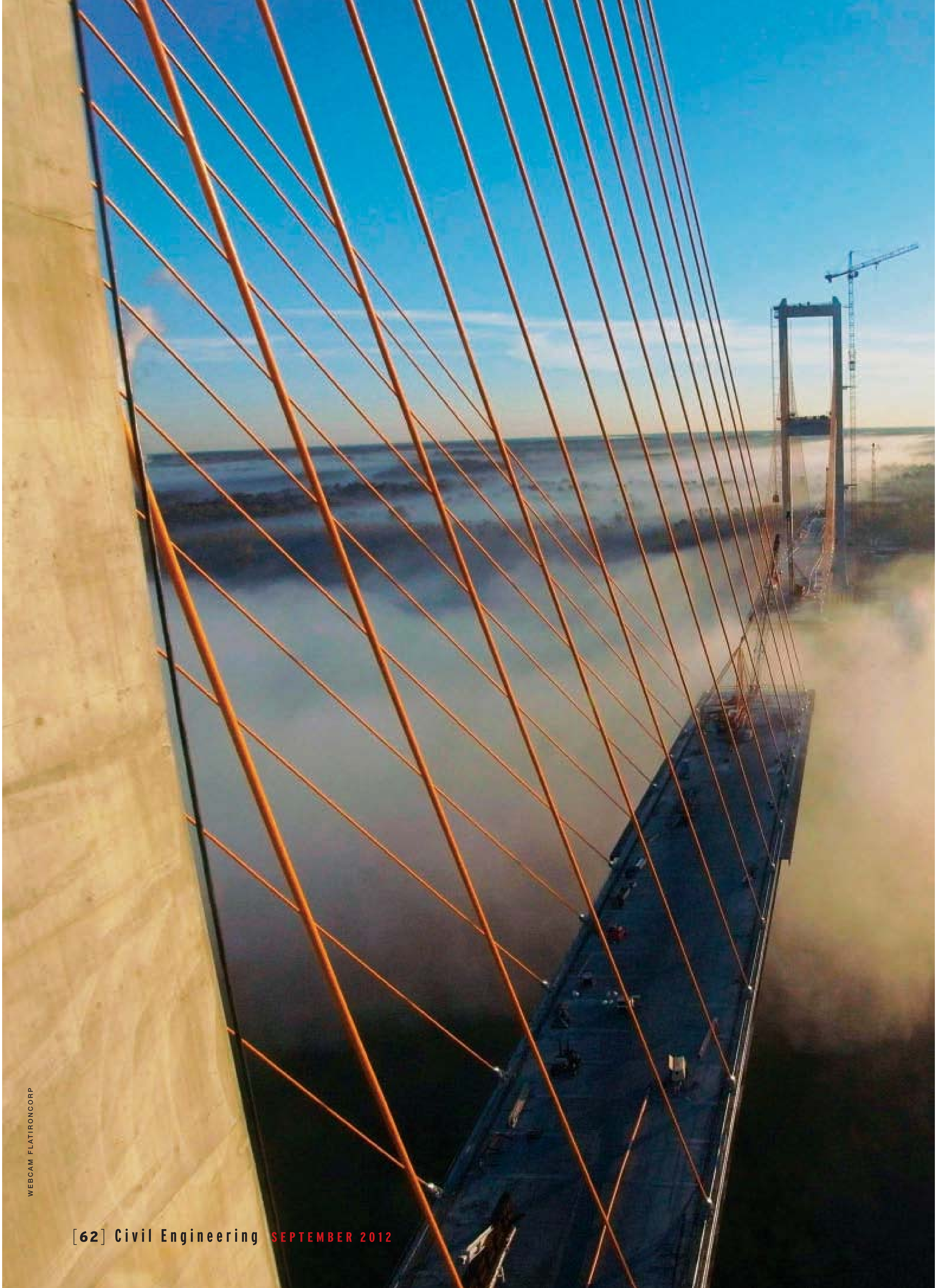

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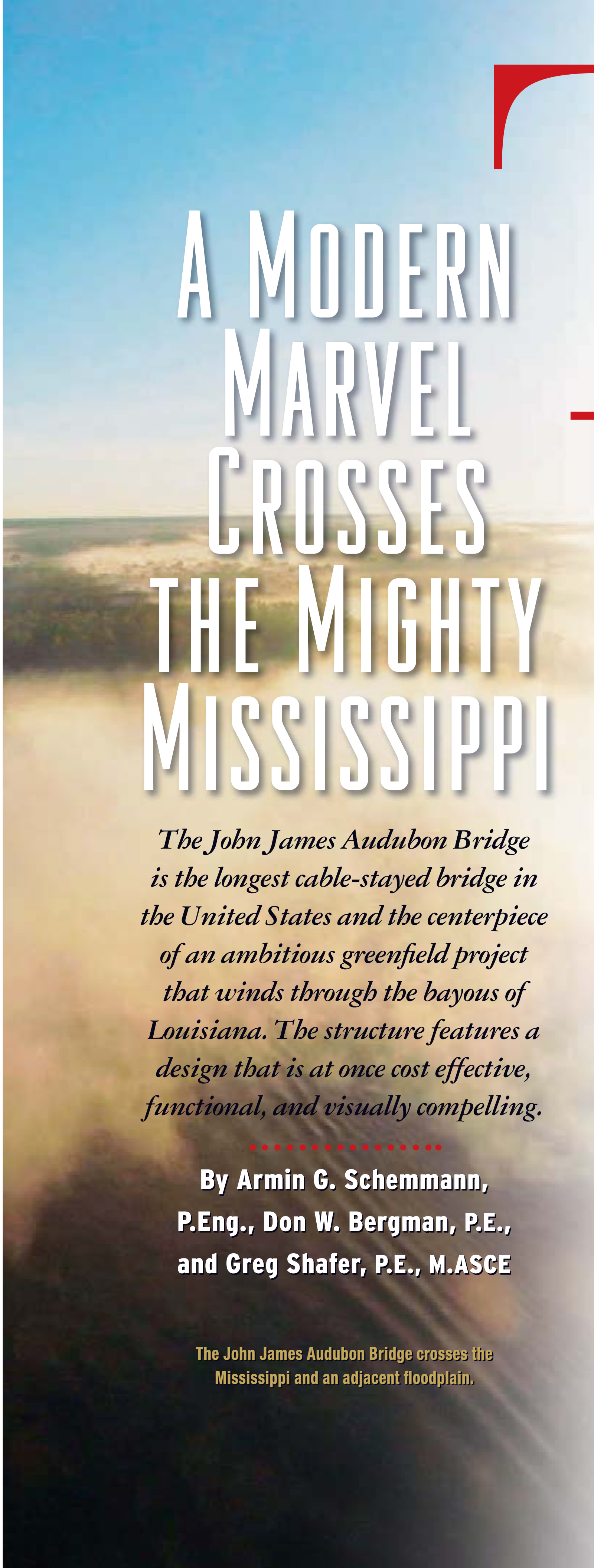
A Modern Marvel Crosses the Mighty Mississippi

The John James Audubon Bridge is the longest cable-stayed bridge in the United States and the centerpiece of an ambitious greenfield project that winds through the bayous of Louisiana. The structure features a design that is at once cost effective, functional, and visually compelling.

By Armin G. Schemmann, P.Eng., Don W. Bergman, P.E., and Greg Shafer, P.E., M.ASCE



The John James Audubon Bridge crosses the Mississippi and an adjacent floodplain.

THE JOHN JAMES AUDUBON Bridge is the longest cable-stayed bridge in the United States. The 2.3 mi long crossing carries Highway 10 over the Mississippi and an adjacent floodplain to connect the Louisiana towns of St. Francisville, on the east, and New Roads, on the west. Distinguished by an ambitious design that decidedly balances form and function, the bridge is the centerpiece of a $408-million greenfield project that includes seven other bridges and extends 11.8 mi through the wetlands and cane fields that surround the river.

