2017 PROJECT OF THE YEAR: THE RITZ-CARLTON RESIDENCES WAIKIKI BEACH, PHASE 1

Waikiki. The Ritz-Carlton. The former is one of the world's premier vacation destinations and the latter is synonymous with luxury development (Fig. 1). The union of these two is The Ritz-Carlton Residences Waikiki Beach. Constructed in two phases, this project is the largest, most recent new resort development and includes approximately 900,000 ft^2 (84,000 m^2) in two complementary 38-story towers, each 350 ft (107 m) tall. Phase 1 of this luxury hotel and condominium project opened to the public in July 2016 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.

The project site posed many challenges. With severe site constraints, building height and envelope restrictions, and a desire to maximize views and sellable

Fig. 1—The Ritz-Carlton Residences Waikiki Beach, Phase 1.

space, structural simplicity was not a priority. Baldridge & Associates Structural Engineering (BASE) rose to the occasion, using post-tensioning throughout the project to create innovative solutions to the structural challenges.


Fig. 2—PT slab under construction.

Fig. 3—Major building transitions.

The resulting building required the following features:

- Optimized thin post-tensioned slabs (Fig. 2);
- Eighteen unique floor types;
- A post-tensioned transfer slab with upturn beams at the roof supporting 32 hanging steel columns;
- Over 50 major wall and column transitions (Fig. 3);
- Seventeen transfer girders;
- One hundred twenty ft (37 m) long post-tensioned concrete truss;
- Sloping W-shaped columns cupped with a post-tensioned restraint beam (Fig. 4 and 5);
- Concrete-wide-flange steel composite columns; and
- Most critical to the occupants, unobstructed views of the Pacific Ocean.

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 needed to be as thin as possible while still maintaining acceptable sound transmission, vibration, and deflection characteristics (Fig. 6). The only way to achieve this was through the use of post-tensioning. The majority of the slab areas are 7 in. (175 mm) thick at parking and residential levels.

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 also reduced.

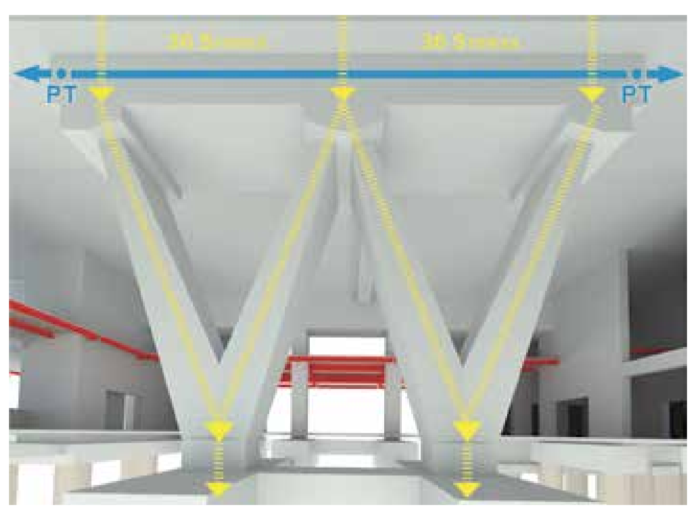


Fig. 4—Load transfer through W column.

GRAVITY SHIFT

Encompassing nearly 500,000 ft² (46,500 m²), the Phase 1 building contains five floors of parking; one floor with back-of-house space; two floors with amenities such as pools, restaurants, and a spa deck; 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 foundations and columns, as well as sloping 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 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 17 post-tensioned transfer girders in both downturn and upturn (Fig. 7) conditions were used to reposition tower-level columns and walls to work with an efficient parking layout and to create a large open space at ground level for the drop-off area located directly below the tower. The ground-floor loading dock space created a major challenge from a structural standpoint due to a number of constraints that would not allow the podium vertical elements over the loading dock to extend down to the foundation level. These constraints included very tight spaces for trucks to maneuver, city utility easements, and access requirements for an adjacent retail loading area. This meant that an area of the building with spans of 50 to 120 ft (15 to 37 m) had to be column-free (Fig. 8). To support the seven floors of podium structure above, four post-tensioned transfer

Fig. 5—W column under construction (top) and completed (bottom).

girders and a two-story post-tensioned concrete truss was designed to span the full 120 ft (37 m). Post-tensioning was used in the top and bottom chords of the truss to help control deflections and vibrations (Fig. 9 through 11).

At the top of the tower, the penthouse units were designed with spectacular double-story atrium spaces (Fig. 12). These were achieved by hanging the penthouse post-tensioned slabs with 32 steel "skyhook" columns from the roof transfer slab (Fig. 13). The roof not only had to support the loads from the hanging columns but also

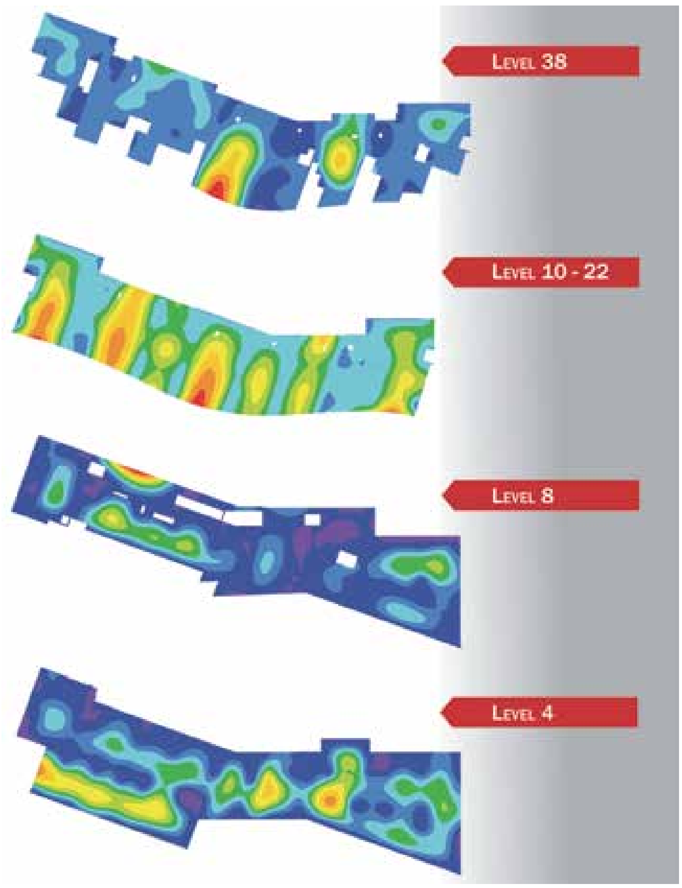


Fig. 6—*Ritz Phase 1 deflection plans.*

Fig. 7—*PT upturn transfer girder.*

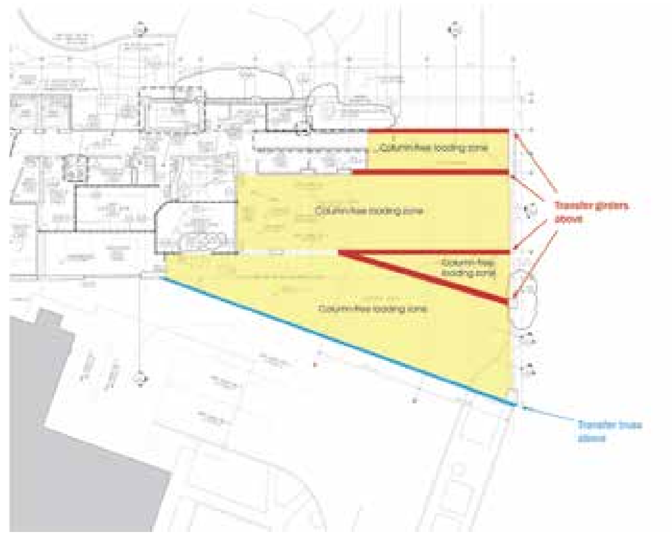


Fig. 8—Drawing indicating four PT transfer girders and 120 ft (37 m) PT concrete truss.

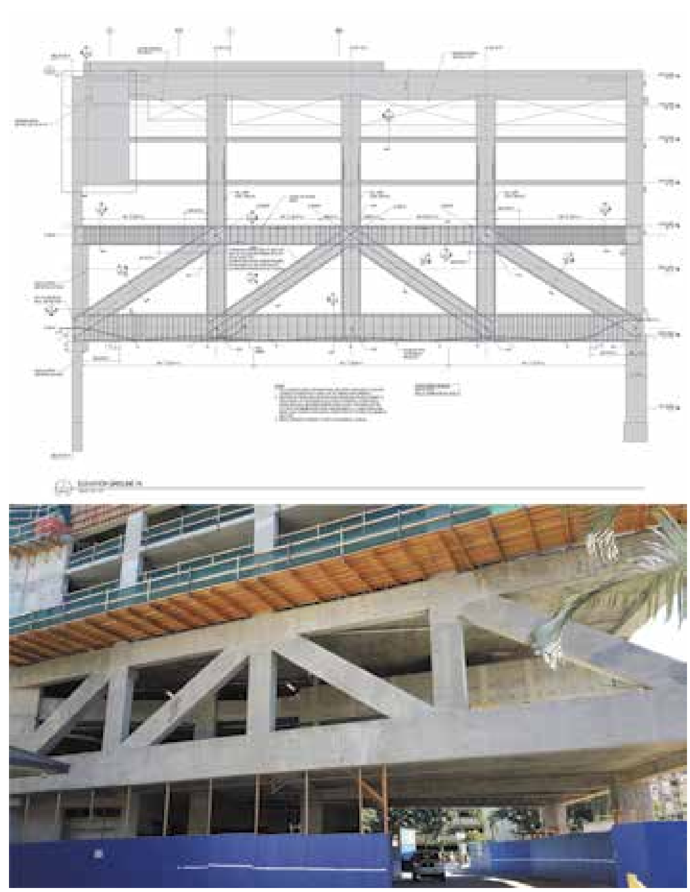


Fig. 9—PT concrete truss drawing (top) and completed truss (bottom).

Fig. 10—PT concrete truss under construction.

Fig. 11—Rendering of PT transfer girder.

Fig. 12—Rendering of double-story penthouse atrium unit.

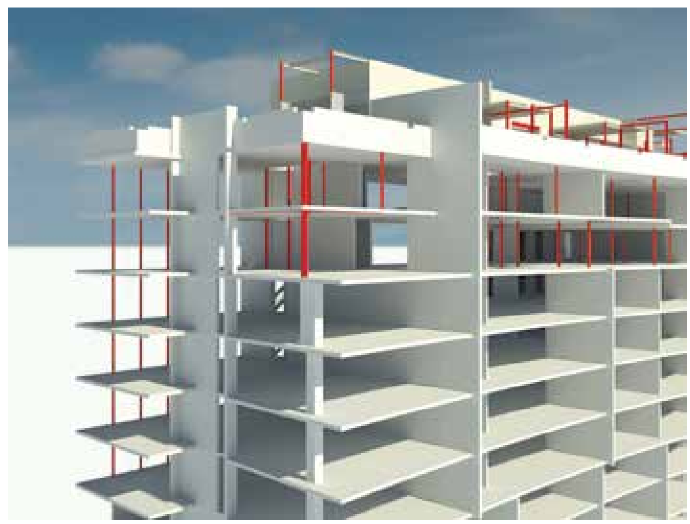


Fig. 13—Revit model of hanging penthouse slabs, PT slabs, and roof PT transfer slab.

Fig. 14—Penthouse unit rooftop pool and terrace.

the loads from heavy mechanical loads in the center and loads from rooftop terraces on the perimeter (Fig. 14). This was achieved by using a post-tensioned concrete slab with post-tensioned upturn concrete beams.

To maximize views for the premium units in the upper floors, more glazing and less concrete wall had to be incorporated into the design of the north (back) face of the tower at levels 30 through 38. A series of small HSS 5 x 3 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 40 ft (12 m) long glass walls at each side of the elevator core.

BASE started working on the conceptual design of the Phase 2 tower 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. During the initial Phase 1 design, BASE collaborated with the architect and contractor 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, HI, 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, with and without the Phase 2 tower, 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. BASE developed composite link beams 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.

CONCLUSIONS

Construction on The Ritz-Carlton Residences Waikiki Beach, Phase 1, was completed in April 2016. 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 an iconic structure worthy of both name and location.

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

Jury Comments:

- The designers were able to use post-tensioning to mitigate the height restrictions by using thinner post-tensioning slabs and transfer girders.
- This building is a wonderful example of the advantages of post-tensioned concrete in tall buildings.
- Without post-tensioning, this building would be much taller, much heavier, much more expensive, and would present many more architectural challenges.

2017 AWARD OF EXCELLENCE: DRESBACH BRIDGE OVER MISSISSIPPI RIVER

Over the main channel of the Mississippi River (Fig. 1), the new four-span Interstate 90 Dresbach Bridge features twin, cast-in-place, post-tensioned, segmental concrete structures with 508 ft (155 m) long main spans deliberately established to minimize river impacts. Its post-tensioned concrete segmental design enabled the bridge and its construction to avoid environmental impacts. It has the fewest possible piers; it was built from above with form travelers in balanced cantilever (Fig. 2), eliminating the need for large ground- and water-based equipment; and it allowed unrestricted commercial and recreational river traffic during construction, a critical need for the U.S. Army Corps of Engineers (USACE) Lock and Dam No. 7 located just upstream. Using efficient long spans permitted by post-tensioning resulted in a smaller permanent bridge footprint for the best environmental stewardship and greatest economy.

