POST-TENSIONING INSTITUTE ANNOUNCES WINNERS OF 2019 PTI PROJECT AWARDS

The Post-Tensioning Institute (PTI) announced 13 winners for the 2019 Project Awards, who were honored during the PTI Awards Presentation at the 2019 PTI Convention on May 6, 2019, at the Hyatt Regency Seattle, Seattle, WA. The awards recognize excellence in post-tensioning applications. Any structure completed or rehabilitated in the past 7 years that uses post-tensioning as a structural component was eligible. Entries were submitted by owners, architects, engineers, contractors, and post-tensioning suppliers. Awardees were selected by a jury of industry professionals and were judged based on creativity, innovation, ingenuity, cost-effectiveness, functionality, constructability, and aesthetics.

The highest honor, "Project of the Year," was awarded to 55 Hudson Yards. This award is given to a project that demonstrates excellence in post-tensioning applications and stands out above all other entries. The project was submitted by WSP, USA, and the structure is located in New York, NY.

The remaining winners were selected from six categories, with an 'Award of Excellence' given in each category, and an 'Award of Merit' presented to other projects deserving recognition. The categories and winners include:

- Bridges
  - Award of Excellence: Sarah Mildred Long Bridge Replacement
  - Award of Merit: Botanical Garden Atocha—La Liria Bridge
- Buildings
  - Award of Excellence: Kinects Tower
  - Award of Merit: The Ritz-Carlton Residences Waikiki Beach, Phase 2
- Industrial/Special Applications
  - Award of Excellence: LA Stadium and Entertainment District
  - Award of Merit: Precast Segmental Post-Tensioned Box-Girder Wind-Turbine Foundation
- Parking Structures
  - Award of Excellence: Segal Visitors Center
  - Award of Merit: Miami Museum Garage
- Repair, Rehabilitation, and Strengthening
  - Award of Excellence: East Link Extension—The Homer M. Hadley Floating Bridge
  - Award of Merit: 25 Beacon
- Slab-on-Ground
  - Award of Excellence: R.H. Johnson Recreation Center—Sun City West, Arizona
  - Award of Merit: Athletic Running Track Renovation

The PTI Project Awards program runs every 2 years. The next round will be held in 2021. To see examples of past winners, visit **www.post-tensioning.org.** If interested in submitting a project for the next awards program, details and an application kit will be available in 2020.

2019 PROJECT OF THE YEAR: 55 HUDSON YARDS

Fig. 1—Overview of 55 Hudson Yards during construction.

The 55 Hudson Yards project is part of the Manhattan Western Expansion project. (Fig. 1) Located on 11th Avenue between 33rd and 34th Street, the 51-story tower reaches 760 ft (230 m) in height and encompasses 1.3 million ft^2 (121,000 m^2) of gross area, including a 10-floor podium.

Among the most significant engineering challenges of the project are the site and load-carrying constraints due to large interaction with existing MTA infrastructure, as well as the maximization of both column-free spaces and flexibility for future modifications. These challenges were successfully addressed by incorporating post-tensioned concrete elements in three distinct ways, and a unique construction sequence aimed at creating a specific vertical load path.

Post-tensioned applications were implemented in: a) 40 to 50 ft (12 to 15 m) span, 9 in. (230 mm) thick slabs; b) bonded tendons draped over a 39 ft (12 m) deep transfer wall; and c) straight tendons on the 9th floor to carry horizontal loads associated with walking columns at the transition between the podium and the tower.

The unique construction sequence was triggered by the need to redirect vertical loads to specific locations in the foundation where ample load-carrying capacity had been identified, and away from the existing MTA infrastructure located directly underneath the building. The required sequence was developed by means of a "construction gap" left open until the redirection of loads had been successfully concluded.

Paramount to the project was the requirement to provide flexibility for potential future changes per the tenant's requests. Provisions for floor layout modifications were achieved by using a banded distribution of tendons creating rather ample tendon-free zones where tenants may create new floor openings. Furthermore, three-dimensional (3-D) laser surveys were carried out during construction to determine the as-built location of all post-tensioning tendons.

The interaction and overlapping of new structures with existing infrastructure is becoming not only more common but also more complex in dense urban environments. 55 Hudson Yards greatly exemplifies the need for innovative design strategies and construction techniques to address the challenges of extensive projects in dense urban settings.

PROJECT DESCRIPTION

Background

Hudson Yards is a 14-acre area located within the west side of Manhattan, currently undergoing a major expansion primarily after the acquisition of building rights above the Long Island Rail Road yards by a well-known developer. As a result of the initial Hudson Yard air rights acquisition and the opening of the Number 7 subway line extension, major development by numerous developers ensued all along the west side of Manhattan and around the site of the railroad.

Project evolution

The 34th Street-Hudson Yards station of the No. 7 subway line and the ventilation building were originally designed in coordination with the former developer of the site to support a steel-frame structure with exterior braced frames reaching more than 50 stories high. Under the original conception, the majority of vertical loads were envisioned to reach the foundation around the perimeter of the site, setting specific load-carrying capacity limits at selected points of the site.

However, during the years of construction of the MTA infrastructure below and next to 55 Hudson Yards, the original developer decided to sell the property and air rights to the current owner. The new developer immediately pursued modifying the design of the tower to meet their new objectives, resulting in a 51-story building with 1.3 million ft^2 (121,000 m^2) of gross area for commercial use. The changes required for the tower resulted in both architectural and structural modifications specific to the tenant base envisioned by the new developer in the full context of the Hudson Yards expansion.

In light of these changes, the steel-based structural solution became less attractive in financial terms, and an alternate concrete-based solution was explored even when a concrete structure of comparable height to the original design would typically be heavier. Meanwhile, the MTA infrastructure and adjacent ventilation building had progressed as planned, so any modifications to the original project would require adhering to the load criteria set forth for the original design.

To overcome the challenges imposed by the existing MTA structures, innovative design strategies and careful control of the construction sequence were implemented as unique structural solutions for the project.

Architectural features and structural challenges

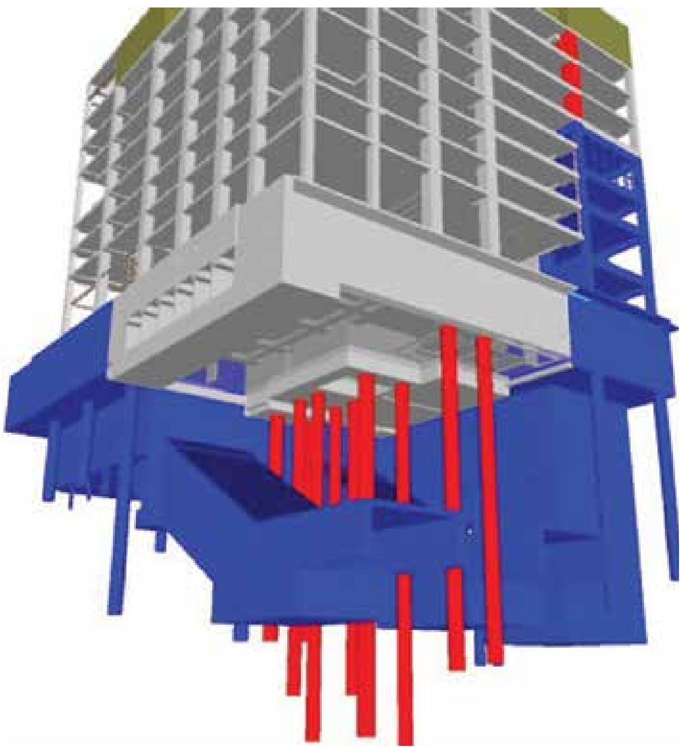
Architecturally, the concrete tower posed some interesting challenges. The tower at 55 Hudson Yards features a 10-story podium and a 41-story recessed tower above. A central utility and elevator core extend for the entire height of the tower, allowing for no interior columns within the podium or the tower. The column-free spaces extend almost 50 ft (15 m) at the podium and more than 40 ft (12 m) at the tower. The transition between the perimeter columns of the tower to those in the podium is cleverly hidden in the mechanical floors located at the top floors of the podium (9th and 10th floors).

As the result of the architectural intent, the structural solution of 55 Hudson Yards was envisioned through a central reinforced concrete core comprised of orthogonally arranged shear walls, an exterior moment frame, and a partial outrigger towards the top of the tower. An ingenious load path and load distribution pattern resulted in 20% of the vertical load of the building load to be carried by a concrete mat foundation, and 40% being transferred from the shear walls in the core to 10 reinforced concrete drilled caissons located in the relatively small gap between the two major escalators leading from the entrance to the platform of the 34th Street-Hudson Yards station (Fig. 2). The drilled caissons have an impressive length of up to 100 ft (30 m). To further reduce the load demand around the core, the remaining 40% of the vertical loads were redirected to the perimeter of the site, where there was ample load-carrying capacity per the original design.

Use of post-tensioned concrete

To achieve the lightest possible concrete structure while directing loads from the central core to the perimeter, post-tensioning systems were used in three different ways. First, the thickness of a typical floor spanning 40 to 50 ft (12 to 15 m) was reduced to only 9 in. (230 mm) using post-tensioned slabs. The weight of the floors was further decreased with the use of 130 lb/ft^3 (2080 kg/m^3) lightweight concrete, a somewhat unconventional practice for post-tensioned floor slabs. The second implementation was the provision of bonded post-tensioned tendons draped over a 39 ft (12 m) tall transfer wall located in the 9th and 10th floors to redirect a portion of

(a) Overview of foundation system with location of MTA escalators.



(b) Revit model showing location of drilled caissons between MTA escalators.

(c) Overview of mat foundation with provisions for mass concrete.

Fig. 2—Details of foundation system for 55 Hudson Yards.

the vertical load of the central core to the perimeter (Fig. 3). Finally, at the change in geometry between the tower and the podium, walking columns were used to transfer vertical loads to the perimeter columns. This solution resulted in significantly large compression and tension forces within the 10th and 9th floors, respectively. The latter were carried by straight post-tensioned tendons (Fig. 4).

Special construction sequence

Aside from the innovative use of post-tensioning systems, a special construction sequence at the base of one of the central core walls was employed to redirect load toward the perimeter. Because the newly built MTA infrastructure located at the center of the site could only carry the loads from the tower of the building and not those of the podium, the pour of a small segment at the base of the wall was purposely delayed, creating a gap between said wall and the existing MTA structure below. The gap was deemed adequate to shed the vertical load of the podium to the perimeter of the structure and away from the central core.

The construction of the building proceeded as usual until the podium was completed. At that point, the gap was closed, creating a positive connection between the wall and the existing MTA structure. As construction progressed, the vertical loads of the tower were transmitted down through the central core, but the loads of the podium remained being redirected to the perimeter (Fig. 5).

Flexibility for future changes

In addition to the site constraints previously described, the architectural intent required the accommodation for potential future changes. Seeing as though 55 Hudson Yards is a commercial building that requires flexibility to address

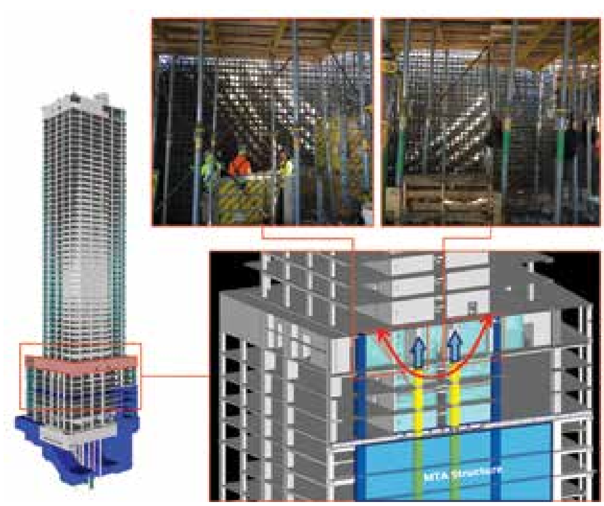


Fig. 3—Location and post-tensioning details of 39 ft tall transfer wall.

specific tenant's requests, the use of a traditional post-tensioned slab system where tendons may not be cut was considered disadvantageous. To allow for future floor layout modifications, the tendons were banded together, creating rather ample tendon-free zones where tenants may create new floor openings. During construction, 3-D laser surveys were carried out to determine the as-built location of all post-tensioning tendons (Fig. 6). These surveys proved to be great sources of information while addressing specific tenants' requests, pointing them to the precise locations to avoid cutting the tendons.

Construction and design in dense urban environments

Building in a dense urban environment often means the overlapping of new structures with existing and newly developed infrastructure. This interaction is becoming not only more common but also more complex due to market forces, and the 55 Hudson Yards greatly exemplifies the need for multiple innovative design strategies and construction techniques to address the challenges of extensive projects in dense urban settings.

The appropriateness of the project in the context of the Hudson Yards expansion is evident when the repurposing of undeveloped spaces in a very dense urban environment is considered. Not only has the economic growth of an otherwise underutilized area of New York City been jump-started by the 55 Hudson Yards project and its neighbors, but also the area is on a clear path of integration with the always-changing cityscape.

The 55 Hudson Yards Project obtained the Gold LEED certification for its numerous features for improved environmental performance and sustainable design.

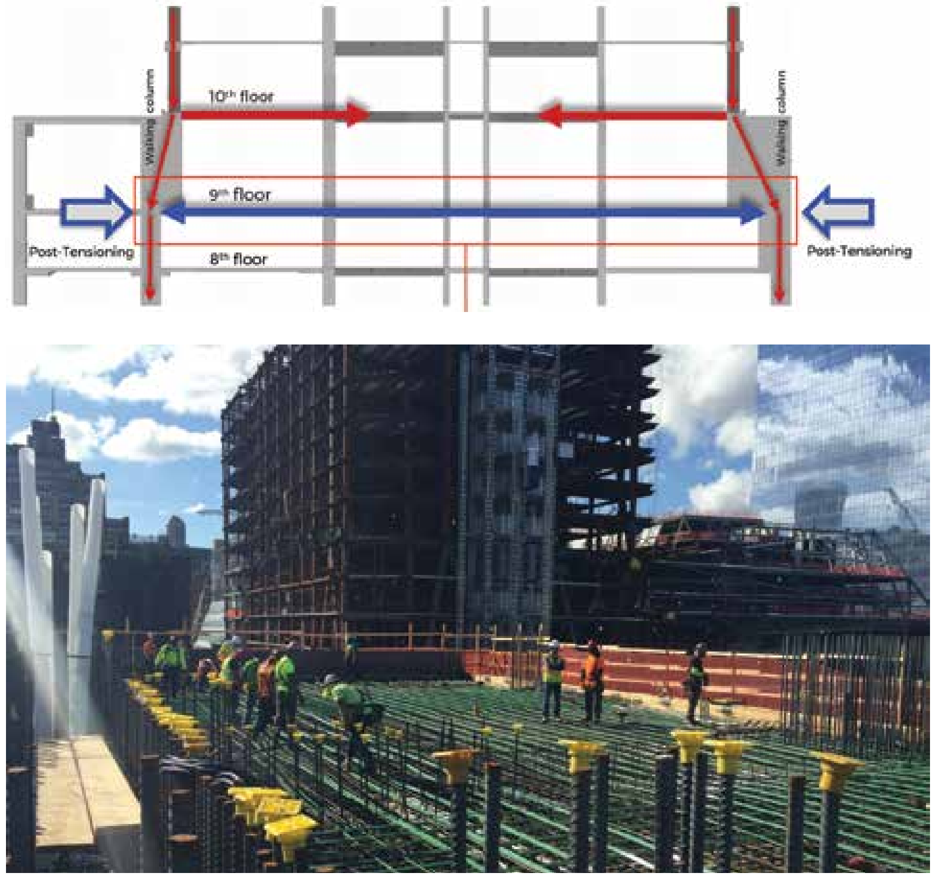


Fig. 4—Load path diagram at podium-to-tower transition resulting in installation of straight tendons on 9th floor.

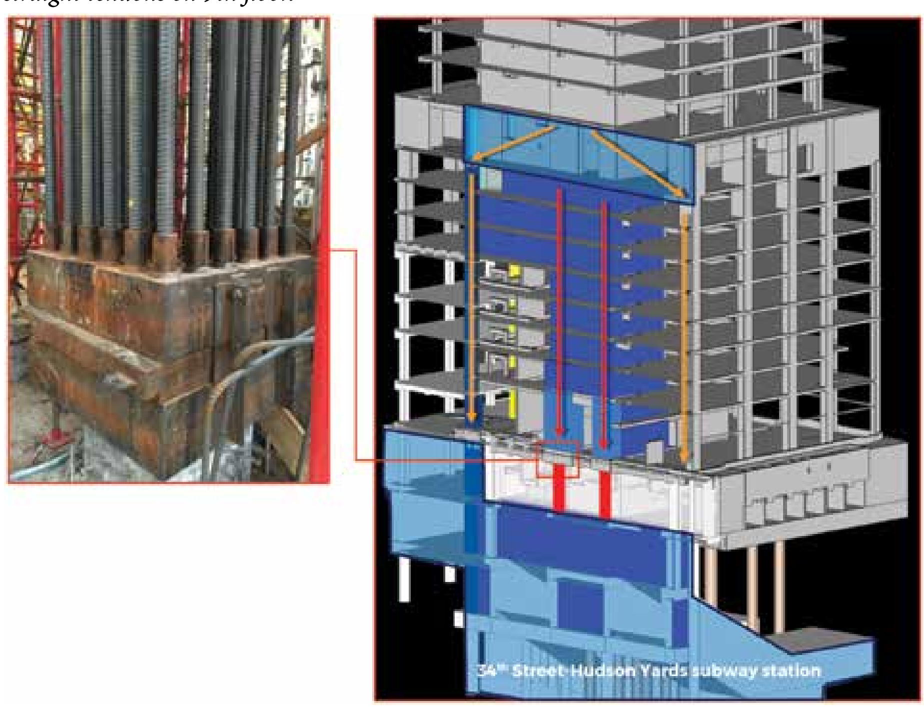


Fig. 5—Vertical load path created through special construction sequence and detail of shimmed connection prior to engagement

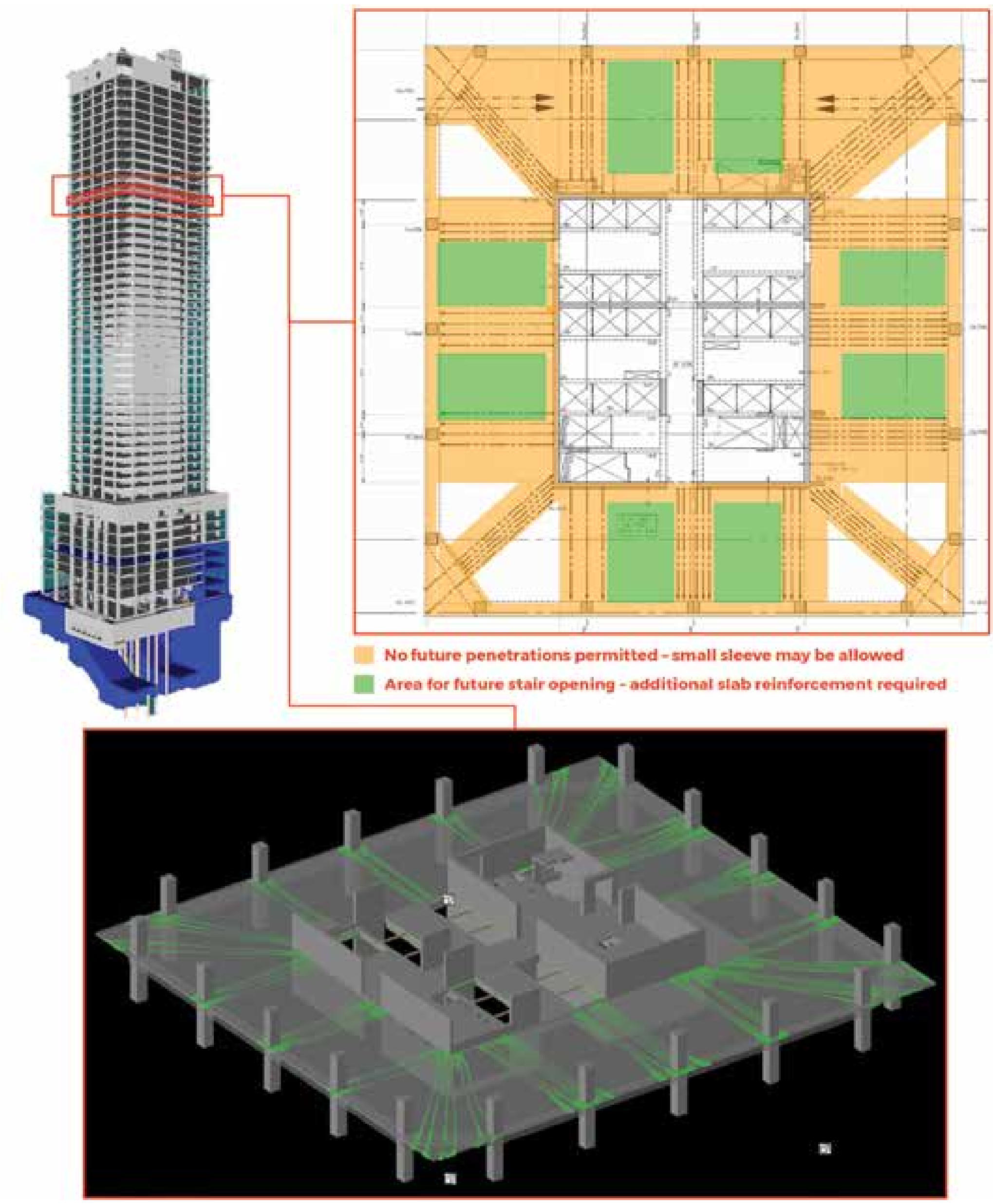


Fig. 6—3-D scans of as-built location of post-tensioning strands for identification of areas for future potential floor openings.

