A Landmark Cable-Stayed Bridge Over the Charles River, Boston, Massachusetts

Many design innovations and technological breakthroughs were used to construct this distinctive bridge, now a key part of the downtown Boston skyline.

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Boston, in the forefront of the American Revolution over two centuries ago, has now risen to the forefront of another revolution — in the field of cable-stayed bridge technology. A highly complicated structure, the Leonard P. Zakim Bunker Hill Bridge, has recently been completed. New technologies and innovations have become hallmarks of this bridge, which crosses the Charles River at the North End section of Boston. This challenging assignment brought engineers and architects, as well as the community, together to provide a gateway to the city. Since the bridge was located at a pre-eminent point where Paul Revere crossed in 1775, warning colonists that the British were coming, and since it was near where the Battle of Bunker Hill took place, it took on special meaning from a historical perspective.

Introduction

The immense Central Artery/Tunnel (CA/T) Project in Boston consists of many kilometers of tunnels, four major interchanges and two long-span parallel crossings of the Charles River, one connecting Storrow Drive and the other a cable-stayed bridge connecting I-93 — the Leonard P. Zakim Bunker Hill Bridge (see Figure 1). This bridge features a hybrid cable-stayed structure that is the first of its type in the United States. The river crossing provides virtually the only access to Boston from the north, straddling the Charles River in that historic area. Accordingly, there was great interest in ensuring that the bridge features a distinctive design while also provid-

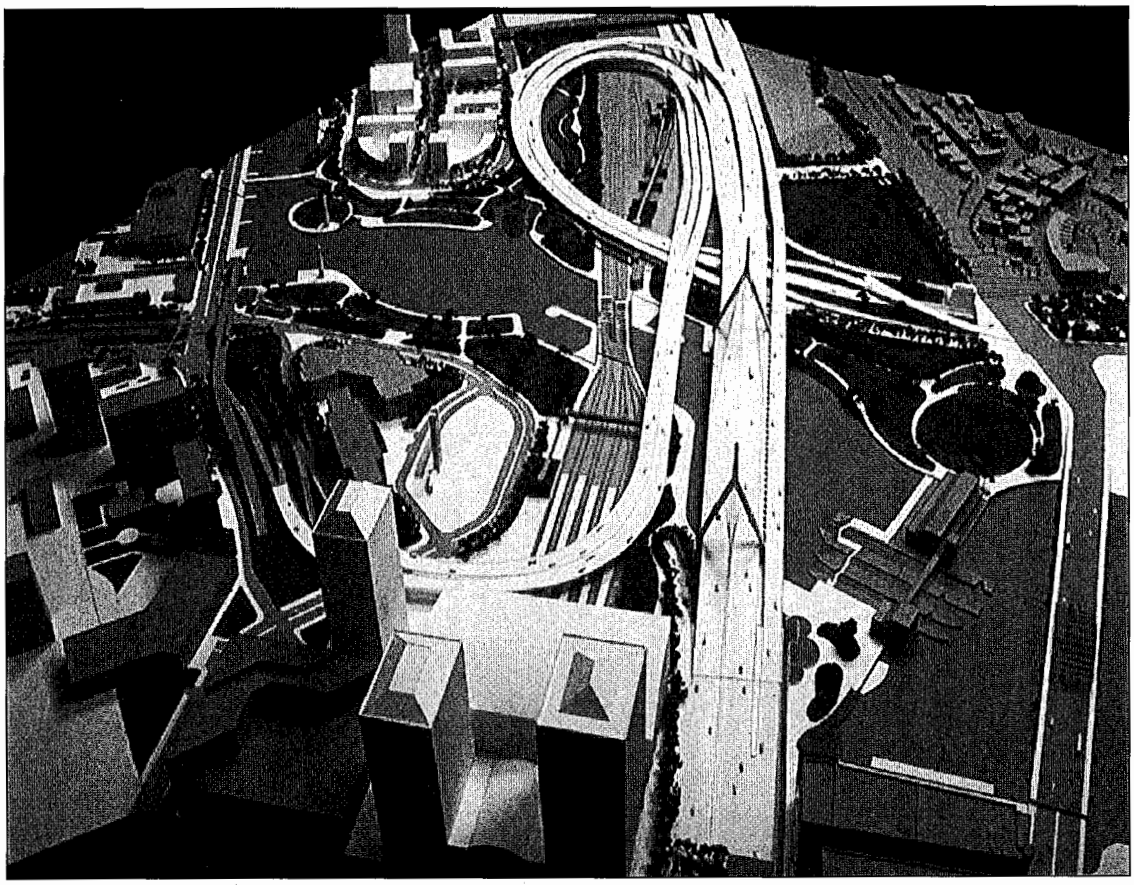

FIGURE 1. Aerial view of a model of the Charles River area.

ing a dramatic gateway to the city's downtown.

Numerous alternatives were studied for the crossing and the interchange configurations on both sides of the river. Eventually, the option known as the "Non-River Tunnel" alternative was chosen, which required a ten-lane crossing of the Charles River. The ten lanes included four lanes each for Interstate-93 (I-93) northbound and southbound as well as a two-lane ramp on the east side. Some impediments the bridge designers had to contend with included:

- the Orange Line subway adjacent to and below the bridge (including an existing Orange Line subway ventilation building adjacent to the south main pier);
- the proximity of the Charles River Lock and Dam abutting the east side of the bridge, and the need to maintain navigation;
- a major 0.92-meter-diameter water main located below the south tower footing;
- a cantilevered two-lane ramp on only one side of the structure;
- the existing Storrow Drive ramps at the south end, dictating the arrangement of the stay cables in the back spans; and,
- a new tunnel at the south end of the structure.

Bridge Type Selection

The Leonard P. Zakim Bunker Hill Bridge had to meet the requirements of numerous state and federal regulatory agencies, including the Federal Highway Administration (FHWA). The bridge design had to present sound engineering solutions to numerous site constraints while also meeting community expectations that the structure create a distinctive signature on Boston's skyline. To fulfill these goals, the project team conducted a bridge type study

and assembled from the CA/T Project a multidisciplinary team of experts in structural engineering, inspection and maintenance, highway design and engineering, urban planning, construction, environmental engineering, architecture, and cost and scheduling control. The team also included representatives from the FHWA, the bridge concept developer and the project owner.

Initially, the project team identified sixteen bridge types, each with a main span measuring 227 meters. The sixteen designs included three arch bridges, four truss bridges, a segmental box girder bridge, a finback concrete bridge, a suspension bridge and six cable-stayed bridges. The engineering team then analyzed each option for a wide variety of variables — including alignment, design, environmental, urban design, constructability, ease of inspection, maintenance and cost — before narrowing the field to seven bridge types for further study. They included a two-hinged arch, a simple span truss and five cable-stayed bridges. The cable-stayed bridge options had both symmetrical and asymmetrical designs. They varied between ten and twelve traffic lanes and featured single- and double-tower configurations. It was also decided that the cable-stayed bridges could be built using steel, concrete or a hybrid of both, and that tower pier shape could be flexible. An evaluation matrix was prepared based on priority factors, as well as a quality rating for the variables previously mentioned.

With the participants addressing all of the aspects, priority factors for those variable items on a scale of 1 to 3 were assigned (1 = necessary, 2 = important and 3 = very important). The priority factors used to evaluate the seven bridge options did not vary between options. Then a quality rating on a scale of 1 to 5 (1 = poor, 2 = fair, 3 = good, 4 = very good and 5 = excellent) was voted for each bridge type. The weighted rating was then computed by multiplying each priority factor times the quality rating. The total score for each bridge type was the sum of the weighted ratings. Ultimately, the concept developed by Dr. Christian Menn was selected as the most appropriate bridge for the I-93 crossing of the Charles River, providing a unique bridge for one of the nation's oldest cities (see Figure 2).