The cable-stayed bridge is named in honor of the world-renowned artist, ornithologist, and naturalist, who painted no fewer than 80 of the famous works in his Birds of America series, a collection of life-size illustrations, while residing in St. Francisville in 1821. Audubon is highly regarded throughout the Pointe Coupee and West Feliciana parishes on either side of the river. Completed in 2011, the 3,183 ft long Audubon bridge comprises a 1,583 ft long main span, 640 ft long side spans, and 160 ft long flanking spans. The main span affords a 1,463 ft wide and 65 ft tall navigational channel to accommodate the large volume of barge traffic that plies the river. The deck is supported by 136 parallel-strand stay cables arranged in a traditional semiharp pattern in two nearly vertical planes. The two concrete H-frame main towers, 1E and 1W, are founded on 8 ft diameter drilled shafts installed to a depth of 180 ft in the riverbed. Anchor piers, 2E and 2W, located on either side of the main towers, comprise twin 8 ft diameter reinforced-concrete columns extending from drilled shaft foundations. Struts between the columns at the water level provide the necessary strength to resist ship impacts.

The bridge deck is a composite steel section comprising 6 ft deep edge girders that accommodate four traffic lanes. The deck is typically 76 ft wide, the exception being the center half of the main span, where additional wind fairings widen the deck to 85 ft and provide a thin leading edge to the deck section for enhanced aerodynamic stability. Cable anchorages are spaced 45 ft 9 in. apart, and floor beams span between edge girders that are spaced at 15 ft 3 in. intervals. The deck is vertically and transversely supported at all piers. Vertical and transverse support at piers 1W and 1E is provided by elastomeric bearings. Pot bearings are located at piers 2W, 2E, 3W, and 3E. The pots are transversely guided at piers 3W and 3E to allow longitudinal movement from temperature changes. Shear keys provide transverse support at piers 2W and 2E at the midspan of the floor beam, which engages the pier cap.

The deck is free sliding at all of the piers except 2W, 1W, and 1E. Longitudinal deck fixity is asymmetric, rigid links at the west tower and lockup devices at the east tower providing restraint. Longitudinal fixity at 2W increases the stability of the relatively slender pier columns. Uplift forces from the backstays are resisted through the combined effects of the flanking span weight, the deck counterweight over the anchor piers, and the uplift restraint brackets attached to the anchor piers. The counterweight was sized so that the pier would not experience net uplift under the service limit state. The uplift restraint brackets are designed to hold down the edge girder bottom flange when uplift forces approach the ultimate strength limit state.

The bridge is located approximately 75 mi from the Gulf of Mexico along flat terrain that experiences moderate to high winds and occasionally winds of hurricane force. Wind was one of the key factors in the design of the towers and in determining deck shape, deck articulation, and cable dampers. The design required the application of a 100-year return period for wind at the deck level with a mean hourly wind speed of 86 mph for calculation of design forces. It also required a 10,000-year return period for flutter stability (equal to a 10-minute mean wind speed of 117 mph). As a result of the compressed design and construction schedule, wind data for the site were obtained through wide-scale climate analysis using the Weather Research and Forecasting Model to correlate data from local meteorological stations to the project site rather than wind measurements at the site or other estimating techniques. (The model, a computer program for forecasting and studying weather, was developed by the National Oceanic and Atmospheric Administration and the National Center for Atmospheric Research, along with more than 150 other organizations and universities around the world.) Aerodynamic stability of the structure for flutter- and vortex-induced oscillation was confirmed by sectional wind tunnel tests carried out on the deck section of the main span. Aerodynamic response for buffeting of the completed and partially erected bridge was verified by wind tunnel testing of the full aeroelastic models.

Wind demands on cable-stayed bridges typically arise from three fundamental effects: mean (static), background (gust), and inertial (buffeting). The mean and background components for this bridge were calculated through knowledge of characteristics pertaining to the site. Inertial effects were obtained from buffeting analyses, which provide peak inertial displacements for the fundamental modes of vibration. Total wind load forces were obtained by modal superposition, whereby the inertial effect of each mode is combined with the mean and background components using the root sum of squares. A wind tunnel test was conducted to confirm the results. Variability in wind direction was considered by simultaneous application of wind load effects in the transverse and longitudinal directions. The magnitude of coexisting transverse and longitudinal effects was determined from the results of the aeroelastic testing.