Post-tensioning allowed for great design flexibility. Aesthetically, the Dresbach Bridge serves as a beautiful companion to the pristine waters and recreational facilities of the Upper Mississippi River. Functional requirements of the crossing are enhanced though context-sensitive design developed with an understanding of the prominence, use, and visual features of the Mississippi River and nearby heavily forested bluffs and islands. Aesthetic design inspiration for the Dresbach Bridge came from the picturesque natural landscape of the surrounding areas (Fig. 3), and thus the greatest aesthetic opportunity came in custom-shaping the bridge piers. The heavily forested surroundings feature old-growth trees that appear to emerge from the land with great size and strength. The piers are shaped to honor the feel of the trees and extend the forest environment across the river (Fig. 4). While providing the strength to support the bridge, the pier shape is that of the trunk of a majestic tree supporting its foliage above.

Post-tensioning is a key component to build a concrete segmental bridge in the balanced cantilever construction method. Long, graceful cantilevers are only possible through the use of post-tensioning, and the Dresbach Bridge contains 19-0.6 in. (15 mm) diameter strand cantilever tendons to maintain flexural and principal concrete stresses. After the structure is constructed in cantilever,

Fig. 1—Interstate 90 Dresbach Bridge.

Fig. 2—Form travelers in balanced cantilever.

Fig. 3—Dresbach Bridge and surrounding landscape.

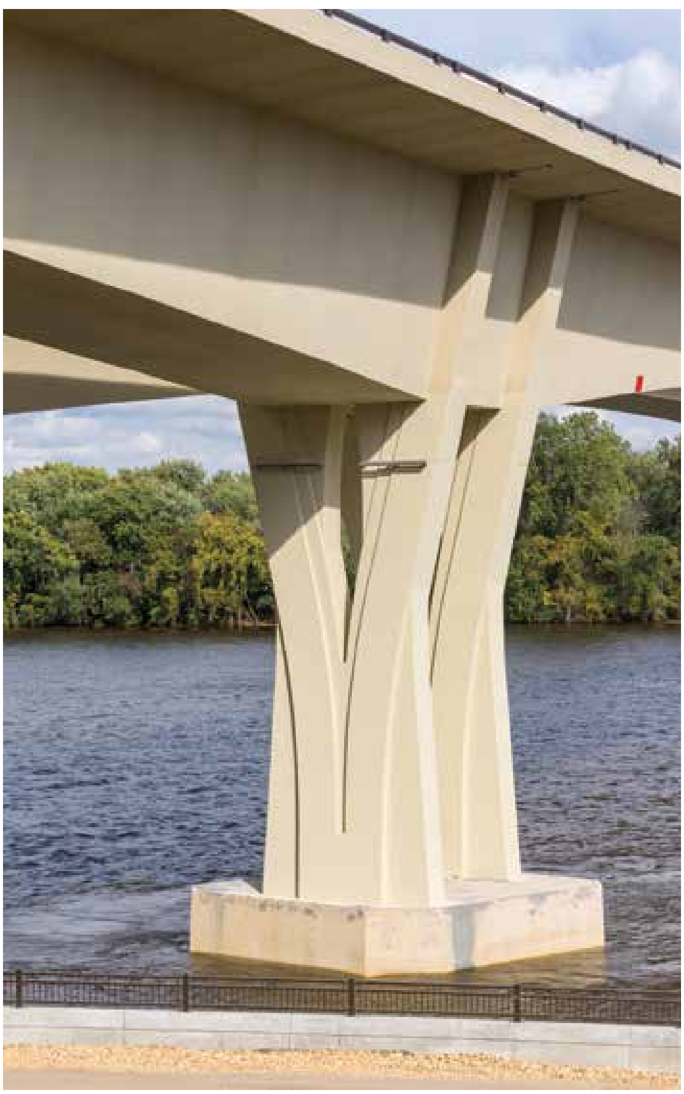


Fig. 4—Bridge piers.

closures are made between adjacent cantilever tips to form a continuous span. The combination of 27-0.6 in. (15 mm) diameter strand external draped tendons and 19-0.6 in. (15 mm) diameter strand bottom slab tendons serve as continuity post-tensioning to resist superimposed dead load, live loads, and the redistribution of the forces as the structure migrates from a cantilever state to a continuous state. Post-tensioning allows for residual longitudinal compression of the deck under normal service loads. Coupled with transverse post-tensioning, this bi-directionally pre-compressed structure minimizes cracking, which enhances the durability of the structure. Structural elements such as pier diaphragms and deviators also include post-tensioning designed to resist service loads and maximize durability.

Location: Dresbach, MN
Owner: Minnesota DOT
Architect: FIGG Bridge Engineers
Engineer: FIGG Bridge Engineers
Contractor: Ames Construction
PT Supplier: Schwager Davis
Submitted by: FIGG Bridge Engineers

Jury Comments:

- This structure is extremely graceful and very fitting for its location.
- This project is just very visually striking.
- This bridge is gorgeous aesthetically and one can imagine the construction challenges that were faced and overcome with the weather in this part of the country.

2017 AWARD OF EXCELLENCE: KELLOGG SCHOOL OF MANAGEMENT

Northwestern University's Kellogg School of Management has been located in the Donald P. Jacobs Center (formerly known as Leverone Hall) since 1972. To maintain its status as one of the world's top graduate business schools, Kellogg has relocated to a stunning new building located between the Allen Center and Lake Michigan in 2017 (Fig. 1). This architecturally ambitious new "Global Hub" helps to achieve Kellogg's goal of remaining first in class.

CREATIVITY OF STRUCTURAL DESIGN TO MEET NEEDS OF OWNER AND ARCHITECT

The structure aims to be an innovative, next-generation business school that inspires new forms of community building and adult learning by combining the physical and virtual in unprecedented ways. Project goals included:

- **Collaborative Learning**—Designed to encourage collaboration by pulling students and faculty out of traditional silos, the building is filled with open, inviting communal spaces that enable dialogue, spontaneous idea sharing, and inspired problem solving.
- **Inspired Design**—Striking 360-degree views deliver the lake to the east, Chicago's skyline to the south (Fig. 2), and Northwestern University's historic campus to the north and west. To energize people within, exterior glass walls will flood the building with daylight, fresh air, and amazing views.
- **Future Flexibility**—As the business world, teaching styles, and research evolve, so must the building. Classrooms and faculty spaces need to be easily reimagined and reconfigured; walls will need to be moved, reshaped, and rewired.

To address these goals, the new 415,000 ft^2 (38,500 m^2) facility consists of four, six-story concrete "lofts" situated around a large central atrium at the first floor (Fig. 3). The atrium is open for three floor levels and is surrounded by a network of pedestrian bridges, cantilevered balconies, monumental stairways, and is accented by a hanging conference room overlooking the space. Above the atrium is a central Faculty Summit area with interconnecting monumental stairs and flanking seminar rooms. A structural steel penthouse structure sits atop each of the lofts and contains mechanical and HVAC equipment.

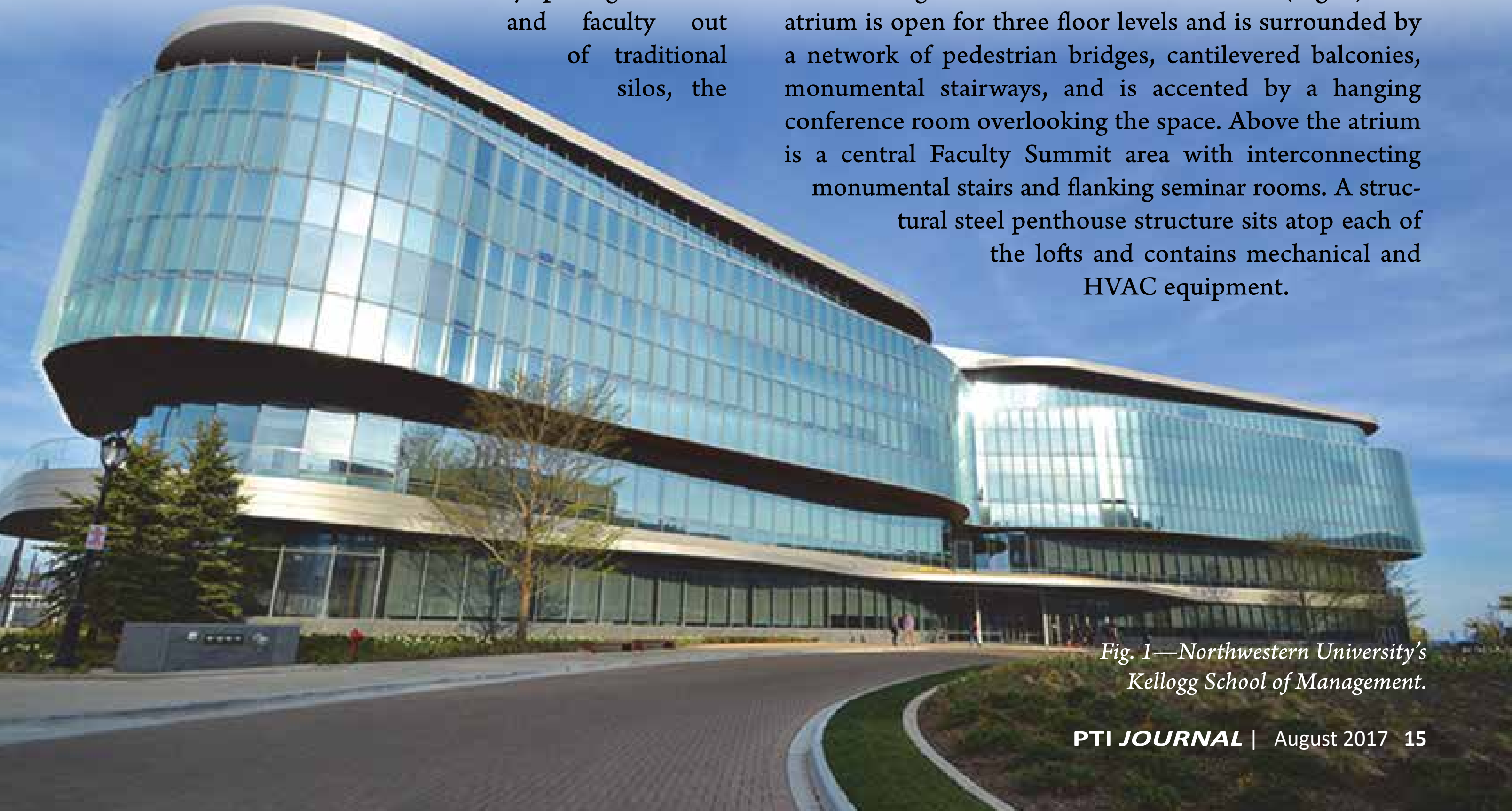


Fig. 1—Northwestern University's Kellogg School of Management.

Additional key features of the building include large open classrooms, a two-story auditorium with dramatic views of Lake Michigan and the Chicago skyline, light courts at each loft that allow light to penetrate down into the interior, and extensive exterior cantilevered terraces and canopies.

SUSTAINABILITY AND THERMAL MASS

To meet the project's sustainability goals, the architect chose to use concrete throughout the superstructure due to its high thermal mass properties. Therefore, to achieve the curving shape of the building, extensive cantilevering, and to accommodate various programming requirements throughout the building, post-tensioning was introduced to minimize structural depth and deflection.

Fig. 2—View of Lake Michigan and Chicago during construction.

Fig. 3—Central atrium framing.

POST-TENSIONING EXPERTISE

Amid numerous structural challenges, the most prevalent was supporting 169 transfer columns in the upper three levels of the building. These transfer columns, which are necessary due to the dissimilar column grids of the lower level classrooms and the upper-level offices, are supported by a network of large post-tensioned transfer beams (Fig. 4 and 5) behaving as "foundations in the air." The transfer columns at the perimeter of the upper levels are set outboard of the levels below, resulting in dramatic post-tensioned transfer cantilevers up to 19 ft (5.8 m) in length (Fig. 4).

The advantage of post-tensioning in these transfer beams is that long spans and cantilevers are achievable with minimum structural depth and deflection, which is a critical concern for transfer columns. The curving nature of

Fig. 4—Cantilevered post-tensioned transfer beam.

Fig. 5—Post-tensioned transfer beams.

the floor plates also creates complicated PT transfer beam intersections (Fig. 5), which were a key constructibility consideration during the design phase. Many of the post-tensioned transfer beams were stressed in multiple stages to manage the varying loading conditions that occurred during construction without overstressing the beams.

Post-tensioning is also used in the lower levels in beams that enable large, column-free classroom space and extensive cantilevering of exterior terraces while minimizing structural depth.

TECHNICAL CHALLENGE AND SOLUTION: MONUMENTAL STAIRS

Five sets of structural steel monumental stairs with long and curving spans required unique analysis and structural solutions. The atrium's "Communication" stair is a curved single span of 55 ft (17 m); two additional sets of curved single spans stretch 35 ft (11 m) each (Fig. 6); and two more sets of nearly identical steel stairs serve as tiered seating. Creative and technical solutions assured openness as well as stability. This and other design challenges were met without sacrificing budget or architectural intent.