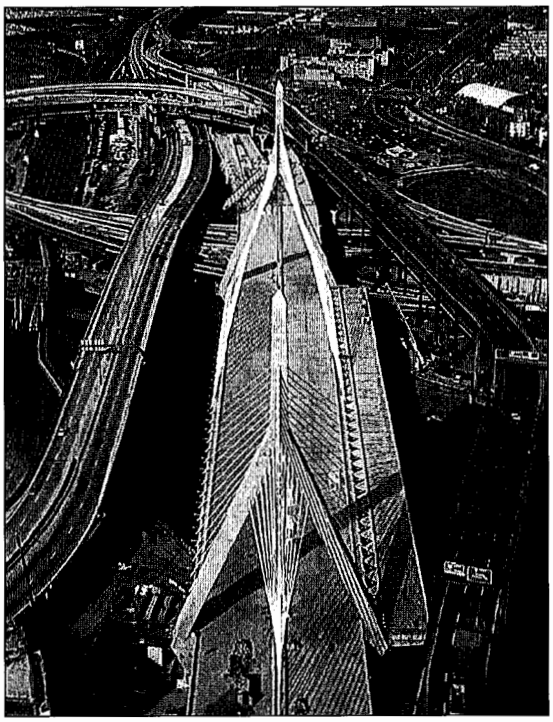

FIGURE 2. Aerial view of the Leonard P. Zakim Bunker Hill Bridge looking north. It replaced the structure to its east.

Preliminary designs were then prepared for steel, concrete and hybrid alternatives for the asymmetrical cable-stayed structure. Even though each alternative posed unique challenges, a committee composed of international bridge experts concurred with the project team that only the hybrid alternative should be pursued. Due to the relatively short south back span (as compared to the main span), it featured "heavy" cast-in-place post-tensioned concrete construction to counterbalance the "light" main span, which was constructed of steel floor beams and edge girders with a precast concrete composite deck. The bridge was the first hybrid cable-stayed structure built in the United States with concrete back spans and composite steel main spans.

Due to the uniqueness of the bridge and its many new concepts — including its extremely wide main span, asymmetry of the structure

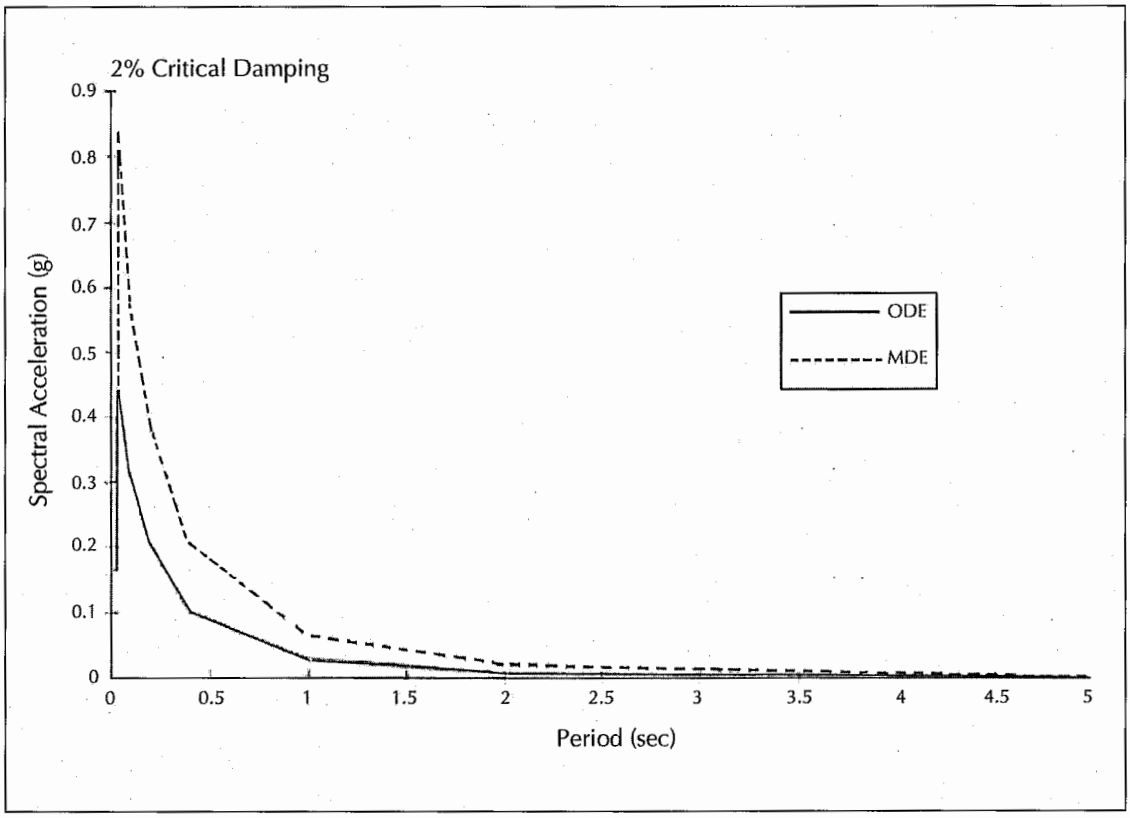

FIGURE 3. Site-specific seismic spectra.

due to the cantilever on one side outside the cable plane, extensive shear lag and torsional effects, the sequencing and method of construction, and its noteworthy cable anchoring details — the FHWA requested the preparation of an in-depth study prior to granting approval for the project to proceed. In fact, this extensive effort, conducted during the preliminary stages, actually allowed the final design to proceed at an expedited pace and, thanks to clearly defined parameters, this effort also contained costs.

Site-Specific Seismic Study

The Leonard P. Zakim Bunker Hill Bridge is a lifeline to Boston. Like all major interstate highway structures, it was considered "important" for seismic considerations. This designation meant that the bridge would have to be serviceable after a design earthquake, sustaining only minor damage. Therefore, due to the structure's critical role and its unusual design features, a site-specific seismic study was undertaken. This study was undertaken because:

- The generic seismic design spectra found in American Association of State Highway and Transportation Officials (AASHTO) specifications are based primarily on ground motions from earthquakes occurring in California and along the west coast of the United States. Earthquakes occurring on the east coast tend to have higher frequency motions and do not attenuate as quickly with distance because the nature of the earthquake source mechanisms and the bedrock are different from those in western states.
- The seismic response spectra that were previously developed specifically for tunnels in the CA/T Project did not totally account for the long period motions that would be encountered in long span bridges.

