TESTING TOMORROW'S TURBINES

The recently completed Wind Technology Testing Center, in Charlestown, Massachusetts, is the largest facility in the world for testing wind turbine blades. Featuring three posttensioned-concrete stands supported by a massive reaction footing on drilled concrete shafts, the center is capable of testing blades up to 90 m long and is expected to greatly facilitate efforts to increase wind power.

By Eric M. Hines, Ph.D., P.E., M.ASCE, and Mysore V. Ravindra, P.E.

THE PAST TWO DECADES have witnessed a proliferation of wind turbine developments, or wind farms, both here and abroad. In keeping with this trend, the U.S. Department of Energy has established a goal of generating by wind 20 percent of the energy consumed domestically by 2030. As domestic energy producers seek to meet this goal and wind farm companies seek to expand overseas, demand is rising for turbines that are larger and capable of producing more energy than ever before. Although 2 to 5 MW per turbine is the current standard, such machines are expected to have a capacity of 10 MW or more during the next 10 years. To accomplish this goal, the turbines themselves must be larger and have longer blades and more robust foundations to support the larger forces they will have to withstand (see "Supporting the Winds of Change," *Civil Engineering,* August 2010, pages 76–85, 94).

But prior to the construction of any new wind turbine, the blades must be tested to ensure that they can endure the forces that will act on them as they operate. Since 1993 the Department of Energy has relied primarily on the National Wind Technology Center—which is located near Boulder, Colorado, and is managed by the National Renewable Energy Laboratory, based in Golden, Colorado—to conduct such testing for blade manufacturers. However, in 2007 the department announced that, even with improvements to the facility, other facilities would be needed to test the larger turbine blades demanded by power companies. Ideally, any new testing centers would have access to a deepwater port to accommodate the larger blades planned for offshore wind farms.

So it was with great excitement that Massachusetts's governor, Deval Patrick, dedicated the Wind Technology Testing Center (WTTC), just north of Boston in Charlestown, on May 18. Located on the grounds of a terminal owned by the Port of Boston, the 47,000 sq ft facility is the largest of its kind in the world and has the capacity to test up to three 70 m long wind turbine blades simultaneously or a single blade up to 90 m in length. It was built to handle blades with lengths up to 1.8 times those handled by the facility in Colorado and with bending moments greater by a factor of more than 5. Funded in part by the Department of Energy through the American Recovery and Reinvestment Act of 2009 and recognized by the White House as one of the act's top projects, the facility is now operated by the Massachusetts Clean Energy Center, a semipublic agency established by the state to accelerate job growth and economic development in the clean energy industry. The laboratory structure was designed by LeMessurier Consultants, Inc., of Cambridge, working with Architerra, Inc., of Boston—the project architect—under a contract with the Massachusetts Port Authority, which acted as the owner's representative for the Massachusetts Clean Energy Center.

The laboratory features three numbered (1, 2, 3) posttensioned-concrete test stands supported by a massive reaction footing on drilled concrete shafts. Individual wind turbine blades are mounted to the test stands as cantilevers and are subjected to both static and dynamic testing. Under this arrangement, the blades can undergo an unprecedented range of loading regimes in vertical, horizontal, and biaxial directions. The blades are mounted to the concrete test stands by means of 74-ton faceplates made of precision-machined steel.

Test stands 1 and 2, *opposite,* were designed as an integrated unit 64 ft wide, 36.5 ft high, and 12 ft thick, creating torsional capacity for horizontal blade testing and increased vertical capacity when the largest blades are tested alone. The test stands and reaction footing were designed with uniformly distributed posttensioning stresses in all three directions to prevent the structures from experiencing tension except under the highest stress loads. Given the large scale of the laboratory, the simplicity of the trussed enclosure can be appreciated for its lightness and sculptural quality. The most complicated joints for these truss frames were developed as elegant structural steel castings.

DAVID W. PEARSON, *OPPOSITE;* LEMESSURIER CONSULTANTS, INC., *RIGHT*

The laboratory is enclosed by 11 steel trussed frames, each 140 ft wide and 82 ft high. Fixed at the base and self-stabilizing in all directions, the three-dimensional frames are 7 ft deep and spaced 30 ft apart. To ensure that they would be able to carry substantial joint loads, facilitate fabrication and erection, and reduce costs, the most complicated joints for these frames were developed as elegant structural steel castings.