Location: New York, NY
Submitted by: WSP USA
Owner: Related Companies & Oxford Properties Group & Mitsui Fudosan America, Inc.
Architect: Kohn Pedersen Fox; and Kevin Roche, John Dinkeloo and Associates
Engineer: WSP USA
Contractor: Gilbane Building Co.
PT Supplier: AMSYSCO, Inc.
Other Contributors: Florian Aalami, PhD - ADAPT Corporation

Jury Comments:

- A truly innovative approach for using the secondary effects of post-tensioning to control the tower's load path. This project has opened the door for the use of post-tensioning in the NYC developer market, and is a huge win for the entire PT industry.
- This significant project has several notable features including a 39-foot transfer wall, use of lightweight concrete, and transfer of the loads at the walking columns.

2019 AWARD OF EXCELLENCE: SARAH MILDRED LONG BRIDGE REPLACEMENT

The Sarah Mildred Long Bridge (SMLB) replacement project (Fig. 1) included removal of the previous steel vertical lift bridge with deck truss, roadway, rail, and approaches having a total length of 2800 ft (850 m). Completed in 1940, the previous bridge provided a critical link across the Piscataqua River between Kittery, ME, and Portsmouth, NH. The previous bridge served as a vital backup route in the event of a disruption of service on Interstate 95; serviced heavy truck transit to and from the commercial service stations along the US 1 Bypass; and included a rail line used to transport heavy freight to the Portsmouth Naval Shipyard, which employs approximately 4200 workers. The previous bridge experienced structural deterioration, creating a need for a new bridge.

Fig. 1—View of the completed bridge over the Piscataqua River – July 2018.

Ultimately, the existing bridge was closed a month before its scheduled closure date due to a mechanical failure that rendered the lift span inoperable.

The Sarah Mildred Long Bridge is one of three bridges that link the states of Maine and New Hampshire over the Piscataqua River. The two states jointly own the three bridges as part of what is known as the "three bridges agreement." The previously mentioned I-95 Piscataqua River Bridge; the SMLB; and another vertical lift bridge downstream, the Memorial Bridge, make up the three bridges. The Memorial Bridge had recently been replaced as part of the agreement and the SMLB was next to be replaced.

DESIGN

The process to design and replace the SMLB proceeded as a "CMGC" project, where a general contractor was selected during the CM phase under a construction management contract to work with the owner and engineer on constructability, schedule, and project costs. At the end of the design phase, negotiations were held for the final price between the chosen GC and the owner, and if an agreed upon price could not be reached, the job would go out to bid as a conventional design-bid-build project. Ultimately, a price was agreed upon and the GC brought on in the design phase was awarded the construction contract.

During the design phase, a few changes from the old bridge would offer a substantial improvement for both the marine traffic using the shipping channel and the vehicle traffic. The existing bridge had two deck levels on the lift span. When seated, a train could cross the bridge simultaneously with vehicle traffic. The rail is only used to service the Portsmouth Naval Shipyard, and they typically only see one or two trains per year. The decision was made to have just one deck on the lift span, which would sit at roadway level, be lowered to rail level for a train to pass, or be raised for marine traffic to pass (Fig. 2). This allowed the roadway elevation to be raised higher because it only had to follow the roadway specifications rather than the railroad specifications for the grades of the approach spans. This resulted in a much higher vertical clearance for marine traffic with the bridge seated at roadway level, which will result in an expected 68% fewer openings. This will reduce traffic congestion, especially in the summer when recreational boating is popular on the Piscataqua River. The other major change was the skew of the bridge to the navigational channel. The existing bridge was at a skew to the navigational channel that reduced the actual clearance between the piers of approximately 215 ft (66 m) to a useable clearance of approximately 185 ft (56 m). There are multiple industrial facilities upriver of the bridge that see large ships, and the usable width of the channel meant that the tugboats escorting the vessels needed to detach before the bridge and reconnect after the bridge. This was not ideal with the strong tidal currents present on the Piscataqua River. By creating a new alignment for the bridge, expanding the length of the lift span, and skewing the tower piers to the bridge, a usable navigational channel width of 250 ft (76 m) was attained, allowing the tugboats to remain connected to the vessels as they passed through.

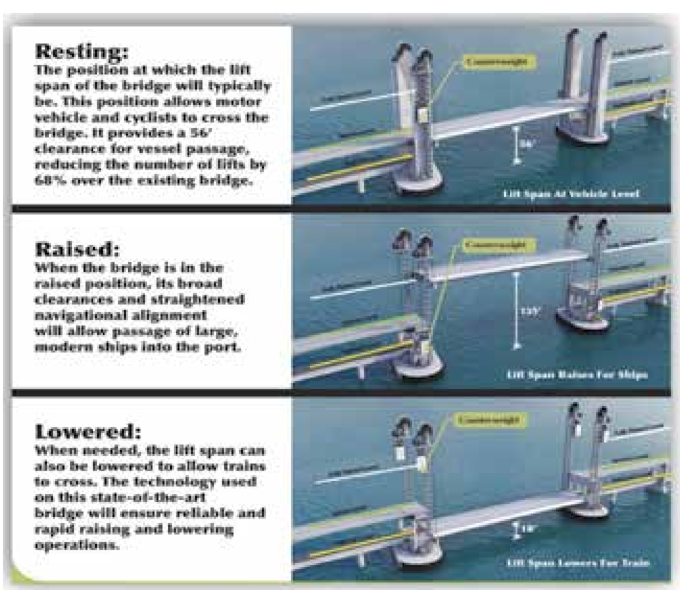


Fig. 2—Lift bridge configurations.

The bridge was designed using post-tensioned concrete elements for the approach bridges and the lift span towers. These design elements allowed for longer roadway spans, fewer piers, and an accelerated construction schedule. It was decided to cast the concrete tower segments on site, and the roadway and railroad segments would be cast offsite.

For the tower foundation, a stay-in-place precast concrete cofferdam was designed, and post-tensioning was designed for the bottom slab to connect the individual pieces. The bridge also required three "shared piers" where the roadway and railroad segments needed to be supported one over the other. Post-tensioning was used on these piers to connect the roadway segment to the concrete columns, which ran up the sides of the segment, allowing the railroad segment to run between the columns and below the roadway segments (Fig. 3). The control room for the lift span was comprised of a precast floor slab and roof slab, which was post-tensioned to the lift tower. The "sheave walls", which cap off the concrete towers and support the 20 ft (6 m) diameter counterweight sheave, were also precast and post-tensioned to the roof slab of the tower. All of these post-tensioning elements allowed components to be precast, which was critical in accelerating the schedule.

The project team also made the decision to install a temporary work trestle. Working on the Piscataqua River can be challenging with the strong tidal currents. The temporary work trestle allowed for crane, truck, and pedestrian traffic to every in-water pier. Although the larger picks still needed to be completed by the larger barge-mounted cranes, most of the day-to-day operations could be supported by the trestle cranes. The trestle greatly reduced the amount of boat and barge work and limited the risks involved with such work.

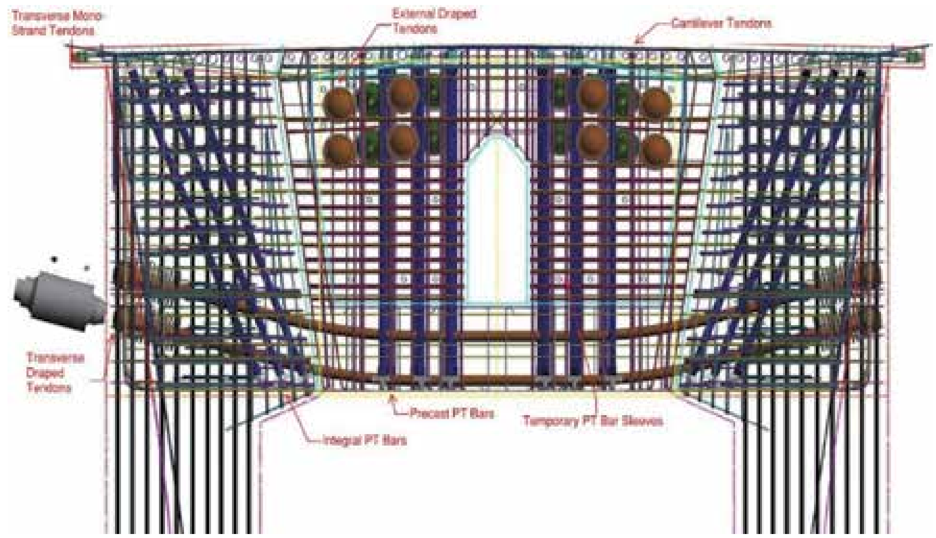


Fig. 3—3-D view of shared pier to aid construction.

Most of the piers were built on drilled shaft foundations, with some built on traditional foundations using cofferdams and tremie concrete seal placements (Fig. 4 and 5).

The mechanical system is considered a "modified tower drive" with the operating machinery at the base of the towers, and a system of operating ropes that either pull the counterweight down to raise the bridge, or pull the counterweight up to lower the bridge. Four counterweights, weighing 1,000,000 lb (450,000 kg) each, hang inside each of the four towers. Counterweight ropes pass over the counterweight sheaves that sit atop the towers and connect the 4,000,000 lb (1,800,000 kg) lift span to each counterweight. The lift span is made up of orthotropic box girders, with a concrete deck, and a composite fairing which reduces icing and wind loading.

Fig. 4—Erecting the first precast cofferdam, which will become a stay-in-place form for the lift tower pier foundation formed on top of eight 10 ft (3 m) diameter drilled shafts – June 2016.

CONSTRUCTION

Construction began in late 2014 with the installation of causeway fill material and temporary work trestles. The project included three separate work trestles, one which spanned Cutts Cove, one that connected the New Hampshire Port Authority Barge Pier to the Portsmouth tower pier, and one which connected the Kittery tower pier to the shore on the Maine side. The three trestles, in addition to a short earth fill causeway, created crane and truck access to the entire bridge, with the only gap at the navi-

Fig. 5—Ten foot (3 m) diameter drilled shaft installation working from access trestle – September 2015.

gational channel. The Cutts cove trestle was 240 ft (73 m) long, the Portsmouth trestle was 660 ft (200 m) long, and the Kittery trestle was 548 ft (167 m) long.

With the completion of the access trestles, substructure work was able to begin. Three of the approach piers on the Portsmouth side were built on traditional foundations. Sheet pile cofferdams were installed, the earth was excavated, and tremie concrete seals were placed inside. The bridge has four abutments: two for the vehicle bridge and two for the railroad bridge. Three of the four abutments were also built inside sheet pile cofferdams in a similar way. The fourth was built in an open excavation. The remainder of the piers were built on drilled shaft foundations. There were three styles of piers supported by drilled shafts. There were piers that supported only the railroad deck, which were built on one drilled shaft. There were shared piers that supported the railroad deck and the vehicle deck, which were built on two drilled shafts. Lastly, there were tower piers that supported the lift span towers and the lift span, which were built on eight drilled shafts. This totaled 29 drilled shafts for the project. The drilled shafts for the shared piers and the tower piers were connected by stay-in-place precast concrete cofferdams. These were filled with reinforcing bar and concrete and created the foundation for the bridge above.

Fig. 6—Railroad balanced cantilever erection – September 2016.

While all the aforementioned was underway, two precast yards were gearing up to produce the precast tower segments, and the precast deck segments for the railroad and the roadway decks. The tower segments were made on site, inside the NH Port Authority, and the deck segments were made off-site and trucked to the site. In total, it took nearly 500 precast segments to make up the foundations, the lift span towers, the control room, as well as the railroad and roadway decks. Segment erection began with the railroad deck, which was installed as balanced cantilevers using the trestle as the shoring to support them (Fig. 6). Temporary post-tensioning bars were used for erection and removed as the permanent post-tensioning strands were installed. As the railroad deck was completed, shoring was set up on the deck to support the vehicle deck-balanced cantilevers above which were installed in the same fashion as the railroad segments (Fig. 7). As each span was erected, closure pours were used to connect each span to the next. At the tower piers, the tower segments were erected to create the four lift span towers. The tower segments were also temporarily secured using post-tensioning bars, and permanently secured using post-tensioning strands. With the towers erected, the control room slabs were installed. The control room cantilevers from the side of one of the lift span towers. The control room was built using a floor slab and a roof slab that are post-tensioned to the side of the tower with post-tensioning bars.

Spanning between the towers are the mechanical room and the electrical room: the mechanical room at the base of the towers resting on the foundation and the electrical room above with a clear span below for the railroad corridor. The roof of the mechanical room is the railroad level, and the roof of the electrical room is the roadway level. The slabs, walls, and roofs were all cast in place between the precast towers. The mechanical room at the base of the tower houses all the machinery that operates the lift span. The electrical room and control room house all the electrical equipment that powers and controls the lift span. Inside each of the four towers is a counterweight. On top of the towers are the counterweight sheaves, which are 20 ft (6 m) in diameter. Counterweight ropes connect to the counterweight, run up and over the counterweight sheave, and connect to the lift span. Operating ropes connect the machinery below to the counterweights to pull them up or down, moving the lift span.

The lift span was assembled on a barge adjacent to the jobsite. The lift span is made up of orthotropic box girders, with a concrete deck, and composite fairing wind screening. The steel and the fairing were installed on the barge. Next, the lift span was floated into position between the lift span towers and lowered onto the railroad bearings (Fig. 8). The concrete deck was placed and the counterweight ropes were attached. This allowed the lift span to be raised by the operating machinery for the first time (Fig. 9). All the commissioning and testing took place next. Lastly, the curbs were placed on the decks, the railings/guardrails were installed, and the deck was paved. All the earthwork at the abutments was completed, and the bridge was opened to traffic in March 2018.

Fig. 7—Vehicle bridge balanced cantilever erection off the shared pier, as well as setting the last counterweight sheave for the lift span – September 2017.

SAFETY

The Sarah Mildred Long Bridge Replacement project was completed with over 600,000 safe work hours, with no lost time injuries and an OSHA Recordable Incident Rate of 0.32. The GC's on-site Safety Management team completed over 900 safety orientations for the owner(s), engineers, subcontractors, visitors, and the GC's project team.

Fig. 8—Float-in of the lift span – October 2017.

Fig. 9—Lift span in operation – July 2018.

Location: Kittery, ME
Submitted by: Cianbro Corporation
Owner: Maine DOT/New Hampshire DOT
Architect: Figg and Hardesty and Hanover
Engineer: Figg and Hardesty and Hanover
Contractor: Cianbro Corporation
PT Supplier: VSL
Other Contributors: McNary Bergeron (Construction Engineer), Case Foundation, Shaw Brothers, West Wind, Unistress, Coastal Precast, Euclid, Atlantic Dismantling, Ninive, Structural Technologies, G&G, On Point, Panatrol, B&B Roadway, A&D Electric

Jury Comments:

- Utilized the segmental construction for vertical lift bridge towers and approach spans. Dual-level vertical lift bridge designed for both train and vehicles.
- Very unique dual-purpose structure (vehicles plus rail) in a lift bridge. Very pleasing to the eye and entirely functional.

2019 AWARD OF EXCELLENCE: KINECTS

Fig. 1—Kinects Tower.

Kinects is a luxury residential tower in the Denny Triangle, one of downtown Seattle's fastest-growing neighborhoods (Fig. 1). The 550,000 ft^2 (51,000 m^2) building includes 356 apartments, a rooftop swimming pool, ground-floor retail, and four levels of parking above grade, with four additional levels below. Three sides of this 41-story project taper outward as the building rises. As a result, the uppermost floors are approximately 50% larger than lower floors, creating more rentable square footage at the top of the building where views are best and rents highest.

Post-tensioning (PT), which was used at all levels, was key to creating the tapered tower. Horizontal thrust forces from slanted columns were resisted by a carefully engineered combination of PT and mild reinforcement. This resulted in a buildable and efficient building that received accolades from many professional associations, as well as from residents who enjoy living there.

KINECTS SIGNATURE STRUCTURAL SYSTEM

Optimized post-tensioned slabs

Kinects' state-of-the-art structure consists of cast-in-place concrete with post-tensioned floor slabs and a shear wall core for seismic and wind resistance (Fig. 2). The gravity system uses long-span, 8 in. (200 mm) thick two-way post-tensioned slabs, with unique 18 in. (460 mm) deep outriggers extending 8 ft (2.4 m) from the sloping columns. The outriggers helped extend the slab cantilevers to 13 ft (4 m) at the perimeter. The optimized slab design eliminated additional internal columns, which resulted in open and spacious interior layouts and unobstructed views of the City, Puget Sound, and Cascade Mountains. At subterranean levels, shotcrete perimeter basement walls followed construction of the post-tensioned slabs (Fig. 3). The delay between basement slab and wall construction allowed unrestrained slab shortening, resulting in a nearly crack-free subterranean garage. It also helped accelerate construction because it removed basement walls from the critical path.

Sloping tower columns and efficient post-tensioned slab design

Kinects slopes outward on three sides, resulting in increased floor area at upper levels of the building where views are best and rents highest. To achieve this, 10 columns on the expanding sides were slanted approximately 6 in. (150 mm) per level. A consistent PT and mild reinforcing layout were maintained by cantilevering the slab 13 ft (4.0 m) at each floor and increasing the interior back span as the floor plates grew. The PT profiles were modified slightly every several floors to maintain consistent slab uplift forces throughout. This kept slab bending moments reasonable while facilitating a repeatable reinforcing layout with only minor variations in reinforcing bar and PT placement above Level 8.

PT slabs reduced the gravity loads and seismic mass

Post-tensioned, 8 in. (200 mm) thick flat-plate slabs were used at all levels, including the subterranean parking levels (Fig. 4). This resulted in smaller columns and reduced mass, which created reductions in foundation sizes, seismic forces, and shear wall requirements. It also lowered the floor to floor height and, importantly, in this modern era of sustainability, reduced the building's carbon footprint. Additionally, the effective use of high-strength concrete and reinforcing bar allowed standardization of column sizes throughout, which maximized formwork productivity and reduced waste.

Highly efficient core-wall seismic system

Due to the International Building Code's 240 ft (73 m) height regulation for shear wall structures in zones of high seismicity, the structural engineer opted for a coupled shear wall core system (Fig. 5) using performance-based design (PBD). The PBD shear wall core minimized the floor area required for the seismic system. Also, the PBD design required less reinforcement than for a code prescriptive design, especially because PT helped reduce the weight of the structure which transferred directly into reduced seismic forces. Working with the structural and geotechnical peer reviewers, the structural engineer used nonlinear seismic analysis to design and detail the core in an efficient and buildable manner.

High-strength concrete and reinforcing bar

Kinects used high-strength concrete to improve structural performance and enhance constructability. A concrete strength of 12,000 psi (82.7 MPa) was specified for tower columns up to Level 8 to reduce the column sizes, allow fewer and smaller columns, and increase the leasable floor area. This also allowed column sizes to be kept constant nearly full height, as loads and concrete strengths diminished, which maximized formwork productivity. Further, Grade 80 reinforcement was used for all tie steel in columns and shear wall boundary

Fig. 2—Cast-in-place concrete with post-tensioned floor slabs and a shear wall core for seismic and wind resistance.

Fig. 3—Subterranean level post-tensioned slab.

Fig. 4—Eight in. (200 mm) flat-plate slabs.

elements (Fig. 6), as well in the mat foundation for flexural reinforcement. This synergistic use of high-strength concrete and high-strength reinforcing bar reduced the overall steel tonnage, minimized reinforcing bar congestion, and reduced field labor.

