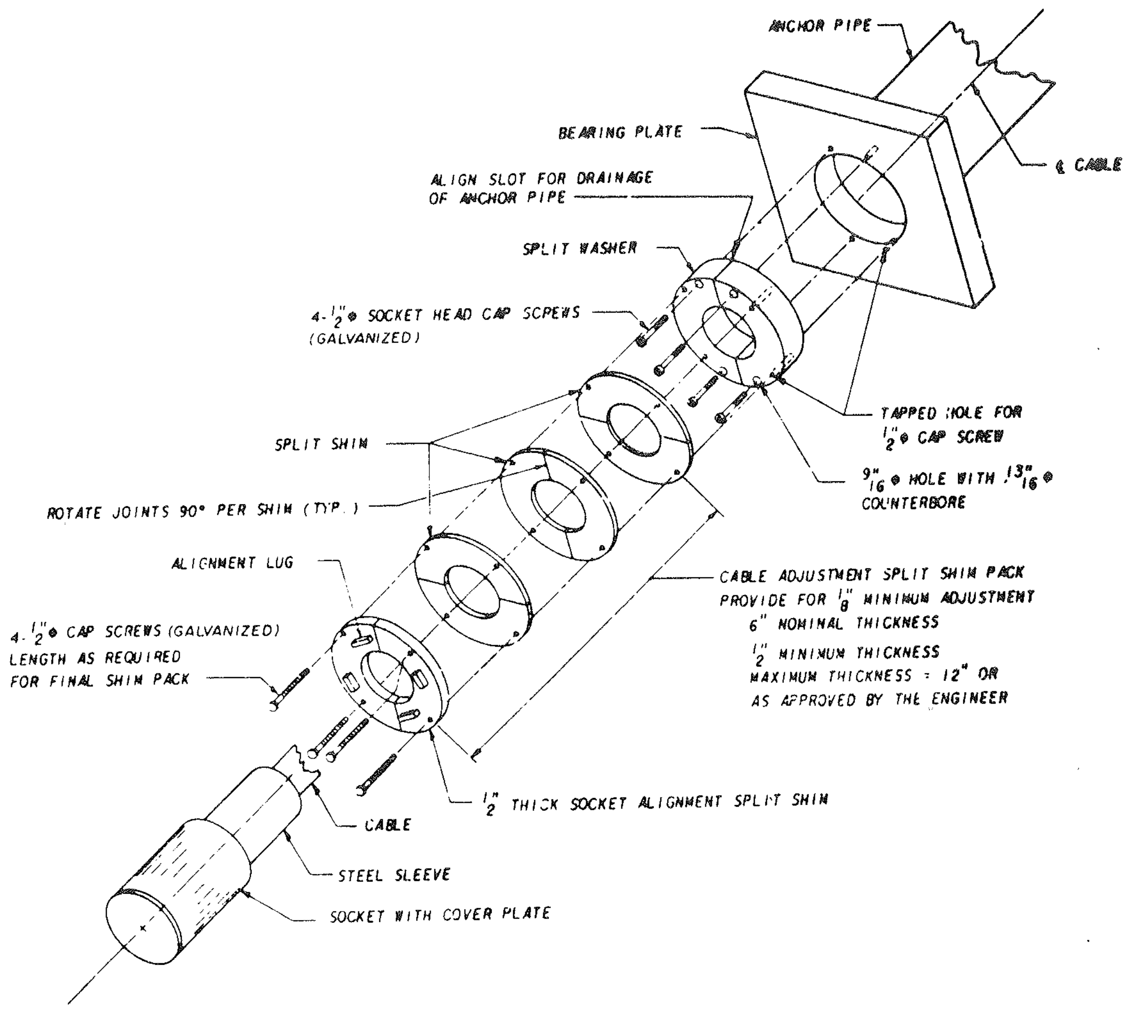

FIGURE 11 Cables: details of edge beam attachment.

have been oversized, either to provide additional stiffness to the structure or to provide for cable removal, and these requirements will also apply to any alternative cables proposed. It is desirable that an alternative cable system use anchor pipes no larger than those presently detailed. If the size of the anchor pipe were to be changed to accommodate a proposed alternative cable, many of the internal details within the edge beam and tower would have to be modified to a larger anchor pipe size.

Typical corbel details in the tower pylon are shown in Figure 12. The anchor pipe, bearing plate

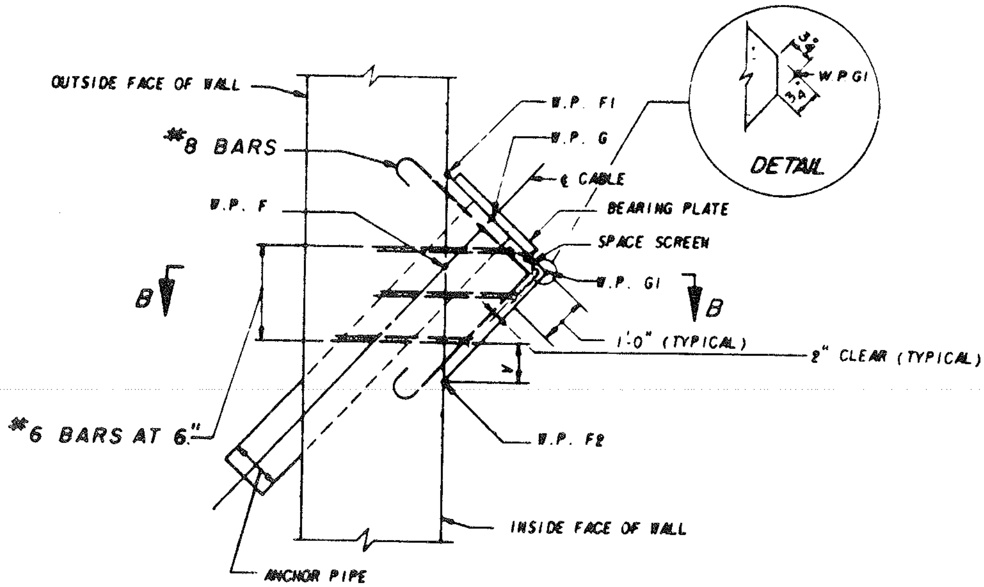

FIGURE 12 Cables: reinforcing at tower corbel.

assembly, and the location of the work points that completely describe the geometry of each corbel are shown. The extra reinforcing required at each corbel, and all other required reinforcing, is completely detailed on the contract drawings.