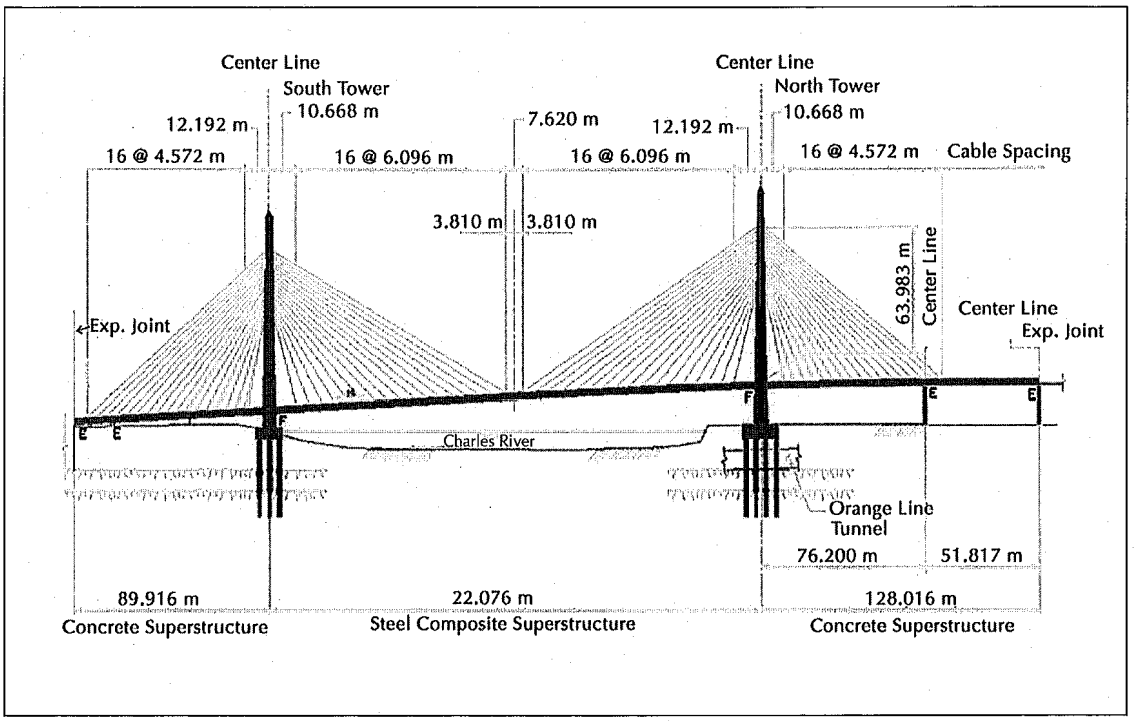

FIGURE 4. Elevation of the bridge.

The study found that source zones from latest developments generally resulted in lower spectral accelerations than the CA/T Project criteria. Thus, it was decided to use the previously established CA/T Project response spectra and develop extensions of the spectra as required for the long span cable-stayed structure. Rock motion time histories and response spectra for the 2,000-year maximum design earthquake (MDE) and 500-year operating design earthquake (ODE) were developed for both 2 percent (see Figure 3) and 5 percent critical damping.

Bridge Configuration

The shape of the tower piers and the cable arrangement (main span cables splayed out from the tower with back span cables centered in the median) evolved principally from technical considerations, with aesthetics playing an important role. The proximity of the existing double-deck ramps at the south end, which needed to remain in service during the construction of the new bridge, dictated that the back span cables be anchored in the median and not splayed out similar to the main span cables. The southernmost cable cleared the underside of an existing ramp with only less than half a meter to spare, and the south back span of the bridge was built encompassing an existing bridge column. Once a replacement ramp was opened to traffic (as part of an adjacent contract), the existing ramp was demolished and the deck opening was filled.

This cable-stayed bridge featured a 56.4-meter-wide, five-span hybrid superstructure with a main span of 227 meters, two south back spans of 34.2 and 39.6 meters, and two north back spans of 51.8 and 76.2 meters (see Figure 4). The tower piers were inverted Y shapes (see Figure 5). The pinnacles of the south and north towers were 89.9 and 98.5 meters, respectively, from the tops of their foundations. The towers had ladders inside their hollow legs to provide access for the inspection and maintenance of the cable anchorages, and to the aviation beacons at the very top. The back spans consisted of multicell cast-in-place concrete box girders 3 meters deep and 38.4 meters wide. Main structural elements included a 3-meter-wide central spline beam with internal floor beam

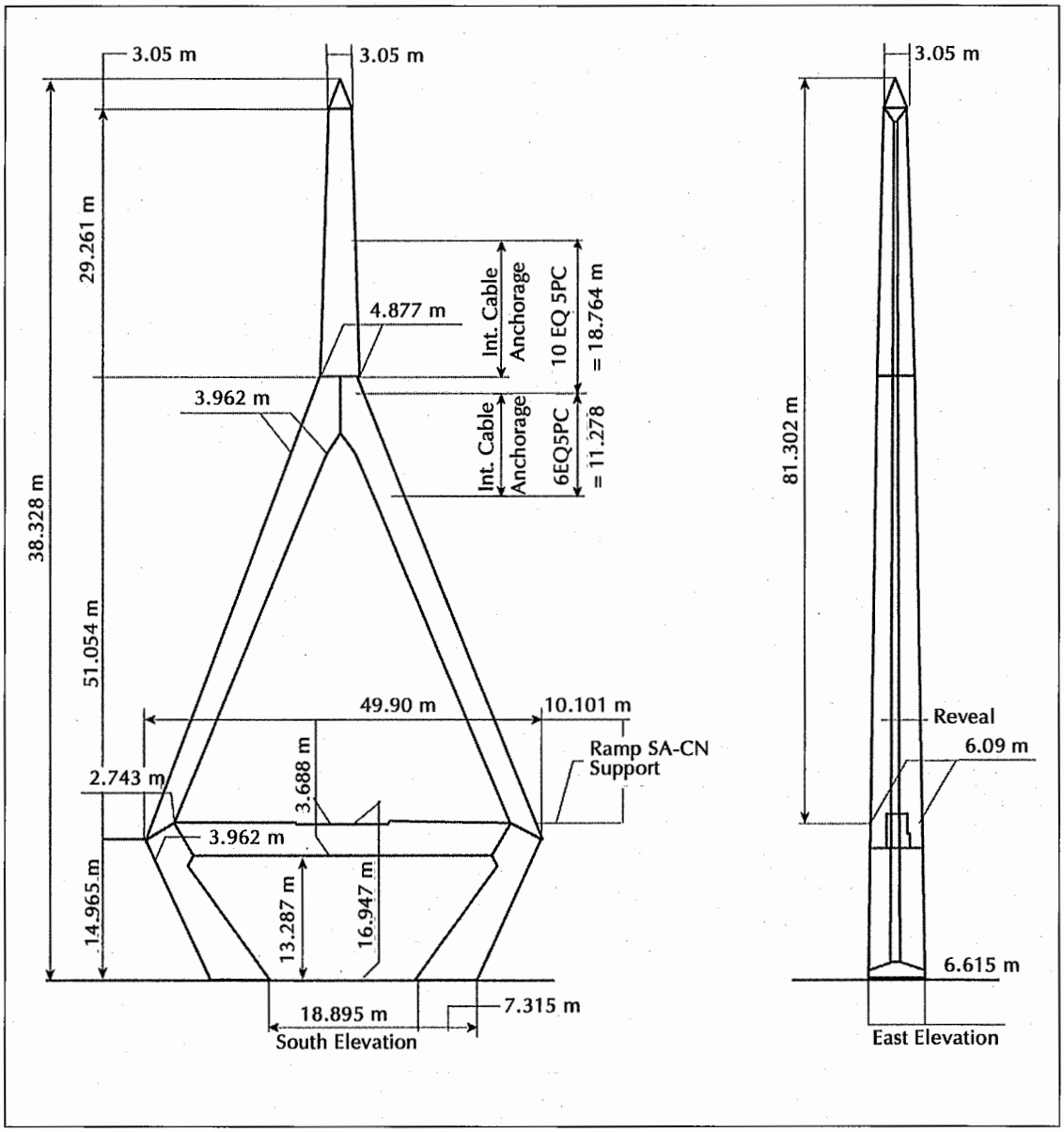

FIGURE 5. North tower pier elevation.

diaphragms at 4.6 meters on center framed with four secondary webs (see Figure 6). The spline beam in turn was supported by a single plane of cables spaced at 4.6 meters.