Bridges across the lower Mississippi are typically founded on dredged caissons. This type of foundation is commonly used because it has the ability to distribute large loads from the columns or towers to the deep bearing soil. The overlying

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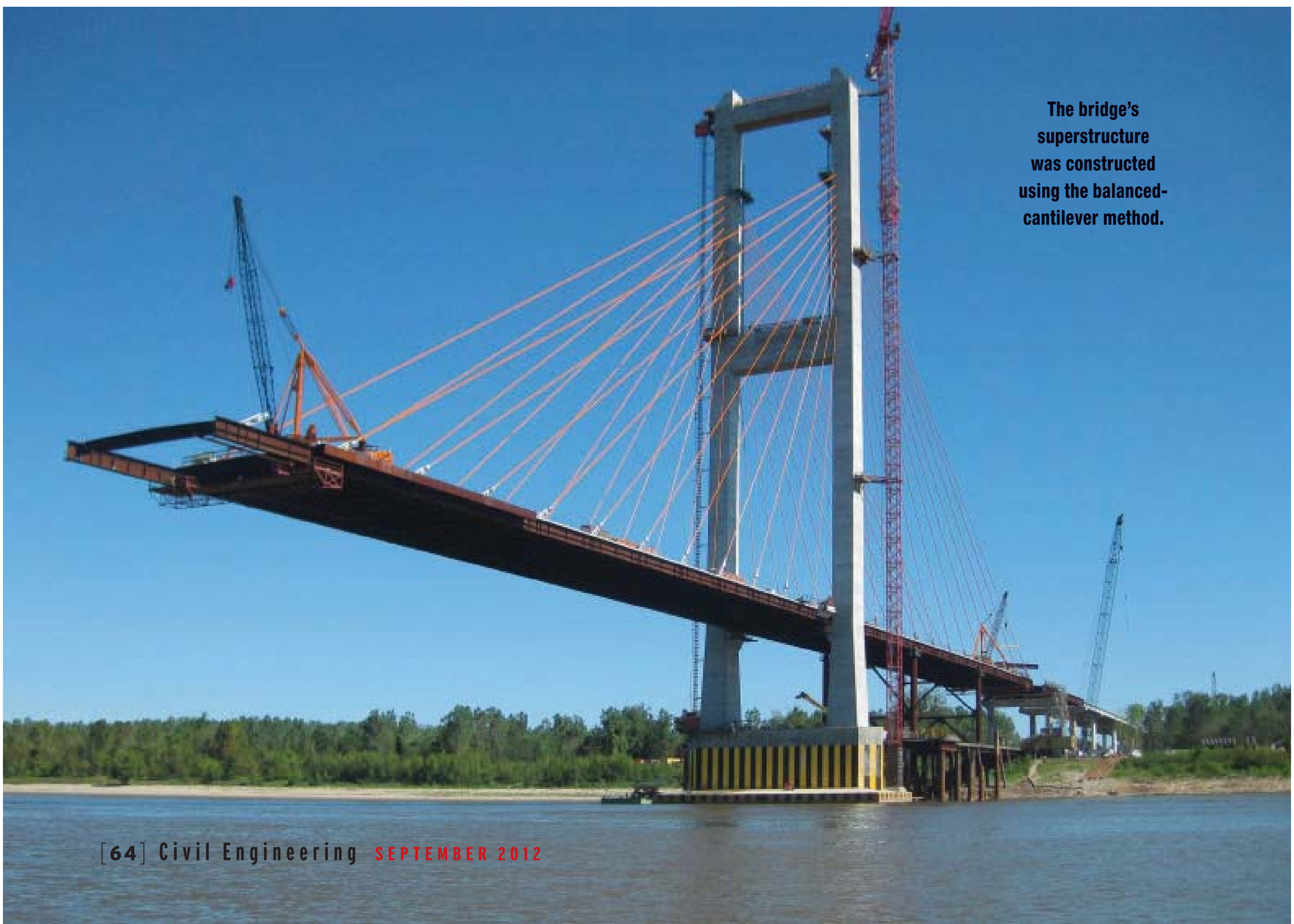
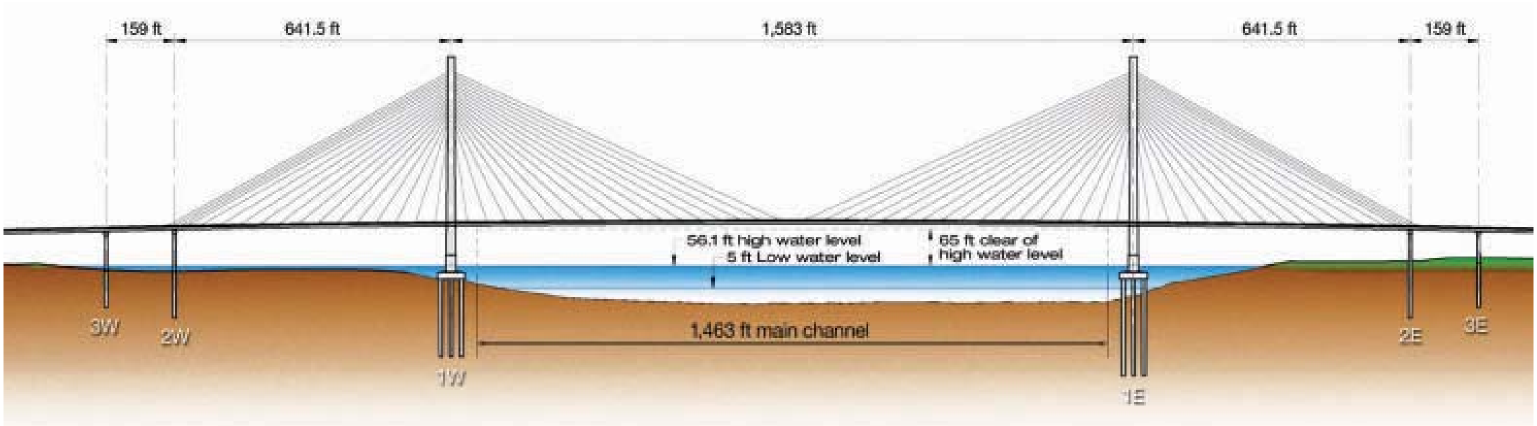