CONCRETE ALTERNATIVE DECK SYSTEM

A general elevation of the concrete alternative bridge is shown in Figure 1. This bridge is of segmental concrete construction. Beginning at each tower, the deck is cast-in-place. Then, proceeding in each direction from the tower, the deck is composed of precast segments. The closing sections at the centerline of the bridge and at the anchor piers are cast-in-place. The cast-in-place sections use 5,000-psi concrete, and the precast segments have a minimum strength of 6,000 psi. In each side span the cables are spaced at 36 ft, 8 in.; main span cable spacing is 36 ft, 0 in.; and generally the segments are the same length as the cable spacing. There are 12 cables on each side of each pylon for a total of 96 cables on the bridge.

A cross section of the deck section is shown in Figure 13. Cables are in a vertical plane, and on each side the edge beam is centered directly under the cables. The floor beams span transversely between the edge beams and carry the stringers and the deck. In this figure edge beams of different sizes are shown in the left and right half sections because the edge beam changes dimensions along the bridge. In the side span there are two major changes in edge beam dimensions. In the first major transition the edge beam is a constant 3 ft, 9 in. wide

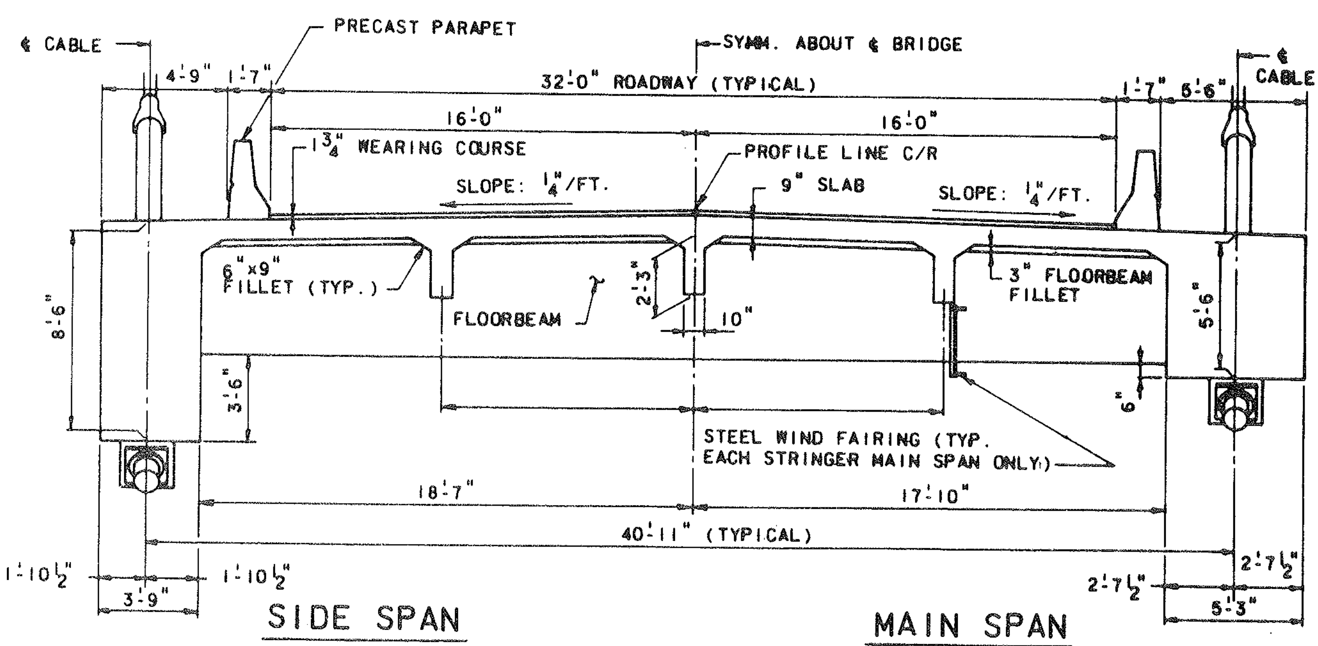

FIGURE 13 Concrete alternative: cross sections.

and varies from 8 ft, 6 in. to 5 ft, 6 in. in depth. In the second major transition the edge beam remains at 5 ft, 6 in. in depth and varies from 3 ft, 9 in. to 5 ft, 3 in. wide. It remains at this latter section across the tower pier to the centerline of the bridge. Thus, although the depth and the width of the edge beam vary at several different locations, only one dimension varies at any given location.

The floor beams are box sections that have a width of 2 ft, 6 in. The concrete stringers are 10 in. wide and 3 ft deep from the top of the roadway slab to the bottom of the stringer. The roadway slab is 9 in. thick, with a 1.75-in. bituminous concrete wearing course.

Steel wind fairing plates (to control wind vibration) are added to the structure beneath each stringer in the main span only; they extend from the bottom of the stringer to the same elevation as the bottom of the edge beam.

Figure 14 is a typical cross section of the deck that shows the mild steel reinforcing and the prestressing bars. There is longitudinal posttensioning in every longitudinal element (i.e., the edge beam, the deck, and the stringers). The amount of the posttensioning varies with the natural thrust in the bridge. At the towers, where the natural thrust in the bridge is the greatest, the amount of the posttensioning is the least. Generally, the longitudinal reinforcement within a segment is the same length as the segment itself. However, in the edge beam a percentage of the mild steel reinforcing is placed in ducts and mechanically coupled to the reinforcement in the adjacent segment. The ducts are then filled with grout, which produces some continuity of mild steel reinforcing between the segments in the edge beam.

A front elevation of the floor beam is shown in Figure 15. This is a reinforced-concrete member with no prestressing in it. Generally, there are two floor beams per segment. In addition to supporting the floor and its loads, the floor beam stabilizes the edge beam. A grid analysis was performed that shows that if a load is placed on one floor beam, that floor beam deflects and twists the edge beam, and as the edge beam rotates the adjacent floor beams are caused to deflect.