The main span consisted of precast concrete deck panels acting compositely with longitudinal steel box edge girders and transverse steel floor beams (at 6.1-meter centers) by means of cast-in-place closure strips (see Figure 7 on page 60). The box edge girders were supported by cables anchored on the outside web at 6.1-meter intervals. On the main span side, the two-lane ramp (SA-CN) was carried on floor beam extensions cantilevered on the east side of the main line deck. Medium-weight precast concrete deck panels were used for the ramp to minimize eccentric dead loads. On the back spans, the ramp was a single-cell concrete box girder, independent from the cable-stayed structure, with roadway joints at the tower interfaces. Open-grid fiberglass closure panels partially covered the

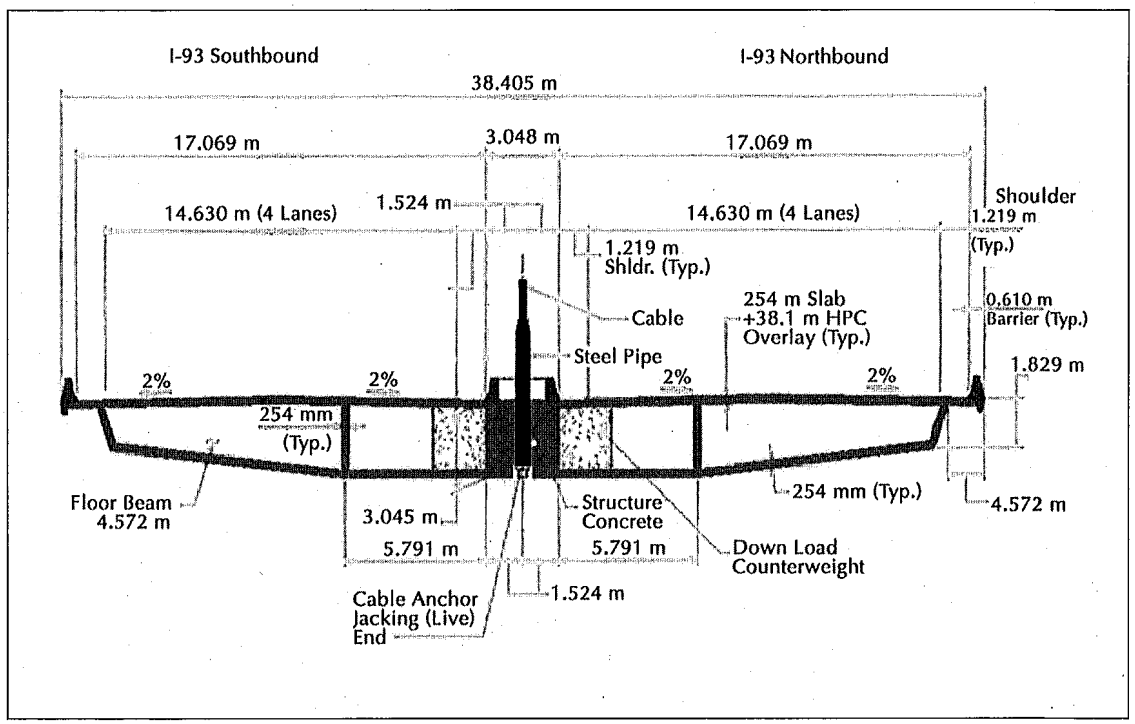

FIGURE 6. Back span cross section.

underside of the main span superstructure to enclose utilities, creating a more aesthetic underbelly while also providing access for inspection and maintenance.

Foundation & Drilled Shaft Design

The tower foundations consisted of footings on 2.44-meter drilled shafts (fourteen at the south tower and sixteen at the north tower) designed to carry a working load of 2,270 metric tons each. At the north tower, the footing and the supporting drilled shafts straddled the Orange Line tunnel. Measures were taken to ensure that the tunnel was not adversely impacted by foundation construction activities. The drilled shafts closest to the Orange Line tunnel were placed outside a 1.52-meter buffer zone. The effect of lateral forces transmitted from the drilled shafts through the surrounding soil to the Orange Line tunnel was investigated. As a result, the shafts closest to the tunnel were installed within 2.74-meter-diameter isolation casings. Meanwhile, the south tower foundation was very close to the existing subway ventilation building on the west side, while also bridging a 0.92-meter-diameter water line. Here also the three drilled shafts closest to the ventilation building were isolated within a 2.74-meter-diameter isolation casing. The gap between the casing and the drilled shaft was unfilled. The south tower foundation slab was 47 meters long by 12.2 meters wide and 4.6 meters thick. The north tower foundation slab was 42.1 meters long by 16.8 meters wide and 4.6 meters thick.

Tension Strut at Tower Piers

The change in direction of the tower legs at the deck level produced large tension forces in the connecting beam, called a "tension strut," between the tower legs. The tension strut also served as the transition from the main span composite steel superstructure to the post-tensioned concrete box girder back spans. In addition, compressive forces from the cables, which were carried in the spline beam of the back spans, needed to be transferred to the edge box girders of the main span and vice versa. The edge girders, in turn, were then attached to the main span cables. As a result, special attention was focused on tension strut design, while shear lag effects and tension

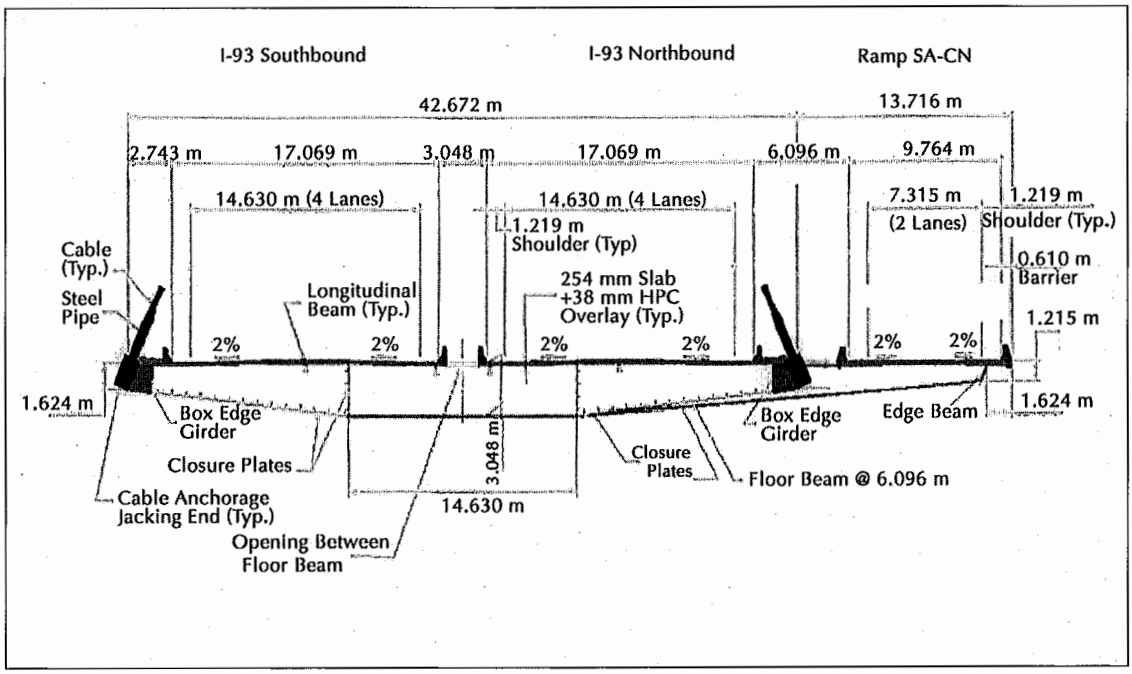

FIGURE 7. Main span cross section.