COST-EFFECTIVE AND TIME-SAVING COORDINATION

The remarkable architectural form created challenges for Kellogg's design and construction teams. The architect-led design team features 20 specialty design consultants, and this collaborative team has been able to deliver on the

Fig. 6—Curving steel monumental stairs.

project's biggest hurdles, helping to achieve Kellogg's long-term goals of remaining one of the world's top graduate business schools.

Post-tensioning was the only solution that could create the unique structural forms and spaces while meeting the sustainability goals of the project. Without post-tensioning's superior deflection control and ability to minimize structural depth, another less-cost-effective structural steel solution such as deeper structure or less thermal mass would have been required.

Location: Evanston, IL
Owner: Northwestern University
Architect: KPMB
Engineer: Thornton Tomasetti
Contractor: Power Construction
PT Supplier: Dywidag Systems International (DSI)
Other Contributors: AEI Affiliated Engineers (mechanical, electrical, plumbing); Erikkson Engineering (civil, geotechnical); Transsolar (energy, climate); Hoerr Schaudt (landscape); HJ Kessler Associates (LEED); Tillotson Design Associates (lighting); Construction Cost Systems (cost); CM Architects (accessibility); Threshold (acoustic, audio-visual); Soberman Engineering (elevator); S20 (food services); Desman (parking, traffic); Brian Ballantyne Specifications (specifications); Cini Little (waste management)
Submitted by: Thornton Tomasetti

Jury Comments:

- This building's visually striking appearance was accomplished only through the use of post-tensioning for the numerous transfer girders necessary to accommodate the geometric changes.
- This project had everything!
- This project had substantial structural challenges that post-tensioning solved just beautifully.

2017 AWARD OF EXCELLENCE: MANHATTAN WEST PLATFORM

INTRODUCTION

New York City is one of the most densely populated areas in the world. Containing a population of more than 8.5 million, the city has about 28,000 people/mile^2 (10,800 people/km^2), a ratio that seems destined to increase. In fact, people keep knocking at the door of the Big Apple to find a place to live and to work; finding all the building space needed to meet this demand is not an easy task.

With buildable land scarce in NYC, the city's premier real estate developers were in search of a creative solution to maximize the limited space available. Looking out at the bridge industry, they discovered a way to use the high-tech bridge construction machinery to make it happen.

FROM STEELY PLANS TO UNPRECEDENTED PRECAST/POST-TENSIONED SOLUTION

Brookfield Office Properties, owner and developer of the Manhattan West Project, contacted Rizzani de Eccher USA (RdE USA) seeking a solution to close a big gap in the middle of their 7,000,000 ft^2 (650,000 m^2) development site (Fig. 1) by somehow spanning this air rift with a platform above the high-traffic-volume railway tracks. Brookfield executives had observed the efficiency of the Launching Gantry (LG) RdE USA used to erect precast bridge segments in rebuilding the Roslyn Viaduct in Long Island, NY.

Originally, the platform design called for structural steel. After collaborating with several consultants on an innovative new concept, RdE USA presented the owners with the following alternative solution:

1. Use precast segmental post-tensioned bridges to span the entire opening;
2. Erect the precast bridge segments with a custom-built overhead LG; and
3. Eliminate the need to place columns in between the railroad tracks below.

Fig. 1—Manhattan West Platform.

ERECTING MONSTER GIRDER WITH RECORD LENGTH

To span the gap, it required an extensive engineering work to come up with a 2400 ton (2200 metric ton), precast/post-tensioned 240 ft (73 m) long segmental girders positioned by a custom-built LG above 15 live rail lines and their electrified power lines (Fig. 2). These girders will support a public plaza and parking structure between two high-rise buildings.

The girders were placed at a record length of 240 ft (73 m) during the early morning hours 55 ft (17 m) above live tracks that run in underground tunnels to Penn Station. Two elements have

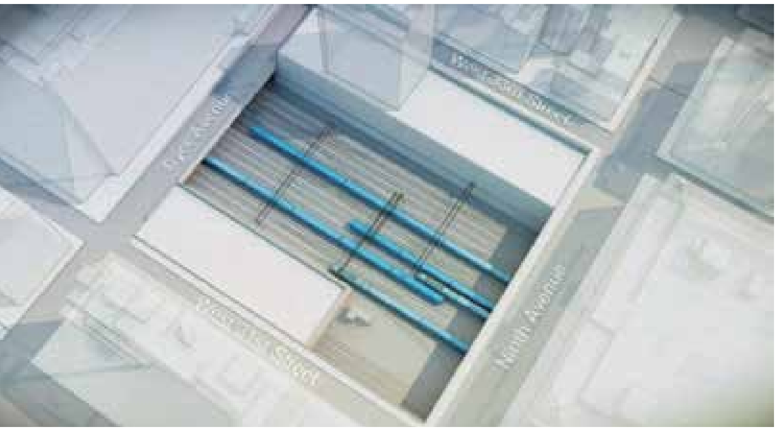


Fig. 2—Schematics with initial layout.

Fig. 3—View of jobsite.

Fig. 4—Underslung system.

particularly contributed to the success of the Manhattan West Project:

- The ability of DEAL, a subsidiary of RdE USA, to manufacture a very high-tech piece of LG equipment with the capacity to smoothly place 16 girders in position; and
- The skills of McNary Bergeron to design a precast concrete segmental box girder that could accommodate 100 tons of post-tensioning strands inside.

Comprised of 1100 tons (1000 metric tons) of steel with a capacity to lift 3600 tons (3300 metric tons), the LG has a 90 ton (82 metric ton) winch for handling the individual segments before they are epoxied and post-tensioned into a single, giant girder. Every component was designed and manufactured in Italy and broken down into more than 90 containers to allow oceanic transport to NYC.

Before placing the first span, some New Yorkers and other contractors were expecting a dramatic scene: perhaps loud impact noises, or the thrill of precariously swinging loads. But when RdE USA erected the first girder, it was so smooth, silent, and precise that many observers on site were somehow disappointed.

LAUNCHED INFRASTRUCTURE AND SAVING WITH PRECAST

The assembly process is very similar to other sequential span-by-span bridges that use an under-slung LG (Fig. 4). Here, the difference is that a platform is being built, so there is no need for a gantry that launches from pier to pier. Rather, a machine moves sideways to place one completed bridge span adjacent to the next. This is a unique setup in Manhattan and one that could be very useful in any situation where, because of busy train tracks or roadways, urban developers are not able to maximize all the potential real estate space.

The most important feature of this type of project is early collaboration during the conceptual phase on construction alternatives that allow placement of columns, slabs, or steel or segmental bridges to maximize design flexibility. In particular, solutions that use precast concrete elements speed up construction and, considering the significant labor rates, keep costs down. Metropolitan Walters played a crucial role in this success. They are a local steel erector with no experience in segmental erection. Nevertheless, the management approach taken by RdE USA has allowed the effective transfer of know-how. Metropolitan Walters has been able to face this challenge with no injuries and completed the work on time and on budget.

The LG solution seems to have been the only economical way to develop these parcels of land. In fact, Brookfield acquired the property in the early 1980s, and it remained undeveloped for decades due to the presence of critical rail traffic.

CONSTRUCTION SEQUENCE

The first activity on-site was the installation of a steel temporary protection platform over the rail lines. It served the dual purpose of protection of the railways and of underslung bed, the "girder factory" from where the LG could be assembled and the girders No. 3 and 4 could be set into place. This temporary protection platform (TPP) was the only structure that needed column touchdown at the track level. The next operation was the assembly of the LG over the TPP. This procedure was similar to a steel bridge incrementally launched. In this phase, the geometry of the structure was essential. The two main girders were composed of eight elements each with more than 24,000 bolts connecting them. Each module had to be perfectly aligned with the previous to maintain the required line and level.

When all the systems were connected, the precast segment erection operations started. All 39 segments for one girder could be stored in a 6100 ft^2 (570 m^2) yard on the site. A straddle carrier supplied by DEAL serviced the staging area, offloading the segments that were trucked in at nighttime and placing them in their temporary storage location. The segments had to be double-stacked for maximizing the storage capacity.

To erect the segments, the straddle carrier lifted each segment and brought it to the LG feeding point. At that location, a trolley was bringing the segment from the dropping point of the straddle carrier to the picking point of the LG winch. The winch picked each segment using a C-hook to minimize PT bar tensioning and connection timing. The C-hook lifted the segment beneath its top slab and carried it into place on top of the adjustable screw jacks, over the two steel I-girders that composed the underslung equipment. The segment joints were then epoxied and the first-stage tendons (24 tendons) were connected with temporary PT bars and tensioned on the underslung bed using the launching gantry. After the first-stage tensioning, the camber generated by the PT force was such that 200 tons (180 metric tons) of counterweights had to be positioned on top of the deck to counteract the uplift generated by the tensioning of the tendons. The span was subsequently moved to a secondary stage area, where the last 10 PT tendons were tensioned and the final grouting was performed. After the grout had reached the initial set, the counterweights could be removed. The last operations were the installation of miscellaneous material and the punch list correction before the girder placement.

To prepare for the girder erection, the lowering system was connected to the pier segments by means of eight threaded bars. When all the connections were secured, the lifting operations could start.

The LG was basically a gantry able to move sideways on rails and its main components were a winch and a lowering system (Fig. 5). The gantry could move 10 ft (3 m) per minute and lowered the girders into place at a rate of 0.5 ft per minute with a 1/4 in. (6 mm) precision. The lowering system consisted of hydraulic cylinders

Fig. 5—Lowering system.

Fig. 6—Post-tensioning on face of pier segments.

Fig. 7—Cast-in-place operations.

with pins and slotted bars (Fig. 5). On each slotted bar, there were two pins that were used alternatively for lifting the girder and for keeping it in position while the jacks restroked. This was a very important feature for the railways' safety: there was, at all times, a positive connection between the bridge and the erection equipment. Even in case of a hydraulic failure, the load would have always been secured by the pin connection. On top of that, the LG was remotely connected with the engineers and technicians in Italy to monitor the functioning and ensure that all the sensor and indicators were working properly. After the first two girders were set, the TPP was removed and the underslung assembly bed was relocated over Span 3 and 4 and used as the stage from which subsequent girders were fabricated.

Each girder required 100 tons (91 metric tons) of post-tensioning supplied by Tensa, using twenty 37-strand and fourteen 31-strand tendons (Fig. 6). The 16 girders created a platform 480 ft (146 m) wide and 12 ft (3.7 m) deep with and a total amount of 612 segments.

The platform was designed as three distinct sections made of seven, six, and three bridges separated by movement joints. The final work, executed by John Civetta and Sons, was placing a 6 in. (150 mm) concrete layer on top of the deck and in between the end anchor segments to create a rigid diaphragm across each of the three solid structures (Fig. 7).

As was expected with the magnitude of compressive stress in the structure, small cracks were observed at locations of high stress concentration, such as at the armored openings after post-tensioning. All cracks were relatively small (<0.01 in. [0.25 mm]) and were sealed with methacrylate crack sealer.

The members of the construction team worked together closely throughout the project to ensure it ran smoothly under challenging conditions in producing a first-of-its-kind structure. Although the structure is not strictly a bridge, the construction of a platform over an active and busy railway addressed many of the same issues that more typical bridge structures encounter, but on a much more massive scale.

COMPLEX GIRDER GEOMETRY

Because of all the geometry control required, precasting segments such as these for the first time was not easy. As a result, RdE USA sent some of its engineers to the precast plant to work side-by-side with Jersey Precast personnel, the precaster that supplied the elements, to ensure an accurate transfer of knowledge for a quality job.

McNary Bergeron designed the precast bridges. Each bridge consists of 37 to 39 match-cast segments individually trucked on site and assembled over the underslung bed.

CONSTRUCTION CHALLENGES

Despite the overall success of the project, there is always room for improvement. For example, the temporary post-tensioning bar configuration was a real challenge because having narrower-than-usual precast segments presented some difficulties in the installation and removal of the rods.

Another significant challenge is the engineering involved in coordinating the design of the bridges and equipment. Because this is such a unique structure, an innovative, seamless approach is required to make it work.

Fig. 8—Staging area.

The logistics of the project are also complex, with work taking place on a staging area effectively the size of a postage stamp (250 x 90 ft [76 x 27 m]) in the middle of Manhattan (Fig. 8). Assembling the LG meant trucking in more than 90 very large and heavy containers to a small, confined area where other contractors were also working.

RdE USA had significant interaction with SOM (the project architect) during the design phase. The most critical issues faced were related to creating openings in the platform to possibly allow future columns for the overhead buildings to touch down at the track level (Fig. 9). The platform's design is very sophisticated and these openings were particularly daunting because of the flow of stresses on the deck.

NEW URBAN FRONTIER

Beyond the structural challenges and record span lengths achieved, there is one element that lays the foundations for a new way of developing the urban environment. The happy marriage between the structural engineering features of bridge construction and the building industry has paved the way to creating real estate where there seems no opportunity to do so (Fig. 10). The ability to cover an area that effectively was unbuildable suddenly opened up a new possibility of developing apartments, offices, and especially green areas.

Fig. 9—Column openings.