An essential structure in America's renewable energy industry, the building itself was designed to limit carbon emissions. The tight deadlines and exacting budget controls established by the American Recovery and Reinvestment Act of 2009 required that several construction packages be released earlier than usual and with minimal change orders allowed.

THE FACEPLATES TAKE THE FORM of 20 ft diameter steel plates 12 in. thick to which blades are mounted via threaded metric bolt circles. The size and thickness of the plates were such that the plates had to be fabricated and shipped in sections to the site. The

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fatigue stress concentrations at the bolt holes were studied in detail by means of three-dimensional finite-element analysis with nonlinear contact elements. Interaction between the threads and concentrations of flexural stress was mitigated by countersinking the threads themselves to a depth of 100 mm beneath the front surface of the faceplates. Each faceplate is posttensioned to its test stand with approximately 22,000 kips of force but is also designed to be demounted and replaced if future uses dictate or if the actual fatigue performance of the posttensioning bars declines during the life of the laboratory. For the safety of the laboratory personnel, special retainer systems were designed for the faceplate bars to dissipate the full elastic strain energy of a bar should a fatigue-induced fracture occur.

The faceplates are angled upward 6 degrees from the vertical on test stands 1 and 2 and 8 degrees from the vertical on test stand 3. These angles, together with pairs of shims that are angled at 3 degrees each, make it possible for the blades to be mounted at any angle up to 14 degrees. This wide range of angles is necessary to accommodate the range of blade types expected in the laboratory, which will vary significantly with respect to their curvature and the amplitude of the positive and negative displacements caused by bending.

The 6-degree shims are 22 ft tall reinforced-concrete wedges cast in place against the posttensioned test stands. The wedges were placed in a single lift of 22 ft from the top of the forms. The many horizontal ducts placed through the wedges for the faceplate posttensioning bars, combined with the 4 ft diameter center holes that provide access for inspecting blade interiors, resulted in a highly congested configuration that posed challenges during wedge placement. For this reason, the wedge concrete was specified as a self-consolidating concrete mix reinforced with steel fibers (200 lb of steel fibers per cubic yard of concrete). Steel reinforcement was provided only at the wedge perimeter.

The concrete contractor, Francis Harvey and Sons, of Worcester, Massachusetts, created two full-scale mock-ups onsite to test the mix with and without vibration. The mock-ups were constructed as 4 ft wide slices of the full wedge and were filled with obstructions similar to the actual wedges. Experience with these two mock-ups demonstrated the following: form vibration was necessary to avoid honeycombing near the surface of the concrete; consolidation of the mix was even and coherent with vibration; and the steel fibers were well distributed throughout the wedge with no apparent segregation. The adequate consolidation of the steel fibers with the mix at full scale was evaluated via visual inspection of cores taken from several elevations on the 22 ft mock-ups.

The posttensioned faceplates, test stands, reaction footing, and drilled shafts were designed to resist high cycle fatigue loads under 100 million cycles—orders of magnitude higher than most common criteria for components in the civil engineering profession. Special tests were conducted on 1.75 in. diameter, 150 ksi, cold-drawn posttensioning bar assemblies with nuts and couplers in order to establish the appropriate mean stresses, half-amplitude stresses, and endurance limits. Both the mean stress levels and the number of cycles for these tests were unprecedented.

Bar stresses and concrete stresses within the test stands were assessed with the help of three-dimensional finite-element models. The test stands and reaction footing were designed with uniformly distributed posttensioning stresses in all three directions to prevent them from experiencing tension, except at the highest stress concentrations. The absence of cracking in the test stands and reaction footing structure made it possible for reliable models to be run with elastic elements. The final model of the test stands included loads representing each bar so that stresses could be assessed in cases in which individual bars had been compromised during construction.

THE FOUNDATIONS OF THE STRUCTURE WERE DESIGNED AS A HYBRID SYSTEM WITH DRILLED CONCRETE SHAFTS SUPPORTING THE REACTION FOOTING AND STEEL PILES SUPPORTING THE NORTH AND SOUTH GRADE BEAMS.