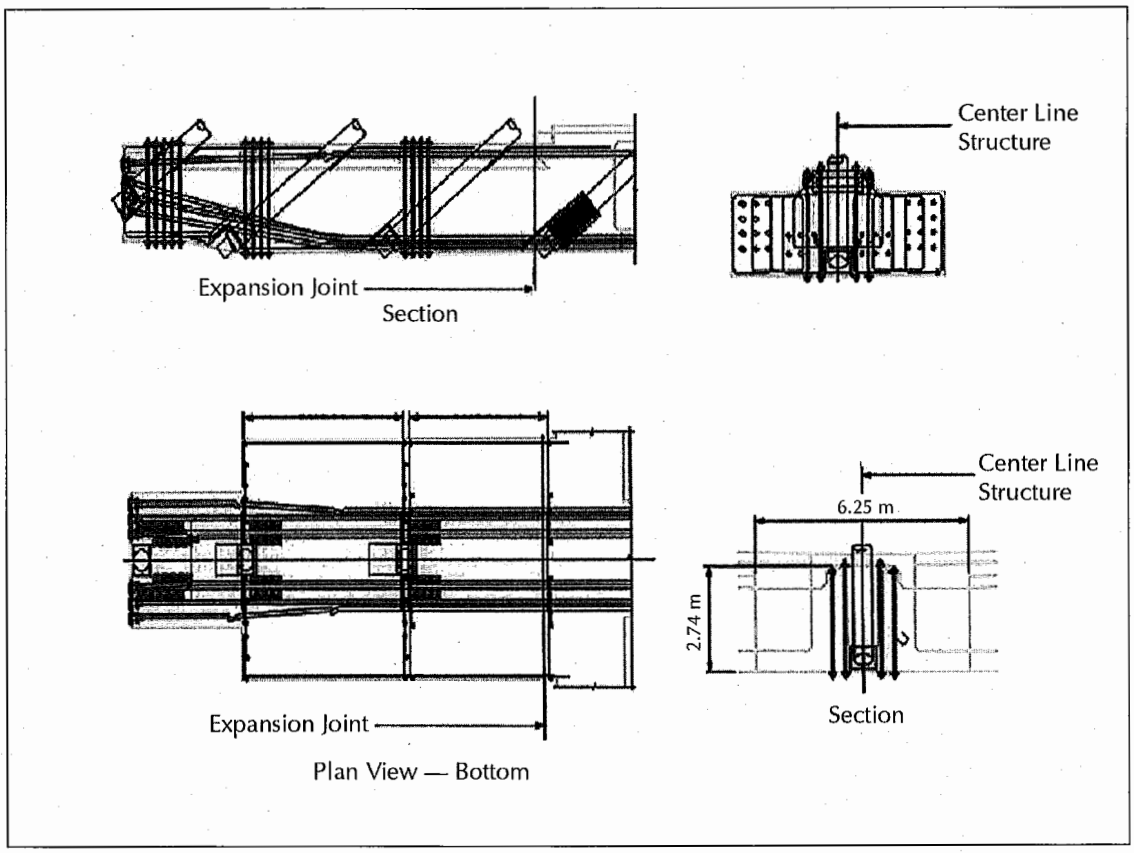

FIGURE 8. Spline beam extension at south back span.

stresses in the concrete were also carefully evaluated.

Additionally, the imbalance of bending, shear and axial load forces in the main span and back spans under different loading combinations produced torsion, bi-axial bending and bi-axial shear stresses in the tension strut. Due to its critical structural nature, the tension strut was post-tensioned in stages to a total jacking force of 25,130 metric tons. Limiting principal tensile stresses to pre-determined values under the working loads was an important design consideration for this element.

Post-Tensioned Concrete Back Spans

Cast-in-place post-tensioned concrete back spans were constructed on falsework for the bridge. Major challenges for the south back span design and detailing were the physical overlap of the plan area of the proposed bridge at the south end with an adjoining tunnel that was also part of the CA/T Project, as well as the proximity of the existing Storrow Drive double-deck ramps connecting to existing I-93, which had to be maintained in service during construction. At the north back span, numerous existing temporary and future ramps posed unique construction challenges. At the south interface, the design solution featured an early termination of the main line bridge deck (by approximately 15.24 meters), while extending the central spline beam the required length as a cantilever to receive and anchor the last three cables (see Figure 8). Heavy-weight ballast concrete with a density of 4,000 kilograms per cubic meter was placed in the box girder cells within the last three floor beam bays to counteract the local reduction in superstructure weight due to the early termination of the roadway. The cantilever extension of the spline beam was housed in a vault.

To reduce the impact of north back span construction to ramp traffic by limiting ramp closures and detours, final design and detailing of the north back span were conducted based on the incremental launching construction method. However, after a detailed evaluation, the contractor proposed and the project team accepted a value engineering proposal to cast in place the north back span using falsework, which spanned over the various ramps.

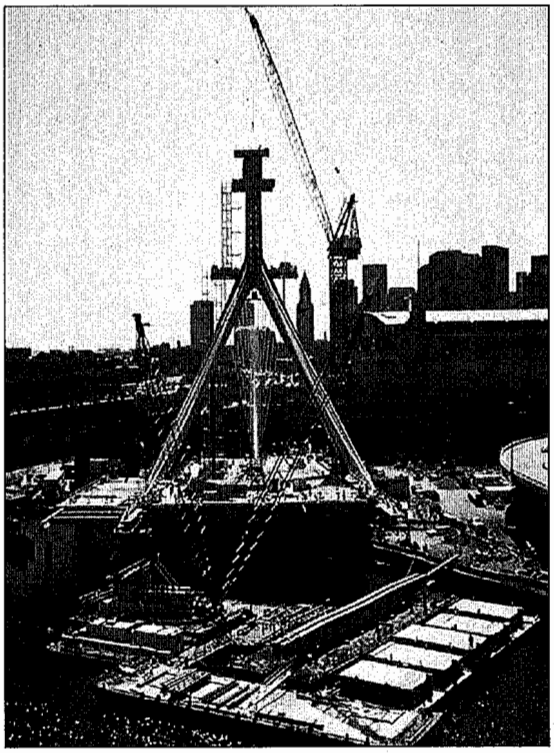

FIGURE 9. Main span under construction.

Steel Composite Main Span

The edge girders were asymmetric steel box sections with an inclined bottom flange and an inclined fascia web (see Figure 9). Typical edge girder sections were 18.3 meters long, supported by three stay cables. To achieve a full moment connection between the tower and the edge girders at the strut level, a base plate connection with 35-millimeter-diameter, 1,030 MPa post-tensioning bars was used. Steel floor beams, spaced at 6.1 meters longitudinally, spanned 42.7 meters between box edge girders. A separate floor beam cantilevered approximately 13.7 meters to carry two lanes of traffic outside the east plane of the stay cables. An edge beam provided at the fascia of the cantilevered section distributed truck loads to multiple floor beams.

The main span composite superstructure construction started with the edge girders (in

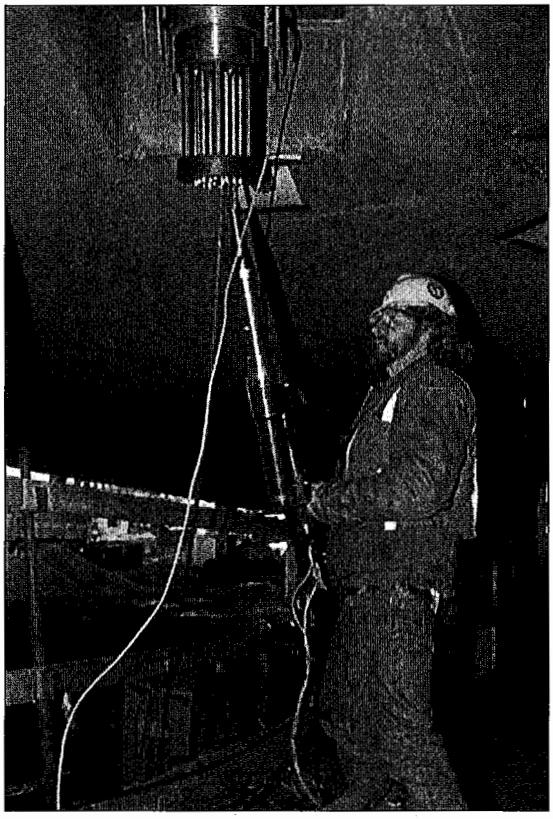

FIGURE 10. Ungrouted stay cable stressing in operation.