Fig. 10—Rendering of future use of platform.

The value of this approach is immense in cities such as New York where more and more people are looking for places to live and work.

This project has drawn a lot of attention and it appears that many other cities are facing the same problems. In fact, it is evident that many cities have railways, roadways, or other types of active traffic below the street level and close to areas that could potentially be developed.

The top-down construction method that was used in the Manhattan West Project is a technology that could be applied to many other scenarios. Different structural solutions might be implemented, but the application of specialized equipment to ensure a fast and safe execution of the project is a method that allows minimizing the work at the level of the area that needs to be covered.

Location: New York, NY
Owner: Brookfield Properties, Inc.
Architect: SOM
Engineer: (1) McNary Bergeron Associates; (2) Entuitive Corp.
Contractor: Rizzani de Eccher USA Ltd.
PT Supplier: TENSA America
Submitted by: Tensa America

Jury Comments:

- Only through the use of post-tensioning was this platform able to be built over 15 live Metro tracks without needing any supports between the tracks.
- The coordination involved in this project was incredible and the span of it is amazing.
- What a feather in the cap of the world of post-tensioning!

2017 AWARD OF EXCELLENCE: MIAMI DESIGN DISTRICT CITY VIEW GARAGE

Tim Haahs worked with Dacra and L Real Estate on the design of a mixed-use 559 space parking facility to serve the Miami Design District (Fig 1). KVC Constructors were the contractors who built the mixed-use parking facility. One of the most unique characteristics of the garage design is the use of multiple façade architects on the project. This allowed for numerous artistic schemes to be prevalent throughout. Two internationally renowned architectural firms worked on the garage façade designs—Leong Leong Architects and Iwamoto Scott.

The innovative master plan for the Design District transformed a once-overlooked Miami neighborhood into a high-end shopping, dining, and cultural destination, attracting over one hundred top retailers and countless domestic and international visitors. The City View Garage includes approximately 22,700 ft^2 (2100 m^2) of retail on ground level (Fig. 2) and 14,800 ft^2 (1400 m^2) of office space on the northeast corner of the project. To complement the District's dedication to the creative experience, it provides an attractive connection between parking and the rest of the development with its vibrant façades (Fig. 3), dramatic lighting, and ground-floor retail spaces that engage the pedestrian. The Leong Leong façade on the west consists of titanium-plated stainless-steel panels with thousands of sections cut and folded out to provide a three-dimensional (3-D) effect. The Iwamoto Scott façade on the east features a digitally fabricated metal screen that wraps around to the corner of the garage and features a blue and silver gradient color pattern that complements that of the surrounding Palm Court buildings. The middle portion of the south façade features a public art piece by John Baldessari. This piece showcases a highly technical artistic application which transforms the pixels from simple ink dots into different diameter cut outs in the steel panels, providing the tone variations needed to create an image that is ambiguous up close but which becomes gradually more visible at a distance. Timothy Haahs & Associates designed the North façade, which is made out of precast concrete panels. Hexagons were chosen as the preferred opening shape with a randomized pattern to provide an interesting mosaic.

Fig. 1—City View Garage.

Fig. 2—Ground-level retail.

Fig. 3—Façade of the City View Garage.

Fig. 4—Ground tier plan.

The garage structure was designed with post-tensioned concrete slabs and beams, which allow the required spans to be achieved and keeps the parking decks column-free while also reducing the curing time required to build such a structure. The structural bays consist of columns on a 24 x 54 ft (7.3 x 16.5 m) grid designed for 65-degree double-loaded parking. The entry and exit lanes include two PARCS lanes each, to allow more efficient entry and egress to the garage (Fig. 4). There is a speed ramp connecting the ground level to the second level where the parking area starts. From that point on, the parking ramps are laid out in a double helix format to allow a quick drive to the top of the garage and back down. Pay on Foot machines are located on both stair towers to reduce wait times when exiting the garage. The garage has wayfinding graphics throughout.

The City View Garage has landscaping and urban scale treatments matching the rest of the Miami Design District projects. The entire mixed-use facility has been treated as a canvas for public artwork. The City View Garage signage was designed by RSM Design, who has designed all the wayfinding and signage in the Design District. For the City View Garage, RSM created 3-D lettering for the marquee signs, identifying the garage itself. RSM created custom neon signs identifying bicycle racks and the parking entrances. They also created custom pedestrian directional signs, column graphics, and wall graphics. The Garage is part of the overall Miami Design District LEED-ND certification and received Stage 2 LEED-ND certification in 2013. The lighting of the garage was designed by Speirs & Major and is mostly comprised of LED light fixtures for energy efficiency. The garage is fitted with security cameras tied into the Design District's security office.

Location: Miami, FL
Owner: DACRA
Architect: Timothy Haahs & Associates
Engineer: Timothy Haahs & Associates
Contractor: KVC Constructors
PT Supplier: Suncoast Post Tension
Submitted by: Timothy Haahs & Associates

Jury Comments:

- Post-tensioning allowed longer spans and thinner elements resulting in a very open garage structure.
- This project bodes a very visually striking exterior in keeping with the Miami scene.

2017 AWARD OF EXCELLENCE: DOLPHIN TOWER EMERGENCY STRUCTURAL REPAIRS & REHAB

Constructed in 1973-1974, the Dolphin Tower Condominium in Sarasota, FL, (Fig. 1) is a 15-story 117-unit reinforced concrete building that provided 36 years of service before a sudden structural failure occurred. The fourth-floor column transfer slab failed with little warning (Fig. 2). Quick installation of shoring was all that prevented collapse. The failure was exceptional in that it was not the result of deterioration or overloading.

Because of the unusual nature of the failure and the prime location on the downtown waterfront, the project instantly became the topic of news reports and casual conversation, as well as superlative scrutiny by engineers, insurance companies, and attorneys. Through it all, a volunteer Board of Directors navigated the course to repair and reoccupation.

The Dolphin Tower Project demonstrates the intensity of the testing, inspection, evaluation, and analysis to determine the root cause of the failure; the unusual collaboration of design engineering and contractor's engineering teams to achieve a cost-effective repair using multiple mitigation strategies; the holistic nature of the repairs using novel methods; the effectiveness and efficiency of the construction team that delivered a complex project with unforeseen challenges on time, and several hundred thousand dollars under budget.

BACKGROUND AND STRUCTURAL CHARACTERISTICS

The building is a 15-story condominium located in downtown Sarasota with the first three levels devoted to parking. To accommodate the parking spaces, the fourth floor is designed to be a 24 in. (610 mm) thick column transfer slab, distributing the 12-story column loads above to the non-aligning columns below. Built in 1974, the building has conventionally reinforced concrete flat slabs supported by concrete columns and pile foundations.

PROBLEMS IN 2010

On June 24, 2010, the resident manager who lived on the fourth floor observed walls within the unit buckling and the tile floors cracking. She immediately contacted the structural engineer with whom the association had previously worked. The engineer confirmed a major failure and incipient collapse and arranged for a contractor to install

Fig. 1—Dolphin Towers.

Fig. 2—Shear crack failure.

emergency shoring immediately. By the time shoring was installed, the crack spanned four column bays and in some areas the concrete floor had lifted as much as 4 in. (100 mm). The building was evacuated.

INSPECTION/EVALUATION AND TESTING

In an effort to identify the cause of the failure and determine feasibility/methods of repair, the building was intensely scrutinized. In addition to the Association's primary structural engineer, forensic engineers were employed by the insurance company and the association's coverage counsel. Following removal of interior walls and finishes, test methods included visual inspection, strength testing, petrographic analysis, surface penetrating radar, and ultrasonic pulse velocity testing. Full-building finite-element analysis was performed to evaluate the stresses in the building and correlate the materials testing results with the observed failures.

Although no "trigger" was identified, the failure was determined to be punching shear caused by a combination of inadequate design for overlapping punching shear perimeters, low-strength concrete, poor-quality concrete, inadequate consolidation, and inadequate reinforcing steel detailing.

A side effect of the intense analysis of the building was the discovery of vulnerabilities in addition to the failed transfer slab. These included inadequate lateral design for hurricane wind resistance and inadequate punching shear capacity at the upper-level slabs.

REPAIRS/REPAIR PROCESS

To address the identified issues, remedial measures were designed, including replacement of the failed fourth-floor transfer slab, column enhancements, installation of new lateral force-resisting elements and associated foundations, enhancement of existing lateral force-resisting elements, and enhancement of upper-floor slab/column joints.

Once the initial analysis and design was completed, a construction manager was retained to obtain bids for the work. Upon completion of initial bidding and subsequent value engineering, a preferred contractor was selected. In an effort to bring the cost of the project within the budget established by the Association, the Contractor was afforded the opportunity to further refine the design, allowing the unit dimensions to be altered. The removal of this design constraint permitted a substantial cost savings and resulted in a construction team that included the Owner, Owner's Representative, Owner's Engineer, Contractor, and Contractor's Engineer. The design evolved from removal and replacement of the fourth-floor slab and installation of exterior shear walls to installation of post-tensioned drop panels combined with a structural overlay and interior shear walls. This novel team approach ultimately resulted in an initial cost saving and completion on time and under budget.

REPAIR IMPLEMENTATION

Sequencing

When work began onsite, there were already 1100 post shores in place (Fig. 3). These posts were originally installed as an emergency measure and were set up in clusters throughout the three levels of the garage. A challenging part of this project was that this shoring needed to be incorporated into the construction sequence and remained in place to support the tower columns above. This shoring also relieved the damaged transfer slab while the repair work was being done. Due to the required shoring, access and storage space within the garage was limited; therefore, safety for the workers in these areas was paramount. Weekly safety meetings were held onsite with all crews and subcontractors to make sure everyone was aware of the specific safety concerns present in their work areas. Work proceeded concurrently on multiple levels of the garage and extra precautions were taken and constantly monitored to make sure workers were safe from falling debris from the extensive coring operation above. Excess water created by the core drill machines was cleaned up on a regular basis to prevent slip and fall hazards.

I – 4th Level Crack Repair & Phase 1 Foundations

The repair was based on multiple phases. The first phase of the repair was to fix the large shear failure crack within the fourth-level transfer slab, which spanned four column bays. The repair comprised conventional selective demolition (Fig. 4) and replacement along with epoxy injection. This would further weaken the already-damaged transfer slab, so additional shoring was installed. In all, 80 gal. (300 L) of epoxy was used in this process. Additional steel was added to the repair areas where concrete was removed, and the floor was repoured to its original level.

Simultaneously, work began on Phase 1 of the foundation strengthening. This phase consisted of areas that fell outside the existing shoring and could easily be accessed. The foundation work consisted of demolishing and removing the slab-on-ground concrete, excavating down to the existing concrete pile caps, installing new steel helical piles (Fig. 5), placing steel reinforcement, and pouring of the new expanded pile caps.

During excavation of the first pile cap location, it was discovered that the existing cap was only 16 in. (405 mm) deep. However, the original construction as-built drawings showed a pile cap depth of 29 in. (735 mm). The same was true for 10 out of 12 of the Phase 1 pile caps. Based on these existing conditions, the structural engineers had to revise their original designs.

II – Level 3 Drop Panels & Upper-Level Exterior Shear Walls

The second phase of the project was to install the post-tensioned drop panels on the underside of the fourth-floor slab (Fig. 6 and 7). The first step was to prep the ceiling using hydrodemolition and then install column enlargements on Level 3 (Fig. 8). Six hundred 8 in. (205 mm) holes were cored through the fourth-level slab to install "shear lug keys" (Fig. 9). These would tie the new drop panel to the fourth-level overlay, creating a new composite slab. The area of the drop panel construction was over 7700 ft^2 (715 m^2) and was split up into four sub-phases. The process took over 2 months, 400 yd^3 (305 m^3) of concrete, 22,000 ft (6700 m) of post-tension tendons, and over 42 tons (38 metric tons) of reinforcing steel to complete. Because access within the formwork was limited, self-consolidating concrete (SCC) was used. To test the SCC material, several columns were poured prior to the first drop panel section. This was done as a mockup to test the material and confirm that it met the design requirements and that it would work within confined forms similar to those that would be used for the drop panel. The steel cages for the new drop panels were hung from the lug keys, and the post-tension tendons were installed within the cage. Post shores were installed underneath the drop panel formwork

Fig. 3—Shoring.

Fig. 4—Level 4 crack repair.

Fig. 5—Installation of helical piles.

Fig. 6—Drop panels with nonprestressed and PT reinforcement.

Fig. 7—Drop panels with nonprestressed and PT reinforcement.

Fig. 8—Column enlargement.

Fig. 9—Shear lugs provide continuity between drop panels and overlay slab.

and across 100% of the tower area on all three levels of the garage. This shoring would support the new construction of the drop panels and overlay. It remained in place until the overlay construction was completed and the post-tension tendons were stressed. In total, 2800 post shores and 200 shoring towers were used for the drop panel and overlay construction.

Also during the second phase, exterior wall segments at the four corners of the building were converted to concrete shear walls (Fig. 10). The existing windows and CMU walls were removed, reinforcing steel was installed, and concrete was poured.