Unique construction challenges

Kinects' small site posed significant design and construction challenges. Due to the proximity to adjacent structures, the tower crane was placed on the sidewalk and supported by a 6 ft (1.8 m) deep pile cap on concrete piles extending to well below the basement excavation. The tower crane loads were carefully analyzed to avoid overstressing basement walls and shoring soldier piles.

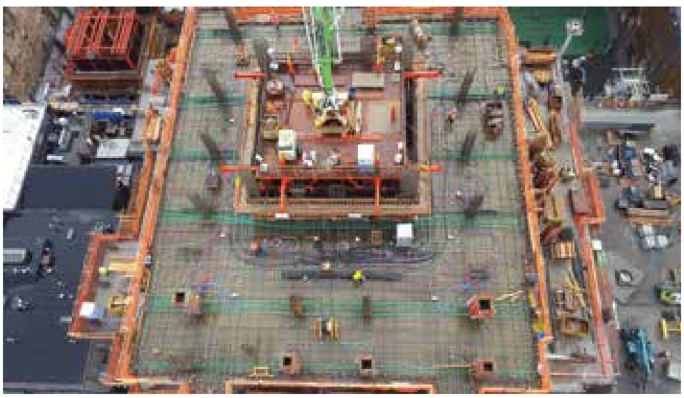


Fig. 5—Coupled shear wall core system.

Fig. 6—High-strength tie steel in columns.

Buildable structural design

The structural engineer worked closely with the owner, architect, and contractor during the design phases to create a post-tensioned structure that not only complemented the architectural intent but also minimized construction cost and shortened the schedule, allowing construction to finish earlier than planned. The state-of-the-art PT design, combined with proactive coordination and a well-executed plan, were essential to maintaining the rapid construction schedule. The slant columns on three sides helped eliminate transfer beams and provided a continuous load path to the foundation (Fig. 7).

ADVANTAGES OF PT

Kinects is an excellent example of how PT can be used to create exceptional buildings. The structural design of Kinects effectively used PT to resist horizontal thrust forces from sloping columns and achieve the project's key design goal of maximizing the floor area at upper levels where views are best and rents highest. PT flat plates were used at all levels, including the subterranean parking. This resulted in reduced mass, which translated to reductions in column size, foundations, and shear walls. It also lowered the floor-to-floor height and, importantly in this modern era of sustainability, reduced the building's carbon footprint.

The slab system used 8 in. (200 mm) thick post-tensioned flat plates with 13 ft (4 m) cantilevers at three sides of the building. This resulted in fewer perimeter columns and optimized slab bending moments. Concrete outriggers 18 in. (460 mm) deep by 8 ft (2.4 m) long were placed at the double cantilever corners to mitigate slab deflection by extending the support points of the cantilevers and allowing PT high points to be located nearly 8 ft (2.4 m) from the columns.

The slant columns, made possible through the use of post-tensioned flat plates, eliminated transfer beams, maximized formwork productivity, and maintained the rapid construction schedule. The PT structural design for Kinects also reduced the concrete volume and reinforcing bar tonnage, improved the formwork efficiency, and facilitated faster construction.

Fig. 7—Kinects Tower under construction.

Fig. 8—Construction of sloping column.

Location: Seattle, WA
Submitted by: Cary Kopczynski & Company
Owner: Security Properties
Architect: Bumgardner Architects
Engineer: Cary Kopczynski & Company
Contractor: Andersen Construction Company
PT Supplier: The Conco Companies
Other Contributors: Ready Mix Supplier – Stoneway Concrete

Jury Comments:

- The elegance and beauty of this project is in the simplicity of the structural design. Carefully positioned columns eliminate the need for fussy transfer elements and the benefits of PT are fully used to achieve the cantilever spans.

2019 AWARD OF EXCELLENCE: LA STADIUM

Fig. 1—LA Stadium and Entertainment District.

The LA Stadium and Entertainment District at Hollywood Park is not only an iconic project in the eyes of the public but it's also an exceptional example of ingenious engineering and proof of the positive impacts that integrating design solutions with construction can bring (Fig. 1).

The stadium will be home to the Los Angeles Rams and Los Angeles Chargers. It will feature approximately 3 million ft^2 (280,000 m^2) of usable space; an estimated seating capacity of 70,000; 16,000 premium seats; and 275 luxury suites.

No matter which team wins at Super Bowl LV (which will be played at the stadium in 2022), it's clear that this project has been a win for all involved.

DESIGN CHALLENGES

In LAX flight path

The project is located in the flight path of the Los Angeles International Airport (LAX), which creates restrictions to the structure's height. The field level of the stadium has been constructed 100 ft (30 m) below existing grade to satisfy FAA regulations.

On an active seismic area (Newport-Inglewood Fault)

The design criteria require that the stadium withstand a 9.0-magnitude earthquake. The complexity of the structural design was increased by the site being located adjacent to the Newport-Inglewood Fault.

Aggressive schedule

The design and construction schedule were determined based on eligibility to host Super Bowl LV in 2022.

DESIGN INNOVATIONS

Mechanically stabilized earth (MSE) wall

An MSE wall around the perimeter of the playing field allowed for excavation of the bowl and construction of the foundation support structures for the stadium roof to be performed simultaneously with steel erection inside the bowl (Fig. 2). Extensive three-dimensional (3-D) analysis was performed for the design of the MSE wall to ensure the displacement imposed by the design earthquake would be sustained. The MSE wall, at 252,000 ft^2 (23,000 m^3), is one of the tallest ever built, standing at nearly 100 ft (30 m) tall with a total of more than 5.7 million ft (1.7 million m) of straps and over 320,000 bolts.

Isolated post-tensioned foundation support structures

To reduce the seismic loads on the structure, the entire foundation support structure is isolated from the adjacent MSE fill—all made possible by innovative use of post-tensioning. At the bottom of the excavation, the structure is supported on a conventional cast-in-place spread footing, which serves as the lower mat foundation. This lower mat foundation is the base for plinth columns that carries vertical loads and is reinforced with PT (Fig. 3). The PT loops through the mat foundation and is simultaneously end stressed from the top of the butterfly cap at grade level. The horizontal loads are carried by lateral struts, also post-tensioned, and anchored at locations of zero displacement. The plinth columns and struts are connected by a cast-in-place butterfly cap; the shape is designed as efficient as possible by minimizing the mass and was referred to as the butterfly cap throughout its construction. These elements are embedded in the MSE fill and isolated. This foundation is the support of the precast blade columns, which are tear-shaped and hollow with external post-tensioning. 7.6 million ft (2.3 million m) or 1450 miles (2300 km) of PT strand is used to reinforce the foundation support structures for the stadium roof.

STRUCTURAL COMPONENTS

Plinth columns

Post-tensioned plinth columns inside the MSE fill support the stadium roof. The plinth columns are cast-in-place with a circular cross section and the tendons loop through the lower mat foundation to be double-end-stressed at the top of the MSE fill. The tendons are composed of three to five 27-strand tendons. The plinth columns are isolated with offset corrugated metal pipe (CMP) casings throughout their entire height.

Jigs were used to install the steel PT ducts for improved geometry control and to expedite the construction schedule of the 40 x 40 x 8 ft (12 x 12 x 2.4 m) lower mat footings (Fig. 4).

Lateral struts

Four x 6 ft (1.2 x 1.8 m) concrete lateral struts that are essentially tiebacks extend 150 to 300 ft (46 to 91 m) to a 30 x 20 ft (9.1 x 6.1 m) dead man anchored in soil behind the MSE fill and encapsulated in soil cement. Each strut is reinforced with four to six 27- to 31-strand tendons. The lateral struts were isolated by sitting on bearings or void forms and covered with a multi-plate isolation casing.

The foundation supports for the roof structure were laid out along the perimeter of the playing field, creating a spiderweb of lateral struts. This network of lateral struts created complexity for construction and detailing of the PT and deviations, as well as the isolation casing system. Detailed construction schedules, constant field coordination, and PT shop drawings were required to ensure efficient and consistent construction activities.

Figure 6 shows the multiple vertical elevations of the lateral struts. The variation in elevations are the result of canyons to provide access to multiple levels in the field. Figure 7 shows a construction aerial of the lateral struts and the complexity of construction sequencing with different levels around the stadium.

Once the struts were built, PT was installed through the deadman anchor up to their second anchorage in the butterfly cap. The tendons were stressed from the deadman anchor and protected with a concrete pour back.

Butterfly caps

At the top of the MSE fill, the plinth columns and lateral struts came together at a butterfly cap (Fig. 8) that housed the anchors for the PT tendons. For efficiency in the seismic design, the shape of this butterfly cap was optimized, resulting in a unique concrete outline. These cast-in-place caps are the support for the hollow precast blade

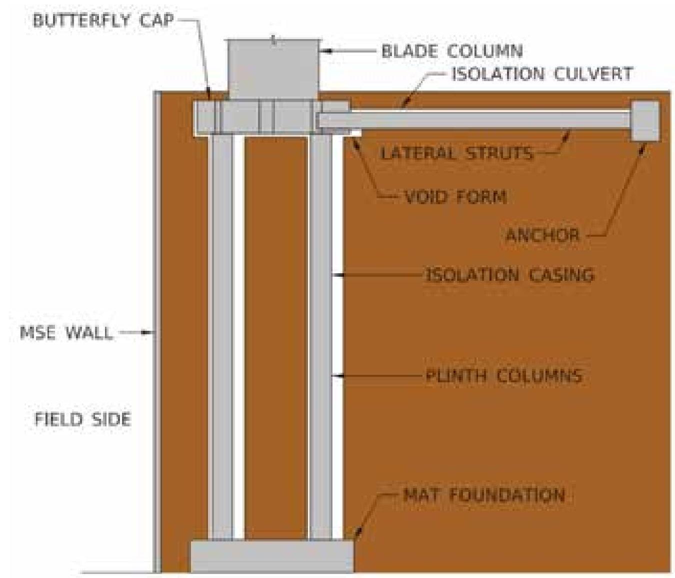


Fig. 2—Mechanically stabilized earth (MSE) wall.

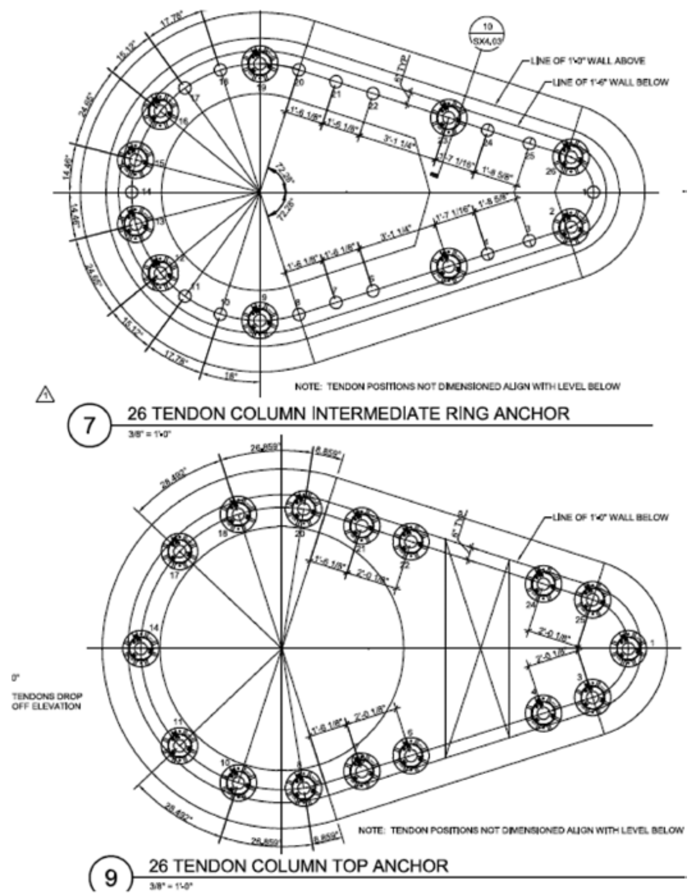


Fig. 3—Column PT layout.

columns that extend from top of grade to the roof structure. The blade columns are reinforced with 26 external PT tendons, making the shape of a J with the anchor point out the side of the butterfly cap. External tendons reduced improved seismic performance by reducing mass, and facilitated concrete placement in the thin-shell columns. Self-consolidating concrete (SCC) was used in the butterfly caps and mockups were performed on the field to troubleshoot the placement of the mild reinforcement, PT,

Fig. 4—Installation of steel PT ducts.

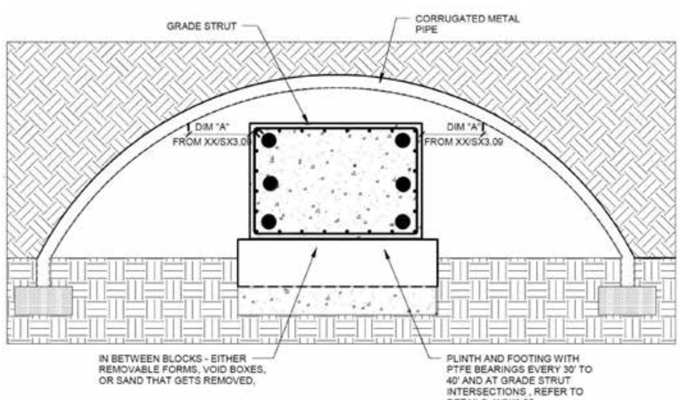


Fig. 5—Lateral strut section.

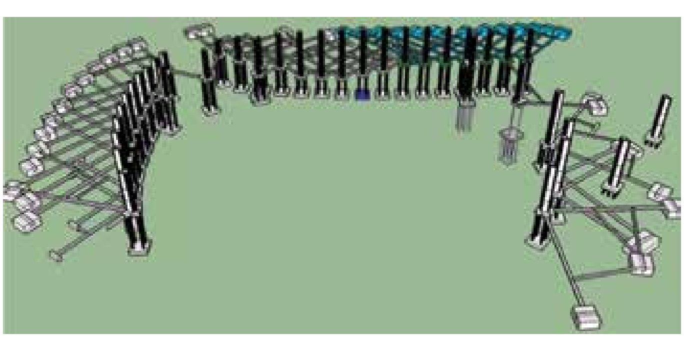


Fig. 6—Rendering showing location of multiple lateral struts.

and concrete pour. A total of 48 PT tendons are anchored in the butterfly caps.

Blade columns

A total of 37 tear-shaped blade columns are the last component of the roof foundation support structures (Fig. 9). These precast elements have external PT tendons inside the hollow portion of the column and were cast on site. Ten tendons are anchored at an intermediate ring segment with the remaining 12 to 16 tendons being anchored at the top of the columns.

CONSTRUCTION AND PT OPERATIONS

The innovative use of the MSE wall and isolated post-tensioned system resulted in savings of more than $100 million and the methodology selected allowed for the aggressive schedule to be met. Coordination was required for multiple construction operations occurring simultaneously, including installation of the MSE (Fig. 10) and straps around the plinth columns, construction of the plinth columns and lateral struts (Fig. 11 and 12), construction of the butterfly caps, erection of the blade columns, and PT operations.

The butterfly cap contained ducts and anchors for the PT of the plinth columns, blade columns, and lateral struts.

Fig. 7—Lateral struts under construction.

Once the butterfly cap concrete was poured and cured, the PT for the plinth columns could be stressed and grout cap components installed. Plastic PT ducts extended past the top surface of the butterfly cap to allow for coupling of the ducts and installation of the PT once the blade column segments were erected (Fig. 13). The vertical faces of the butterfly cap had recessed blockouts to house PT anchorages for J-shaped vertical tendons of the blade columns and horizontal tendons of the struts.

An additional construction activity occurring on site was the segment casting and storage for the blade column precast segments. Four casting cells and seven different forms were used to cast the 374 precast segments. Proximity of the casting yard to the erection site facilitated efficient coordination of segment transport. Construction became an iconic view for incoming LAX travelers.

The blade column segments were transported to their respective locations, placed on top of the butterfly caps, and stacked to the ring segment (Fig. 14). After verifying the geometry, the segment bases were grouted and 10 to 27 strand tendons were installed from above; each tendon running vertically through the column and exiting out the side of the butterfly cap to the anchors, and stressed from the ring segment. The remaining segments for the blade columns were

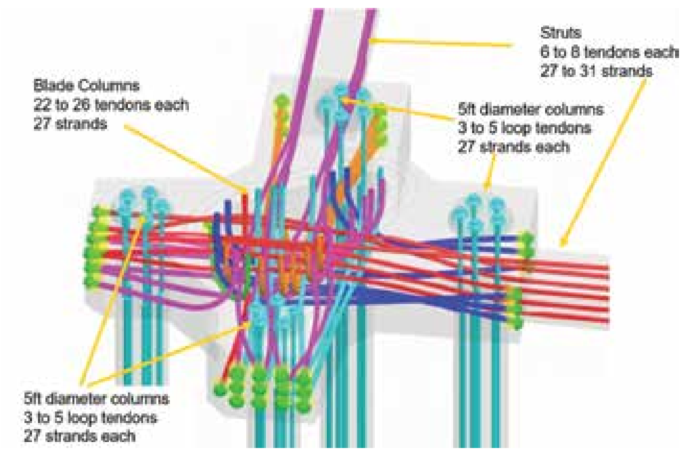


Fig. 8—Butterfly cap PT reinforcement.

Fig. 9—Blade column.

Fig. 10—MSE (Bottom left) and construction of plinth column caps.

Fig. 11—Construction of plinth column caps.

Fig. 12—Stressing of lateral struts.

Fig. 13—Staging of blade column segments ready for erection.

Fig. 14—Erection of blade column segments.

Fig. 15—PT stressing platform.

Fig. 16—Roof structure.

Fig. 17—Aerial photo showing blade columns along the perimeter of the stadium.

Fig. 18—Aerial photo after PT operations complete with remaining roof and stadium work to be completed.

erected up to the top cap segment, with sixteen 27-strand tendons installed from above and stressed from the top.

To safely and efficiently post-tension the columns at heights up to 165 ft (50 m) off the ground, a self-contained PT platform was used (Fig. 15). It housed all the equipment for the operation while also eliminating the need for tie off and protecting from dropped objects. The platforms were designed to easily sit on top of the ring or cap for one quick mobilization by the crew, using the erection crane after the last segment had been placed. The efficiency of the precast system and reduction of the construction schedule was the result of the design optimization by integrating design, engineers, and the construction team from the early stages of the project.

When the blade columns were completed, a cast-in-place parapet wall was built to support the isolator bearings and the roof structure (Fig. 16). Each parapet wall is unique to capture the slope of the roof depending on the location along the perimeter of the stadium. The completed blade columns received a fog finish to achieve the architectural vision for a monolithic column inside the stadium.

The future home of the Los Angeles Rams and Los Angeles Chargers will be one of the most spectacular venues in the country, made possible by the use of PT (Fig. 17 and 18).

Location: Inglewood, CA
Submitted by: Kiewit Infrastructure West Co.
Owner: Kroenke Sports & Entertainment
Architect: HKS
Engineer: Walter P Moore/Kiewit Infrastructure Engineers
Contractor: Turner AECOM-Hunt NFL JV
PT Supplier: Schwager Davis, Inc.
Other Contributors: Kiewit Infrastructure West Co. and McNary Bergeron & Associates

Jury Comments:

- The LA Stadium project exemplifies a very innovative use of post-tensioning systems in a very large-scale application and in an extremely high seismic design location.
- This project is a prime example of using the longevity benefits of post-tensioning paired with base isolation to minimize risks in high seismic zones. Advances in these technologies enable project teams to execute amazing projects that were not feasible just a few years ago.
- An incredibly attractive, creative, multi-faceted use of multi-strand PT technology in order to solve various complex engineering challenges on an iconic, futuristic structure.

2019 AWARD OF EXCELLENCE: SEGAL VISITORS CENTER

Fig. 1—Completed Segal Visitors Center Parking Garage.

The newly completed structure stems from Northwestern University's objective to push parking to the perimeter of the campus to enhance the internal pedestrian paths and open space. Out of this objective emerged the idea of combining a new parking structure with a university visitor center to form an entry element to the campus from the south (Fig. 1).

The building sits on the beach near Lake Michigan at a significant bend in a road. This location allows the structure to celebrate the campus's unique location on the water, as well as give it prominence from the perimeter of the campus. The architecture responds to these two different conditions—campus and lake.

The mass of the building relates the curve along the drive to the west and to the shifted grid of the southern portion of the campus on the north, south, and east façades. The overall height is limited by zoning restrictions, as well as a desire to minimize the presence of the parking structure on the site. An arcade along the western façade helps tie the building to the campus at a pedestrian level (Fig. 2). The structure occupies the southern end to maximize its visibility.