The foundations of the structure were designed as a hybrid system with drilled concrete shafts supporting the reaction footing and steel piles supporting the north and south grade beams. There are no deep foundations supporting the diaphragm slab. With a thickness of 18 in., this slab covers the 105 ft width in front of the reaction footing and runs 256 ft between the reaction footing and the east grade beam. The reaction footing is supported by 15 drilled concrete shafts 4 ft in diameter and 170 ft long. The bottommost 10 ft of the shafts are socketed in bedrock, which here is moderately weathered argillite. Samples from borings at the site revealed that the average unconfined compressive strength of the argillite was roughly 2,000 psi. The site generally consisted of 53 ft of fill and organic materials above 105 ft of clay, which in turn was located above 2 ft of glacial till. To accommodate the fatigue loads and provide superior long-term performance, the concrete shafts were designed to remain in compression under the highest (110,000 kip-ft) overturning forces induced on the reaction footing. This condition results under proof testing of a 70 m blade on test stand 1, an 80 m blade on test stand 2, and a 70 m blade on test stand 3.

The reaction footing is posttensioned with 80 tons of 270 ksi cable comprising 19 strands, each 0.6 in. in diameter. This arrangement uniformly distributes compression forces of 720 psi in the east–west direction and 480 psi in the north–south direction, preventing the footing from experiencing tension. Vertical posttensioning from the test stands into the footing was enhanced by additional vertical posttensioning

The laboratory is enclosed by 11 steel trussed frames, each 140 ft wide, 82 ft high, and 7 ft deep. Spaced 30 ft apart and fixed at the base, the three-dimensional frames are self-stabilizing in all directions.

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bars with custom pocket formers that will enable the footing surface to act as a strong floor for the attachment of additional fixtures in the future if they should be required.

Site conditions dictated that all of the footing's posttensioning be lifted into place from above. Because of this requirement and the complicated system of horizontal cables and vertical bars, the contract documents for the footing described a detailed erection sequence that included the use of supplemental reinforcement to stabilize the posttensioning itself. The posttensioning elements were set into fixtures fastened within a tolerance of 0.125 in. to a stiff concrete working surface.

The 1,633 cu yd footing was placed continuously during a single day between 2 AM and 4 PM. The low-heat concrete, which was 50 percent fly ash, took many hours to begin setting, and its sticky consistency proved challenging to finish and required 12 additional hours of work after the placement was complete. The footing was then covered with insulating blankets in accordance with a detailed temperature control plan that was to be followed for 28 days after the placement. After the blankets were removed but before the posttensioning of the steel, no cracks were visible on the footing surface. A balanced sequencing of the posttensioning work between the footing and the test stands ensured a high level of compatibility between these integrated structural elements and minimized cracking in the test stands near their interface with the footing.

To delay the final posttensioning of the reaction footing while permitting construction of the laboratory floor and enclosure, a small section of the south grade beam and a 15 ft wide area of the diaphragm slab were left open to afford access to the posttensioning cable ends. Concrete was placed in these areas after the posttensioning was completed.

Test stands 1 and 2 were designed as an integrated unit 64 ft wide, 36.5 ft high, and 12 ft thick that creates torsional capacity for horizontal blade testing and increases the vertical capacity when the largest blades are tested alone. Test stands 1 and 2 were located near the rear of the reaction footing to provide additional weight on the west side in order to help counteract the greatest tensile forces from overturning. Test stand 3 was

located near the front of the footing so that large-scale component testing could proceed behind the stand without interfering with blade testing. In addition to the custom pocket formers described above, 331 steel pipes were cast into test stands 1 and 2 in the east–west direction so that objects other than wind turbine blades could be mounted and tested if required.

The reaction footing is structurally isolated from the rest of the laboratory to allow for a slight rocking under vertical fatigue testing regimes. However, horizontal testing requires the use of the laboratory floor as a diaphragm to stiffen the reaction footing in torsion. Maintaining structural separation in the vertical direction while providing integrated structural performance in the horizontal direction required special shear keys and bearing plates on the north and south ends of the reaction footing.