A cross section of a typical floor beam is shown in Figure 16; it is a hollow box, 2 ft, 6 in. wide with 5-in.-thick walls. Diaphragms are located at the stringer and intermediate points. At the center of the main span some of the floor beams are solid

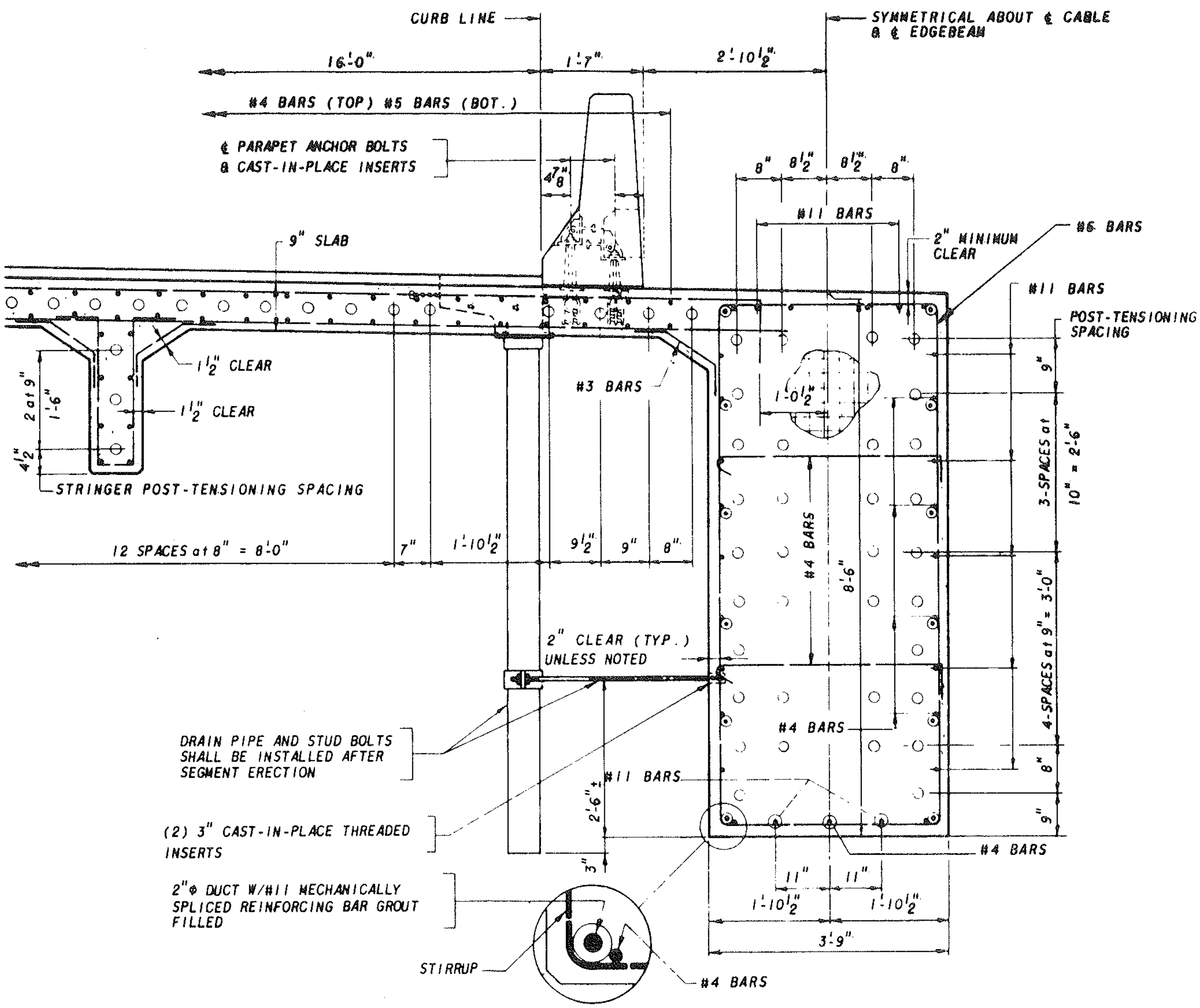

FIGURE 14 Concrete alternative: prestressing and reinforcing details.

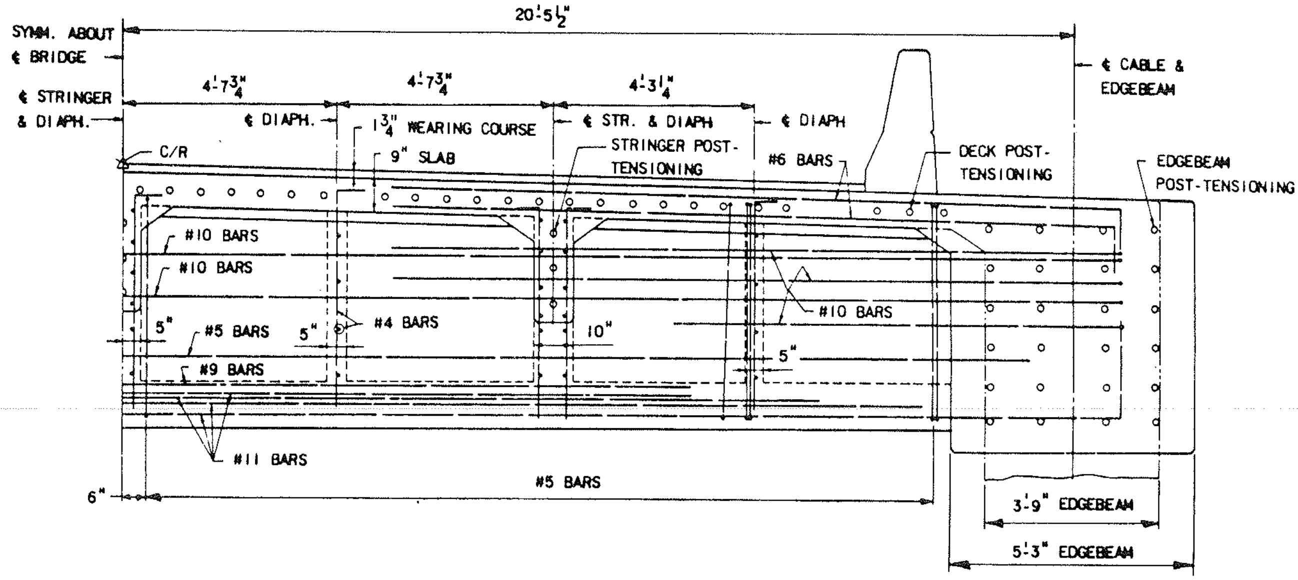

FIGURE 15 Concrete alternative: floor beam reinforcing details.

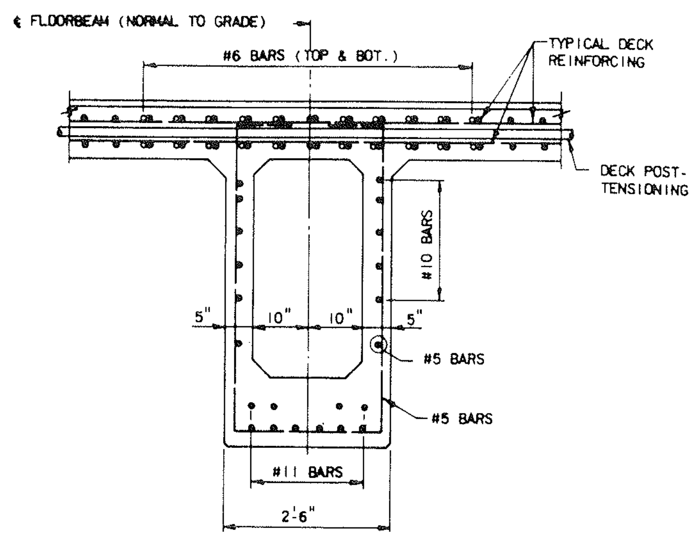

FIGURE 16 Concrete alternative: floor beam cross section.

where additional weight is required to balance the bridge.