III – Level 4 Overlay Slab, Interior Shear Walls, Upper-Level Shear Repairs

The third phase commences with construction of the fourth-level overlay slab (Fig. 11). The overlay was roughly 13,500 ft^2 (1250 m^2) split into two sub-phases. Over 68 tons (62 metric tons) of reinforcing bar and 300 yd^3 (230 m^3) of concrete was used. Additionally, because the floor height was being increased by 6 in. (150 mm), the two elevator doors required relocating. Patio sliding glass doors were replaced into the new openings and ADA-compliant ramps with railings were constructed at the exits of public spaces.

After the overlay slab was completed, interior shear wall construction began starting on the fourth level and continuing up to the penthouse. In conjunction with the installation of new interior shear walls (Fig. 12), upper-level slab overstress was relieved through jacking. Once the concrete cured, the shoring would be released, redirecting stresses in the floor to the new shear wall. This is an example of how cooperation and open communication within the construction team resulted in a solution that would address the structural issues while also saving the owner money.

To supplement lateral wind load capacity and address upper-floor punching, existing CMU walls were filled with grout and 108 steel "T-columns" were installed at various locations. Originally, these columns were to be poured-in-place concrete, much like those done on the garage levels. However, with input from the contractor, the structural repair engineer was able to design these as steel columns, which greatly reduced the cost to the owner and also reduced the time required to complete the repair.

IV – Phase 2 Foundations

The final stage of work was to finish the second phase of the new foundation pile caps—those areas that were previously inaccessible due to the substantial amount of shoring required for the fourth-level repairs. For these foundations, the existing pile caps were 4 ft (1.2 m) or more below the upper slab-on-ground, resulting in excavations extending 2 ft (610 mm) or more below the water table. Consequently, undermining of the slab surrounding the excavation areas was a major concern. Temporary sump-pump pits were created to lower the water table level and reduce soil erosion. Several large concrete piers buried within the existing foundations were discovered, making it very difficult to install helical piles. Excavation was required to remove these piers. The final foundation area to be completed was within a subgrade mechanical pump room. Due to limited headroom, holes were punched in the ramp slab above to install the helical piles. Because of the depth of this excavation, the water was again an issue and undermining of the surrounding slab needed to be controlled. Therefore, soil grouting was used to maintain support of the existing mechanical units and prevent further erosion.

CONCLUSIONS

The Dolphin Tower project was very challenging and used many interesting and groundbreaking design concepts. Constant communication between the contractor, design engineer, and owner was a necessity to adapt to continuously changing conditions that required quick solutions to maintain the aggressive schedule. Despite many challenges, the repairs were completed on schedule and under budget.

Location: Sarasota, FL
Owner: Dolphin Towers Condo Association
Engineer: Morabito Consultants
Contractor: Concrete Protection & Restoration, Inc.
PT Supplier: PTE Systems International, LLC
Other Contributors: (1) Karins Engineering Group (2) CEMEX
Submitted by: Concrete Protection & Restoration, Inc.

Jury Comments:

- Post-tensioning saves the day and the building.
- Only through the innovative use of post-tensioning was this building salvaged.
- What they did here was just spectacular.

Fig. 10—Exterior shear walls.

Fig. 11—Overlay reinforcement.

Fig. 12—Interior shear walls.

2017 AWARD OF EXCELLENCE: JAMES PASCOE GROUP DISTRIBUTION CENTER

INTRODUCTION

The James Pascoe Group distribution center in Laidlaw Way, East Tamaki, New Zealand (Fig. 1) consolidates warehousing and distribution for all of the group's brands, which include Farmers, Whitcoulls, Stevens, and Pascoes. Constructed over the course of 2014 and 2015, the new building (designated JPL), which is an extension to an existing warehouse, comprises a total floor area of 485,000 ft^2 (45,000 m^2). This includes a 270,000 ft^2 (25,000 m^2) ground floor with a minimum 66 ft (20 m) roof height, 155,000 ft^2 (14,500 m^2) of a mezzanine floor level, and a multi-story car park and photographic studio (Fig. 2). The internal volume of the building is six stories high and it covers an area the size of three rugby fields.

Acting as both client and project manager, James Pascoe Group was focused on long-term business performance and not development profit. The goal of the project was to create a state-of-the-art distribution center to minimize costs and maximize value for customers by maintaining the highest "in-stock" position possible for core merchandise. This was supported by introducing the latest warehousing systems and technology, including 52 ft (16 m) high, very narrow aisle (VNA) racking (Fig. 3). The VNA racking and material handling equipment (MHE) required a level of floor flatness that is considered the highest in the world.

As the working surface in the distribution center, the concrete flooring played a pivotal role in achieving the

Fig. 1—The James Pascoe Distribution Center.

Fig. 2—Interior of the finished James Pascoe Group distribution center.

Fig. 3—James Pascoe Group introduced cutting-edge logistics technology to create a state-of-the-art facility. A key feature was the 52 ft (16 m) high VNA racking – the highest ever installed in New Zealand.

twin goals of durability and material handling efficiency. These two priorities created a complex set of challenges and required the development of new technology mixed with the refinement of existing global best practice. This led to the achievement of a series of New Zealand firsts, including the only internationally certified true superflat VNA floor in the country, and the use of a movement joint detail that was specifically developed to mitigate slab curl.

A CUTTING-EDGE LOGISTICS FLOOR

To maximize the capacity of their new facility, James Pascoe Group introduced the latest warehouse systems. Conslab Limited was the party responsible for designing, delivering the completed floor, and providing the technical content of this article, (Walker 2015) and worked closely with client James Pascoe Group and BBR Contech, PT supplier, to address all of the challenges. The key feature of the facility was the use of nearly 1.2 miles (2 km) of 52 ft (16 m) high VNA racking aisles. VNA racking allows for much denser product storage compared to a conventional system by using significantly higher racking combined with a narrower aisle width. Material is handled by special equipment that operates semi-autonomously and is wire-guided. In comparison to standard material handling equipment, VNA equipment will run on a fixed wheel path—akin to a train running on tracks—as opposed to traditional equipment that will operate more like a car does on the road. This characteristic of a VNA system, coupled with the narrow aisle width and high racking level, require a different approach to floor flatness.

Floor flatness

These systems require exceedingly tight floor flatness to function. Any imperfection in the floor surface is magnified multiple times over when a forklift mast is operating at a 52 ft (16 m) height and attempting to place a 1650 lb (750 kg) pallet precisely onto two rack beams (Fig. 4). In fact, a 0.08 in. (2 mm) difference in level across the narrow 5.6 ft (1.7 m) forklift wheel base will create enough lean at 52 ft (16 m) that it becomes physically impossible to place a pallet. The current New Zealand standard for floor flatness, NZS3114:1987 (Standards New Zealand 1987), isn't suitable for a modern logistics facility. As such, an alternative internationally recognized specification was adopted for the JPL project, DM1 to TR34. The UK Concrete Society Report TR34 includes a special section covering floor flatness for VNA facilities (The Concrete Society 2013). The flatness categories have been developed based on real-world performance of VNA warehouses and are accepted by all the key parties involved in delivering a successful VNA warehouse: the racking, material handling, and concrete flooring industries.

Adopting a proven standard for floor flatness is critical to the success of a modern warehouse project. Compared to NZS3114:1987, the flatness standard presented in TR34 has many benefits. It is calibrated to the actual requirements of the material handling industry, the flatness tolerances are achievable provided the correct construction methodology is adopted, and the surveying process is clearly defined and repeatable.

For VNA floors flatness is measured only along the specific wheel path the vehicle runs along and controls for the tilt and "bumpiness" both along and across the aisle. These properties are measured using specialized equipment called a Profileograph (The Concrete Society 2013) (Fig. 5). For the JPL project, the equipment was sourced from FACE Consultants Pty in Australia. The Profileograph gives precise data of the profile of the floor at 2 in. (50 mm) increments along each wheel path (Fig. 6).

Achieving "superflat" VNA floor flatness tolerances

To achieve the tight tolerances required in a VNA facility, the traditional construction methodology is to build a single aisle at a time, leveling the floor off accurately placed formwork set on either side of the aisle. This approach was

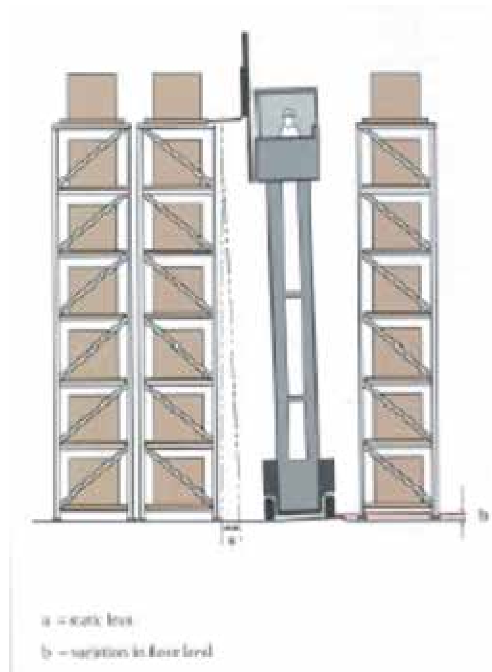


Fig. 4—Small variations in floor level can cause large leans in VNA systems (The Concrete Society 2013).

Fig. 5—FACE Consultants Profileograph surveying of the JPL Distribution Center.

not well suited to the new James Pascoe Group Facility. It would lead to the introduction of a plethora of joints and saw cuts that would require a lifetime of maintenance, and create the potential for joint curl to disrupt traffic after the facility was operational. Joints and saw cuts are weak points in a floor that tend to spall and fail under heavy use. This traditional narrow strip construction method (Fig. 7) is also exceedingly slow and would not meet fast-track program requirements. It would also require the use of a less efficient floor design, which would require a much larger volume of concrete and therefore present a less sustainable solution.

CREATING A DURABLE FLOOR WITH VNA FLATNESS

To meet the dual requirement for a durable and maintenance-free floor with exacting VNA flatness requirements, the project team developed a new world first hybrid approach to VNA floor construction. The floor was constructed as a series of large bay post-tensioned (PT) floors (Fig. 8), coupled together so that there were only two opening joints located within the 270,000 ft^2 (25,000 m^2) ground floor. It was constructed and ready for racking in under 100 days, saving approximately 20 to 30 days compared to a conventional solution. Combined with the use of a new system for concrete floor jointing—the Rhino joint—this created a surface that would require little to no maintenance during the life of the structure, thus saving in excess of $500,000 of maintenance costs over a 20-year period. The use of PT allowed for an efficient and relatively thin 9.4 in. (240 mm) thick floor to cater to the 27,000 lb (120 kN) back-to-back rack loading, thereby reducing the volume of concrete by at least 20% and subsequent CO_2 emissions compared to a traditional floor system. In contrast, the traditionally designed floor would likely require a thickness greater than 11.8 in. (300 mm) (CCANZ 2001). The combination of faster construction time and a more efficient use of materials resulted in an approximate 25% reduction in construction cost.

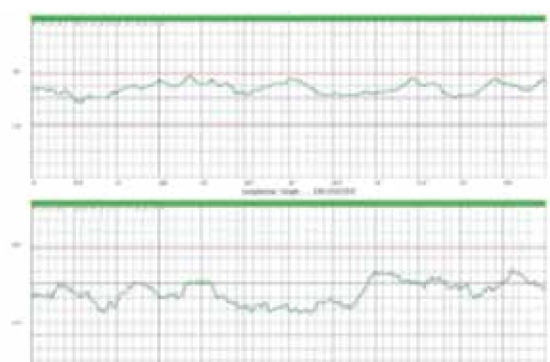


Fig. 6—Sample data from a Profileograph survey.

The layout of floor pours was optimized to suit both the racking layout and the placing and finishing process that the flooring contractor developed for the floor. This process involved the refinement of Somero Laser Screed techniques to strike off and level the floor, along with a tightly controlled system of straight edging and ride-on power trowel finishing. The concrete mixture design was optimized to support this process by providing the best possible window of finishability while still meeting the structural and durability requirements of the project. Attention was paid to limiting the variability as much as possible and so the slump (used as a proxy measurement for set time) was controlled rigidly for every load of concrete delivered to site, and the floor construction was programmed to take place in a weathertight and windproof building to limit environmental variability.

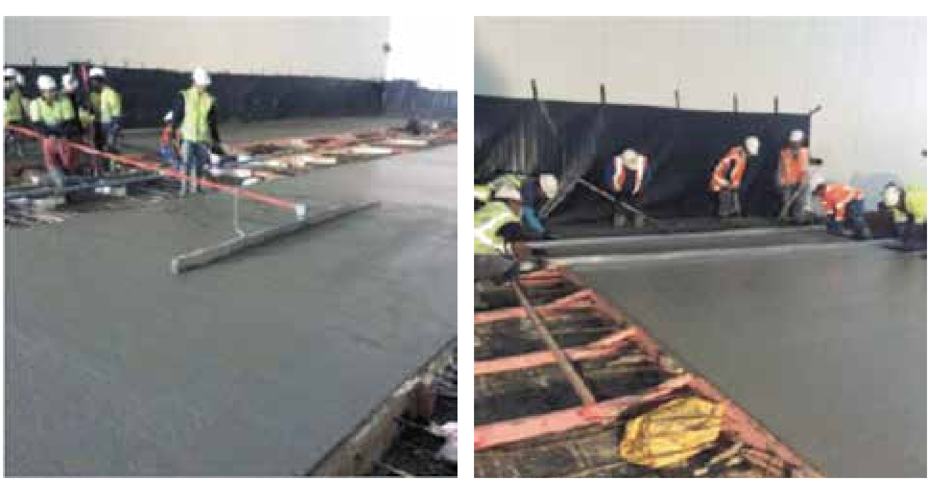


Fig. 7—An example of traditional narrow strip construction methodology for a VNA facility in Christchurch, New Zealand.