The 435-car parking structure is framed by planes of limestone—the historic material of the campus. On the north and east façades, vertical fabric fins (Fig. 3) infill the limestone to help screen the visibility of the garage while remaining open, thus eliminating the need to mechanically ventilate the parking areas. The undulating fins facing the lake animate the view from the beach and relate to the adjacent sailing center. With the understanding of the University's preference for an enclosed parking structure to reduce its presence, the south and west façades of the parking structure, which are visible from the campus and its perimeter, are clad with a fritted glass curtain wall. This, along with upturned concrete bumper walls that conceal headlights, screens the visibility of the cars. The parking entry is located at the north end of the site to minimize its presence from outside the campus. All sloped parking ramps are held to the eastern side to further disguise the parking structure in its context to the north, south, and west (Fig. 4).

The structure's interior space is carved out of the lower two floors of the mass with a double-height hall and auditorium at the south end, made possible by a U-shaped speed ramp to the parking levels. A wraparound terrace along the south and west façades celebrates the location

Fig. 2—Arcade along western façade.

Fig. 3—Internal view of the parking garage showing vertical fins on east façade.

Fig. 4—Parking garage under construction showing sloped ramps on eastern side.

Fig. 5—Encapsulated PT system used.

of the campus. The tall curtain wall enclosing the structure takes advantage of the views of the beach and the lake and also provides a transparency and visibility into the space from a prominent road, reinforcing its function as an entry to the campus.

A post-tensioned system allowed the structure to exceed the owner's expectations while fulfilling all the objectives. It was the ideal solution considering the site restrictions, geometry, and a higher level of durability requirements.

USE AND ADVANTAGE OF POST-TENSIONING IN STRUCTURE

The post-tensioned parking structure has many challenges. The long span beams with the curved west façade follow the roadway. The overall height limitation is due to the city zoning requirements. Supporting a very appealing architectural façade with vertical fabric fins infill the limestone on the north and east side and cladding with fritted glass curtain wall on the south and west side. A portion of the north end of the building was built over an existing parking deck.

To meet the aforementioned challenges, a post-tensioned floor system was used to reduce the member's sizes with 5.5 in. (140 mm) thick post-tensioned slabs supported by 36 in. (910 mm) deep beams with PT anchors at the curved edge of the slab. To meet the aggressive exposure zone requirements, a fully encapsulated PT system was used (Fig. 5). The pour strip was closed with a more stringent mixture design with fibers and a corrosion inhibitor. The shear wall pour was delayed by 28 days to allow for post-tensioning and to alleviate the restraining caused by the volume change movements.

Considering the site restrictions, geometry, and a higher level of durability requirements, a post-tensioned system was an excellent option that not only met the owner's objectives but also exceeded their expectations.

Location: Evanston, IL
Submitted by: Walker Consultants
Owner: Northwestern University
Architect: Perkins + Will
Engineer: Walker Consultants
Contractor: Power Construction
PT Supplier: Dywidag-Systems International, Inc.

Jury Comments:

- This project is a great example of combining the efficiencies of PT construction with striking architectural expression in a mixed-use parking structure application.
- Well-conceived design which integrates well with the building enclosure.
- Beautiful integration of the parking and visitor's center with the site and surrounding landscape.

2019 AWARD OF EXCELLENCE: EAST LINK EXTENSION—THE HOMER M. HADLEY FLOATING BRIDGE

USE AND ADVANTAGES OF POST-TENSIONING

Seattle, WA, is situated on a series of hills in a lowland area on the Puget Sound's eastern shore between the Olympic Mountains and the Cascade Mountains. The metropolitan area is lodged between Lake Washington to the east, Lake Union and Portage Bay to the north, and divided from West Seattle by the Duwamish Waterway. Expanding from Seattle in any direction other than southward requires crossing a body of water.

Part of the regional transit authority's expansion plans includes extending Light Rail Transit (LRT) east, from Seattle to the City of Redmond, requiring crossing Lake Washington. The closest existing structure from downtown Seattle that bridges Lake Washington is the I-90 freeway, comprised of the Homer M. Hadley (HMH) and Lacey V. Murrow (LVM) floating bridges.

The center roadway of the HMH Bridge was originally designed and built to accommodate high-capacity transit; however, the bridge was built under older design codes and without the knowledge of the LRT system's weight impacts. To further undermine the original design assumptions, more modern climatology studies increased the loading on the bridge from wind storms not anticipated in the original design. Therefore, with these structural concerns, the Transit Agency agreed to strengthen the bridge with post-tensioning.

Fig. 1—The HMH Bridge looking Eastward (HMH is on the left).

The bridge is comprised of individual concrete pontoons that are connected with bolted joints and an overlay placed on top, which acts as the roadway surface. The post-tensioning would keep the concrete pontoons in a compressed state, increasing the structure's durability and longevity. With the addition of post-tensioning, the bridge can endure stronger wind events, ensuring a more reliable transit system by reducing service interruptions caused by high wind events, in addition to increasing the life of the bridge (Fig. 1).

PROJECT OVERVIEW—E130: SEATTLE TO SOUTH BELLEVUE

The logistically and technically difficult E130 project consists of 7 miles (11 km) of LRT beginning at the existing International District Station (IDS), along an existing elevated roadway, on the Interstate 90 center roadway to the interface with the E320 project in South Bellevue (Fig. 2).

As a result of the length of the post-tensioning, the retrofit nature of the application, and the installation on an active floating bridge, nothing was routine in nature

> With 20 tendons containing over 1 million ft (305,000 m) of steel strand; 35,000 gal. (130,000 L) of grout, running 3600 ft (1100 m) through the middle segment of the floating bridge, this was one of, if not the longest, applications of post-tensioning ever completed.

and each operation had problems and challenges that required unique solutions. During stressing, the strands in the tendons were elongated approximately 20 ft (6.1 m) to create a force of 615,000 lb (280,000 kg) on each of the 20 reaction frames at the east and west ends of the bridge and as a result, when post-tensioning was completed, the bridge had shrunk approximately 3 in. (76 mm).

The reaction frames installed at the east and west ends of the post-tensioning operation weigh approximately 41,000 lb (19,000 kg) and contain approximately 1800 individual pieces that were hand delivered through hatches in the bridge deck and transferred to the far ends of individual pontoons. In total, 500,000 lb (230,000 kg) of weight was added to the bridge. Although offsetting ballasting was performed, at completion of post-tensioning, the bridge now sits lower in the water (an anticipated loss of freeboard that was closely monitored during every stage of construction).

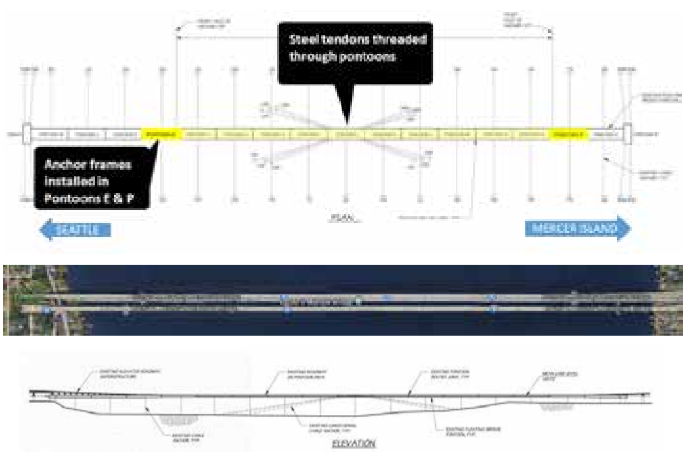


Fig. 2—Layout of the HMH Bridge, PT, and location of reaction frames.

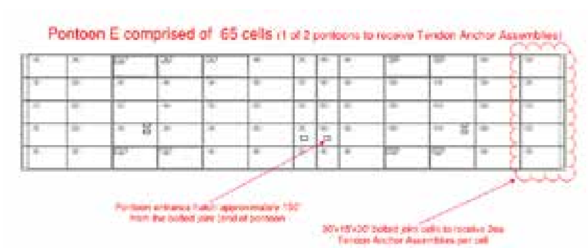


Fig. 3—Layout of pontoon cells.

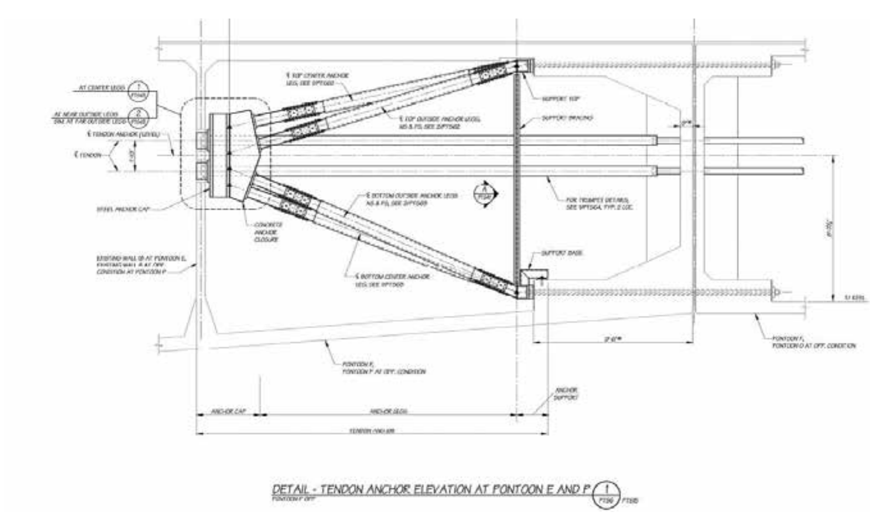


Fig. 4—Reaction frame elevation at Pontoons E&P.

Post-tensioning system design challenge

The design objective was to provide a post-tensioning system which would impart an additional minimum uniform pre-compression stress to the pontoon cross section of 300 psi (2.1 MPa). The rationale for this value was based on providing an additional 40% pre-compression to the keel slab (750 to 1050 psi [5.2 to 7.2 MPa]). Likewise, the pre-compression in the deck slab, which is a non-prestressed element, would improve from 0 to 300 psi (0 to 2.1 MPa).

The post-tensioning system consists of three main elements: continuous 3600 ft (1100 m) steel tendons, grouted ducts encapsulating the tendons, and reaction frames to anchor the tendons and transfer the compression into the pontoons.

Each pontoon is subdivided into cells, with five rows of cells running the entire pontoon length. Each row received four tendons for a total of 20 tendons running between Pontoons E and P (Fig. 3).

The reaction frames were designed to anchor the tendons and transfer the post-tensioning forces into the pontoons (Fig. 4). The reaction frame configuration provided the required stability and load distribution and, just as importantly, allowed for each of the frame elements to be transported from the roadway deck down into the pontoons and to their final locations. While the reaction frame concept addressed a number of the critical objectives, it also raised several design challenges—one of which was that the frame leg reaction at the bolt beam could not include any vertical components. The existing bolt beams located in the reaction frame cells were primarily designed for horizontal reactions and analysis showed that applying a vertical force at the face of a bolt beam may cause cracking of the pontoon. Therefore, to ensure that no vertical force was imparted into the bolt beams, a very low friction bearing pad system was designed. The design specified a testing regime to ensure that these bearing pad assemblies would perform as designed.

One other challenge brought on by the reaction frame concept was the extremely tight construction tolerances for the frame installation. Load eccentricities had to be minimized, as any additional weight required to accommodate these eccentricities would have resulted in an unacceptable bridge freeboard loss.

Adjustments required to be made for weight and freeboard management

The most important aspect of a floating bridge is that it stays afloat. Freeboard, the distance from the waterline to the bridge deck, is a critical component to ensure the bridge stays buoyant and is loaded consistently while putting minimum stress into the joints between pontoons. During construction, permanent material and transient loads were constantly measured. Freeboard measurements were taken and analyzed, and ballasting was undertaken to achieve the recommended freeboard.

CONSTRUCTION

Reaction frame assembly

The post-tensioning retrofit of the floating structure—10 pontoons, each approximately 355 ft (108 m) in length—required the installation of twenty 15-strand continuous tendons, just under 3600 ft (1100 m) in length. To transfer the jacking force of the hydraulic post-tensioning rams into the top and bottom slabs of the pontoons, structural steel reaction frames (tendon anchor assemblies) were designed as anchorage points for the tendons at both ends of the floating bridge. Each end of the floating bridge received 10 reaction frames. Each frame is comprised of 90 steel members weighing a total of 7 tons (6.4 metric tons) and has a corresponding frame on the opposite end of the floating bridge. Two full-length tendons travel between each reaction frame. To allow for the tendons to travel full length through 10 separate pontoons, over one-hundred-twenty 6 in. (150 mm) diameter holes were cored through transverse concrete pontoon walls every 30 ft (9.1 m). In all, over 2500 holes were cored to allow for the 20 tendons to run continuously between the reaction frames inside the bridge.

LIDAR mapping

Once assembled, the structural steel members that make up the reaction frames take up a significant portion of the available space provided in each of the pontoon end cells. Variances made each of the 10 tendon anchor assemblies' cells dimensionally unique. The variations in cell dimensions, however minor, required the detailed analysis of each cell's as-built condition. To collect and perform detailed survey analysis, the Contractor enlisted the assistance of an engineering company to perform LIDAR mapping (light detection and ranging) of each cell (Fig. 5). The drafting company began the 3-month shop drawing development process using this precise data (Fig. 6).

Fabrication overview

Once the shop drawings were approved, the fabrication process began for each assembly and required approximately 10 weeks to complete. The only access into each pontoon is through a 2 x 4 ft (0.6 x 1.2 m) hatch opening located near the midpoint of each pontoon. All structural steel members, ranging in weight from 200 to 4000 pounds, had to be fabricated to fit through the 2 x 4 ft (0.6 x 1.2 m) hatch openings and then transported 150 ft (46 m) to their destination cells. All manipulation of steel was done by hand with the aid of manual chain hoists and hand tools.

Reaction frame erection was even more challenging than loading the structural steel. The resources needed to manipulate, hoist, and set steel in its final position had to be hand packed into the pontoon and operated by hand during construction. The lack of headroom available within the pontoon cells and the fact that crews were working under live traffic on the floating bridge prevented the use of traditional hoisting methods to set the steel members. Custom-engineered hoisting devices, manual chain hoists/lever hoists, hand-operated hydraulic jacks, and temporary aluminum shoring towers were a few of the many resources used.

CORE DRILLING OVERVIEW

The 20 post-tensioning tendons anchored at paired reaction frames on either end of the floating bridge run continuous throughout the full length. For that to be possible, each tendon had to pass through more than 100 transverse walls within the bridge. This required drilling over 2500 6 in. (150 mm) diameter core holes inside the floating structure. All penetrations needed to be sealed to prevent any communication between cells. Temporary plugs in the core holes as penetrations were made to ensure that the pontoons maintained their watertightness.

To eliminate the risks from the confined space, transport of the welding equipment through cells and down ladders, and alignment requirements, the duct was welded into 175 in. (4.4 m) full-length sections and installed into the pontoons through the top hatches using a system of electric hoists and rolling sheaves.

A welding qualification procedure was developed requiring daily production fusion weld testing of each duct assembly to a pressure of 125 psi (0.86 MPa) on all assemblies. Passing sections were sealed to protect from the moisture until installation.

Vapor phase corrosion-inhibiting powder (VPI) was installed in the ducts prior to loading into the pontoons. Some of the concerns raised regarding blowing VPI inside the pontoons was the ability to contain it without jeopardizing the confined spaces, as well as the increased risk of creating potential blockages in the tendon ducts with the length at which the VPI would have to travel within the complete system. The VPI was installed inside the ducts prior to installation of steel strand, allowing the duct to be evenly coated with the powder to eliminate the concern of blowing powder into the pontoon confined spaces and helping alleviate the risk of the VPI leading to a plugged duct.

Fig. 5—Photograph of pontoon cell 13E (left) and image of pontoon cell 13E from LIDAR scan (right).

Once a length of duct was ready to be installed, hatch rollers were placed over the top hatch and inside the pontoon to guide the duct into the first cored hole of the transverse wall adjacent to the anchor gallery wall.

A total of 600 cells in Pontoons F through O received temporary duct supports, all having to be hand packed into the pontoons through top deck hatches in pieces for assembly. Each temporary duct support was installed to match the tendon profile, which lead to non-typical support heights to handle the variances from cell to cell.

Once stressed and grouted, all temporary duct supports were removed. Where duct spans were greater than 20 ft (8.2 m), permanent duct supports were also required.

Strand installation

Post-tensioning for the floating bridge included twenty 15-strand tendons each approximately 3600 ft (1300 m) in length. Each pontoon is five bays wide and each bay received four tendons spanning from reaction frames at the ends.

To limit the penetrations added to the pontoons, the plan evolved by using the existing top deck hatches and ballast ports to install the strand from the top deck of the end pontoons. A series of rollers and sheaves were temporarily installed through the pontoon allow conveyance down the

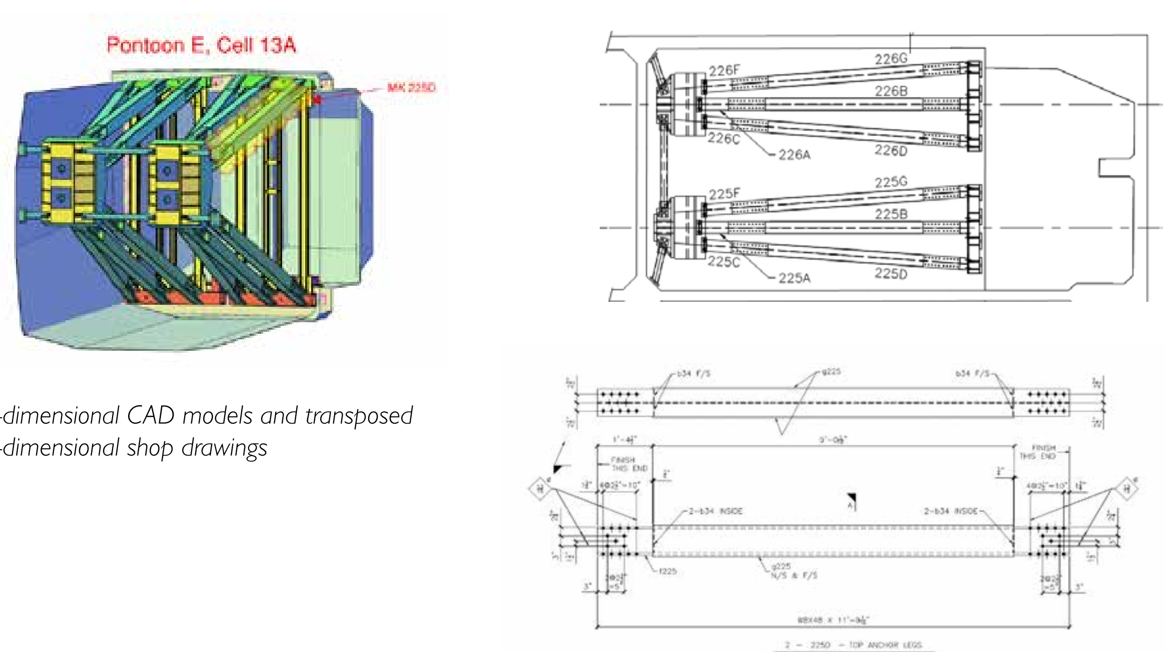


Fig. 6—3-D CAD models and transposed two-dimensional (2-D) shop drawings.

main access catwalk to the necessary cells, guiding the winch line into each duct. A winch was staged at each end and used a cable glider secured to the winch line to measure the length and force on the line, and the speed the line was traveling.

After running messenger and winch lines, two strands were attached from two separate strand packs that were staged on the top deck inside protective pack frames/payoffs. Swivels allowed the strand to uncoil and twist, releasing potential energy stored during the manufacturing process and eliminating the risk of kinking or damaging the strand. Typically, strand was pulled through a tendon at a rate of approximately 140 ft/min (43 m/min), at a force of approximately 1 lb/ft (1.5 kg/m) of strand pulled. The plate was pulled approximately 10 ft (3.1 m) into Pontoon E once it reached the end of the tendon. The plate was then disconnected from the winch line and strand and flipped around to pull in the opposite direction. The strand was then cut 10 ft from the trumpet in Pontoon P. Once the plate was flipped around, the winch lines were reattached to the plate and two new strands in Pontoon E were connected to the plate. Strands were then pulled in the opposite direction back towards Pontoon P, repeating the process. Each strand pack contained two full strands, allowing packs to be replaced in each payoff every two pulls from each end. This process was repeated, back and forth, for seven pulls to install 14 strands. The final pull was typically the single strand to make 15 strands in each tendon.