18.3-meter lengths) being placed and field spliced, followed by the cantilever floor beams and the fascia edge beam. The first of these floor beams between the edge girders was subsequently brought in and connected to the edge girders. The stringers connecting the floor beams were then installed. The precast deck slabs closest to the previously erected section were installed and first stage stressing of the main span and back span stay cables was performed. The next set of precast panels were then erected and corresponding stay cables were stressed to their first stage stressing. This process was repeated for the third set of precast panels, after which longitudinal post-tensioning was installed, followed by placing transverse closure strip concrete over the floor beams. When the closure strip concrete attained 24 MPa strength, the slabs were longitudinally post-tensioned. After the longitudinal closure pours were made over the edge girders and stringers, second-stage stay cable adjustments were made to all cables in the unit.

Stay Cables

Project design documents required stay cable strands to be either greased and sheathed or epoxy coated. Flexibility was provided concerning the type of anchorage (wedge or socket or wedge/socket) that could be utilized. After the project was bid, the successful contractor proposed using ungrouted stay cables employing a stressing method that had not been previously used in the United States (see Figure 10). The contractor also successfully proposed using coextruded high-density polyethylene (HDPE) pipe with a spiral bead to reduce potential stay cable rain- and wind-induced vibration.

This method of stressing allowed the stay cables to be installed one strand at a time. Based on the calculated installation force for the stay cable, the force required for the first or reference strand was calculated. This force had to account for the weight of the HDPE sheathing, which was temporarily supported by the single strand and structural deformations that occurred due to stay stressing. After the reference strand was installed, each subsequent strand was installed to match the force in the reference strand. Forces were measured by load cells on the reference strand and the strand being stressed, with electronic control stopping the stressing operation when forces were equalized. As strands were added, the weight of the HDPE sheathing was shared equally by the strands, slightly reducing the force in the reference strand. Additional deformations of the deck and tower reduced the force in the reference strand as each subsequent strand was stressed. When the stressing operation was complete, the force in each strand multiplied by the number of strands equalled the calculated installation force. The final dead load force in the stays ranged from 1,330 to 7,100 KN, while the live load force ranged from 60 to 1,330 KN.

The assessment of the potential for cable vibration, considering the use of coextruded HDPE pipe with a spiral bead to reduce rain/wind vibration and ungrouted stay cables, resulted in the need for cross-ties to

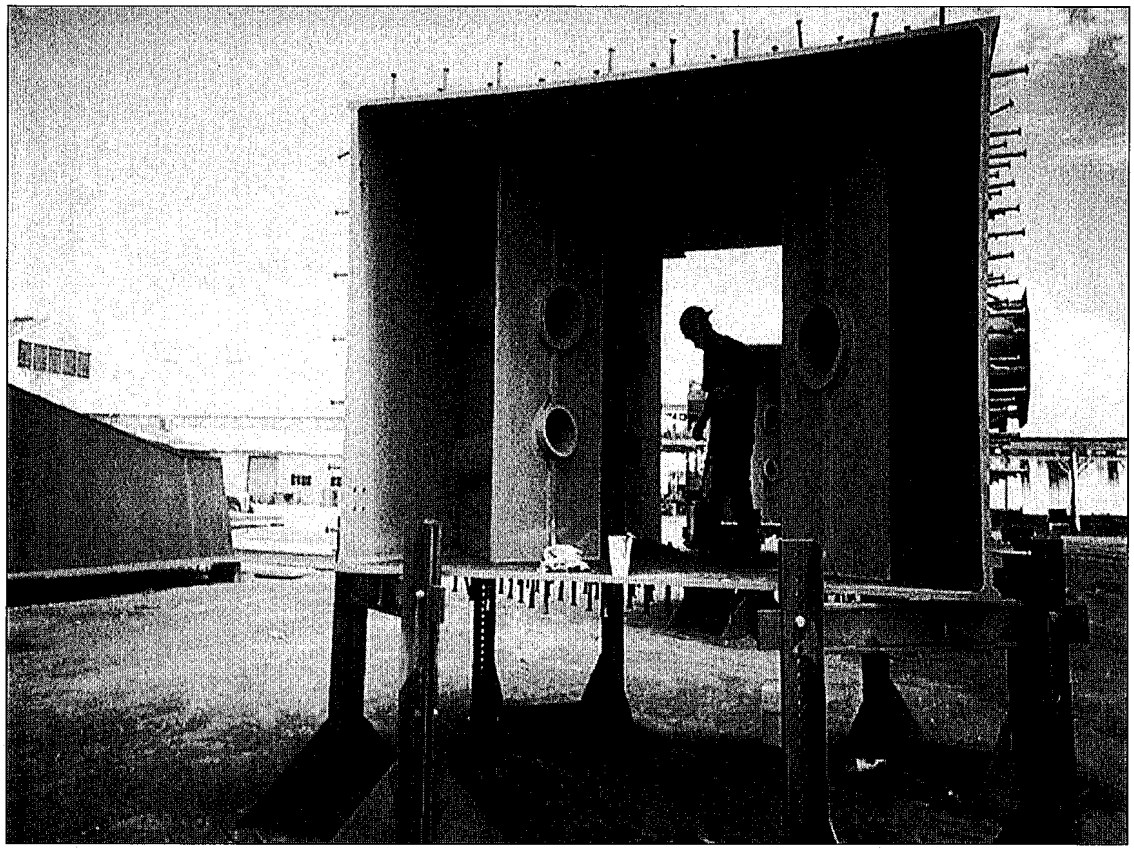

FIGURE 11. Tower anchorage unit on its side.

offset any potential galloping. After a study of the viscoelastic dampers was conducted, it was concluded that providing dampers at all lower anchorages, coupled with a cross-tie arrangement for longer cables, would best meet project needs.

Cable Anchorage at Towers

The vertical leg at the tower top varied from 4.9 meters at the base to 3.2 meters square just beneath the peak. Because of the limited room to anchor cables, a prefabricated steel anchor box was built into the tower, acting compositely with the exterior concrete by means of shear connectors. The cables were anchored by bearing at the inner end of structural pipe sections built into an anchorage stiffener (see Figure 11). This detailing offered the following advantages:

- reduced torsional moment due to closer transverse spacing of the cables;
- improved geometry control of the cable anchorages;
- elimination of complicated forming of the inside walls;
- elimination of post-tensioning in the tower cross section; and,
- ready access for inspection and maintenance.