The trade off of constructing the floor in large bays was that the finishing process could no longer be optimized to focus on individual VNA aisles. The benefit of the traditional approach of constructing VNA floors in long narrow strips is that the floor flatness can be controlled meticulously in the one location that really matters: the aisle. In contrast, when constructing using a large bay approach, the floor can achieve a very high overall

Fig. 8—Large bay post-tensioned floor pour at JPL.

standard of flatness, but will still contain areas in aisles that fall outside of a VNA specification. These areas were addressed by grinding along the fixed MHE wheel paths to bring them into tolerance. While grinding the surface in these locations can be unsightly and reduce surface durability, this downside is more than made up for by the key benefits of using a large PT floor—a more efficient structural design, superior MHE performance due to minimal construction joints, and the complete elimination of saw cuts, which are the most common long-term maintenance issue in floors.

The initial goal was to construct the floor to the highest level of "general" flatness classification in TR34-FM1. This classification applies to the floor as a whole, and not just the specific VNA aisles. The tighter VNA tolerance was then applied specifically to the aisles. This was the DM1 specification from TR34. Due to the high-quality construction of the original surface as little as 20 to 30% of the aisle length required grinding to achieve the optimal surface profile.

Mitigating curl

Slab curling can present a significant issue in a high-quality warehouse floor by creating an out of tolerance surface at joints or reducing load capacity when a curled joint loses contact with the subbase (Garber 2006) (Fig. 9). Curl is the result of differential moisture and temperature profiles in a floor (Garber 2006). In New Zealand, where internal floors are constructed on an impermeable damp-proof membrane, floors curl upwards at joints. Like drying shrinkage, curl in concrete slabs-on-ground should be seen as a given (Garber 2006); however, in the author's experience, the relatively high-shrinkage concrete in New Zealand (due to the use of elastic aggregates) exacerbates the issue compared to other markets. Contrary to a commonly held misconception, the use of dowels will not mitigate curl at construction joints, although they will help reduce the risk of failure due to loading of a curled joint (Garber 2006).

For a facility such as the JPL distribution center, curl would be a significant issue—it would compromise the floor flatness required for the cutting edge VNA system and could present a significant maintenance burden over the lifetime of the building. To mitigate the issue of slab curl, the number of opening joints in the floor was minimized by using a PT floor design. The joints that were required were then installed in the lowest traffic areas possible, and a special New Zealand designed joint system called "Rhino Joint" was installed (Fig. 10). The Rhino joint

Fig. 9—Slab curl in a typical New Zealand floor.

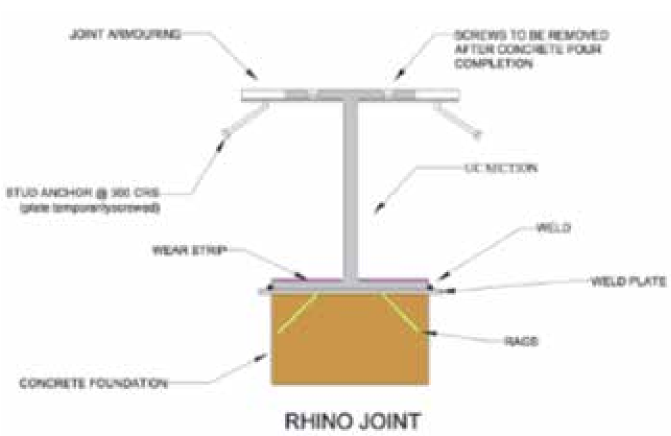


Fig 10—The NZ designed "Rhino Joint" is a patented movement joint system that prevents slab curl by restricting upwards movement.

resists and prevents the typical upward curl through by creating an anchoring force on the top edge of the slab.

A New Zealand First – A True Superflat VNA Floor Surface

The current New Zealand standard for floor surface finish and flatness (NZS3114:1987) (Standards New Zealand 1987) is not suitable for the requirements of a modern logistics warehouse. To meet the client's exacting requirements, the project team:

- Introduced a European standard (The Concrete Society 2013) that integrates floor flatness, racking, and MHE tolerances to control the interface risk between these elements and specify a floor finish that would support 52 ft (16 m) high VNA racking.
- Used special profiling/surveying technology to map the exact pathway that MHE would run down in every aisle to be able to confirm compliance.
- Refined a Somero Laser Screed placing and finishing process, and carefully planned and controlled key variables that impact surface finish, such as concrete mixture design, environmental conditions, and the sequencing and detailing of pour layouts.
- As a result of this process, the floor achieved the highest general flatness category of FM1 in key racking areas, and only required approximately 20 to 30% aisle grinding to achieve the highest recognized VNA flatness standard in the world: DM1.

REFERENCES

CCANZ, 2001, *Technical Manual: Concrete Ground Floors & Pavements for Commercial and Industrial Use, part two, specific design,* Wellington, New Zealand.

Garber, G., 2006, *Design and Construction of Concrete Floors*, second edition, Burlington, MA.

Standards New Zealand, 1987, "Specification for Concrete Surface Finishes (NZS3114:1987)," Standards New Zealand, Wellington, New Zealand.

The Concrete Society, 2013, *Techincal Report 34: Concrete Industrial Ground Floors; A Guide to Design And Construction*, fourth edition, The Concrete Society, Blackwater, Camberley, UK.

Walker, T., 2015, JPL Distribution Centre – World Leading Concrete Flooring, The New Zealand Concrete Industry Conference 8-10 October, Roturua, New Zealand.

Location: Auckland, New Zealand
Owner: James Pascoe Group
Architect: TSE Architects
Engineer: BGT Structures
Contractor: James Pascoe Group
PT Supplier: BBR Contech
Other Contributors: Conslab Ltd.; Concrete Structures
Submitted by: BBR VT International Ltd.

Jury Comments:

- The use of post-tensioning significantly minimized the number of construction and expansion joints and allowed for precise control of floor flatness.
- A facility such as this one really benefits from the advantages of post-tensioning.

2017 AWARD OF MERIT: SOUTH NORFOLK JORDAN BRIDGE

Dedicated in October 2012, the new South Norfolk Jordan Bridge (Fig. 1) is a modern, high-level bridge crossing over the Southern Branch of the Elizabeth River in Virginia. The new 5370 ft (1640 m) long bridge connects Chesapeake, Portsmouth, and the Hampton Roads region, restoring a vital transportation link in the regional transportation network. The bridge's 385 ft (120 m) main span over the navigation channel (Fig. 2) of the Elizabeth River is a high-level fixed span, allowing major ships to pass below the bridge while vehicular traffic flows across the bridge. Using the advantages of precast segmental construction with post-tensioning for both the superstructure and the substructure, the bridge was constructed in just 20 months.

The column segments for the piers were precast at the adjacent casting yard and trucked (land columns) or barged (river columns) to the site. The column segments were then stacked, using epoxy joints, and temporarily supported with vertical post-tensioning bars. Columns up to 100 ft (30 m) tall could be erected in 1 day, saving time in the compressed construction schedule. Afterwards, vertical post-tensioning tendons were installed and grouted before the superstructure was erected.

To build the bridge's long main spans over the river, the team used balanced cantilever construction with a barge-mounted crane (Fig. 3 and 4). This method allowed the heavily traveled river (part of the Intracoastal Waterway)

Fig. 1—South Norfolk Jordan Bridge.

Fig. 2—Elizabeth River navigational channel.

Fig. 3—Balanced cantilever construction.

to remain open to navigational traffic while the bridge was constructed. Up to six cantilever segments were erected per day.

Fig. 4—Barge-mounted crane.

Fig. 5—Post-tensioned segmental concrete construction.

The constant depth approach spans were match-cast, trucked to site over the previously erected spans using the span-by-span erection method, and post-tensioned. The superstructure was post-tensioned bidirectionally (transversely and longitudinally) to minimize microcracking and increase long-term durability. Crews were able to complete up to two approach spans per week. Using multiple erection headings, superstructure erection took just 14 months. Thanks to post-tensioned segmental concrete (Fig. 5), the bridge was cast and constructed using local labor and materials. Post-tensioning minimizes cracking, maximizes durability, and provides residual longitudinal compression of the deck under normal service loads.

Location: Chesapeake & Portsmouth, VA
Owner: United Bridge Partners
Engineer: FIGG Bridge Engineers
Contractor: FIGG Bridge Engineers
PT Supplier: VSL/Structural Technologies
Submitted by: FIGG Bridge Engineers

Jury Comments:

- The use of precast post-tensioning segments for columns and superstructure allowed for faster construction with better quality in a challenging location.
- This is a really impressive, monumental high bridge and just an excellent project all around.

2017 AWARD OF MERIT: METRÔ LINE 15 (SILVER) MONORAIL

Metropolitan São Paulo, Brazil, has a population of 21.1 million, and is the largest economy by gross domestic product in Latin America and the Southern Hemisphere. The São Paulo Metrô system carries 4.7 million passengers daily. In 2009, the decision was made to use monorail technology in the extension of the Metrô System for Line 15 (Silver Line) when the Government of the State of São Paulo and the Prefecture of the Municipality of São Paulo signed an agreement of Technical and Financial Cooperation to substitute the original project, a dedicated bus line, for a monorail line (Fig. 1).

The elevated line snakes its way through the densely populated outlying areas of eastern São Paulo (Fig. 2). The Silver line connects passengers to Metrô Rail Line 2 (Green) and Line 10 (Turquoise), and connects to surface bus transportation at major transfer facilities. The alignment is placed in the median or on the shoulder of existing roadways (Fig. 3). The elevated stations have already seen growth nearby, as residents and businesses are eager to be near a dependable transportation option.

The total project was divided in two large phases. The first phase, until São Mateus, with a length of 6.8 miles (11 km),

Fig. 1—Metrô Line 15 (Silver) Monorail, São Paulo, Brazil.

Fig. 2—The elevated line snakes its way through the densely populated outlying areas of eastern São Paulo.

Fig. 3—Monorail piers in the median of an existing roadway.

10 stations, has an expected ridership of 340,000 daily passengers. The second phase, until Cidade Tiradentes, with a length of 16.5 miles (26.6 km), includes 18 stations and has an expected ridership of 501,000 daily passengers using 54 trains consisting of seven cars per train.

Construction of infrastructure of the first 1.8-mile (2.9 km) segment of the first phase between Stations Vila Prudente and Oratorio began in 2009, prior to the selection of the vehicle supplier. In 2010, Bombardier Transportation was selected as the vehicle provider and final design of the guideway beams began. In 2011, construction of the guideway beams and the infrastructure for the balance of the first phase commenced. By 2014, 8.1 miles (13 km) of dual guideway beams had been completed, as well as an impressive maintenance and storage facility. By 2018, it is expected that the eight remaining stations will be built, and commercial operation started.

The first 1.8-mile (2.9 km) system between Vila Prudente and Oratorio began on August 30, 2014. The section opened commercially to the public on August 10, 2015. Currently, approximately 8800 passengers use this line daily to access the remaining Metrô system from Station Vila Prudente.

The three-story stations (Fig. 4) have colorful pedestrian bridges that connect them to elevator and escalator access structures located on the sidewalks. The station has ample mezzanines with ticketing and support rooms. Escalators and elevators lead the users to platforms on the top level, where they board trains via full-height platform gates.

Just beyond Oratório Station, a spur track provides access to a remarkable maintenance facility called Patio Oratório, one of the two maintenance facilities planned to support the line (Fig. 5). There are eight switch structures that branch out to create 25 maintenance and storage track spurs; seven tracks extend into the maintenance facility buildings and one track serves as a vehicle wash bay. Each guideway beam in the maintenance yard is post-tensioned,

Fig. 4—Three-story monorail station.

Fig. 5—One of the two maintenance facilities.

while several of the tracks are simple span, requiring only one stage of post-tensioning due to the shorter span arrangement in this area. Administration and control rooms complete this maintenance facility.

The monorail structure is elegant and refined. An impressive "Superman Logo" type pier designed by Metrô São Paulo (Fig. 6) was chosen for use along the length of the entire project. The pier's flared shape tapers nicely to limit the footprint, while the larger top allows for easy placement of the dual guideway beams. In the first 1.8 miles (2.9 km), the average height of the top of beam is approximately 56 ft (17 m), above the tree line, to match in scale with numerous apartment buildings in the area. For the balance, shorter columns approximately 43 ft (13 m) were chosen to reflect the two- to three-story high buildings in the vicinity of the line.

The beams consist of attractive hunched hollow beams with an average length of 100 ft (30 m), with a parabolic profile consisting of cross sections varying from 7 x 2.3 ft (2.1 x 0.69 m) at the ends to 5.3 x 2.3 ft (1.6 x 0.69 m) at midspan. The beams are cast with varying horizontal and vertical geometry, with the radius of horizontal curvature as tight as 150 ft (46 m), which is ideal to be used in urban areas.