The general deck arrangement is shown in Figure 17, which is a half plan for a typical superstructure segment. The following elements are shown: the deck, the edge beam shown at the bottom, and a cable anchor pipe passing through the edge beam. The stringers are shown running horizontally, and the floor beams are shown as vertical lines, with their diaphragms being horizontal.

Figure 18 shows an elevation view of the edge beam with the reinforcing and the embedded cable anchor pipe terminating at the steel bearing plate under the edge beam at the corbel. Detailing of the reinforcing and the prestressing to maintain clearances for the cable anchor pipe was a problem.

The hardware associated with the anchorage of the cable at the edge beam is shown in more detail in Figure 19. A neoprene boot is fastened to the anchor pipe and to the cable. An elastomeric washer is located at the upper end of the steel anchor pipe. The cable and the anchor pipe pass through the edge beam to the bottom of the corbel where the anchor pipe is welded to a bearing plate. The bearing plate supports the split washers, the shim pack, and the cable socket.

The edge beam cross section in Figure 20 shows the posttensioning and the embedded bearing plate and cable anchor pipe. The shear rings welded to the cable anchor pipe are also shown. This figure shows the tight clearances that exist between the posttensioning and the cable anchor pipe with its welded shear rings.

Erection of the bridge is basically the contractor's responsibility. However, an erection sequence has been assumed and analyzed on a preliminary basis. The contractor may choose to be guided by the procedure shown or may develop a totally different erection procedure.

The first stages of the erection of the superstructure are shown in Figure 21a. The tower construction has been completed. The auxiliary erection stays have been tied off to the top of the tower, and falsework has been erected adjacent to the tower

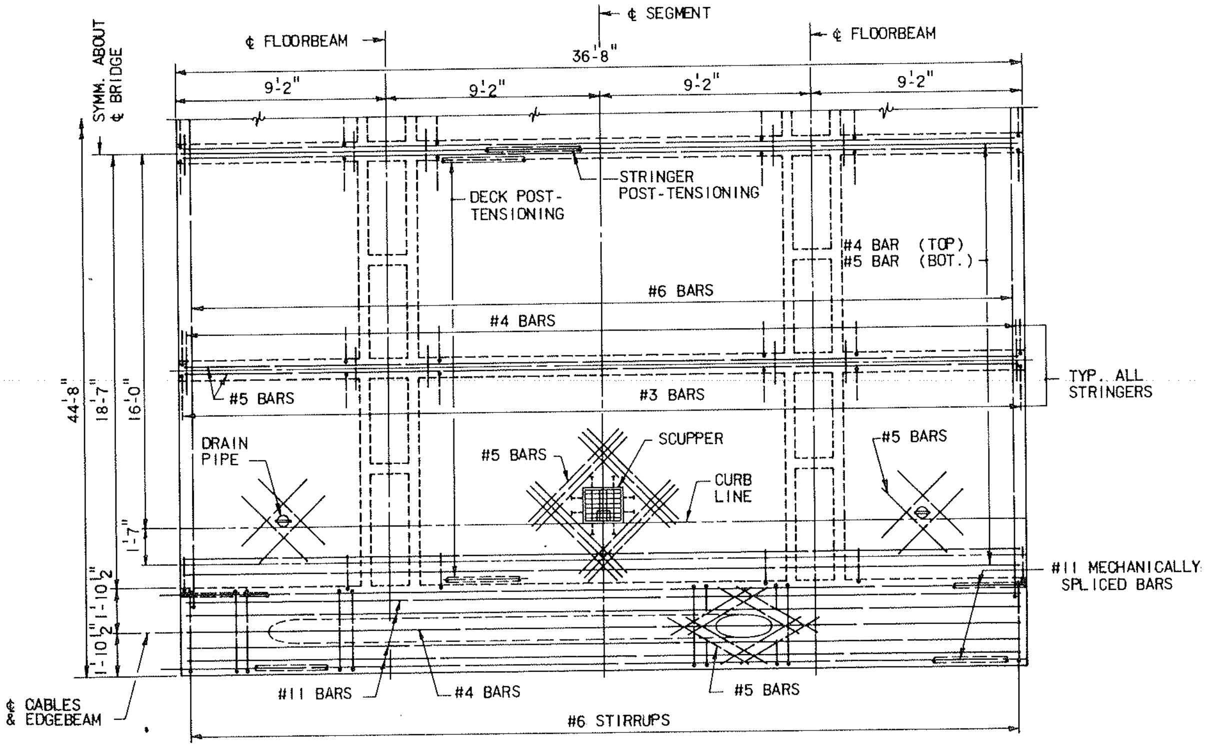

FIGURE 17 Concrete alternative: segment reinforcing details.

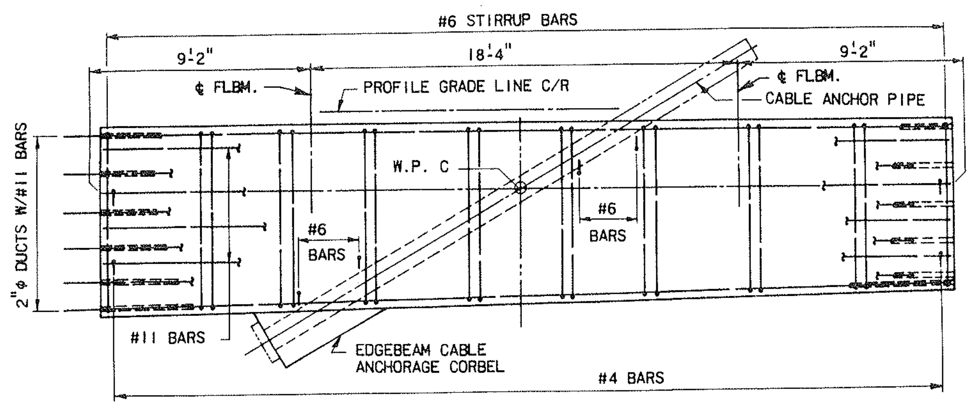

FIGURE 18 Concrete alternative: edge beam reinforcing details.