However, torsion in the tower leg due to the cantilevered ramp on one side posed a challenge. The eastside cables of the main span carried a 30 percent greater load than the westside cables. During the preliminary design stage, the induced torsion due to the cantilever ramp on one side was overcome by offsetting the back span cables by 140 millimeters as they entered the tower. However, during the final design, the dead load torsion was overcome by rearranging the location of the main span cables as they entered the tower. The back span cables were kept at the center-

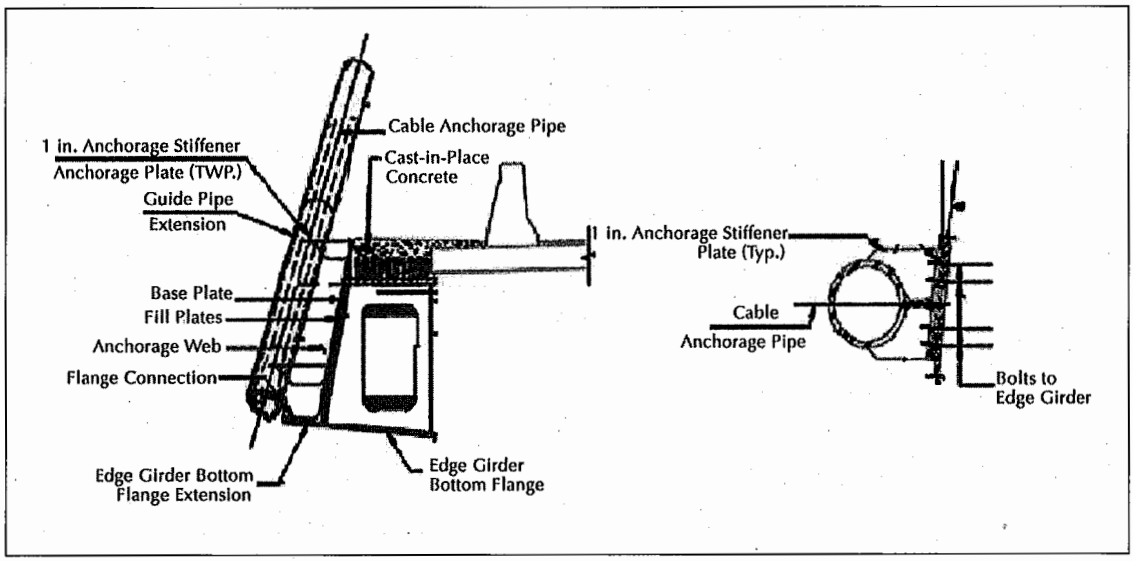

FIGURE 12. Stay cable pipe anchoring to the steel edge box girder.

line of the tower. In addition, medium weight concrete (density equal to 2,000 kilograms per cubic meter) for the deck slabs of the cantilevered ramp was used to reduce the torsional load. To avoid external cable anchorages and their related maintenance issues in the inclined legs of the tower, a non-uniform cable spacing scheme was adopted.

Pipes that were part of the prefabricated anchor box were custom formed by a specialist fabricator for a variety of cable pipe sizes (ranging from 275 to 430 millimeters in diameter and from 20 to 40 millimeters in thickness).

The steel tower box was fabricated in several sections with high-performance steel (ASTM A709, Grade HPS 70W), with horizontal field splices. The contractor requested additional splices from those shown on the contract plans to accommodate the lifting capacity of their tower crane. Unfortunately, the first shipment of three sections was not unloaded from rail to truck in Framingham (located 32 kilometers west of the project site in Boston) and driven as planned to the bridge site. As a result, the rail car and sections got jammed under the Southampton Street Bridge near downtown Boston. An expert metallurgist advised that minor damage could be repaired by heat straightening and that severely bent sections could be repaired by cutting and splicing in new material. The damaged sections were trucked back to the fabrication shop in Colorado and repaired.

Girder-to-Cable Anchorages

The cable anchorages on the main span box edge girders were mounted on the outside webs and were detailed as a pipe assembly bolted to the side of the girder (see Figure 12). The cables were then passed through the anchor pipe, with the cable anchor bearing against the lower end of the pipe. The pipe was connected to a base plate with a single web plate. This detail was selected due to its visual appeal over typical box-type cable anchorages, fabrication and erection considerations, and because it allowed easy access for inspecting all critical welds and bolts. High-performance steel (ASTM A709, Grade HPS 70W) was also selected for the cable anchorage pipe connections to the edge beams. Finite element design showed high tensile stresses would have resulted in pipe thicknesses too problematic for fabrication if lower grade steel were used.

Pipes were custom formed for a variety of cable pipe sizes (ranging from 275 to 420 millimeters in diameter and from 20 to 30 millimeters thick). Fabrication involved forming the pipes by progressively bending flat plate

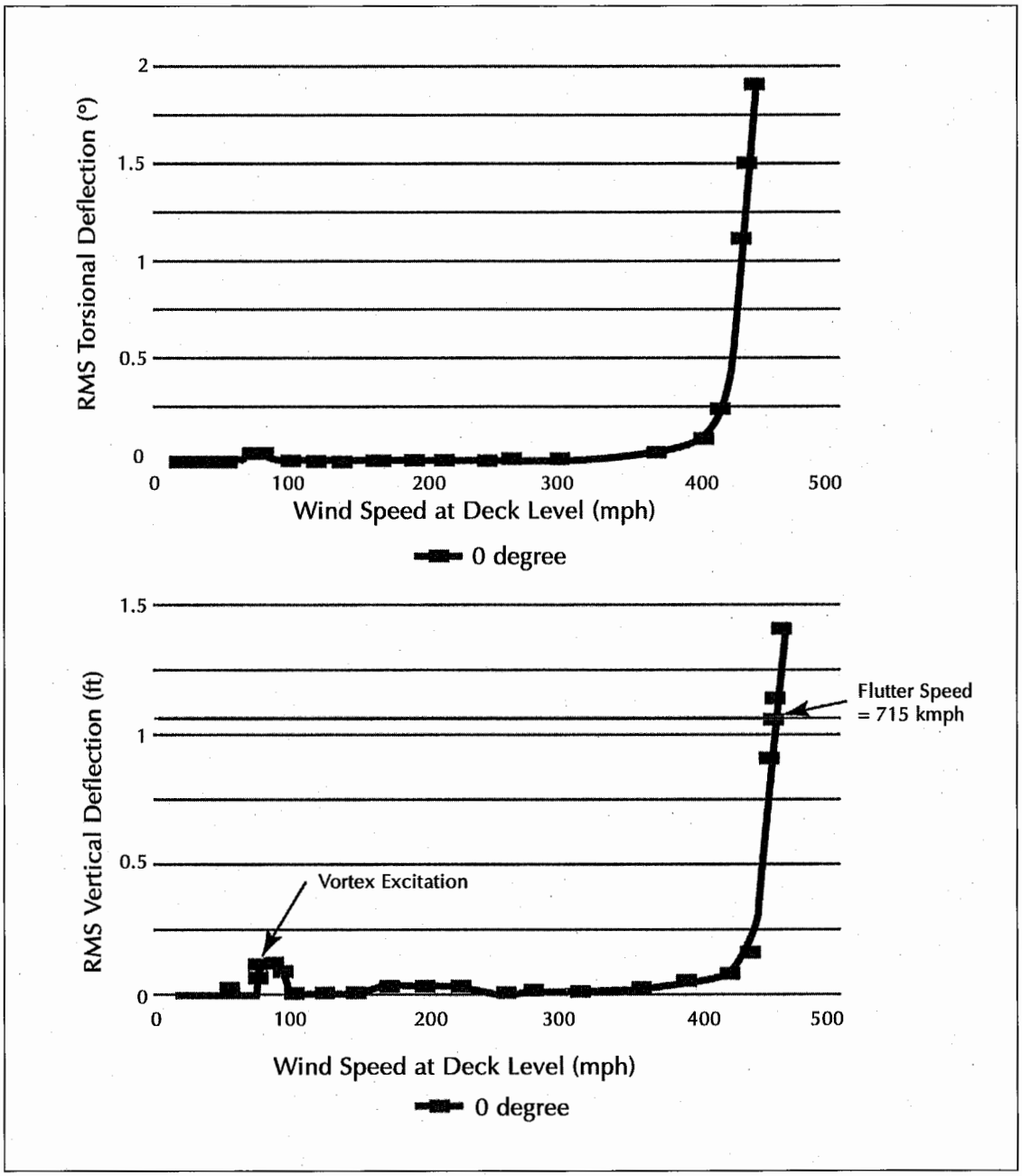

FIGURE 13. Results of wind tunnel testing.