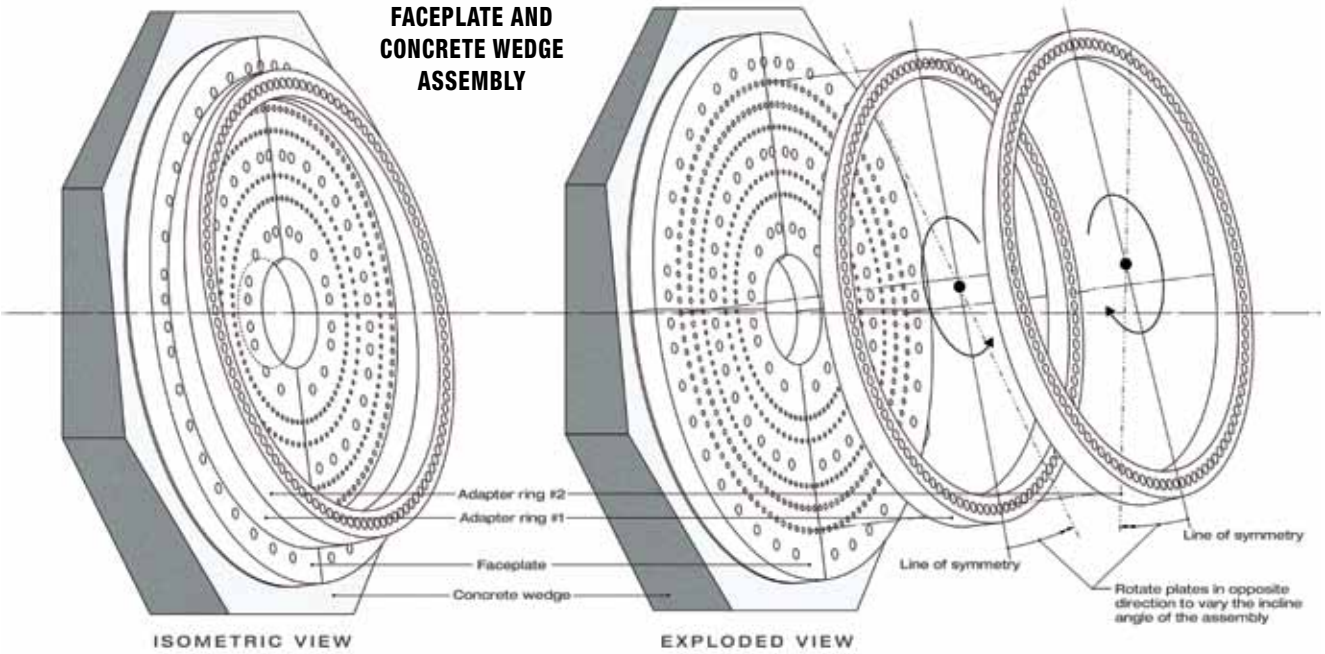
The shear keys were placed at the neutral axis of the reaction footing and shaft system so as to maintain structural separation in the vertical direction. Together with Teflon bearing plates embedded in the reaction footing, the shear keys create an interaction between the laboratory floor and the footing resembling that of a wrench and nut. The shear keys were designed to be replaceable should they become damaged by fatigue or by movement of the monolithic laboratory floor caused by long-term shrinkage.

The diaphragm slab was designed without deep foundations under the assumption that the slab could settle up to 2 in. during the 30-year life of the laboratory. This diaphragm slab provides a horizontal link between the reaction footing and the grade beams and can also accept postinstallation tie-downs for future test fixtures placed on a 1 by 1 ft grid. Reinforcing and splices for the diaphragm slab were placed with precision inside 4 in. wide lines in each direction, providing 8 by 8 in. clear areas spaced 1 ft apart on center. Since the locations of the reinforcement are tied to hard points in the laboratory, technicians will be able to install future tie-downs without having to scan the slab for reinforcement conflicts.

Adding office space outside of the main laboratory footprint would have required additional deep foundations. To avoid the expense of such foundations, the owner and the design team opted to lay out offices and mechanical spaces as three mezzanine floors between the trussed frames on the north side of the structure. Arranged around the trusses as sculptural elements, the offices afford panoramic views of the Mystic River outside while offering rich perspectives of the laboratory space.