The bridge's superstructure was constructed using the balanced-cantilever method.



JOHN JAMES AUDUBON BRIDGE ELEVATION

soil in the region is soft, and driven piles are impractical because of the lack of lateral support in the upper regions. However, sunken caissons present such inherent risks as limited control of geometry and lack of flexibility. Other challenges include working at depth with pressurized air and handling the delivery of large precast-concrete elements. The request for proposals allowed the bidders on this project to use either dredged caissons or drilled shafts. Given the particular conditions of the bridge site and the risks that would be encountered, the design/build team selected drilled shafts.

Each tower is supported by 21 drilled shafts measuring 8 ft in diameter. The shafts are joined by an 18 ft thick reinforced-concrete pile cap that runs 160 ft along the longitudinal axis of the bridge and 64 ft transversely. The bottom of the pile cap is located at minimum low water elevation, 5 ft above mean sea level, to prevent vessel impact on an individual shaft. A pedestal that measures 24 ft in the longitudinal axis of the bridge and 140 ft transversely extends 5 ft above the maximum high-water level to 61 ft above mean sea level to prevent vessel impact on the hollow sections of the towers supported above. The shafts themselves are not conventional and required sophisticated tools and procedures for construction. Each shaft comprises two distinct elements: the lower portion is of conventionally reinforced concrete, and the upper portion is reinforced with an integral steel casing. The casing in the upper portion of the shaft serves as a form for the concrete when the shaft is placed in open water or soft soils; it also serves as a load-carrying element to enhance the lateral capacity of the shafts.