stock through rollers. The partially completed pipe sections were shipped to the fabricator, where single-bevel groove welds were made along the seams, using ceramic backup bars. Radiography was used to confirm the quality of the initial welds. Subsequently, once the selected weld procedure was proven, ultrasonics (UT) was adopted for quality assurance of the remaining welds. One hundred percent UT testing was specified for the lower 600 millimeters of the pipes that were in tension, while random 25 percent UT testing was performed for the remaining welds in compression. Completion of the girder-to-cable anchorages involved welding the pipes to stiffened single web plates, which in turn were

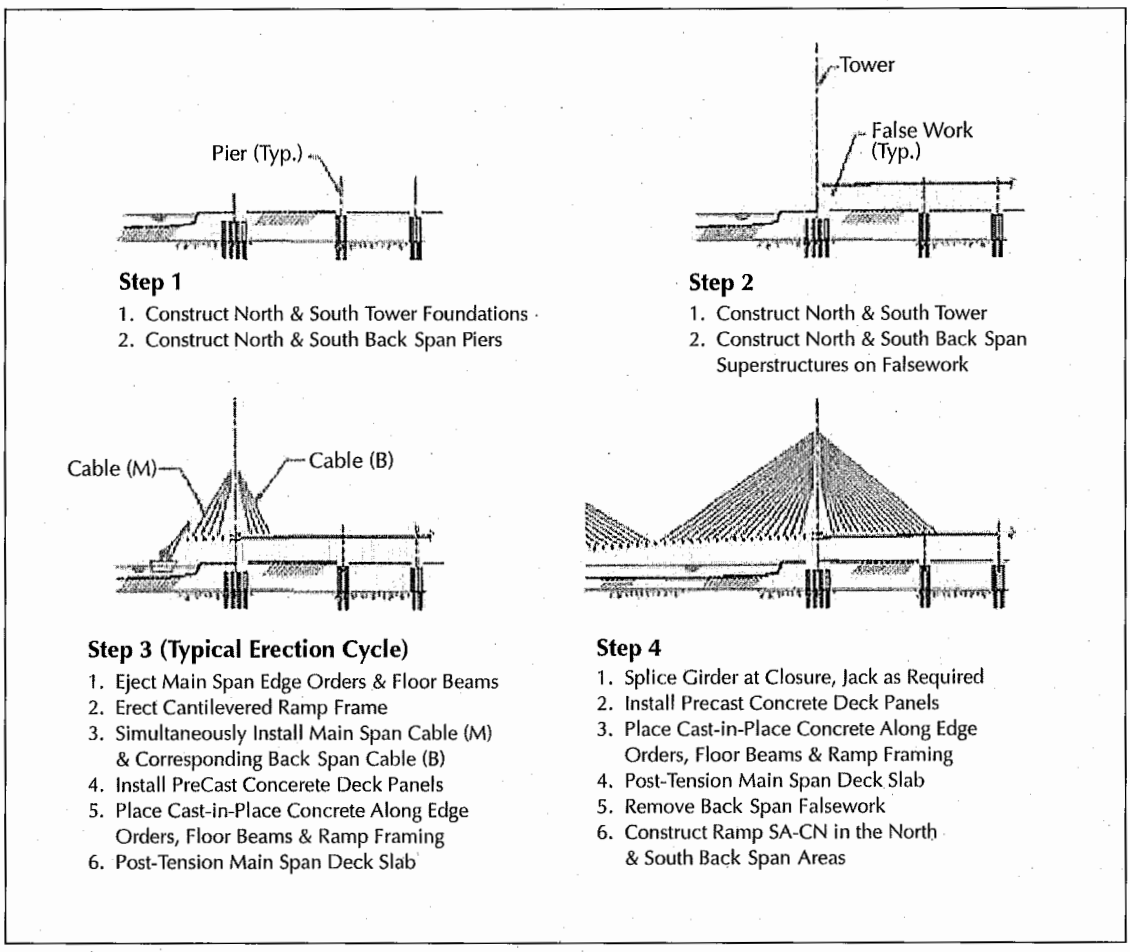

FIGURE 14. Sequence of bridge construction.

welded to connection plates bolted to the webs of the box edge girder.

Aerodynamic Evaluation

Wind tunnel tests of both the sectional and aeroelastic models were performed for the final structure as well as for intermediate construction stages. Vortex excitation occurred at about 128 kilometers per hour (kmph), which was within criteria. Flutter speed was measured at 715 kmph, well above the requirement of 210 kmph (see Figure 13). Smoke flow visualization tests also indicated that wind flows were not significantly altered by changes to the deck section, such as deck openings and open mesh closure panels on the underside.

Construction

The cast-in-place back spans were constructed on falsework concurrently with tower construction. The tension strut at the tower piers was post-tensioned in stages. Afterwards, the superstructure of the main span was erected in a cantilever fashion (see Figures 14 and 15).

The main span of the bridge consisted of thirteen field sections. Twelve field sections were erected by cantilevering from the two towers, with one field section connecting the cantilevers at mid-span closure. A typical field section was 18 meters in length and comprised two longitudinal box edge girders, three transverse floor beams, one longitudinal edge beam and three transverse cantilever beams. All steel met AASHTO M270, Grade 50, specifications. All splices and connections were field-bolted using 22-millimeter-diameter mechanically galvanized ASTM A325 bolts. Installation of the stay cables, their first-stage stressing, pre-

FIGURE 15. Bridge nearing completion, after mid-span closure looking from west to east.

cast concrete slabs, infills between precast slabs, second-stage stay cable tensioning along with post-tensioning of the deck slabs, followed in a sequential procedure.

Conclusion

The Leonard P. Zakim Bunker Hill Bridge is a unique structure featuring many engineering innovations and breakthroughs. The concept of a hybrid cable-stayed bridge (composite steel main span, concrete back span), use of ungrouted stay cables, use of the novel method to stress the cables, use of high-performance steel for the cable-stay anchor boxes in the towers and for anchorages at the superstructure level, and use of viscoelastic dampers (to name just a few innovations) all extended cable-stayed bridge technology in the United States.

Construction of the bridge started in September 1997. The total cost of the bridge, including change orders and savings derived from value engineering, was approximately $105 million. The superstructure mid-span closure of the main span was accomplished in May 2001. The bridge is scheduled to open to traffic in three phases: I-93 northbound opened in December 2002, I-93 southbound opened in December 2003 and the cantilevered ramp on the east side will open in December 2004.

ACKNOWLEDGMENTS — *The owner of the project was the Massachusetts Turnpike Authority; management consultant was Bechtel/Parsons Brinckerhoff (joint venture); bridge concept was developed by Dr. Christian Menn; the bridge type study and preliminary engineering were conducted by Parsons Brinckerhoff; final design of the bridge was performed by HNTB, Inc., with FBE as a major subconsultant; construction management was supervised by Bechtel/Parsons Brinckerhoff; contractor for the project was Atkinson/Kiewit (joint venture); the cable-stay and post-tension supplier was Freyssinet; structural steel was supplied by Grand Junction Steel; and wind tunnel studies were performed by Rowan Williams Davies & Irwin (RWDI). Freyssinet supplied the viscoelastic dampers. Greased and sheathed strands were used for the cables. Freyssinet developed and*

used the Iso-tension method of stressing ungrouted stay cables. Pipes were custom formed by specialist fabricator Houston Blow Pipe of Houston, Texas. An earlier version of this article was previously published in Structural Concrete (Journal of the fédération international du béton), *Vol. 3, No. 4, December 2002. Reprinted with permission.*

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