The laboratory is enclosed by 4 in. thick insulated metal panels spanning between tube steel girts and a 7.5 in. thick metal roof deck. The girts and the roof deck span the distances between the 11 trussed frames. The triangular trussed frames required no additional bracing besides the roof deck and their connections to the grade beams. The minimization of framing and bracing members outside of the trusses simplified erection, especially for the 82 ft high roof. These trussed frames are composed of 8 in. tail pipes and 10 in. nose pipes with 4 in. diagonal members. The frames could be very light, thanks to their continuity and the fact that their bases could be fixed to the grade beams. The roof trusses themselves weigh 6 psf, while the frames as a whole, including the roof and column trusses, weigh 12.9 psf. To facilitate drainage and increase the midspan bending capacity, the roof trusses taper linearly in depth from 7 ft at the ends to as much as 8.5 ft at the midspan.

LeMessurier designed the steel castings for the corner nose pipe and tail pipe joints, as well as for joints at the midheight splices of the truss columns. The midheight castings and the tail pipe castings were fitted with attachment points for drag elements made from 8 in. diameter pipe that runs the length of the laboratory and helps stiffen the truss columns in torsion. The drag elements enable the building to resist wind in the east–west direction as a unit. For such a light structure, however, it was important to design the system with some redundancy. Therefore, the trusses on either end were outfitted with X bracing between the tail pipes for additional



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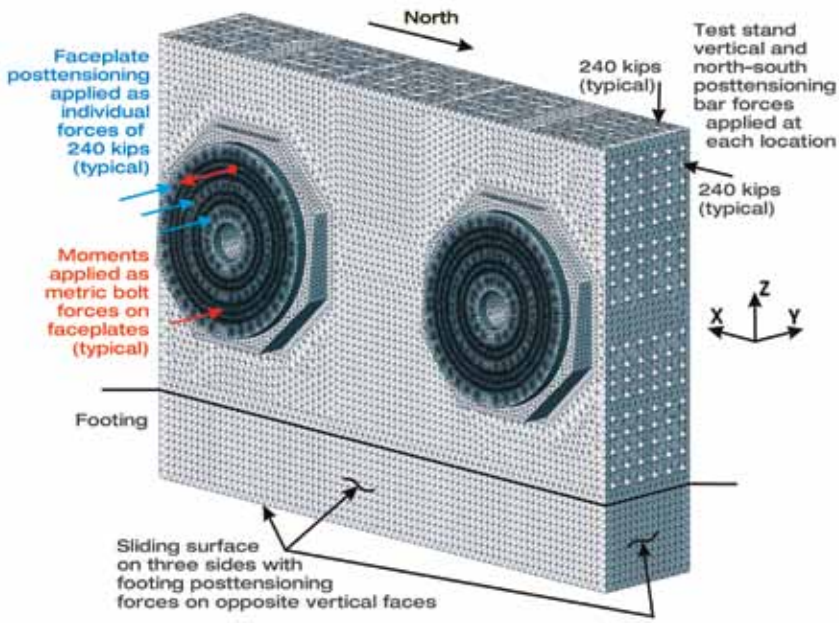
torsional capacity. These end trusses are equipped to carry the entire wind load on the east or west face of the building. The X connections in these trusses also were designed as steel castings.

Trusses were fabricated in the shop in four sections: north column, south column, north roof truss, and south roof truss. The roof trusses were spliced with full-penetration field welds on-site before they were lifted into place on the truss columns. Field connections between the roof trusses and the column trusses also took the form of full-penetration field welds between the castings (connected to the roof truss) and the truss column pipes. The truss column diagonals at the highest elevation between the nose pipe castings and the column tail pipes were installed last. Once all of the trusses were erected, drag elements were installed between them and welded into place after truing.

THREE-DIMENSIONAL FINITE-ELEMENT MODEL



The 12 in. thick, 20 ft diameter steel faceplates were so large that they had to be fabricated and shipped in sections to the facility. The faceplates for test stands 1 and 2 are angled upward 6 degrees from the vertical, and the faceplate for test stand 3 is angled at 8 degrees. These angles, together with pairs of shims that are each angled at 3 degrees, make it possible for the blades to be mounted at any angle up to 14 degrees to accommodate the range of blade types that are to be tested at the laboratory.