The John James Audubon Bridge deck is supported by 136 parallel-strand stay cables arranged in a traditional semiharp pattern.

BUCKLAND & TAYLOR LTD., *TOP;* PHOTO BY JOE DUNN, COURTESY OF AUDUBON BRIDGE CONSTRUCTORS, *BOTTOM*

The lateral design of the shafts was dictated by the possibility of vessel impact. In anticipation of the combination of two concurrent extreme events, the shafts were designed for the maximum anticipated vessel load in combination with half of the 100-year design scour depth. Live load and wind were combined with the full 100-year design scour depth. To resist the high vertical loads on the drilled shafts—a factored resistance of approximately 5,000 tons—the shafts were extended deep into the dense sand layer at the bottom of the Mississippi. A conventional drilled shaft of this type would normally develop resistance primarily through skin friction. However, skin friction would have required additional length that would have extended the shaft into a hard clay layer that had significantly less bearing capacity. Therefore, the tips of the shafts were kept in the dense sand layer and a "tip grouting" technique was used to enhance the end bearing capacity. By injecting the tip of the shafts with pressurized cementitious grout after the shaft concrete had cured, the soil at the tip was effectively recompacted to restore the end bearing capacity lost by excavation of the shaft. The effectiveness of this tip grouting was demonstrated by the performance of several static load tests using an Osterberg cell embedded in the concrete. The final tip elevations of the shafts were 175 to 180 ft above mean sea level.

The drilled shafts were constructed using an oscillator system that installs a temporary casing to the drilled shaft tip elevation by twisting the casing while applying downward vertical pressure. The soil near the interior of the casing was excavated using several apparatuses, including a submersible pump, an air lift, and an auger. The air lift received the most use on this project. Once the temporary casing was installed and the soil excavated to the tip, the reinforcing cage with the tip grouting apparatus was installed. A tremie pipe was used to place the drilled shaft concrete underwater while the temporary casing was progressively removed.

The pile cap that ties the drilled shafts together is made of conventional reinforced concrete. The cap had to be constructed inside a cofferdam because the river's water level varied greatly throughout the project. The cofferdam system comprised precast-concrete elements joined to form a box large enough to accommodate the outside dimensions of the