The post-tensioned concrete beams weigh between 60 and 80 tons and were fast to fabricate and erect. The beams were manufactured using American-made specialty forms in a dedicated casting yard (Fig. 7), where the first stage of post-tensioning took place (Fig. 8). The beams were then transported to the jobsite and lifted into position (Fig. 9 and 10). The structures were made continuous over four spans by casting closure pours on top of the columns and a second-stage post-tension was then applied. An end closure pour with fingered expansion plates then completed each frame. Unique pintels were designed for this project to keep the expansion locations aligned as the monorail vehicles pass from one frame to the next, providing displacement compatibility. The resulting continuous frame provides a very versatile method to bridge long distances with an efficient structural system.

One unique feature for monorail construction is that the precast beam is also the vehicle riding surface. There is no opportunity to correct any imperfections in the beams with a wearing surface or with a cast-in-place deck of any type. This requires precision during the casting of the guideway beams using specifically developed concrete

Fig. 6—Monorail piers.

Fig. 7—Post-tensioned concrete beams in the casting yard.

Fig. 8—Post-tensioned concrete beams at first stage stressing.

Fig. 9—Erection of PT beams.

Fig. 10—PT beams during construction.

Fig. 11—Green space below completed monorail.

forms. Computer programs developed by the designers created shop drawings for use in the precast yard. The exact measurements needed for setting the forms were provided on the drawings, including the location of all inserts needed for power supply systems and emergency walkway supports. The first-stage post-tensioning was balanced precisely to deliver guideway beams that were as flat as possible. Long-term creep and shrinkage were also considered to provide a smooth ride for the design life of the project. The second-stage post-tensioning was provided to take the live load forces of the monorail vehicles and their passengers. Aside from the fast guideway construction, the system is quite efficient in another way: the ratio for an empty/fully loaded train to beam dead load is approximately 0.5:1 or 1:1, respectively. It would not be feasible to provide these benefits without the use of post-tensioned beams. It is the post-tensioned beams that allow the structures to perform, where form follows function, in a creative, innovative, and ingenious cost-effective structure.

For the first phase, the trip from Station Vila Prudente to São Mateus by car during rush hour can routinely take over an hour. It will reliably be reduced to under 40 minutes by monorail.

There are two additional monorail projects currently under development in Brazil: Line 17 (Gold) and Line 18 (Bronze). Post-tensioned concrete monorail structures continue to be cost-effective solutions for urban and rural applications with moderate temperatures around the world.

It is in this way that the graceful shape of the post-tensioned monorail structures blend nicely with the landscape, providing green space with bike lanes (Fig. 11) beneath much of the alignment, uniquely contributing to the quality of life of citizens in densely populated cities around the world such as São Paulo.

(A very special gratitude is extended to Embrafoto and Sergio Mazzi for sharing their wonderful photos of the São Paulo Line 15 Monorail.)

Location: São Paulo, Brazil
Owner: Metrô São Paulo, São Paulo, Brazil
Architect: Metrô São Paulo, São Paulo, Brazil
Engineer: (1) Planservi, SP, Brazil (2) Proenge, SP, Brazil (3) Innova Technologies, Las Vegas
Contractor: (1) Bombardier Transportation (2) Construtora Queiroz Galvão (3) OAS Engenharia
PT Supplier: Protende Sistemas e Métodos de Construções Ltda
Other Contributors: Planvia, SP, Brazil; Setepla SP, Brazil; Zamarion e Millen Consultores; ENGETI Consultoria e Engenharia; Condutix-Wampler, Omaha, NE; Helser Industries, Tualatin, WA.
Submitted by: Innova Technologies, Inc.

Jury Comments:

- This is an excellent example of the ability of post-tensioned structures to meet structural, functional, and aesthetics demands.
- This is spectacular; just the sheer magnitude of this project is quite impressive.

2017 AWARD OF MERIT: ROY & DIANA VAGELOS EDUCATION CENTER

INTRODUCTION

The Roy and Diana Vagelos Education Center (Fig. 1) is a 107,000 ft^2 (9940 m^2), 15-story state-of-the-art medical education facility that links students and teachers, function, and experience in an interactive, interdisciplinary learning environment, all while creating a new identity and focal point for Columbia University's Washington Heights campus. It aspires to be an iconic facility that heralds a new era in modern medical and graduate education, attracting the world's top medical students in the process.

ARCHITECTURAL CONCEPT | THE STUDY CASCADE

In response to Columbia University's lofty ambitions for their new flagship facility, DS+R and Gensler envisioned a vertical campus of intertwined, cascading outdoor terraces and social and study spaces that fosters an environment of open collaboration and intellectual exchange. The result is a structurally remarkable and aesthetically alluring southern façade, dubbed the "Study Cascade," that contains vertically interconnected study and social spaces that creatively blend function and experience. The façade is a highly articulated all-glass system that is crucial to the building's expression. In contrast, the northern half of the building is largely uniform floor-to-floor, and is organized for classrooms and administrative space, in addition to a mid-tower mechanical space that supports the building's Anatomy Labs.

CASCADING CANTILEVERS | EFFICIENCY IN STRUCTURAL DESIGN

The main structural design challenge was to find vertical load paths through the Study Cascade while respecting the varied spatial planning of the stacked, two- to three-story atrium-like clusters of diversified social spaces. To minimize the structure's impact on these spaces, the cascade floors are supported by a pair of inclined composite concrete columns (Fig. 2) that are architecturally exposed and cast with high-strength self-consolidating concrete

Fig. 1—The Roy and Diana Vagelos Education Center at Columbia University's Washington Heights campus.

Fig. 2—Inclined columns from the foundation level up to the eighth floor.

Fig. 3—Bonded post-tensioning system.

Fig. 4—PT and void formers.

(10 ksi [69 MPa]). These two exposed columns slope from the foundation level up to the eighth floor, directing loads around a column-free auditorium at the base of the Cascade. The thrusts that result from the changes in direction of the sloping columns are resisted by in-floor trusses constructed with post-tensioning and high-strength reinforcing bar.

A bonded post-tensioning system (Fig. 3) was critical to achieving the long, open floor spans of the cantilevers with minimal structural depth (tapering to as thin as 8 in.) and no perimeter columns, ensuring that the high-strength (8000 psi [55 MPa]) concrete flat slabs successfully met the stringent deflection performance requirements of the all-glass façade. Void formers, manufactured by Cobiax USA, are placed between bands of post-tensioning in the slabs to create long-span, beam-like framing with flat formwork and to reduce the structure's self-weight (Fig. 4).

The flexibility of the post-tensioning system simplified detailing at areas that required tight coordination with the multi-story façade and other trades. Post-tensioning stressing anchor positions were detailed to avoid placing anchors in congested areas, such as the Study Cascade's south slab edge. At critical post-tensioning locations, the structural drawings provided façade embed adjustment recommendations to avoid interference with the post-tensioning tendons.

The building's structural design achieves efficiency by embracing the layout of the stacked neighborhoods. The structural system of the Cascade leverages the natural interconnections that come from the unique arrangement of the program spaces of the vertical campus. Single-story walls and ramps connect and stiffen the cantilevered slabs, allowing for savings in the slabs' post-tensioning, reinforcing bar, and concrete quantities.

ARTICULATED FAÇADE | ATYPICAL CURTAIN WALL

The curtain wall of the Cascade is arranged in multi-story planes that do not align in plan between the stacked neighborhoods and are not parallel with the slabs' edges, presenting difficult detailing challenges for both the structure and façade. Through a design assist phase, anticipated deflections from the slab curvature were coordinated at each glass mullion with the curtain wall contractor, Gartner, and together the project team agreed upon an acceptable long-term deflection limit of 1.25 in. (30 mm) for cantilevers up to 26 ft (7.9 m). The bonded post-tensioned slabs were tuned to meet these performance requirements and were detailed to accommodate a range of façade attachment methods.

Long-term column shortening of the two Cascade columns will eventually cause amplified deflections at the tips of the cantilevers they support. To account for this phenomenon, a staged construction analysis was performed and summarized into a simple planar cambering schedule for the Cascade slabs. Super-elevated slab positions were determined to ensure that the slabs would be level after long-term column shortening occurs. Accordingly, curtain wall installation procedures were arranged to account for the slabs' super-elevated positions.

FROM RENDERING TO REALITY

Completed in 2016, the Roy and Diana Vagelos Education Center stands as a nearly identical realization of the architects' vision—rarely does a completed building so accurately reflect its original renderings (Fig. 5). The success of the project team in bringing such a unique integration of architecture and structure to life is a testament to their commitment to an early and open dialogue between the owner, design team, contractors, and builders, as well as to the remarkable advantages of using a bonded post-tensioning system in complex concrete formwork. In combination with Cobiax void formers, the use of post-tensioning proved crucial to achieving long-span cantilevers that maintained a thin slab edge profile while meeting stringent deflection performance requirements of the all-glass façade.

Fig. 5—Architect's rendering (left) and the completed building (right).

Location: New York, NY
Owner: Columbia University Medical Center
Architect: Diller Scofidio + Renfro (lead designer), Gensler (executive architect)
Engineer: Leslie E. Robertson Associates
Contractor: Sciame Construction LLC
PT Supplier: VSL/Structural Technologies
Other Contributors: Urban Foundations/Engineering; Difama Concrete; Bethlehem Precast; Jenna Concrete; Raul Herrera; COLE Technologies; Steve Zimmerman; Titan America; STI Construction; Tilcon New York, Inc.; Roanoke Sand & Gravel Corp.; BASF the chemical company; Euclid Chemicals; DOKA; EDC; CFS Steel Reinforcing Accessories Manufacturer-CFS Steel, Weidlinger Associates Inc., Cobiax USA, United Structural Works

Submitted by: Leslie E. Robertston Associates

Jury Comments:

- This project has an amazing structural system and appearance made possible only through the use of post-tensioning.
- This is perhaps one of the most spectacular buildings I have ever seen.
- This structure was tremendously and technically challenging from an engineering standpoint and so aesthetically pleasing from an architectural standpoint.

2017 AWARD OF MERIT: ADAMS PRECAST SEGMENTAL TOWER POST-TENSIONED PRECAST SEGMENTAL TOWERS FOR HIGH WIND TOWERS

Fig. 1—Adams County precast tower.

Large-scale commercial wind farms are a significant source of energy generation today in the United States and internationally. The initial cost of construction typically determines the competitiveness of wind energy in comparison to the energy sources of others. The costs associated with transporting large steel tower sections between the tower fabricator and the jobsite are high and often require modifications to existing roads and bridge infrastructure to accommodate. A utility-scale wind farm project commonly has 60 or more towers installed. This creates the opportunity for on-site industrialization in production of the precast segments and significant savings in tower transportation costs. This industrialized approach to constructing wind towers on site provides many of the same benefits found in the construction of large precast segmental bridge structures, where many standardized segment geometries also exist. Wind Tower Technologies (WTT) developed a patented precast segmental tower system for high wind towers in 2011. The company licensed the tower technology to Siemens Energy in 2013 and formed a commercial partnership together with Siemens to bring it to market with a focus on North and South America.

The Adams County precast tower (Fig. 1) is the highest tower constructed in the United States in steel or concrete. The hub height is 377 ft (115 m) and supports a 2.3 MW turbine and 354 ft (108 m) diameter wind blades, resulting in a total height of 605 ft (169 m). The new tower system was fabricated on site, eliminating the disruptions and costs inherent with transporting large steel tower sections over roads and bridges from remote locations to the project site. Using on-site concrete fabrication, the tower base diameter is nearly unrestricted, thereby providing the height of the tower to be limited only by zoning permits and erection equipment.

The significance of taller towers is higher energy production in many geographic markets where increased and more sustained wind speeds exist with height. The market opportunity for concrete wind towers in North and South America is high as the wind market trends towards taller towers.

ON-SITE SEGMENT PRECASTING

Besides the removed limitation on the base diameter and corresponding tower height, additional benefits to on-site concrete precasting includes the use of local labor and locally sourced materials such as reinforcing steel, aggregates, cement, and admixtures. Further cost-saving opportunities can exist when the foundation contractor and concrete tower contractor can coordinate to source materials from the same batch plant.

To achieve the required speed of construction, the concrete tower segments are match cast together, resulting in a tight fit between segments when installed. The geometry of the tower is therefore largely set in the casting yard with minor provisions for alignment adjustments during erection.

Fig. 2—On-site segment precasting.

Fig. 3—Precast segments are match cast.

Fig. 4—Cast-in-place foundation.

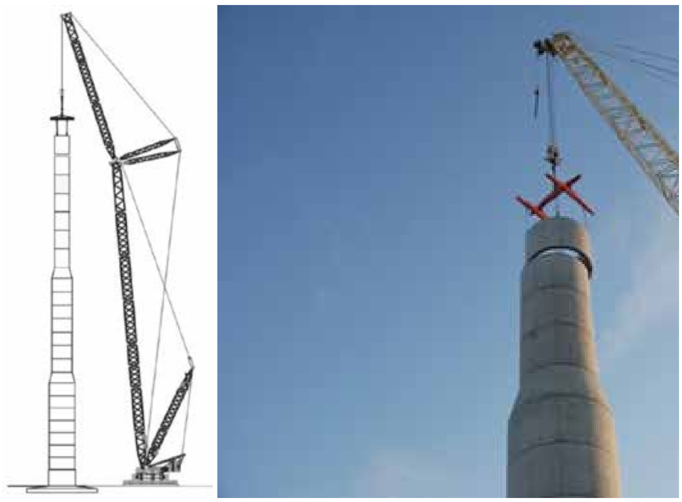


Fig. 5—Precast segmental tower erection.