The laboratory is outfitted with two 50-ton cranes for handling blades. The east crane has a single trolley with a 50-ton capacity, while the west crane has two trolleys, each with a capacity of 25 tons. Working together, these three trolleys will enable laboratory personnel to rotate blades on their axis or to mount them for different testing regimes. The crane rails are supported on continuous runway beams, which in turn are supported by 10 in. diameter pipe columns. The crane columns are stabilized by the trussed frames via pipe column connections to the nose pipes at the panel points. The attachments allow for vertical slippage so that the crane columns are not subjected to overturning forces from the trussed frames. In the plane of the runway beams, the crane columns are supported by the bending stiffness of the attachment and nose pipe assemblies. The investigation of tolerances with regard to the eccentricity of the crane rails and runway beams required detailed study of the interactions between crane girders, crane columns, and trussed frames. This study addressed elastic system stability, including second-order effects, crane runway beam stability using direct analysis methods to account for second-order effects, and inelastic system stability accounting for maximum crane column eccentricity and second-order effects.

THE LABORATORY IS CAPABLE of certifying through the International Electrotechnical Commission's standard IEC 61400-23 that 90 m long blades conform to design specifications. It can also test 70 m long blades to failure for research purposes. At breakage, a 70 m blade is expected to deform up to 34 m. Such a test would be conducted by pulling on the blade horizontally in the flapwise, or weak axis, direction by

means of winch towers supported by the south grade beam. This deflection defined the overall width of the laboratory. Cables are run horizontally from the 6 m high winch towers to yokes on a blade that is anchored to test stand 1 and arranged for horizontal deformation in the flapwise direction. A cable runs over a pulley at the top of each tower and is routed down to the tower base, where it is wound on a winch. Distributed loads on the blades would be simulated by pulling with up to nine winches along the length of the blades. The winch grade beam was also designed to support forced-displacement, dual-axis fatigue testing of blades up to 60 m in length. This design required additional steel piles to be driven in areas where higher fatigue demands from such testing are expected.

Similarly, the height of the laboratory was defined by large proof-test and fatigue-test deflections on a suite of possible blades. The blades are also tested dynamically as cantilevers under resonant loading with amplitudes in excess of 10 m created by shakers attached to the blades near their midspans. The expected bending moments at the roots of the blades constitute the loading criteria to which the test stands and foundations were designed.

The loads from the blades on the test stands were determined by DNV Global Energy Concepts, of Seattle, which worked directly with the design team to discern the nature of loading and its effects on the design of the test stands and the laboratory. The defining loads for 80 and 90 m long blades required thorough investigation, as well as discussion with the Massachusetts Clean Energy Center and the National Renewable Energy Laboratory, because such blades are longer than any that have so far been designed or constructed.

The 47,000 sq ft Wind Technology Testing Center, in Charlestown, Massachusetts, has the capacity to test up to three 70 m long wind turbine blades simultaneously or a single blade up to 90 m long. Its three posttensioned-concrete test stands—including stand 3, *far left,* and stand 1, *far right*—are supported by a massive reaction footing on drilled concrete shafts. The laboratory is outfitted with two 50-ton cranes for handling blades. The crane rails are supported on continuous runway beams, which in turn are supported by 10 in. diameter pipe columns.

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ENVIRONMENTAL STEWARDSHIP also played an important role in the design of the building itself. The building was constructed on a brownfield site contaminated with polychlorinated biphenyls and thus required testing and disposal of hazardous fill. Within the grade beam footprints, an existing 18 in. thick cap made of roller-compacted concrete was crushed and used as recycled stone in the building foundations. But the entire laboratory was elevated 18 in. above the site in order to avoid excavation of the roller-compacted concrete beneath the diaphragm slab. Some 75 percent of the laboratory's 6,000 cu yd concrete contained 50 percent fly ash to control the heat of hydration and conserve cement.

A series of energy models developed from the earliest design phases and continued through the construction documents called for the building to be enclosed by insulated, energy-efficient metal panels 4 in. thick, a reflective roof, and operable windows. The laboratory floor is lit by a combination of daylight and low-energy light-emitting diode (LED) bulbs that are produced locally. Offices and laboratory spaces have customized lighting controls. Heat for the building is largely recovered from the hydraulic equipment used for blade testing. In place of air-conditioning, the laboratory is cooled by a combination of natural and mechanical ventilation.