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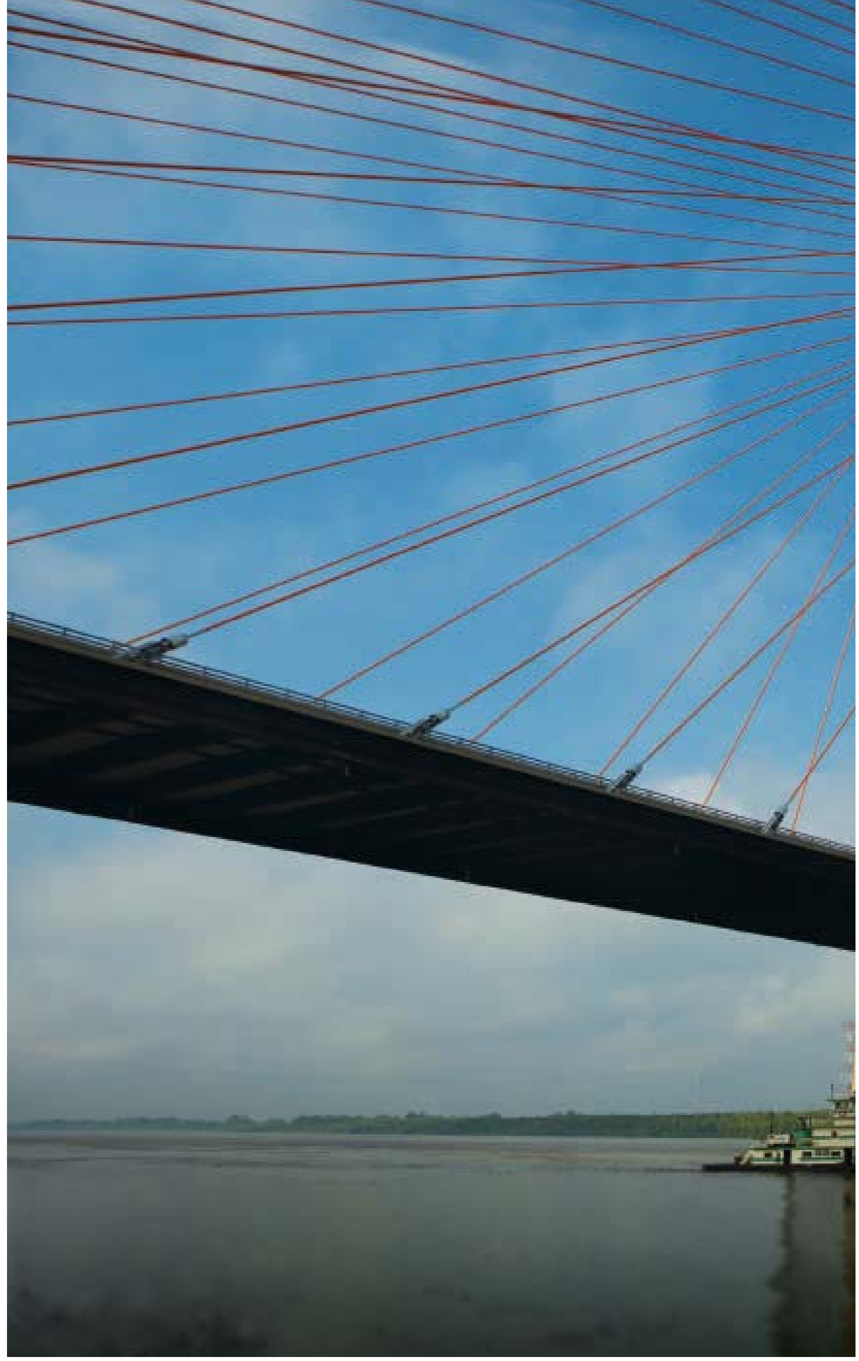
pile cap. The box became a permanent part of the structure. A temporary steel follower constructed of sheet-pile elements extended the height of the cofferdam to the required elevation, allowing continuous construction during river levels up to 48 ft above mean sea level. A substantial steel frame provided lateral support so that the cofferdam box and follower could resist large amounts of hydrostatic pressure. The entire cofferdam was erected on the completed drilled shafts at an elevation well above the anticipated river level. Once the box was constructed, a hydraulic system was used to lower the entire cofferdam over the drilled shafts. The cofferdam was then sealed with a concrete slab and dewatered. The hydraulic lowering system was developed expressly for use on this project to evenly lower the cofferdam without overloading the elements. It took 31 workers approximately one month to complete the lowering process.



The towers are a dominant element of the cable-stayed structure. This status derives not only from their prominence on the landscape but also because of their effect on the overall construction schedule and cost. The primary objective of the conceptual design was to provide a highly constructable tower that would satisfy all of the technical, cost, and scheduling requirements. The final tower design was developed during the prebid design phase and evolved through progressive refinement of no fewer than 12 concepts. Recognizing the benefits conferred by vertical tower legs in the area of constructability, an H-frame arrangement with a single crossbeam above deck was the initial choice. This decision was based on the fact that the vertical legs would have allowed efficient jump form construction with minimal requirements for geometry control while also permitting significant overlap between the tower and deck construction. Moreover, the crossbeams would have stiffened the towers in the transverse direction, thereby reducing bending demand in the towers' legs, and the below-deck crossbeam could have been positioned to provide clearance for an underslung maintenance traveler.