Once all strand was installed in a tendon, strand tails were cut to 5 ft (1.5 m) in length and the anchor caps and wedges were installed to prepare the tendon for stressing.

In total, 150 strand packs were installed in the 20 tendons, totaling over 1,090,000 ft (330,000 m) of strand. It took approximately 6 weeks to complete the installation of the strand into the bridge.

While the strand installation operation was in progress, additional pontoon inspections were completed during off-shift hours because the guide pipes and winch lines running into and through the pontoons would not allow the hatches, doors, or plugs to close fully. This meant the bridge could not be sealed watertight unless the installation equipment was removed. During these off-shift periods, additional plans to secure the bridge in less than 1 hour were made to ensure watertightness of the pontoons could be achieved in case of a major storm event or emergency.

STRESSING

Prior to stressing tendons, four 600-ton capacity hydraulic rams were lowered into the pontoons and moved to the stressing cells on both sides of the bridge.

Specifications required double-ended stressing of tendons to verify the forces in both ends of each tendon. Tendons were also required to be stressed in sets of four (each set called an order) starting at the center of the bridge and working north and south in pairs of two (Fig. 7).

Each order, when stressed, had to remain within 310 kip (140,000 kg) of loading about the vertical centerline of the bridge and 154 kip (70,000 kg) between the two tendons attached to any one anchor. To stay within these requirements, four tendons in each order were stressed simultaneously. Tendon orders one and two, the center eight tendons, were required to be stressed under a full bridge closure, allowing for necessary safety hold points to inspect how the bridge was reacting to the forces being applied. Due to the coordination effort needed to close a major interstate, the stressing date was selected 6 months in advance and an allowance of three 8-hour nighttime closures was provided to complete this work.

Once stressing of all 20 tendons was complete and forces and elongations approved, the strand tails were cut to approximately 3 in. (76 mm) in length and permanent grout caps were installed over the anchor heads. Each tendon was air tested determine large leaks for repair. If a tendon would not hold pressure, the entire tendon length was walked to identify the leak and repair it. Once repaired, the tendon was tested a second time and the process was repeated until it passed. Once the initial air tests were complete, a 100 psi (0.7 MPa) air test was performed to determine if there were any high-pressure leaks.

All tendons passed the 100 psi (0.7 MPa) positive pressure test prior to grouting operations.

GROUTING

Prior to the HMH Floating Bridge, the post-tensioning contractor had not previously grouted tendons 3600 ft (1100 m) in length; however, they had experience with grouting tendons approximately 1500 ft (460 m) in length. To mimic a shorter tendon, it was decided to grout from Pontoon K near the midpoint of the bridge, which was a low point for the tendons. Grout would then travel approximately 2000 ft (610 m) to reaction frame anchors in Pontoon E and 1600 ft (490 m) in the opposite direction to the reaction frame anchors in Pontoon P. Grouting equipment was staged including three high-shear colloidal grout mixers (one as backup), one 500 gal. (1900 L) holding tank, and two grout pumps (one for backup) all in secondary containment pans. Due to the large footprint of the equipment, especially once pallets of grout were

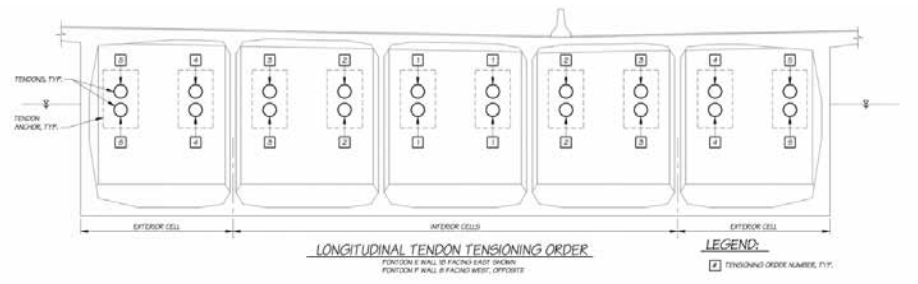


Fig. 7—Longitudinal stressing order.

staged, the grout operation was performed on night shift to minimize congestion. The equipment was plumbed together and trial batches of grout were mixed to find the proper proportioning to stay within the manufacturer's recommendations and within the specification requirements.

Due to the length of the tendons, it was determined the best method to grout would be vacuum-assisted grouting. Per the specification, all tendons were originally required to have a maximum vent spacing of 200 ft (61 m) with additional vents at all high and low points. The risk of leaks and need to achieve a vacuum led the team to approve one vent in each pontoon below the top deck hatches spaced at approximately 350 ft (107 m). If a vacuum could not be achieved and pressure grouting without the vacuum assist became the only option, additional vents would be added at all high and low points of the tendons. All vents had to be closed along the length of the tendon to achieve vacuum prior to grouting and could not be opened during grouting—otherwise, the vacuum would be lost. Vacuums were set up on the tendons several hours prior to grouting and all tendons were able to achieve a nearly perfect vacuum prior to grouting. The vacuum method was also planned to assist with pulling grout from the injection point to the tendon ends at lower pressures than what could be achieved with just pressure grouting.

Once grouting began on a tendon, the process was not allowed to stop until completion. Prepackaged grout was continuously mixed, allowing for multiple tendons to be grouted each night. One of the first major challenges was controlling the temperature of the grout. The water supply was through fire hydrants nearby on the project. To get water to the mixers, the most cost-effective solution was using water trucks. Water being supplied from the hydrants was approximately 68°F (20°C), which was not cool enough for the mixing process. To control the water temperature, a separate water tank was placed between the water truck and the mixers. The trucks filled the tank, ice was added until the water reached approximately 50°F (10°C), and the water was pumped to the mixers. This allowed a smaller volume of water to be closely monitored and adjusted to maintain the desired temperature. A trailer of ice was kept on the bridge to support the operation. Once temperature control was resolved, the process was simple to maintain throughout the operation.

When the grout holding tank reached roughly 200 gal. (760 L) of grout, the grout was tested to meet specifications (temperature, mud balance, flow cone) and then pumped into the tendon. Mixing of new grout would continue and the process would repeat until completion. If pressures higher than 120 psi (0.8 MPa) were observed at the pump, grouting was required to stop per the specifications. For all tendons, grout reached the end of Pontoon P first, approximately 15 to 20 minutes before reaching Pontoon E. Grout samples were collected at both ends of each tendon and the same tests were completed. Once grout was received at both ends of the tendon, final valves were closed and a minimum pressure of 100 psi (0.7 MPa) was applied and held for 10 seconds. The injection port was then closed and the next tendon would be started. Pumping took approximately 1.5 hours for each tendon. While the hoses were being moved, testing was being performed on the grout at the pump prior to starting the next tendon to ensure it was still within the manufacturer and specification requirements. Theoretical volume of grout for each tendon was approximately 1741 gal. (6600 L), equivalent to approximately eight pallets of grout. All tendons used within 3% of the theoretical, except for one tendon in which a leak was observed and plugged during grouting.

The grouting operation required a large amount of labor compared to typical operations. At the grout plant, four people broke bags and mixed grout; one person was stationed at the pump to monitor flow, volume, pressure, and perform tests; one person was at the injection point; one forklift operator moved grout pallets; and one additional person monitored the water volume, temperature, and eco pans. There was also one person in both Pontoons E and P to receive the grout at the tendon ends, shut valves, and perform final tests. To be proactive, personnel was staged in each Pontoon F through O to walk the tendons as grout was being pumped to either stop any small grout leaks that were found or call for help and stop the pump if there were any major failures. One person roved from Pontoon E to K and one person from K to P to transmit radio updates to the tendon ends and the pump throughout the process.

Two tendons had minor issues during grouting. One had a pressure failure in the duct while grouting. A repair boot was placed over the leak and the grouting continued to fill the tendon. A small void was later found in one of the grout cap ends of the same tendon and was repaired by filling the void with grout. A second tendon experienced a small leak from one of the Victaulic couplers that repaired itself and stopped leaking during grouting, leaving no void. Both tendons were completely sounded to ensure no voids were present along the length of tendons following the grout hardening.

In the end, all 20 tendons were successfully grouted in 6 days. Once all tendons were grouted, post-grouting inspections were performed at all grout caps and vents to ensure all were filled. Each cap and vent had a ball valve that was opened to perform an inspection. All angle changes in the tendons were also sounded for potential voids, as these were identified as critical locations. In total, five locations were determined as potential voids with the largest measuring 1 in. wide by 8 in. (25 mm wide by 200 mm) in length located at the top of a tendon near an angle change. Of the five locations, one was identified to be investigated further and was repaired. The remaining four locations identified were left without opening the system ensuring no avenue for introducing oxygen to the system was possible with the fully sealed system.

SAFE DELIVERY

As a result of the unique challenges involved in the construction of the post-tensioning, the work was complex and very atypical routinely performed off-shift and around-the-clock. Early in construction, lead was found in some of the pontoons and a lead testing and abatement program was implemented for construction to continue. With delivery, hand loading, and erection of large steel members through tight portals and within constrained pontoon cells, work was challenging and unrelenting (Fig. 8). Over 150 tons of structural steel was delivered and erected with zero safety incidents.

As a result of the training employed, skill and experience of the workforce, stellar safety program, and above all meticulous planning, the work was executed safely. Zero recordable injuries and no lost-time accidents occurred during the entire post-tensioning operation.

Fig. 8—Future LRT on the HMH Bridge.

Location: Seattle, WA
Submitted by: Jacobs and Schwager Davis, Inc.
Owner: Sound Transit
Engineer: Prime: WSP; EOR for HMH Post-Tensioning: KPFF Consulting Engineers
Contractor: Kiewit Hoffman (JV)
PT Supplier: Schwager Davis, Inc.
Other Contributors: Jacobs (Construction Management) Washington State Department of Transportation (Bridge Owner)

Jury Comments:

- This project represents a tremendous scale. Careful planning was obvious in the ultimate success. Also the use of current PTI/ASBI best practices was noted.
- Great use of PT to preserve an important transportation link.
- Creative application of post-tensioning to use precompression as a way to increase the robustness of the existing pontoon bridge structure and extend the life by at least 25 years.
- The planning, coordination, and executing the project with minimal interference to the traffic is commendable. Very creative in providing solution for anchorage reaction frame, stressing, and grouting one of the longest tendons with almost no defects in the grouting process is great. No reported safety incident is another plus point to the construction team.

2019 AWARD OF EXCELLENCE: R.H. JOHNSON RECREATION CENTER—SUN CITY WEST, ARIZONA

Sun City West is a Del Webb master-planned community for active adults located on the outskirts of Phoenix, AZ. It was started in the early 1970s and was completely built out by 1998. It has 18,200 housing units and a population of approximately 30,000 people.

The Sun City West tennis courts, located at the R.H. Johnson Recreation Center, were closed on March 26, 2018, to undergo a $1.8-million, 7-month renovation project. The main focus of the project was the resurfacing of the 15 tennis courts within the complex (Fig. 1). The complex reopened on November 29, 2018 (Fig. 2).

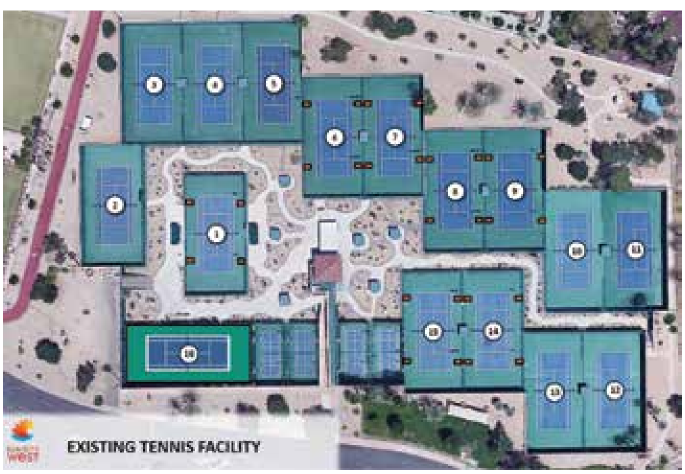


Fig. 1—R.H. Johnson Recreation Center before reconstruction.

Fig. 2—Reconstructed R.H. Johnson Recreation Center.

The plans called for the overall resurfacing of the 15 tennis courts with post-tensioned concrete slabs. The existing asphalt courts were less than 20 years old and were cracking and deteriorating (Fig. 3). This caused cracks to form in the Pro-Bounce surface, leaving dead spots and affecting playability. Most importantly, the courts had become a hazard to player safety.

The choice was to use a post-tensioned concrete slab over the existing asphalt courts (Fig. 4). The design called for a 3 in. (76 mm) layer of road base material placed directly on the asphalt courts, graded to a 1/4 in. (6 mm) tolerance, and topped with a 5 in. (130 mm) post-tensioned slab.

The Reserve Consultant hired by Sun City West to investigate the financial impact of replacing the courts versus making cosmetic repairs to the existing asphalt courts determined:

- The cost of replacing the Pro-Bounce playing surface multiple times over a 30-year period would be $2.2 million;
- Replacing asphalt and Pro-Bounce is more costly over time than concrete and lasts only one-third as long;
- Other Sun City and local facilities have moved to a post-tensioned concrete system—either overlay or new

Fig. 3—Condition of existing asphalt courts.

construction—because it has an expected service life exceeding 40 years; and

- It was appropriate to upgrade to current industry standards and technology when repair and/or replacement was being considered; the current industry standard is now post-tensioned concrete.

Construction began in early April 2018. The courts were constructed in two different sizes; single courts were approximately 60 x 120 ft (18 x 37 m) and double courts were approximately 120 x 120 ft (37 x 37 m). Tendon spacing was approximately 30 x 36 in. (760 x 910 mm) for the single courts, and 30 x 30 in. (760 x 760 mm) for the double courts. The 15 courts were constructed as three single courts and six double courts.

Unbonded 1/2 in. (13 mm) diameter PT tendons were used to produce a minimum 100 psi (0.69 MPa) compression at the center of the court slabs (Fig. 5). Special details were provided for the fence post and net post locations to ensure they would not restrain the free movement of the post-tensioned slabs (Fig. 6). A plastic vapor barrier was also used to further reduce subgrade frictional effects.

A leveling course of road base material (a blend of crushed rock and sand) was placed on top of the existing asphalt in lieu of placing the concrete directly on top of the asphalt (direct overlay method). This is required because the asphalt is not true enough in its plane or elevations to give the uniform thickness required for the concrete slab. It is important that the type of leveling material used is fine enough that it does not contain angular or sharp edges that will puncture or tear the vapor barrier. The leveling course is graded with a tractor-drawn, dual-mast, laser-guided grade box that is capable of producing subgrades with a tolerance of ±1/4 in. (6 mm). A uniform subgrade allows

Table 1—Area facilities which have moved to a post-tensioned concrete playing surface

Sun City—10 new courts/new complex with post-tensioned concrete
Sun City Grand—four courts renovated with post-tensioned concrete
Surprise Tennis Complex—25 courts with post-tensioned concrete
Phoenix Tennis Center—25 courts renovated with post-tensioned concrete
Kiwanis Tennis Center—renovating 15 pro bounce courts with post-tensioned concrete

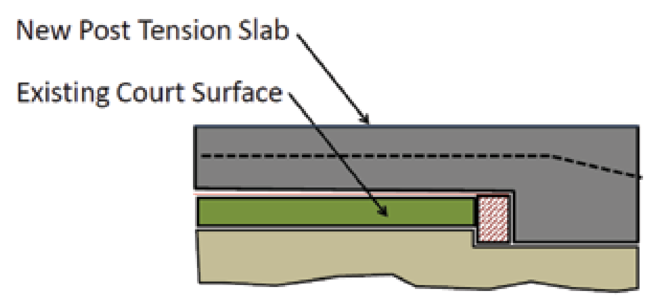


Fig. 4—Design detail showing new post-tensioned slab over the existing court surface.

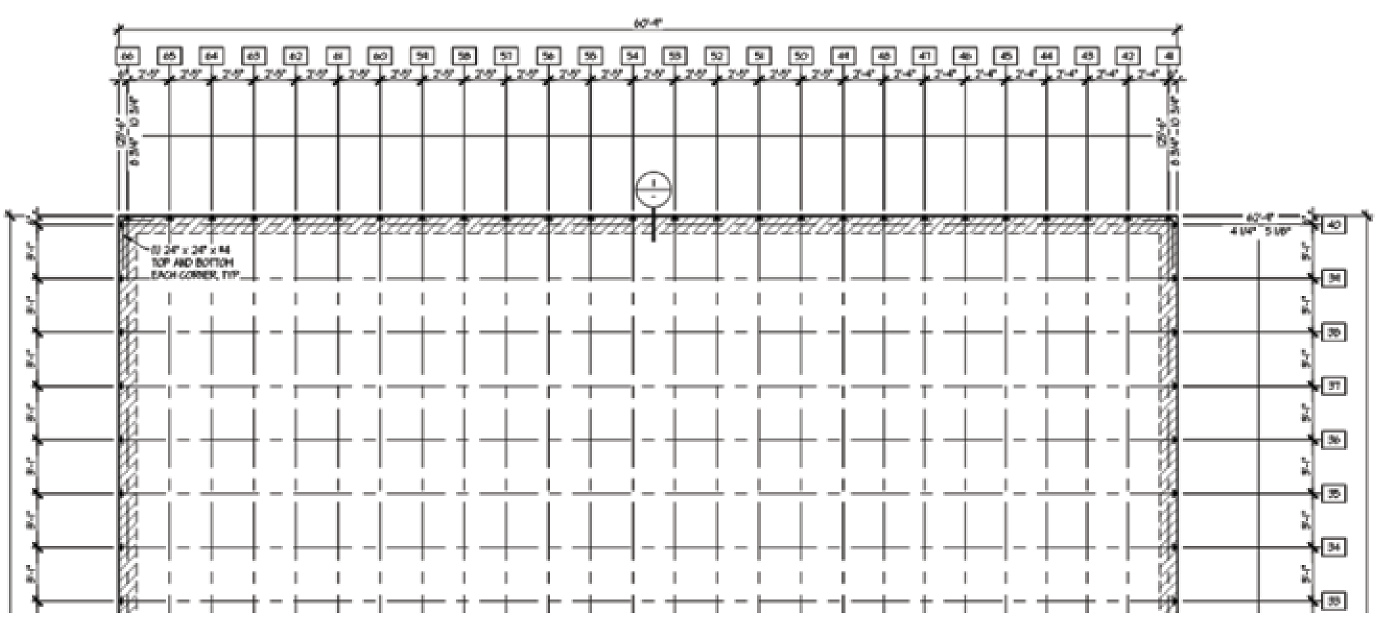


Fig. 5—PT layout.

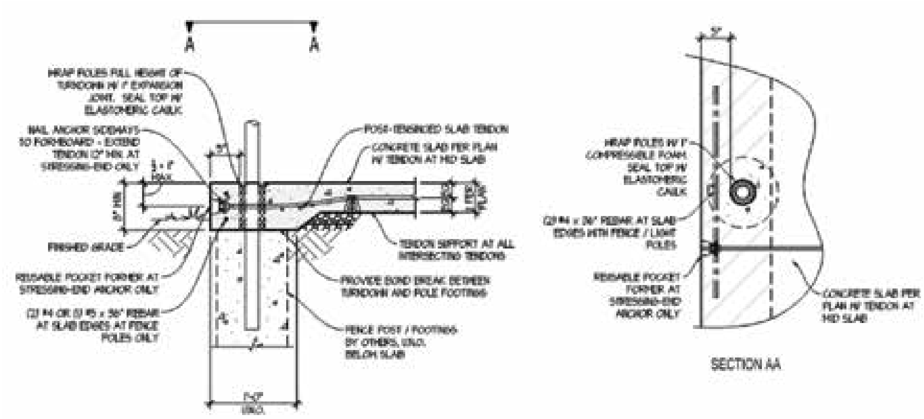


Fig. 6—Details of slab perimeter fence posts and mid-slab net posts.