Tight deadlines and budget controls made it necessary to let several construction packages early, including parts of the foundations and building structure and items with long lead times. For example, some of the 170 ft deep foundations, including the drilled concrete shafts at the reaction footing and the steel H-piles at the north and south grade beams, were already constructed by the time the bid documents related to constructing the building envelope and outfitting the building interior were issued. The reaction footing had been placed by the time the original documents for the faceplates were issued. Before releasing the bid documents for the fabrication of the faceplates, the owner, the design team, and the contractor worked together to redesign the 6-degree faceplate wedge angles as reinforced concrete rather than use steel spacers on the back side of the faceplates. This work saved several hundred thousand dollars without extending the construction schedule.

Construction began in December 2009 and was completed this past May, in keeping with the strictures of the American Recovery and Reinvestment Act of 2009. The laboratory was completed within budget, the construction cost being approximately $26.1 million. While every member of the design and construction teams used the most advanced technology and methods of delivery, the key to the project's success was the constant personal communication between the design team, the contractor, and the owner. The designers oversaw the construction personally, sometimes visiting the site daily to ensure that conflicts were resolved within minutes or hours.

IF WIND POWER IS to continue its growth as an energy source both domestically and internationally, further research and development will be of cardinal importance. To this end, the WTTC can be expected to serve as a catalyst for the growth of the wind power industry. Additionally, the facility will play a vital role in educating a generation of engineers who are devoted to sustainable development and energy independence. Students from nearby Tufts University have already benefited from the design and construction of the WTTC through multiple site visits and projects. As testing proceeds, the laboratory will continue to help educate students on issues related to cutting-edge technologies involved in wind engineering. Located slightly more than 1 mi from downtown Boston, the WTTC will play a vital role in the life of the city and its state.

Visitors from all over the world have expressed excitement about the scope, versatility, and appearance of the laboratory. Even within a tight budget, the WTTC was designed and constructed to be more than an industrial structure. The owner and the design team took advantage of the limitations imposed by the site and the program to create a structure that expresses elemental power with refinement and grace.

Given the grand scale of the laboratory, the simplicity and coherence of the trussed enclosure can be appreciated for its lightness and sculptural quality. Visitors who walk among the trusses on the laboratory floor or sit beside them in the offices on the north side will delight in discovering perspectives that bring to mind both the sublime quality of massive wind turbines and the fine detail of precision-machined components that convert wind into clean energy.


Hines


Ravindra

Unprecedented in scale and disciplined by its site, budget, and schedule, the WTTC confidently faces the technical challenges of our future green economy, assuring us that we need not sacrifice our sense of what is human and beautiful in harnessing the technology that will create a better world for our children. **CE**

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PROJECT CREDITS **Owner:** Massachusetts Clean Energy Center, Boston **Owner's representative:** Massachusetts Port Authority, Boston **Operator:** Massachusetts Clean Energy Center in cooperation with the U.S. Department of Energy's National Renewable Energy Laboratory **Project architect:** Architerra, Inc., Boston **Structural and foundation engineering:** LeMessurier Consultants, Inc., Cambridge, Massachusetts **Geotechnical and foundation engineering:** Haley & Aldrich, Boston **Wind technology consultant:** DNV Global Energy Concepts, Seattle **Civil engineering:** Vanasse Hangen Brustlin, Inc., Boston **Environmental engineering:** GEI Consultants, Woburn, Massachusetts **Marine engineering:** Childs Engineering Corporation, Medfield, Massachusetts **Mechanical, electrical, and plumbing:** RDK Engineers, Boston **Energy modeling:** DMI, Inc., Wellesley, Massachusetts **Construction manager:** Turner Construction Company, Boston **Concrete contractor:** Francis Harvey & Sons, Worcester, Massachusetts **Posttensioning supplier:** DYWIDAG Systems International USA, Inc., Bolingbrook, Illinois **Steel fabricator:** Cives Steel Company, Augusta, Maine **Steel castings:** Steel Cast Connections, LLC, Seattle **Steel erector:** Daniel Marr & Son Company, Boston **Drilled concrete shaft construction:** New England Foundation Company, Boston **Pile construction:** H&L Pile Driving Corporation, Brockton, Massachusetts **Site work:** J. Derenzo Company, Brockton, Massachusetts **Exterior wall:** Ipswich Bay Glass Company, Inc., Rowley, Massachusetts