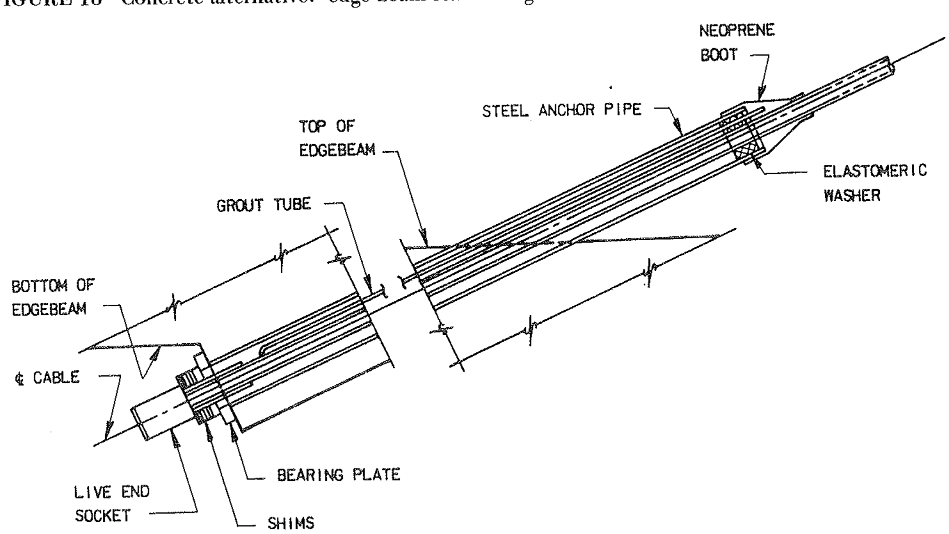

FIGURE 19 Concrete alternative: edge beam cable attachment details.

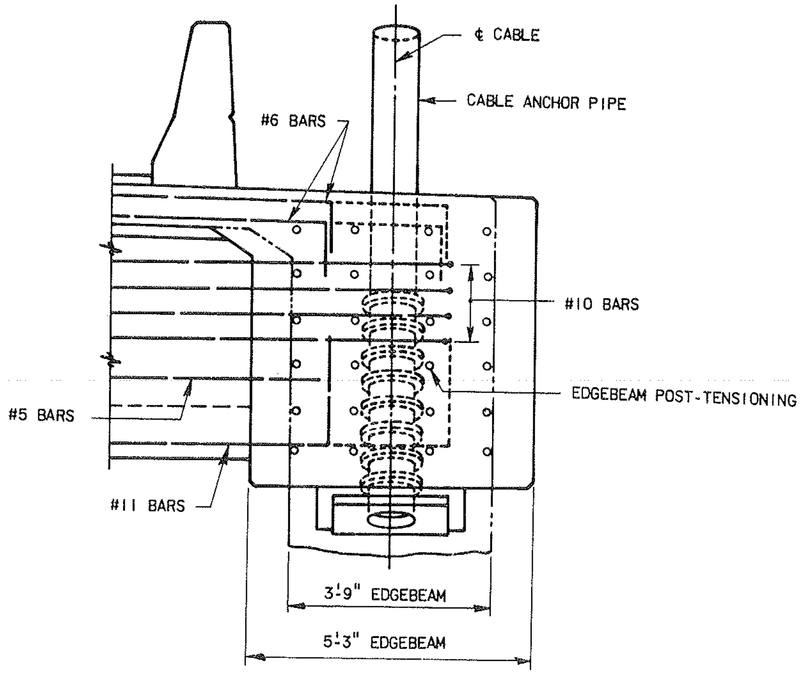

FIGURE 20 Concrete alternative: edge beam reinforcing and cable pipe details.

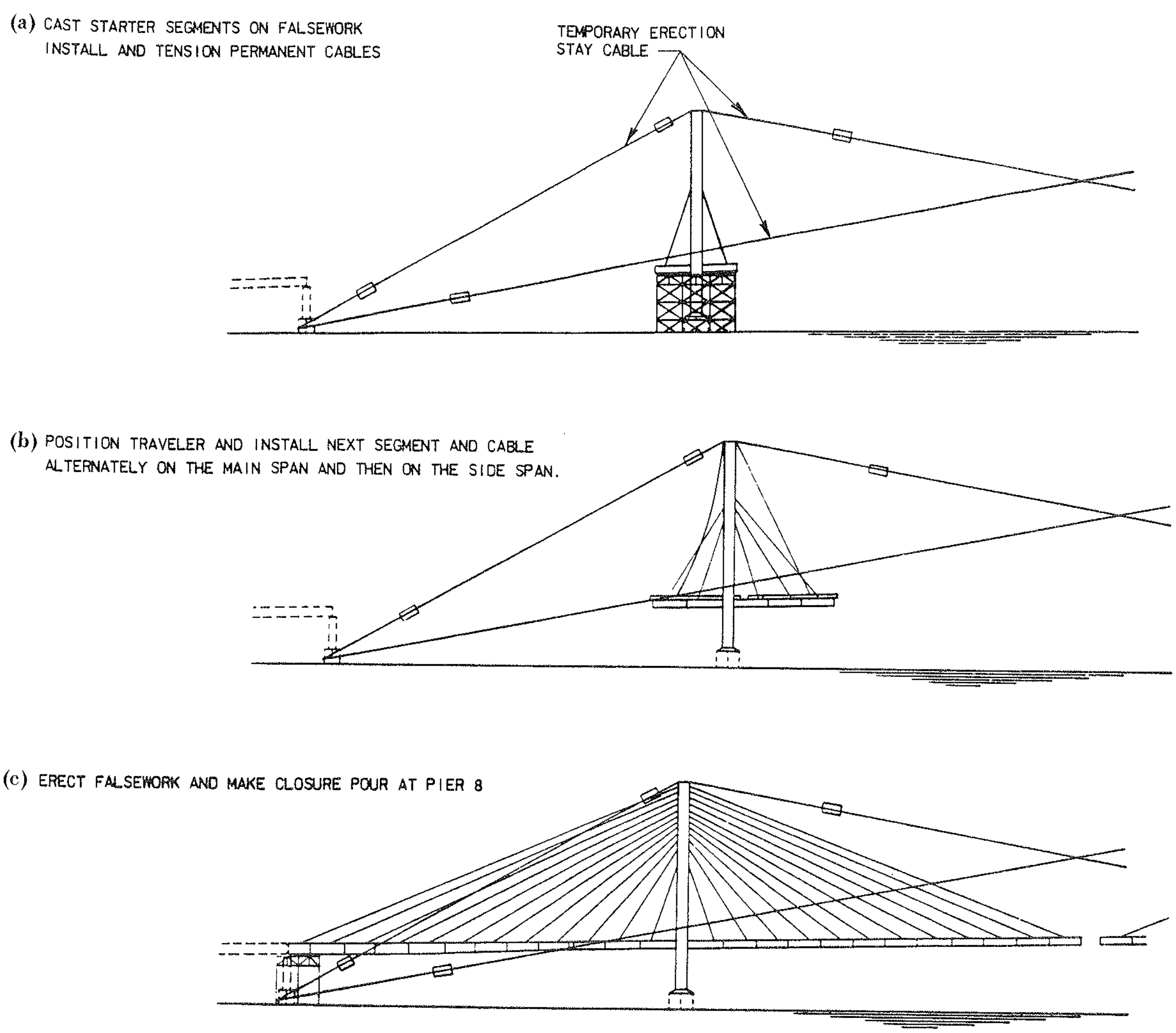

FIGURE 21 Concrete alternative: selected stages of erection.