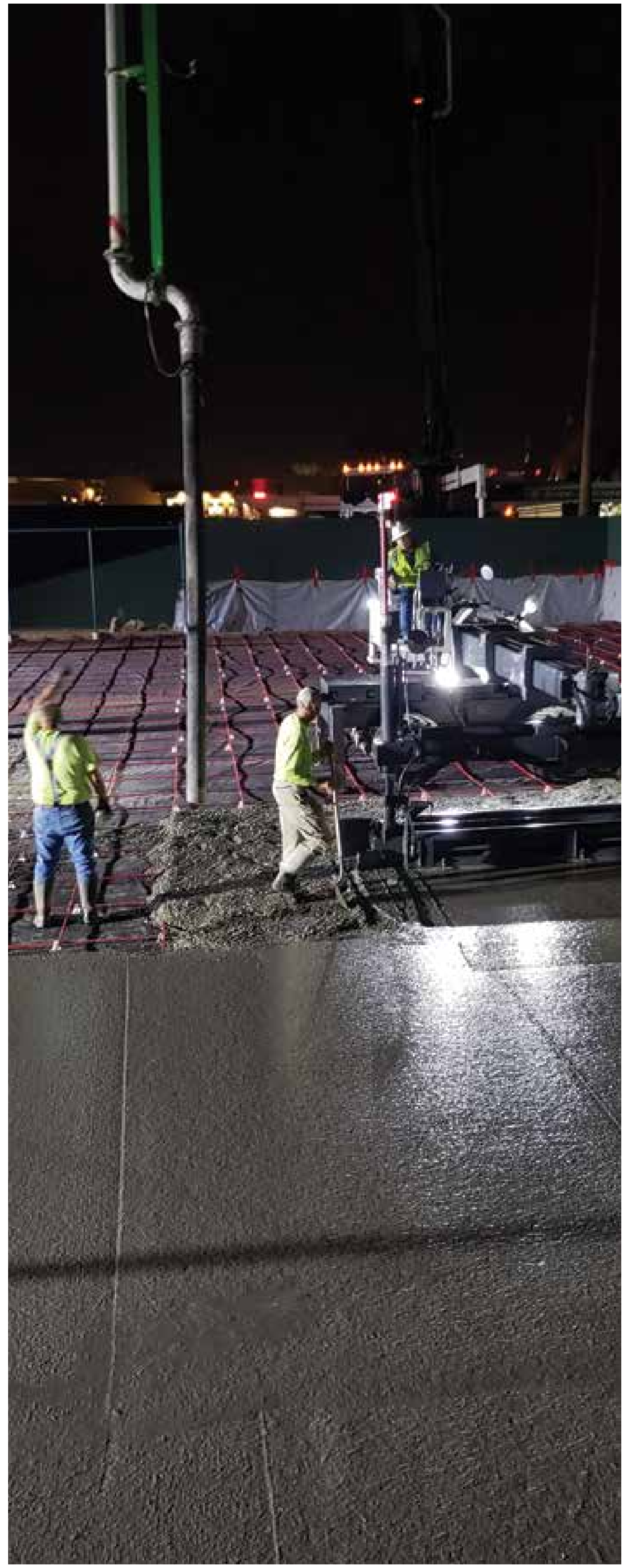
Fig. 7—PT sport court ready for installation of post-tensioning.

Fig. 8—Court ready for concrete placement

for uniform concrete thickness. This ensures that the prestressing forces provided by the post-tensioning system yield a uniform compression stress distribution throughout the entire slab.

On top of the leveling course, a 15 mil (0.38 mm) vapor barrier (Fig. 7) is placed that meets the requirements of ASTM E1745, Class A. If the gradation of the leveling course material results in a subgrade that is angular or sharp, it is recommended that a heavy non-woven geotextile fabric be placed on top of the subgrade prior to placing the 15 mil vapor barrier. The geotextile fabric will help prevent the vapor barrier from being punctured; this layer should extend all the way to the edge of the slab. Sport court coating manufacturers will generally not provide a warranty for their coatings unless a vapor barrier, in compliance with ASTM E1745, is installed beneath the slab to prevent subgrade moisture from transmitting through the concrete. It is important that screed pins or other accessories used for placing the concrete do not penetrate the vapor barrier. Placing concrete directly on top of a vapor barrier without the geotextile fabric has a tendency to increase the potential for surface cracking due to the bleed water being trapped at the bottom of the slab during the initial set of the concrete. The concrete contractor has concluded that using the geotextile fabric provides significant benefits during concrete placement, finishing, and curing, resulting in the concrete acting similar to being placed directly on the leveling base material. Furthermore, the geotextile fabric helps prevent damage to the vapor barrier from the laser screed during the concrete place-

Fig. 9—Laser screeds finishing concrete.



ment. The fresh concrete integrates and bonds with the surface of the fabric and helps to prevent early cracking. This layer of geotextile fabric should stop approximately 12 in. (305 mm) from the outside edge of the court to prevent wicking of moisture under the edge of the slab.

Special tendon installation procedures were followed during construction to ensure that the tendon sheathing was not cut, thus avoiding the potential of PT tendon coating escaping and transmitting through the top of the concrete.

The concrete mixture was a 3000 psi (21 MPa) flowable mixture with a fly ash content of approximately 17%. Because construction took place during the summer months when Phoenix daytime temperatures were often above 110°F (43°C), all of the concrete placing operations started between 1 and 2 a.m. (Fig. 8). Concrete was placed by boom pump and laser screed to avoid damaging the subbase, vapor barrier, or PT tendons (Fig. 9). This process eliminates potential damage from trucks depositing concrete directly onto the prepared subbase. It also allows for the accurate and rapid placement of the concrete while eliminating cold joints in the slab. The average placement was 10.5 yd of concrete every 6.5 to 7 minutes.

Fig. 10—Concrete for the last court was placed on September 28, 2018.

Fig. 11—The completed complex opened on November 28, 2018.

Many sports court coating contractors will not warranty coating bond if any type of curing agent is used. Such curing agents can leave chemical residue and laitance from wet cures that effect bonding of the playing surface coating to the concrete. Therefore, it is imperative the procedures previously outlined are strictly followed to allow the concrete to naturally cure without the use of curing agents.

Location: Sun City West, Arizona
Submitted by: Suncoast Post-Tension, Ltd.
Owner: Sun City West Property Owners and Residents Association
Architect: Dig Studio (Landscape Architect)
Engineer: Felton Group
Contractor: K.L. McIntyre General Contractor L.L.C.
PT Supplier: Suncoast Post-Tension, Ltd.
Other Contributors: Weaver Concrete Inc.

Jury Comments:

- A great example of life-cycle cost improvements and high-end aesthetic appeal using a well-thought-out, PT slab-on-ground solution.
- Detailing of the net posts and its interaction with PT court slab is very creative.
- Promotes effective use of PT for sport court slabs. Use of PT provides long-term durability.
- Post-tensioning proves once again to provide enhanced life cycle, providing lower overall cost of ownership.

2019 AWARD OF MERIT: BOTANICAL GARDEN ATOCHA—LA LIRIA BRIDGE

In one of the most colorful countries in the world, Ecuador, in a small town known as Ambato, you can find two beautiful antique buildings, La Quinta Juan León Mera and La Liria. These two historical buildings form part of the botanical garden Atocha – La Liria, which contain one of the greatest collections of plants and flower species in Ecuador.

Sometimes when the Ambato River flooded, the path between the two main buildings was closed for security reasons and visitors missed half of the experience. Therefore, the Ambato municipality decided to build a pedestrian crosswalk to improve the access (Fig. 1).

The owners wanted a pathway that is integral with the environment while offering safety of the pedestrians and in the possibility of seismic events. The small budget of the botanical garden, the necessity of limited construction disruption, and restriction on equipment size to preserve the flora and fauna of the site made this project difficult.

The engineering team designed an ingenious bridge taking advantage of the canyon of the river and the rock formations in both abutments of the bridge. The final design was a suspended bridge with two main ribs

Fig. 1—Botanical garden bridge.

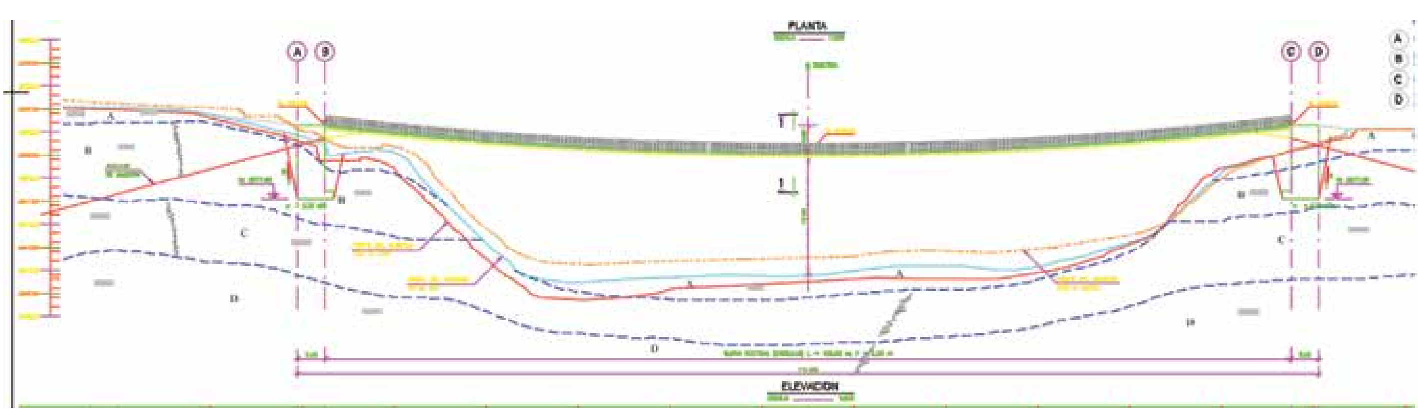


Fig. 2—Profile of the bridge from the design drawings.

that worked as the principal tension members and also as the railway for sliding the precast concrete segments (Fig. 2 through 5). Using this method of construction, cranes and other large equipment were not necessary and the environment was minimally disturbed.

This unique design and construction procedure allowed the botanical garden to minimize the impact of the work in the operation of the park and reduced the environmental impact.

Fig. 3—Two main cable ribs attached to the abutments as a railway for the precast segments.

Fig. 4—Precast concrete segment placement.

When all the segments were in place, the cable path was filled with concrete to build the main rib beam and to obtain the required stiffness of the bridge for seismic events (Fig. 6 through 8).

The lateral aerodynamic profile keeps the bridge safe for winds in the canyon and makes the bridge part of the environment, impacting it as little as possible.

Finally, this bridge meets all the requirements of the owners and also works as a reference for other engineering projects in the country.

Fig. 5—Sliding of a precast segment.

Fig. 6—Most segments in place and preparing the rib beam for concrete placement.

Fig. 7—Rib beam poured and path lighting installed.

Fig. 8—The bridge almost finished, showing the vertical curve and low profile.

Location: Ambato, Ecuador
Submitted by: STUP Latino America
Engineer: Planing Cia Ltd, Eng. Rodrigo Salguero Isch
Contractor: Eng. Ivan Acevedo
PT Supplier: STUP Ecuador

Jury Comments:

- An innovative way to use PT for a suspension pedestrian bridge with precast segment installation. An economic way to take advantage of the material and blend into the environment.
- Unique and innovative application of P-T technology resulting in a striking bridge - perfect for its location.

2019 AWARD OF MERIT: THE RITZ-CARLTON RESIDENCES WAIKIKI BEACH, PHASE 2

With the success of The Ritz-Carlton Residences Waikiki Beach Phase 1 tower, the project's developer was enthusiastic to construct the Phase 2 tower (Fig. 1). The completion of this second tower represented the largest new resort development in Waikiki in recent times and includes approximately 900,000 ft^2 (83,600 m^2) in two complementary 38-story towers, each 350 ft (107 m) tall. Phase 2 of this luxury hotel and condominium project opened to the public in October 2018 and has become one of Hawaii's most sought-after luxury residential addresses, offering the ultimate in resort-style living with unobstructed views of the Pacific Ocean, world-class design, and access to legendary amenities and services provided by one of the world's best hoteliers.

While not intending to upstage its neighboring Phase 1 tower, the Phase 2 tower was designed with a unique geometry. As the tower reaches the mean height of approximately 240 ft (73 m) in urban Waikiki, the floor plan rotates east to face the landmark Diamond Head crater located at the far end of the district. To achieve this, a large portion of the floor plan needed to cantilever as much as 30 ft (9 m) beyond the floor plan below. This

Fig. 1—The completed Phase 2 tower on the right.

was achieved by creating a grid of long span trusses with the primary truss spanning 60 ft (18 m) to a supporting truss that cantilevers 22 ft (6.7 m) off the tower's core. This truss was designed to carry 11 occupied floors along with a useable landscaped penthouse roof deck. The design of these final 12 post-tensioned slabs had to take into account global changing deflections of the truss, as each level was added along with localized deflections that might impact finishes and the exterior glazing.

Accommodating the rotating floor plan, however, wasn't the only challenge. This remaining parcel was wedged against and over an existing city-owned sewage pumping station. Vehicle circulation from an adjacent city service road and the city's pumping station parking lot needed to be accommodated through the building and another 10 underground easements, including the large sewage pump station, had to be worked around as well. With severe site constraints, building height and envelope restrictions, and a desire to maximize views and sellable space, structural simplicity was not a priority. Post-tensioning was used throughout the project to create innovative solutions to the structural challenges. The resulting building required the following features:

- Optimized thin post-tensioned slabs;
- Eighteen unique floor types;
- A post-tensioned transfer slab with upturn beams at the roof supporting a useable landscaped penthouse deck and 23 hanging steel columns to create grand two-story atriums in the penthouse units;
- Over 50 major wall and column transitions;
- Eleven transfer girders;
- A transfer cantilever truss system at levels 27 through 29; and
- Most critical to the occupants, unobstructed views of the Pacific Ocean and at the upper floors an additional rotation to add majestic views of iconic Diamond Head (Fig. 2).

HEIGHT RESTRICTIONS

Squeezing 38 floors into a 350 ft (107 m) height limit was no easy task, especially considering additional floor height requirements for public and premium levels and mechanical transitions at the transfer floors. To meet all of the project requirements, the slab system had to be as thin as possible while still maintaining acceptable sound transmission, vibration, and deflection characteristics. The only way to achieve this was through the use of post-tensioning. The majority of the slab areas are 7 in. (180 mm) thick at both parking and residential levels (Fig. 3).

One unexpected benefit of the height restriction is the structural efficiency created by the use of a thin post-tensioned floor system. Overall structural weight was reduced by as much as 30%, reducing column, wall, and foundation requirements. As seismic load is proportional to the structure's weight, the lateral load requirements were reduced as well.

Fig. 2—Rotated and cantilevered floors.

Fig. 3—Typical residential floor with iconic Diamond Head in the background.

Fig. 4—PT transfer girder.

Fig. 5—Two-story atrium penthouse unit with "skyhook" columns.

GRAVITY SHIFT

Encompassing 357,205 ft^2 (33,200 m^2), the Phase 2 building contains a loading dock and retail space at the ground floor, a vehicular drop-off space at the second floor, three floors of parking, one floor with back-of-house space, one floor with amenities such as a pool and meeting rooms, and 30 floors of condominium units for a total of 38 levels plus a usable roof deck. This diverse use leads to a total of 18 structurally unique floors.

As is typical with vertical mixed-use projects, the optimum column and wall layouts for each use rarely match the supporting levels below. Offset columns were incorporated throughout the project to shift support locations through varying floor layouts. In this tower, more than 50 major transitions were required for the vertical elements, with no tower columns going to the ground in their original location and some elements shifting in plan several times throughout the height of the building.

Adding to the challenge was a lack of structural depth, which in many cases prevented the use of conventional transfer girders except at the podium floors where 11 post-tensioned transfer girders (Fig. 4) were used to reposition tower-level columns and walls to work with an efficient parking layout and to create large open spaces at ground level for the loading dock, retail space, and to avoid City & County of Honolulu easements.

As previously discussed, one of the most challenging elements of the project was rotating the floor plan some 240 ft (73 m) above ground. This meant that a portion of the tower of approximately 2200 ft^2 (204 m^2) per floor at levels 27 and above had to be supported by a transfer truss system hovering over a sewage pump station below. This system included a three-story 22 ft (6.7 m) cantilevered supporting truss, an exterior 67 ft (20.4 m) long truss, and an interior 58 ft (17.7 m) long. These three trusses provide support for six concrete and structural steel columns that in turn support the post-tensioned floors above. The three trusses are supported by concrete walls at the elevator and stair cores.

At the top of the tower, the penthouse units were designed with spectacular double-story atrium spaces. These were achieved by hanging the penthouse post-tensioned slabs with 23 steel "skyhook" columns from the roof transfer slab (Fig. 5). The roof not only had to support the loads from the hanging columns but also the loads from heavy mechanical loads in the center and loads from rooftop terraces on the perimeter. This was achieved by using a post-tensioned concrete slab with post-tensioned upturn concrete beams.

Deflections would occur as floors were added above this truss, which needed to be considered in the design of the supported post-tensioned slabs. Truss camber along with slab studies of floor stresses created by truss deflection were reviewed. The added compression of the post-tensioning in the slabs helped to mitigate the additional stresses caused by incremental truss deflection.

To maximize views for the premium units in the upper floors, more glazing and less concrete wall were incorporated into the design of the north (back) face of the tower at levels 34 through 37. A series of small HSS 5x3 steel columns were designed to replace a solid concrete wall found in the lower floors. The HSS columns were designed to be hidden in the glazing mullions so as to give the appearance of a long glass wall at one side of the elevator core. The added punching shear capacity provided by the slab's post-tensioning allowed these small columns to support the floor without the addition of drop heads or panels.

Conceptual design of the Phase 2 tower began just as construction commenced on the Phase 1 tower. The Phase 1 and Phase 2 towers share common spaces at the podium floors but are separated by seismic expansion joints (Fig. 6). During the initial Phase 1 design, collaboration with the architect and contractor was necessary to ensure that the expansion joint was designed and built with corbels along the floor edge beams and columns to support the future Phase 2 tower even though its design was only in its very early stages. This proactive approach avoided the need to provide new columns and new foundations along the expansion joint that might appear to be an afterthought or retrofit to support the final Phase 2 building configuration.

LATERAL SYSTEM CHALLENGES

The project is located on the island of Oahu, Hawaii, a moderate seismic zone subject to hurricanes. The size, shape, and significance of the project justified a more rigorous approach to determining both wind and seismic loads on the structure. A wind tunnel study modeling the surrounding area was performed by RWDI Consulting Engineers & Scientists to determine the impact of both normal trade and hurricane winds on the building. Unique soil conditions found at the site justified a more in-depth seismic shear wave velocity test that was performed by the geotechnical engineer to determine seismic site class classification.

With the height restrictions on the building, shear wall link beams typically used as part of the lateral system needed to be rather shallow, reducing their effectiveness.

Fig. 6—Expansion joint at pool bridge.

Composite link beams were developed using standard concrete link beams combined with embedded strengthening steel plates. These plates were configured to allow slab post-tensioning to be placed through the links along with embedded conduit and pipe sleeves.

Due to the height of the structure, the relatively light weight of the post-tensioned floor system, and the results of the seismic shear wave velocity testing, seismic forces were reduced to the point where wind forces governed the design of nearly every lateral element, creating a very balanced and economical lateral system considering all of the vertical transfers in both gravity and lateral load resisting elements.

Construction on The Ritz-Carlton Residences Waikiki Beach, Phase 2 was completed in July 2018 (Fig. 7). Through the use of post-tensioning along with the hard work and innovation of the architect, the structural engineer, and the contractor, Waikiki now has iconic fraternal twin towers worthy of both name and location.

Fig. 7—Completed aerial view of the Ritz development from the south. Phase 2 is on the right.

Location: Honolulu, HI
Submitted by: Baldridge & Associates Structural Engineering, Inc. (BASE)
Owner: PACREP 2, LLC
Architect: Guerin Glass Architects, PC
Engineer: Baldridge & Associates Structural Engineering, Inc. (BASE)
Contractor: Albert C. Kobayashi, Inc.
PT Supplier: Suncoast Post-Tension, Ltd.
Other Contributors: PT Installer, CMC Rebar

Jury Comments:

- The Ritz Carlton project is a flagship example of the amazing advantages of post-tensioned concrete over other structural system solutions, and a very creative and attractive building solution of the doors PT can open to the entire project team.
- This project met the challenges of height limits, easements, a large wastewater pump station, and truck maneuvering areas under the building. These required long spans that could only be efficiently achieved through post-tensioning. Add to this hanging column with a roof level transfer slab, transfer girders and a transfer truss. Visually striking cantilever rotated and cantilevered floors at the upper levels.
- An extremely challenging site and program made possible by the use of post-tensioning.

2019 AWARD OF MERIT: PRECAST SEGMENTAL POST-TENSIONED BOX GIRDER WIND-TURBINE FOUNDATION

In January 2018, a Minnesota wind farm was contracted for the wind industry's first precast post-tensioned wind-turbine-foundation structure (Fig. 1). A precast segmental technology had been under development for several years by a startup company, and the wind farm owner was motivated to explore an industry problem. With cast-in-place technology, foundation structures cannot be taken apart or moved without destructive demolition. The owner wanted the ability to move the structure either prior to turbine tower erection or after 20-year decommissioning. Wind projects sometimes require late-stage changes in turbine locations or to the turbines themselves. This can even occur after foundation work has already begun.