That concept was briefly undermined, however, when the dynamic analysis of the design revealed that the ratio of the fundamental deck torsional frequency (ω_t) to the vertical natural frequency (ω) was less than 1.58. The deck has a width to depth aspect ratio of 12.7. Therefore, the low ω_t/ω ratio was expected to result in a relatively low critical wind speed of 116 mph, the speed at which the onset of aerodynamic instability could be expected. The fundamental torsional frequency had to be increased to address aerodynamic stability requirements. A delta leg tower was considered because it would have dramatically increased the ω_t/ω ratio, to 2.22, with a predicted critical wind speed of a comfortable 172 mph. Although it may be more aesthetically appealing than the H-frame option, the delta leg option was dismissed as too costly to construct. It would have required more time to construct and afforded less opportunity to overlap the pier table and tower construction to keep the construction schedule as short as possible. Economics taking precedence over aesthetics, the most cost-effective solution was determined to be a modified H-frame tower design.

As a result of the limited benefit of the below-deck crossbeam, the early H-frame concept was modified by essentially relocating the below-deck crossbeams to the tops of the towers, where the beams would remain clear of the cable anchorages and provide the badly needed boost to the torsional frequency. The ω_t/ω ratio increased to 1.79, leading to an expected critical wind speed of 130 mph. The towers' legs and crossbeams were designed as hollow sections, the lower crossbeams partially posttensioned. Concrete diaphragms, which permit the passage of internal tower elevators, were provided at the leg-to-crossbeam interfaces of the towers, creating a robust joint design as well as inconspicuous anchorage of the lower crossbeam posttensioning within the tower legs.

PHOTO BY JOE DUNN, COURTESY OF AUDUBON BRIDGE CONSTRUCTORS

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The tower legs were moved outward to permit clear passage of the edge girders, which are supported on posttensioned-concrete corbels. The upper crossbeams provided an added benefit in that they resolved the lateral loads from the cable inclination, which was required to allow the deck to pass through the vertical tower legs, and from the cable eccentricity in the upper legs of the towers, which was required to accommodate the passage of the elevators and access stairways.

The interface between the deck and towers proved to be one of the more challenging details to design and construct. An underdeck crossbeam would have accepted longitudinal loads directly from the edge girders, but underdeck corbel supports were used instead. Since the corbels could not accommodate the longitudinal shear from the girders, the longitudinal shear had to be delivered through posttensioned bearing brackets cantilevering from the deck and engaging each of the tower legs. Fixed linkages at pier 1W and lockup devices at pier 1E that work in compression and tension provide longitudinal fixity between the deck and the towers. Several components tying to the restraints are posttensioned to provide effective load paths under conditions of stress reversal and cyclic loading. Posttensioned diaphragms were installed in the tower legs at the deck level to prevent the cross sections from warping unacceptably as a result of the eccentric longitudinal deck shear. To improve constructability, the towers were designed so that the crossbeams, diaphragms, and corbels could be cast after the passage of the basic tower leg jump forms.

The bridge's cables are anchored in pairs using steel anchor trays, or boxes, cast into the walls of the tower legs. The anchor trays serve as tension ties for opposing cables and are secured by shear studs located on three sides of the box. The eccentric positioning of the anchor trays serves a twofold purpose: it provides space for the tower elevators and offers a direct load path for unbalanced cable forces into the longitudinal walls of the towers. The bridge is designed to handle unbalanced cable forces in the event of sudden accidental cable loss. Light box-shaped, stay-in-place forms, which were lifted into place by a tower crane, provide continuity between the anchor trays. The steel forms served as a construction aid, allowing up to four anchor boxes to be preassembled and erected together. The connection between the sections is typically a simple butt plate splice, which is accessible and bolted from the inside of the tower sections. Three adjustable splices are located over the height of the anchorage zones to correct the geometry of the stacked anchorages. The continuity between the anchor trays and the stay-in-place forms allowed the complete cable anchorage area to be fabricated in the shop, maximizing geometric control of this critical element.

The superstructure was constructed using precast-concrete deck panels placed on a steel grid. The deck panels were made composite with the steel grid by casting concrete infill strips over the floor beam and edge girder flanges. The deck panels are $9\,^1/_2$ in. thick and made of *(Continued on Page 85)*