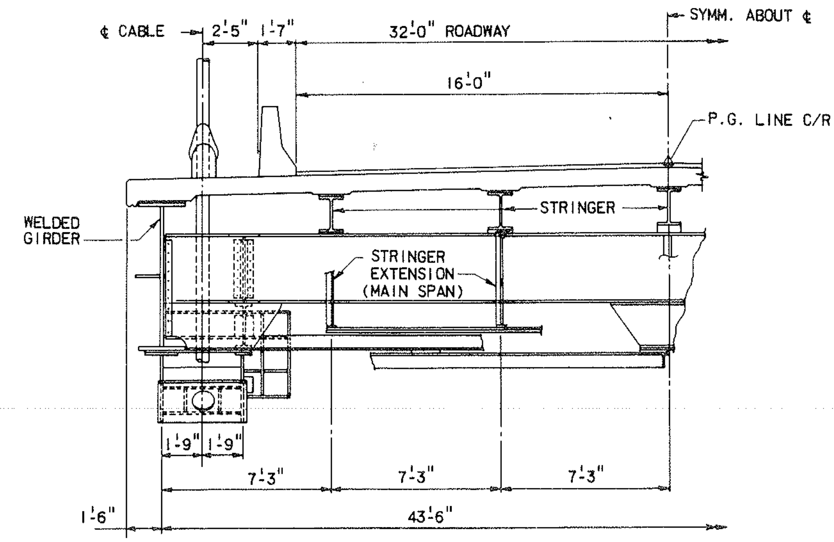

FIGURE 22 Steel alternative: typical cross section.

to support the initial cast-in-place deck segments, which have been cast and are supported on their stays.

Figure 21b shows an early stage in the erection with two travelers in position on the deck. The left traveler is shown in the rest position and the right traveler is erecting a precast segment.

An elevation view of the bridge with all the precast segments erected is shown in Figure 21c. Falsework has been erected at the anchor pier, and the cast-in-place segment has been constructed.

STEEL ALTERNATIVE SUPERSTRUCTURE

A general elevation of the steel bridge layout is shown in Figure 2. The span lengths are the same as the concrete bridge. The structural arrangement includes two single-web welded steel girders, rolled section floor beams, and rolled stringers. The system includes a precast, composite, concrete deck. There are 7 cables on each side of each of the pylons, for a total of 56 cables in all. The cables are spaced at 60 ft in the main span and 63 ft in the side spans.

A typical cross section is shown in Figure 22. The deck slab is detailed to be precast full width. The two main girders are made up of 72-in. constant-depth webs and are spaced at 43 ft, 6 in. The stay cables are inboard of the girders at 40-ft transverse spacing.

Typical floor beams are W36x260 sections. The stringers are two different sizes, depending on location: either W18x119 or W18x97. The stringers are heavier than might normally be required because the deck is under compression and the stringers are required to function as compression members.

Generally, the fabricated steel members are shop welded, and all field connections use high-strength bolts.

Certain details are somewhat unconventional, partly because of the combination of features built into the structure. The composite section is required to accommodate both the bending and compression stresses that the structure loads create. The strength requirements were not obtainable by posttensioning the slab section after it was composite with the steel. A system of precasting the deck slab unit, then continuously posttensioning it, and subsequently making it composite with the steel was devised to meet all strength requirements.

Certain aerodynamic features were necessary on the 900-ft main span, but not on the side spans. Plates were added beneath each stringer, which in effect extend the stringer webs down to the level of the bottom flanges of the girder, as shown in Figure 22. The plates are 0.375 in. thick and are appropriately stiffened and braced.

The wind tunnel testing also led to the addition of wind fairing plates located outside of the main girders. These plates are 30 in. wide and are positioned horizontally in the plane of the bottom flanges of the girders.

A partial cross section view is shown in Figure 23. The W36x260 beam spans longitudinally between floor beams only in the panels that contain cable anchorages. Its purpose is to react the vertical component of the cable. Because the cable is eccentric with respect to the main girder, this beam is required to carry half of the vertical component of the load in the cable.

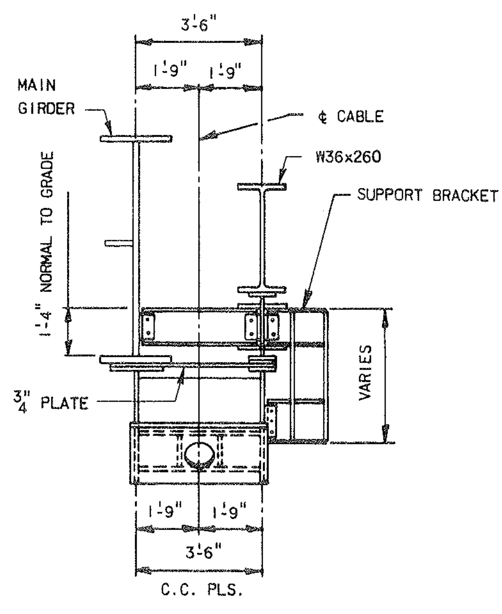

FIGURE 23 Steel alternative: framing at cable locations.

Figure 24 shows a plan and elevation of the deck framing at a typical cable anchorage. The horizontal component of the cable load is applied out of the plane of the main girder, and the transverse framing provides a means of stabilizing the forces without overstressing the girders. The frame is also effective in accommodating cable loads that are unequal in the two cables in a pair on opposite sides of the roadway.

Figure 25 shows a detailed plan view at a cable attachment location. The 0.75-in. plate, which is connected to the girder and the longitudinal W36 beam, helps to react the horizontal component of the cable load. The connections on all four edges of the plate transmit the loads into the horizontal frame.