WIND TURBINE BACKGROUND AND TYPICAL DESIGN

Land-based wind farms in the United States are usually constructed in remote locations such as the Midwestern plains or high mesas of the southwest. Often, western mountain passes and northeastern mountain ridges are locations for economically viable wind resources ranging 260 to 330 ft (80 to 100 m) above ground, where turbines are typically held aloft now. In southeastern states, new studies are showing economically viable wind at 460 ft (140 m) and higher. In the 2000s, the typical power rating for utility-scale wind-turbine generators was 1.5 megawatts (MW). Currently, the typical power rating is approximately 2.3 MW and is increasing rapidly with the top turbine manufacturers now marketing 3 to 4 MW machines. Because ground characteristics cannot be modified easily, increases in turbine hub height and rated power will correspond directly to the need for geometrically larger foundations that keep soil bearing pressures in the same range as current designs. As the overall proportions of foundations increase, the limits of conventional cast-in-place construction will be tested and demand for new methods of design and construction will emerge.

Fig. 1—Wind turbine with precast segmental post-tensioned box-girder foundation.

The current cast-in-place slab foundations are solid mat construction with typically 300 to 700 yd^3 (230 to 540 m^3) of concrete and 30 to 40 tons (27 to 36 metric tons) of mild reinforcing steel. When the excavation is completed, a seal slab is typically placed to produce a level surface and protect the subgrade in case of rain. The seal slab is normally low-strength concrete that must achieve strength before use. Seal slab cure is followed by reinforcing cage and formwork erection that lasts 2 or 3 days and involves two or three ironworkers and a crane operator. The foundations are comprised of a large slab buried in soil and a much smaller pedestal which protrudes above grade and supports the tower. Usually, there is no specific design for a placement joint (cold joint) in the slab, although there may be one between the slab and pedestal. Slab concrete placement times vary with batch plant location and productivity and number of concrete trucks available, but they can last as long as 8 hours. Concrete placement typically uses half a dozen concrete placers and finishers in rotation in addition to pump or belt trucks. The operations and final product are subject to weather-related issues during placement and curing. Slab placement is followed by a day or more of initial curing and then placement of the concrete for the pedestal. Once the pedestal and slab are sufficiently cured (typically two-thirds of design strength), the formwork is removed and backfilling can begin. When full design strength of the pedestal is reached, the tower can be erected and the tower anchor bolts can be post-tensioned.

Fig. 2—Assembly of foundation segments in the excavation.

Fig. 3—Multistrand tendons.

PRECAST POST-TENSIONED FOUNDATION SOLUTION

The project's 500 ft (150 m) wind-turbine and tower impart significant design loads on the foundation: 584,000 lb (260,000 kg) vertical dead load and 45,000,000 ft-lb (6,200,000 m-kg) overturning moment live load. The large overturning demand provided a design challenge and great opportunity for post-tensioning technology. The structure was designed by a combined group of segmental-bridge and wind-turbine-foundation designers. The prototype design was completed within a short schedule and required rigorous use of standards from both industries. The segments were match-cast with 10,000 psi (69 MPa) self-consolidating concrete at a southwest Minnesota facility and then transported 30 miles (48 km) to the wind farm site. The 12 girder segments were assembled and post-tensioned in the excavation (Fig. 2). The post-tensioning system is comprised of 16 tendons, eight in each direction. Two tendons for each girder are harped high at the central hub while the remaining six tendons travel low, horizontally, for maximal effect under wind load. Each tendon includes

19 strands, each 0.6 in. (15 mm) in diameter, and the strands terminate in the standard multi-strand trumpet anchorages used in bridges (Fig. 3). Approximately 8 million lb (3.6 million kg) of compressive force are applied in each of two span directions of the 61 ft (19 m) diameter structure.

After the hub was cast, the post-tensioning tendons were placed. With the central hub partially cured, the tendons were stressed, drawing the segments together at their joints (Fig. 4).

After tendon stressing, the foundation was backfilled with compacted soil—the same technique for a cast-in-place foundation. The space between the underside of the girders and the seal slab was also injected with controlled low-strength material to produce a uniform bearing condition at the underside of the foundation. The wind-turbine generator tower was erected and topped with the nacelle and rotor and energized in November 2018.

PROJECT COST AND COST-EFFECTIVENESS

The project represents the first time precast segmental bridge methods have been applied to support a wind-turbine generator tower. The one-time cost to complete design, formwork, precasting, and installation was \$950,000, \$80,000 of which was post-tensioning equipment and work. The market value of the standard cast-in-place technology is \$180,000. At production-scale, the precast post-tensioned system would reach price and design-schedule parity due to repeated use of forms and consistency of the structural design for a range of towers and ground conditions.

USE AND ADVANTAGE OF POST-TENSIONING IN STRUCTURE

The typical shallow foundation for wind turbines is cast-in-place reinforced concrete. It ranges from 300 to

Fig. 4—Assembled and stressed foundation.

700 yd^3 (230 to 540 m^3) of continuously placed concrete and requires transportation of reinforcement and erection in a remote location. A shallow foundation built with precast concrete segments and post-tensioning, however, permits all concrete work to be completed off-site and limits both road impacts and the time crews spend working inside the excavations. Other advantages include corrosion resistance and ease of reuse and removal. The wind energy industry is pursuing taller and more powerful turbines with larger foundation loads. The precast segmental post-tensioned form of shallow foundation will be able to grow in size more easily for larger loads than the tapered-thickness mat foundations which predominate in the United States now. Although the geotechnical investigation and design requirements are identical for both types, the demand for larger volumes of continuously placed concrete in remote areas and the concrete heat of hydration are not factors with a precast foundation. While foundation total costs of cast-in-place versus precast may be difficult to compare in some cases, an essential benefit to the levelized cost of energy (LCOE) is realistic and is expected to be demonstrable with a precast post-tensioned foundation solution.

Location: Chippewa County, MN
Submitted by: RUTE Foundation Systems, Inc.
Owner: Palmer's Creek Wind, LLC
Architect: RUTE Foundation Systems
Engineer: Barr Engineering Co. (Article written by Kirk Morgan)
Contractor: RUTE Foundation Systems and Fagen, Inc.
PT Supplier: Structural Technologies
Other Contributors: Marvel Bridge Engineers; Beton Consulting Engineers, LLC; K&M Rebar, Inc.; Armeni Consulting Services, LLC; American Engineering and Testing, Inc.

Jury Comments:

- A creative and innovative use of existing PT construction techniques, brought to bear on a different structural application. A timely and efficient solution, a fast-growing infrastructure/energy sector construction industry.
- New and innovative market for Post-Tensioning. Solves many constructability issues and provides the opportunity for reuse. Awesome!

2019 AWARD OF MERIT: MIAMI MUSEUM GARAGE

The brand-new Miami Design District Museum Garage is a product of a highly collaborative effort between architects and artists from all over the world (Fig. 1). As a unified front, the design team blended their creative minds to turn their conceptual designs into realistic, life-size artwork. Five distinct façades were crafted to breathe life and vibrancy into the Miami Design District.

Fig. 1—The Miami Design District Museum Garage.

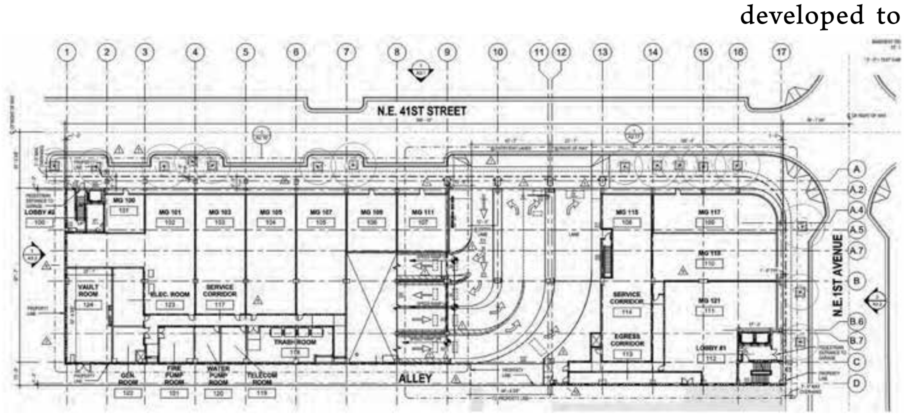


Fig. 2—First-floor layout.

The architectural design of this 927-space, seven-story, cast-in-place post-tensioned garage includes a basement level with car lifts, 22,000 ft^2 (204 m^2) of retail spaces on the ground floor including a restaurant, and 32 electrical vehicle charging stations (Fig. 2).

The structure was uniquely designed to create a seamless transition for visitors to access and reach their destination.

Another layer of complex criteria were the retail/restaurant space live loads and vast utility network requirements. Average parking garages typically require a 40 lb/ft^2 (1900 Pa) live load, but the levels integrated with mixed-use components were designed for 100 lb/ft^2 (4800 Pa) live load. Beams were also designed to be deeper with extra block-out room for MEP utilities to prevent any intersection with post-tensioned strands.

The stair and elevator towers were designed to be as open as possible to provide passive security (Fig. 3).

Miami's relatively high water table posed a challenge for the construction of the basement level. The basement slab had to be designed as a hydrostatic slab because it is below the water table. Auger cast piles were used for both load bearing and uplift loads (Fig. 4).

A slew of creative structural design techniques were developed to support the artistic façade designs. Slab edges were reinforced to address the eccentric loading generated by the façade components. A system of vertical mullions was also developed to transfer the loads of façades to the slab edges. This enabled a secure solution for façade constructability that was also able to deal with the requirements of the High Velocity Wind Zone.

The plans for the garage were developed using BIM to improve

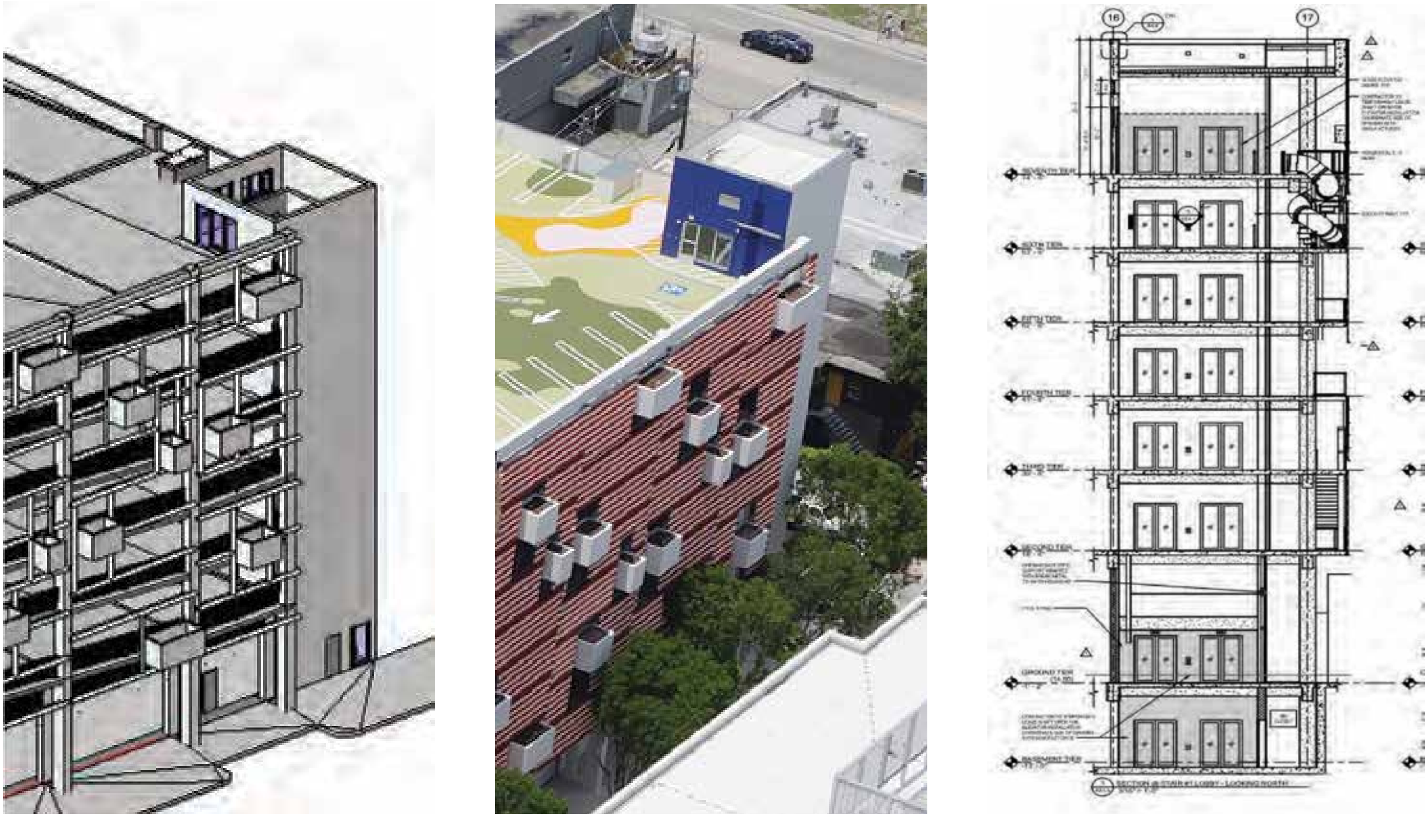


Fig. 3—Elevator tower.

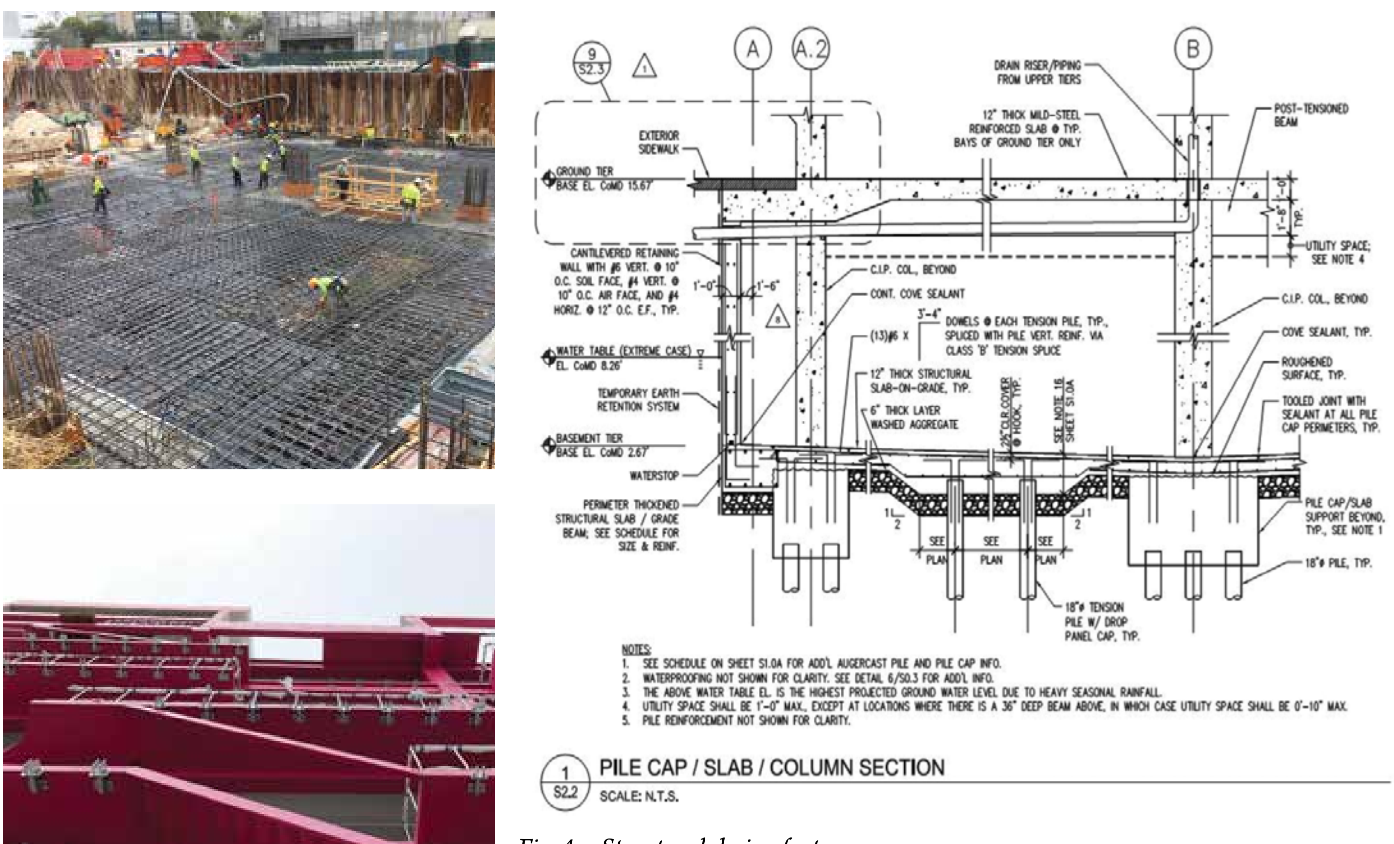


Fig. 4—Structural design features.

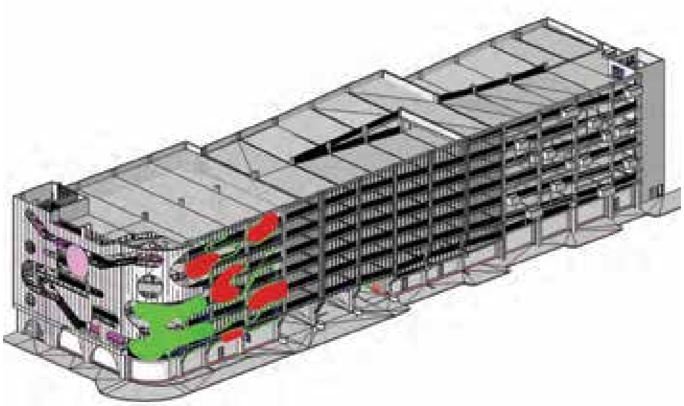


Fig. 5—BIM model.

Fig. 6—Roof deck with hurricane radar graphic.

coordination between the trades and reduce unforeseen conflicts during construction (Fig. 5).

The roof deck was transformed into a mesmerizing graphic of a hurricane that was painted on and inspired by Doppler radar imagery (Fig. 6).

The concept of the façades were borrowed from the French surrealist parlor game "Exquisite Corpse", in which artists would draw different parts of a body without knowing what the rest of it looked like. The five façade designers worked on their portion of the building without knowing what the other four designers were doing.

To complement the District's dedication to the creative experience, this unprecedented structure will provide an attractive connection between parking and the rest of the development. With its vibrant façades, dramatic lighting, and ground floor retail spaces, the Museum Garage was designed to engage all users and pedestrians (Fig. 7).

ANT FARM FAÇADE DESIGN

In an ant colony-inspired display of human activity, miniaturized public spaces and their connecting circulation spaces appear and disappear behind a perforated metal screen that provides visual contrast, shade, and protection (Fig. 8).

This design celebrates social interaction, sustainability, art, music, and the landscape with its garden, lending library, art space, and playground.

XOX (HUGS AND KISSES) FAÇADE DESIGN AND FABRICATION

In an effort to portray a gigantic, interlocking puzzle piece design, this portion of the structure is emblazoned with striping and bright colors. This strategy helps to recall the aerodynamic forms of automotive design and appear to float above the sidewalk below (Fig. 9).

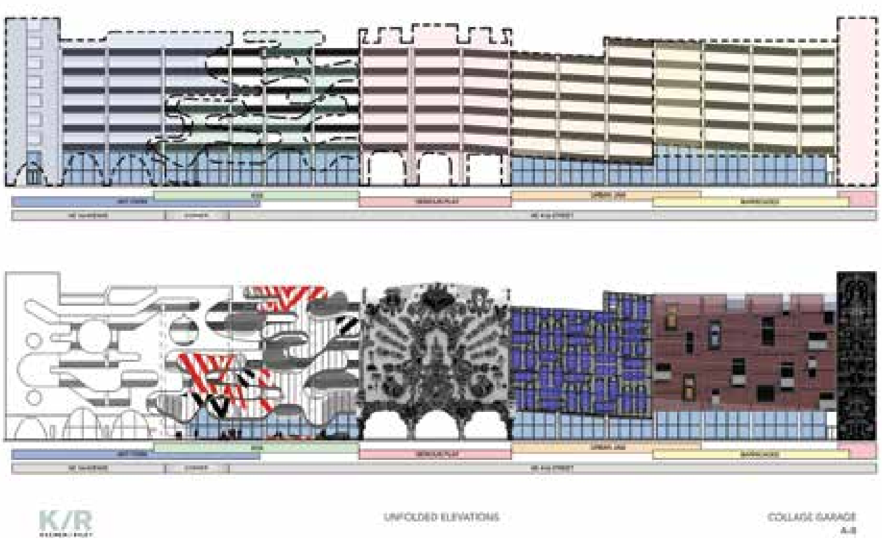


Fig. 7—Façade plan.