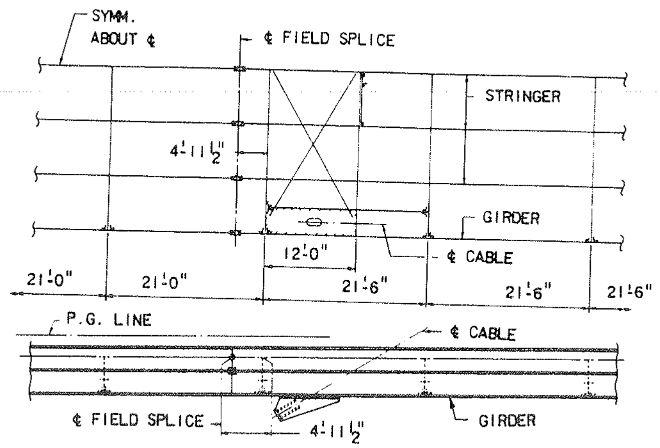

FIGURE 24 Steel alternative: plan and elevation of deck framing at cable anchorage.

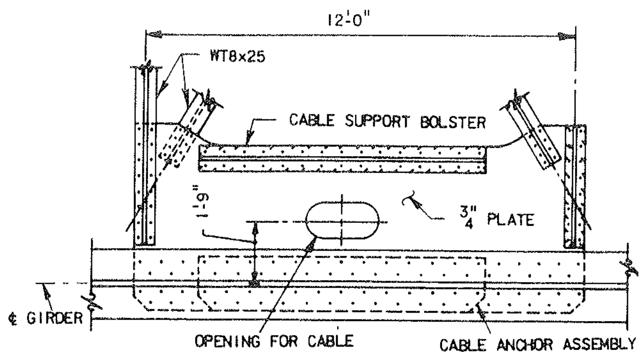

FIGURE 25 Steel alternative: cable frame attachment details.

The connection of the cable to the girder is accomplished by a weldment that accommodates the cable socket. A typical cable anchorage is shown in Figure 26. The weldment is bolted to the bottom flange of the girder and to the framing members inboard of the girder. Shims are provided to allow for initial adjustment of the assembly to assure that it is in proper orientation with respect to the vertical plane of the cable.

Figure 27 shows an elevation view of the lower end of a typical stay cable. The hardware for this type of cable is basically the same as shown previously in Figure 11 for the concrete bridge. The cable penetrates the deck slab through a steel pipe that is embedded in the precast unit.

Details of a typical girder field splice are shown in Figure 28. The girder functions in both bending and axial compression, which requires that the splice details include a heavier connection in the web than is normally required in a typical beam splice. The girder web is considerably heavier than is typical for the same reason. The splices are designed to be consistent with the load factor design for the member and are specially designed for the combined state of stress that occurs at the specific location of each splice. Depending on the erection scheme, it may be possible to erect longer sections and eliminate some of the splices shown on the plans. The splice locations shown on the contract drawings are consistent with the assumptions that were made for the preliminary erection scheme shown on the plans.

Typical features of the precast deck slab units are shown in cross section in Figure 29. The slabs are detailed to be cast full width and full thickness. It is specifically required that the deck erection must follow closely behind the steel erection, because the superstructure design is based on the composite section carrying most of the compressive dead load that is caused by the horizontal components of the cable loads. The slab units cannot assume compression loads until the slab is made composite with the longitudinal girders because the stress occurs initially in the girder and is then distributed into the slab.

The deck units are full width and vary from 9 to 11 ft in length. The slabs contain adjusting screws that allow the units to be placed on the supporting

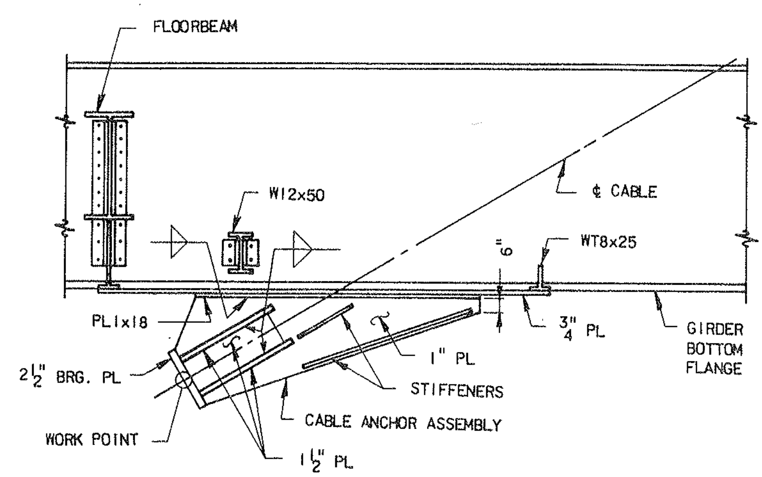

FIGURE 26 Steel alternative: cable anchor assembly details.

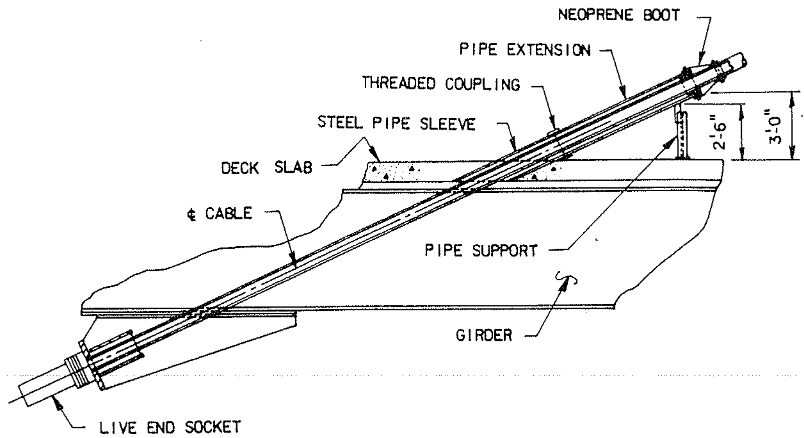

FIGURE 27 Steel alternative: cable details at deck level.