Smaller volumes, covered in metal screens, project outward and are activated with embedded light at night.

SERIOUS PLAY FAÇADE DESIGN AND FABRICATION

This façade serves as an entrance and exit to the garage, rendering a unique experience for all patrons. Constructed with a dark perforated metal backdrop, the façade includes a

Fig. 8—Ant farm façade design.

Fig. 9—XOX (Hugs and Kisses) façade.

variety of diverse 2-D and 3-D elements crafted from laser-cut metals and fiber resin plastic (Fig. 10).

At street level, the façade features four, 23 ft tall, full 3-D caryatids standing astride the garage's arched entrance and exits. Like the caryatids below, the composition above reflects a passion for video games and Japanese animation. Thus, the result is an unexpected juxtaposition of anime, tokusatsu, and manga.

URBAN JAM FAÇADE DESIGN

Where old structures and discarded spaces have been revived by architectural and urban designs, the Urban Jam façade draws from the rebirth of urban life in the Miami Design District.

Urban Jam suggests a "repurposing" of very familiar elements, using 36 gravity-defying car bodies rendered in metallic gold and silver. In effect, the styles of years past

Fig. 10—Serious Play façade.

Fig. 11—Urban Jam Façade.

Fig. 12—Barricades façade.

gain a second life as lux sculptural objects, caught in a surreal vertical traffic jam (Fig. 11).

BARRICADES FAÇADE DESIGN

The design is inspired by Miami's automotive landscape—particularly its ubiquitous orange and white-striped traffic barriers. In this case, the faux-barriers are turned right side up and form a brightly colored screen.

The façade has 15 "windows" framed in mirror stainless steel, through which concrete planters pop out above the sidewalk (Fig. 12).

Location: Miami, FL
Submitted by: Timothy Haahs and Associates, Inc.
Owner: Miami Design District Museum Garage
Architect: Timothy Haahs and Associates, Inc.
Contractor: KVC Constructors, Inc.
PT Supplier: Suncoast Post-Tension, Ltd.
Other Contributors: Curator: Terence Riley
Façade Designers:

- WORKac (New York, NY)
- J. MAYER H. (Berlin)
- Nicolas Buffe (Tokyo)
- Clavel Arquitectos (Miami, FL & Murcia, Spain)
- K/R (New York, NY and Miami, FL)

Façade Fabricators:

- A. Zahner Co (Kansas City, MO)—Facade Fabrication (Metals)
- Entech Innovative (Rockledge, FL)—Fiber Resin Casting

Design Team:

- Jamian Juliano-Villain (East Facade Mural)
- Island Planning Assoc. (Roof Mural)
- Bromley Cook (Specialty Structural Engineer)
- Ford Engineers (Civil Engineers)
- Florida Engineering Services (MEP)
- TLC Engineering (Technology/LV Engineers)
- Speirs + Major (Lighting Consultants)
- Green Space Strategies (Landscape Designers)

Jury Comments:

- PT allowed for an efficient and open garage structure coupled with a visually impressive exterior that will very likely become a point of interest in its own right within the Miami Design District
- Beautiful structure fitting its location. Post-tensioning provided solutions for the geometry, a clean structure and supporting the weight of the external elements such as the planters and architectural elements.
- This fantastic project puts a new, fun spin on parking by engaging the neighborhood and the arts district.

2019 AWARD OF MERIT: 25 BEACON

Beacon Hill in Boston is a conservation area characterized by narrow streets and heritage-protected Federal-style façades.

Numbers 6 and 7 Mount Vernon are two townhouses, which have been refurbished as luxury 21st century properties, along with a building to the rear, which was converted into luxury condos as part of the same project (25 Beacon).

The plan for the development was to add value with a parking basement that would serve both the townhouses and the condos in a location where on-site parking is extremely rare and on-street parking is not permitted. The basement had to be constructed in a way that would protect the above-ground structure during excavation and post-construction, along with the adjacent properties, which share structural walls. The design of the structural solution for the basement also needed to address the requirement for vehicle movements and a proposed capacity of 14 parking bays (Fig. 1).

The conservation area location also involved design and buildability challenges. Modifications to the external aspect of the properties had to be in keeping with the local architectural context and original design, and the narrow streets severely restricted vehicle movements, the equipment that could be used, and any cranage requirements.

Fig. 1—Beacon Street ground level entrance with elevator to basement. (Photo courtesy of richardgaylephotography.)

To overcome the heritage restrictions relating to exterior modifications, the basement parking lot was designed with a ground-level entrance and an elevator to take vehicles below the building (Fig. 2).

INITIAL PROPOSALS

The original design proposal was to use structural steel beams and underpinning to support the above-ground structure, with needling of all foundation walls to shore up the building while the underpinning and excavation works took place.

This plan involved a high level of risk to the integrity of the structure and neighboring properties, along with significant logistical, cost, and extended construction period issues. Access for the heavy construction equipment required for the underpinning was severely restricted and the size and weight of the steelwork would also have made it extremely difficult to deliver materials to site and maneuver them during the build.

Fig. 2—Elevator providing access to basement parking garage. (Photo courtesy of richardgaylephotography.)

Fig. 3—Lower side of the slab showing cable markings and excavation beneath (top-down).

The time and cost implications were also extremely onerous. Underpinning works would have taken up to 6 months and there are very few specialist subcontractors available in the Boston region, which could have caused additional delays for availability of the right expertise. The cost of the steel itself would also have been prohibitive due to the weight and number of structural beams required.

The proposed steel framework option would have required several support columns in the basement, which would have restricted vehicle movements.

POST-TENSIONING SOLUTION

The construction contractor began brainstorming alternatives and proposed the idea of a top-down construction program that would enable the above-ground structure to be supported without underpinning during the excavations. The contractor brought in the post-tensioning specialist, CCL, to advise on feasibility and design a suitable solution.

Combining mini piles with a post-tensioned slab tied into the structural walls, the project team designed a solution that supported the above-ground structure during the excavation and continues to support it post-construction. To avoid the need for a U-beam underneath the slab and embracing the existing structural walls, a post-tensioned two-way transfer slab solution woven through the walls was developed. The solution provided adequate capacity to support the self-weight and the movements of the above-ground structure while allowing for top-down construction.

The intent was once the slab reached its capacity, the structure above would be free from its original foundations, allowing these foundations (concrete and reinforcement) to be cut from beneath the slab (Fig. 3).

This two-way slab solution was very appealing to the project team because it leaves an open-plan parking garage with just two internal columns, essentially yielding a transfer slab of 32 ft (9.8 m) spans intertwined with the existing walls.

EXECUTION

The original wooden ground-level floor was removed and an opening was created in the wall to enable the piling works. Closely drilled 30 ft (9.1 m) mini piles were drilled around the internal perimeter of the external wall, enabling excavations to a depth of 10 ft (3.0 m)—sufficient head room for the parking basement—while providing a further 20 ft (6.1 m) foundation in the ground.

The top of the piles was braced by the post-tensioned slab, thereby providing a structure that could support the force of the earth during the excavations and carry the weight of the existing above-ground structure.

To tie the post-tensioned slab into the existing structure, openings were created in the existing structural bearing brick walls at regular intervals to the height of the proposed slab, creating a castellated effect with gaps in the brickwork for concrete (Fig. 4).

The size and location of these openings was modeled to ensure that the risk of structural wall movement and deflection of the slab were managed. All structural walls were also monitored throughout the works to corroborate the expected design values. In addition to the computer-aided design calculations, extensive hand calculations were carried out to verify the feasibility of this unique approach and demonstrate to the project team that the solution would deliver against all structural and risk management considerations.

With the castellated openings in place, the post-tensioning tendons and steel reinforcing bar reinforcement were installed into the brickwork and woven through the gaps. The positioning of the longitudinal and latitudinal post-tensioning tendons was carefully mapped out and some of the post-tensioning tendons were crossed through the wall openings, ensuring that post-tensioning of the slab was tied into the existing structure (Fig. 5).

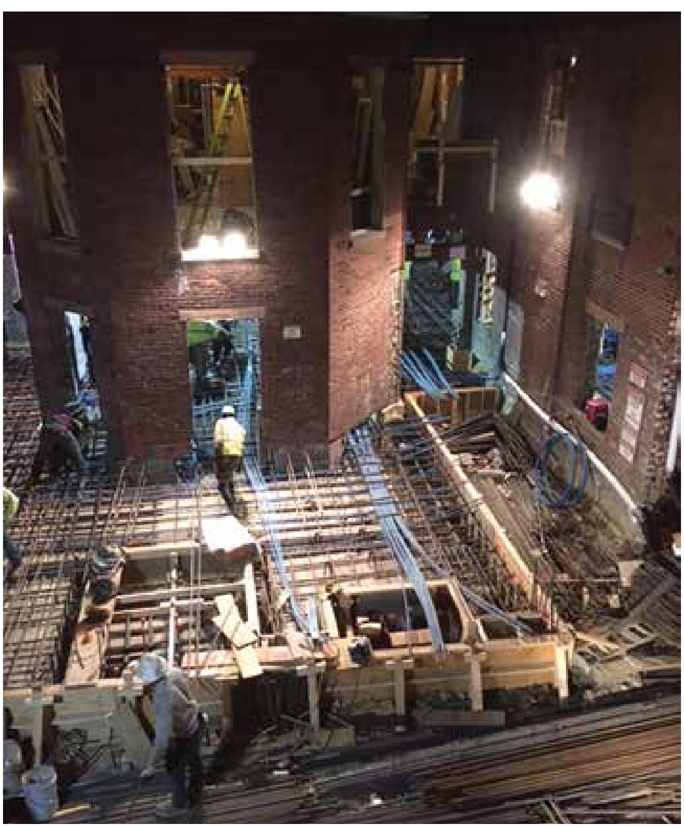


Fig. 4—Post-tensioning layout with historic building above.

Fig. 5—Interwoven post-tensioning tendons.

Fig. 6—Concrete being cast within a finished structure.

The concrete pour was carried out in two phases, filling some of the wall openings with concrete, and then casting the slab (Fig. 6). When all the concrete had cured to the required strength, the tendons were stressed, resulting in a self-supporting slab that was fully connected and running through the existing walls.

With this in place, the upper structure was no longer relying on the existing foundation, allowing for a complete disconnection between above ground and underground. Therefore, excavations could begin, with earth removed and all foundations of existing structure cut from underneath the post-tensioned slab to create the new level opening that would later become the car park entrance.

Shotcrete walls were created around the perimeter of the excavated area connecting the (now exposed) upper third of the mini piles and ensuring a permanent and effective retaining structure. Foundation walls and the basement slab were then also put in place to complete the parking basement structure (Fig. 7).

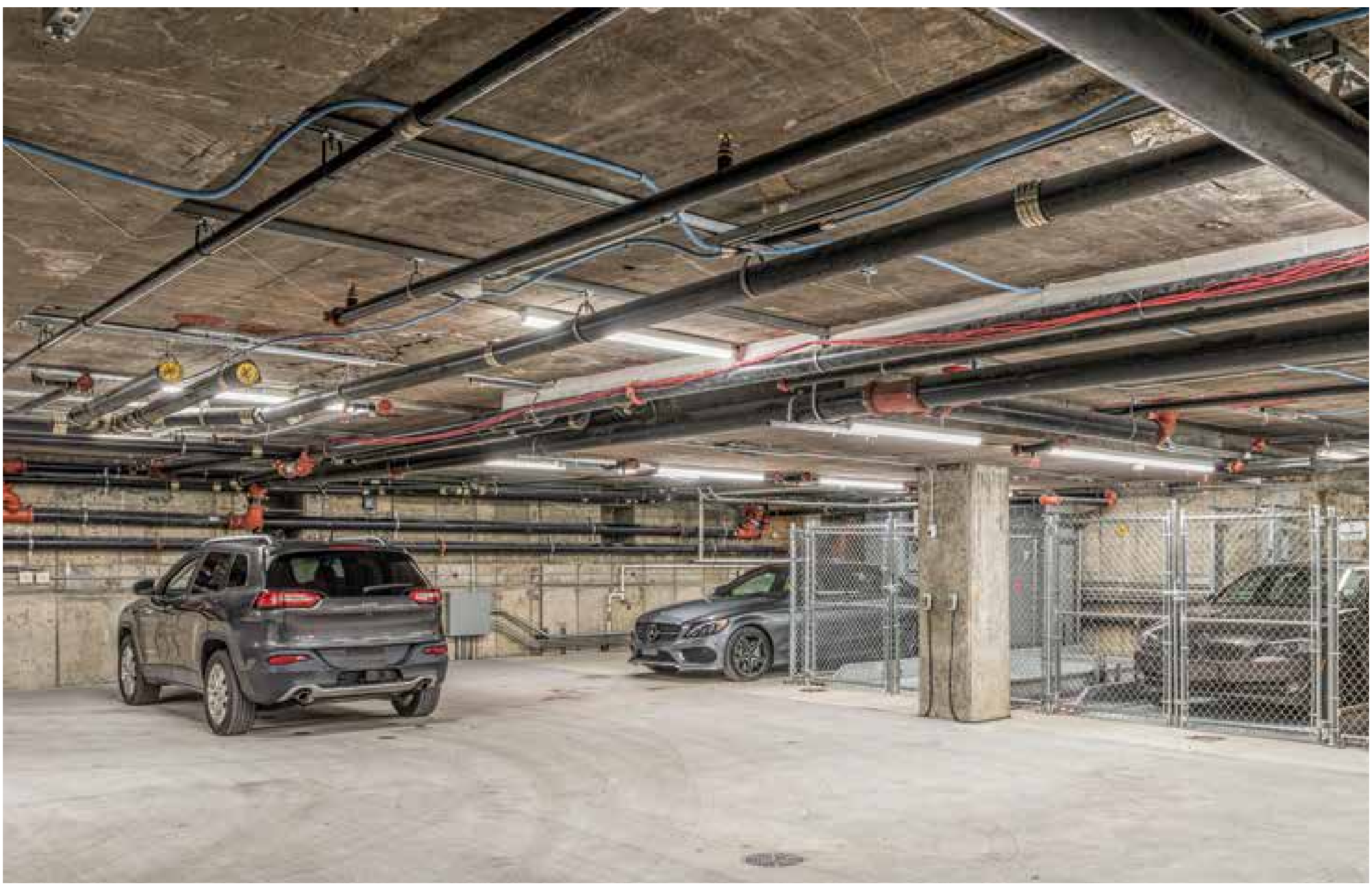


Fig. 7—Beacon Street parking garage interior showing one of only two supporting columns. (Photo courtesy of richardgaylephotography.)

Location: Boston, MA
Submitted by: CCL USA Inc.
Owner: SDC-DLJ Beacon Hill, LLC
Architect: CBT Architects
Contractor: SEA-DAR Construction
PT Supplier: CCL USA Inc.

Jury Comments:

- This was a creative solution in a very tight area and keeping with the historical architectural requirements.
- Innovative use of post-tensioning for challenging below-grade construction.
- Innovative solution utilizing PT to simplify required site stabilization, enabling top-down construction to create a parking basement in a historic area where parking is hard to find.

2019 AWARD OF MERIT: ATHLETIC RUNNING TRACK RENOVATION

Classic Turf Company installed four post-tensioned (PT) tennis courts for Region 10 School District in Burlington, CT, approximately 5 years ago. The PT system was explained and the client understood the advantages of the system. At this time, it was proposed that when the school was ready to rebuild the track and field facility, PT should be considered due to the increased durability and lower maintenance costs as compared to an asphalt subbase. This school district and town are smaller than most others in the area and they do not have the opportunity to raise the amount of money it takes to rebuild athletic facilities every 10 years. They wanted to do something that would last a lifetime. After 3 years of debating and raising money, the PT system was selected for the track reconstruction (Fig. 1).

The Classic Turf Company performed the conceptual design of the PT system, the finished track surface, and the construction of both. The overall scope of the project was to construct a new PT track with new perimeter fencing and the installation of new sod for the football field on the interior of the track. The existing track was asphalt, which had significant cracking. The planarity of the existing track, size, and slope were all within acceptable limits for the base of a new athletic running track. Therefore, the new PT concrete track could be installed directly on top of

Fig. 1—PT running track.

the existing without removing any existing material. When designing the tendon layout for the track, it was desirable to have compression in both directions, providing a monolithic slab with no expansion joints. The track was designed to have a residual compression of 120 psi (0.83 MPa). This was achieved by having transverse tendons to provide compression in the short direction and longitudinal tendons running around the circumference (Fig. 2). Four groups of boxouts were placed in around the circumference for tendon stressing (Fig. 3). The track concrete was placed in one continuous pour. The total size was approximately 45,000 ft^2 (420 m^2) and took just over 650 yd^3 (500 m^3) of concrete. After the concrete had cured, forms were stripped and the tendons were stressed (Fig. 4). The longitudinal cables were stressed from each of the boxout locations and anchored with center-stressing couplers (Fig. 5). These center-stressing couplers resulted in continuous tendons circling the entire track, resulting in one monolithic concrete placement with no joints. The tendons inside the boxouts were in tension so when the boxouts were filled in after the stressing was completed, the concrete bonded to the tensioned cables. The way this track was designed and constructed, the entire slab moves as one without any joints or individual sections being able to move independently.

After the concrete was completed, a 4 ft (1.2 m) tall perimeter chain-link fence was installed around the entire track. The fence posts were installed outside the PT slab. Because the existing track elevation was raised by 4.5 in. (115 mm), additional topsoil was need for the interior field. Approximately 200 yd^3 (150 m^3) of addi-

Fig. 2—Longitudinal and transverse tendons.

Fig. 3—Stressing boxouts.

Fig. 4—Track after concrete placement.

Fig. 5—Stressed longitudinal tendons.

tional topsoil was added, the field was regraded, and new sod was installed.

The last step was the installation of the track surface (Fig. 6). The track surface, Classic Elastic Track, is made up of a prefabricated styrene butadiene rubber (SBR) base mat that is attached to the concrete slab with a moisture-curing polyurethane adhesive. The top of the rubber mat is then sealed with a polyurethane pour sealer followed by the wearing course. The wearing course consists of pigmented polyurethane applied at a designed thickness and then covered with 40 to 120 mil (1 to 3 mm) in size colored EPDM. When complete, this surface is 100% waterproof, which provides extreme protection of the PT slab from Mother Nature's elements in New England.

With the PT concrete base and the Classic Elastic Track surface, this track will serve this community for many years to come with only routine maintenance being necessary.

Use and Advantage of Post-Tensioning in Structure

There are so many problems with using asphalt as a base material for athletic running tracks in the Northeastern United States. We are seeing newly installed asphalt developing cracks when used for tennis courts or running tracks just after several years. The surface material used for running tracks is extremely expensive—sometimes twice the cost of asphalt depending on what surface system is chosen. The running track surface material has a life span of up to 20 years if recoated on time and properly. When asphalt is used for the base of these tracks, the owner never has the opportunity to use the track surface to its maximum potential because the asphalt base develops structural issues even before the track surface needs replacement. This results in the demolition of the entire track and then reconstruction. The use of PT concrete in athletic running tracks is ideal. When designed and installed correctly, the PT base under the running track surface has the capability of lasting longer then the track surface itself. This makes the initial investment by the owner into the facility worthwhile. There will never be a need to reconstruct the entire track from the ground up, only the routine recoating of the track surface itself.

Fig. 6—Installation of track surface.

Location: Burlington, CT
Submitted by: Classic Turf Company
Owner: Region 10 School District
Architect: Region 10 School District/Classic Turf
Engineer: Classic Turf Company/Jerry Kunkel
Contractor: Classic Turf Company
PT Supplier: Builders Post Tension
Other Contributors: Stockmeir Urethanes, MEI

Jury Comments:

- Use of PT for running tracks without any joints and stressing them using center-stressing couplers (dog-bone) makes it unique
- Creative use of PT slabs to meet the client's needs. Durable, low-maintenance solution.
- Glad to see that a small school district had the vision to consider full life cycle costing in order to reduce long-term costs and ensure an extended functional lifespan with minimal maintenance.