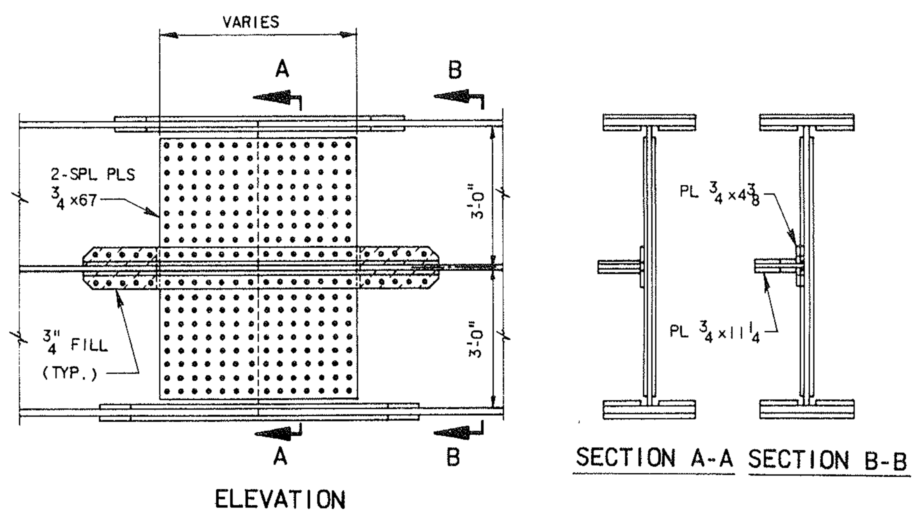

FIGURE 28 Steel alternative: typical splice details for girders.

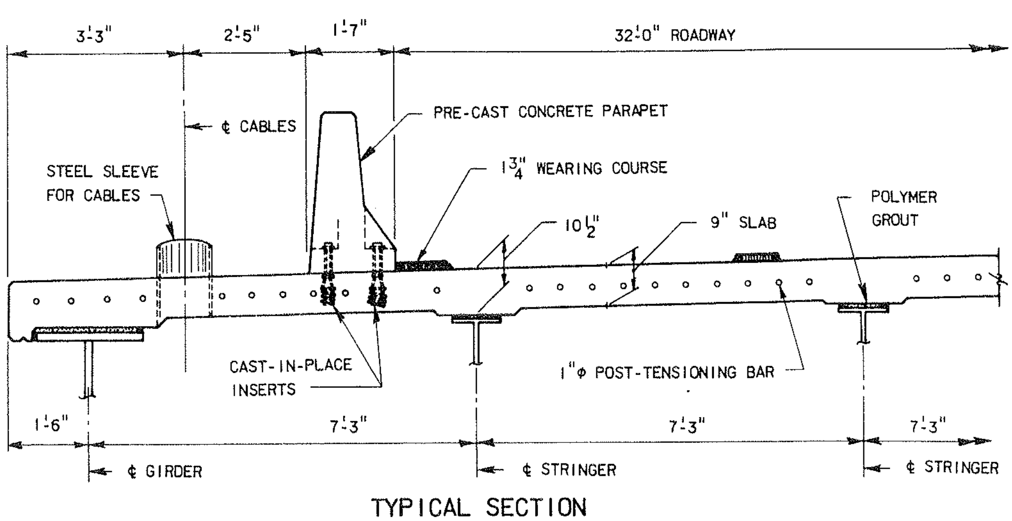

FIGURE 29 Steel alternative: details for precast slab panels.

steel members with a space between the slab and the flange that is later filled with grout to complete the haunch. This sequence permits the slab to be posttensioned before making it composite with the steel beams. The slabs are reinforced both longitudinally and transversely and are posttensioned longitudinally in groups of three to five panels. The posttensioning takes place after the slabs are erected on the steel supporting elements, but before they are made composite with the girders and stringers. The slabs are cast with openings to allow placement of the steel studs. The studs are attached to the beams after the slab has been erected and finally positioned. The openings at the stringer locations allow enough room for normal application of the studs, but the openings over the girder flanges are restricted, and special testing procedures are required for the studs. The normal bend test would require a larger opening than the available space allows.

After the studs are installed and grouting of the openings and the haunch between the underside of the slab and top of the beam is completed, the grout hardens and the slab becomes composite with the steel girder.

An investigation was made of an assumed erection procedure to determine the feasibility of the scheme shown on the contract plans. Computations generally indicated that no major reinforcement of the members will be required, but adjustment of some cables may be necessary to avoid overstress as the erection proceeds. A general sequence similar to that shown for the concrete bridge was assumed, except that the individual pieces are not as heavy, which allows somewhat greater flexibility in maintaining balance with respect to tower bending. The concept is based on installing temporary stays on the tower top and erecting the initial steelwork at the tower, which is supported by appropriate falsework. The deck slab at the tower is cast-in-place and posttensioned after it cures. The typical sequence outlined in the following paragraphs is then followed in a balanced-cantilever arrangement.

The details shown in Figure 30 outline the items comprising one section of deck erection. The process is then repeated for the other sections. The typical required sequence, referring to Figure 30, is described as follows.

- The first step is to erect the steel framing members, cantilevered from the previously completed section.
- The precast slab panel at the cable location is then placed in its required position, as shown in step 1.
- Step 2 is erection of the cable. Because the slab at that location is already in place, the cable must be pulled through the opening in the slab.
- In steps 3-5 the precast panels are placed. With the cable in place, the additional dead loads are supported by the cable and are not dependent on cantilevering.
- At step 6 the posttensioning bars are placed and spliced to the preceding bars at the location of the cast-in-place joint.
- At step 7 the cast-in-place joint is placed and allowed to cure.
- The bars are posttensioned and the studs are placed through the openings in the slab at step 8. The grout is then placed to fill the stud openings and the haunches between the slab and the tops of the flanges. At this point the section becomes composite and the sequence can be repeated for the next section.

At the final closure stage at the center of the main span, first the steel members are closed and the joints bolted, and then a 4-ft section of slab is cast-in-place to complete the deck section. The remaining items can then be completed, such as completion of the installation of parapets, placement of the roadway wearing surface, and final adjustment of the cables.

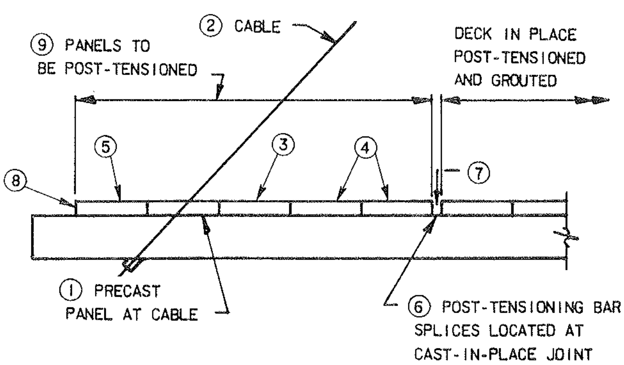

FIGURE 30 Steel alternative: deck panel erection sequence.

CONCLUSION

Both alternative structures were advertised for bid in November 1983. Bids were taken in February 1984. The winning alternative was the steel bridge, which was bid at a total cost of $17,230,461.

ACKNOWLEDGMENT

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