Fig. 6—Stepped concrete tower design.

CAST-IN-PLACE FOUNDATION

The cast-in-place foundation for the tower incorporates an annular pedestal wall that enables the precast tower to connect to the foundation using a grouted joint to provide a uniform transition of forces across this joint. The connection, located close to ground level, is the only grouted joint in the tower.

The concrete tower weight exceeds that of a steel tower for the equivalent load-carrying capacity, thus reducing overturning moments and resulting in a reduction in foundation quantities for a concrete tower in comparison to that for a steel tower. The precast tower system is installed onto the foundation's pedestal wall and secured to the foundation using post-tensioning. This connection has benefits over steel tower connections, whereby mechanical anchor bolts through the steel flange plates, reducing future maintenance of these mechanical components.

INSTALLATION OF TOWER SEGMENTS

With wind towers increasing in height and carrying larger rotors and turbines, innovative concrete tower designs and construction techniques are becoming increasingly important.

The precast segmental tower solution is changing the wind industry's landscape. During the development of the WTT/Siemens solution, the importance of construction speed is critical. The use of match-cast segments greatly expedited the stacking of segments (Fig. 5).

Fig. 7—Nineteen-strand external tendons around perimeter of tower.

The concrete tower design was stepped to optimize the use of formwork. The transition from concrete to a steel tip adaptor near the top of the tower allows for a standardized steel top section for the yaw attachment and cabling platforms.

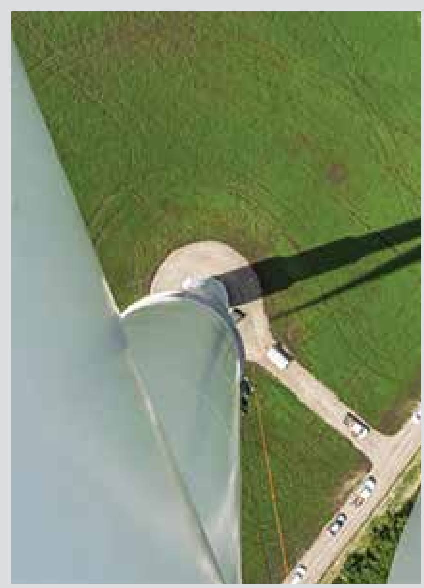
POST-TENSIONING

The 19-strand external tendons located around the perimeter of the tower run the full height of the concrete tower. This configuration provided the design team and contractor with the ability to stack all segments prior to installing and stressing the tendons. Taking this operation off the critical path resulted in significant savings of time and took the post-tensioning off the critical path of the crane activities.

Each tendon was prefabricated offsite and delivered to the wind farm protected from the environment and ready to install and stress (Fig. 8).

Fig. 8—Prefabricated tendons shipped to site ready for installation.

VIEWS FROM THE TOP OF TOWER



Location: Adams County, IA
Owner: MidAmerican Energy Company
Architect: Wind Tower Technologies, LLC
Engineer: Wind Tower Technologies, LLC
Contractor: (1) Siemens Wind Energy (2) Baker Concrete
PT Supplier: Schwager Davis
Other Contributors: EFCO Forms, Thornton Tomasetti Engineers, International Bridge Technologies
Submitted by: Wind Tower Technologies, LLC

Jury Comments:

- This project displays a unique and potentially game-changing use of post-tensioning for wind tower applications.
- This project has the potential to shed some light on a new market for post-tensioning in wind towers.
- This is a trend-setting structure in the United States.

2017 AWARD OF MERIT: GBMC DAFFODIL GARAGE EMERGENCY REPAIRS AND WATERPROOFING

BACKGROUND

The Daffodil Garage is a three-and-a-half-level parking structure located on the Greater Baltimore Medical Center campus in Towson, MD (Fig. 1). The original garage was built in 1976 as a post-tensioned reinforced concrete flat-plate structure. One unusual feature of this garage was that the entrance and exit were on the top level and access to the lower levels was made possible by up/down ramps located on the east and west ends of the structure. A second phase of construction took place in 1993, which involved the addition of a helical access ramp on the west end of the garage and the installation of post-tensioned "infill slabs" at the locations of the original ramps. After years of neglect and due to some odd design features and questionable decisions on the part of the facilities management at the time, the garage was in poor condition and in serious need of repair (Fig. 2) to assure the continued life/safety of all garage occupants.

Fig. 1—Daffodil Garage at the Greater Baltimore Medical Center, Towson, MD.

ISSUES

The footprint of the garage runs from column line 1 (west end) to 16 (east end) and A (north end) to G (south end) with an expansion joint at column line 9, which splits the garage into east and west halves. For an unknown reason, this expansion joint does not run in a straight line and jogs out and around the center column at D9. This expansion joint was originally designed as a slip joint with the concrete beam on column line 9 widened and notched out to receive the sliding post-tensioned slab. However, the original expansion joint was constructed without a sliding polytetrafluoroethylene (PTFE) pad and stainless steel bearing plate assembly that would allow for movement of the post-tensioned concrete slab that rests directly on top of the beam below. In addition, the expansion joint assembly was complicated by the fact that the north sliding slab was east of column line 9 and the south sliding slab was west of column line 9. All these design and construction issues resulted in damage to the concrete slabs, beam, and columns along column line 9 and failure of the expansion joint gland that allowed intrusion of water into the beam, columns, and parking structure below (Fig. 3).

In the mid 2000s, the hospital management made repairs to the Daffodil Garage, which included some concrete repairs at column line 9 and the installation of a traffic-bearing waterproofing membrane on the top concrete deck of this structure. Unfortunately, this traffic membrane was not maintained, which resulted in many slippery surface areas where the non-slip

Fig. 2—Column line 15 before repairs.

aggregate finish had worn due to normal vehicular garage traffic. Over time, ice accumulated on the top deck during winter months, especially after snow and ice storms, which contributed to slip and fall injuries. Unfortunately, in lieu of installing a new topcoat over the existing traffic membrane, the hospital facilities department hired a contractor to use a milling machine to remove the existing traffic membrane. The milling process not only removed the traffic membrane but also removed 1/4 to 1/2 in. (6 to 13 mm) of the concrete slab surface that left deep gouges throughout the concrete deck, which reduced the concrete cover over the existing mild-steel reinforcing bars and PT tendons; exposed the slab reinforcement in numerous areas; reduced the live and dead load capacity of the top deck of this structure; increased the top deck deflection in numerous areas that allowed water to pond on the top deck surface; weakened the exposed concrete surface of this concrete structure; subjected the top deck reinforcement to salt and water intrusion; allowed water penetration throughout the entire garage structure; and dramatically reduced the durability of this concrete post-tensioned structure. After years of deterioration that compromised the life/safety of all garage occupants, the hospital was forced to make emergency repairs to fix this parking garage. The work included concrete slab, beam and column repairs, and the installation of a new epoxy leveling coat and traffic-bearing waterproofing membrane coating with a slip-resistant surface.

2015 REPAIRS

The focus of the 2015 repairs was to make the Daffodil Garage watertight by November, which would protect and prevent future damage to the lower levels of the garage. The work began in March with the emphasis the concrete repair of the top deck ramp infill slab at column line 15 (Fig. 4 through 6), structural repairs to the PT beam below the top deck at column line 9 (Fig. 7 through 11), partial-depth and full-depth concrete repairs throughout the top deck (Fig. 12 and 13), repair of damaged/broken PT cables within the top deck slab, and the installation of new watertight expansion joint at column line 9. After the top deck was structurally restored to satisfy all code-required live and dead loads, an epoxy leveling coat was installed to provide a structurally sound and level surface capable of supporting a traffic-bearing waterproofing membrane that had a service life of 10 years (Fig. 14). Deck drains were added at all areas experiencing ponding water to immediately remove all surface water from the top deck, which will help prevent ice buildup during winter storms. These repairs would ensure continued use of the parking structure and more importantly prevent further damage to structural elements on the lower levels of this facility until additional phases of repair could take place.

Fig. 3—Column line 9 before repair.

Fig. 4—Column line 15 demo.

Fig. 5—Column line 15 new reinforcing bar and PT cables.

Fig. 6—Column line 15 reinforcing bar, PT top deck.

PHASING/REPAIR PROCESS

Phasing for this garage was somewhat difficult because the only entrance to the garage is on the top level, which is also where the majority of the work was to take place. Therefore, the phasing would need to be coordinated carefully to maintain access to the entrance and levels below. The repair crew worked a combination of weekday, night, and weekend hours to minimize impact on hospital operations. The project team of the owner, engineer, and contractor met on a bi-weekly basis to discuss phasing conflicts and schedule to make sure the facility and doctors were aware of the work and could notify their patients of any shutdowns or phasing changes.

Phase 1 was to make repairs to the post-tensioned support beam that supports the former access ramp PT infill slab at column line 15. The existing support was several steel angles, which had become severely rusted and deteriorated due to water intrusion through the damaged top deck slab. The scope was to remove these deteriorated steel angles and add a concrete corbel to the east side of the existing concrete beam, which would support the slab (Fig. 15). The repair work also included repairing 43 PT tendons that were also damaged from water penetration.

Phase 2 involved repairs to the PT beam and east PT slab located at the expansion joint on column line 9, which were both severely damaged due to water penetration and a failure to properly construct a sliding slab connection for the expansion joint to properly function. These repairs took place in two separate phases to allow continuous flow of traffic in the levels of the garage below. During demolition of the spalled beam concrete, it was discovered that five 0.5 in. (13 mm) diameter PT tendons were broken,

Fig. 7—Column line 9 new 0.6 in. PT cable.

Fig. 8—Column line 9 new steel and new 0.6 in. PT cable.

Fig. 9—Column line 9 new 0.6 in. PT cable.

which required replacement with three 0.6 in. (15 mm) diameter PT tendons, which needed to be placed along the entire 120 ft (37 m) length of this PT beam. To allow these new PT tendons to be placed and poured in two stages, it was decided to widen the PT beam to provide a place for the new PT tendons. This approach saved time and money for both the contractor and owner. To assure that the east PT slab was allowed to move under thermal and moisture/volume changes at the expansion joint, it was necessary for the contractor to install a new sliding assembly consisting of a PTFE pad and stainless steel plate under the existing east slab, which required a one-weekend shutdown of the garage to complete the installation. This installation process required that the east slab be shored for two garage levels. A hydraulic jack assembly was developed that lifted the slab approx-

Fig. 10—Column line 9 formwork.

Fig. 12—Top deck PT repairs.

Fig. 11—Column line 9 finished repair.

imately 1 in. (25 mm) high in 5 ft (1.5 m) sections that was then immediately followed by shoring posts that support the slab to allow the hydraulic jack assembly to be lowered and moved to the next section. After the slab was lifted, the repair crew sandblasted/cleaned the underside of the slab and top of the PT beam ledge and installed the new sliding bearing assembly that was epoxied to the PT beam ledge. Soon after the sliding assembly was in place, the east slab was lowered to assure a good bond was created between the PT beam and sliding assembly.

Phases 3, 4, 5, and 6 involved over 2400 ft^2 (220 m^2) of partial-depth and full-depth concrete repairs to the top-level slab, which also included over 100 repairs to damaged/broken PT tendons. Again, these phases were planned and coordinated with the hospital management to keep drive lanes open to the lower levels.

Fig. 13—Top deck PT and concrete repairs.

Fig. 15—Before repair, metal angles supporting slab.

Fig. 14—Before and after concrete resurfacer.

Fig. 16—Column Line 9 new expansion joint and coating.

The final phase was to install the new watertight expansion joint gland on column line 9 and at the entrance bridge and the heavy-duty vehicular traffic coating (Fig. 16). This work was split into three sub-phases to maintain drive lane access to the parking levels below and it was completed over weekends to minimize impact on the facility. As previously discussed, the exposed concrete surface of the top deck of this garage was severely compromised during the milling process that was used to remove the previous traffic membrane. Although the best way to restore the structural integrity would be to install a new bonded concrete overly, this option proved cost prohibitive for the owners. As an alternative, the engineer and contractor suggested that a leveling coat consisting of a fluid epoxy/sand mixture be installed to fill in the gouges in the concrete deck and provide an acceptable surface for the installation of a new heavy-duty traffic-bearing waterproof membrane. This coating system was a four-step process: epoxy leveling coat, waterproofing base coat, heavy-duty epoxy intermediate coat, and UV-resistant topcoat with heavy aggregate to create a slip-resistant surface (Fig. 17). A series of mockups were performed to confirm the necessary coverage rate for the epoxy resurfacer and determine which coverage rate would provide an acceptable surface for the waterproofing system. Pull tests were also conducted to confirm that the epoxy resurfacing material could double as the primer coat, saving the contractor much-needed time and the owner substantial money. The new coating greatly improved the appearance of the top deck while also creating a watertight and slip-resistant surface that would protect the structural elements on the levels below.

Fig. 17—Coated top deck.

CONCLUSIONS

The GBMC Daffodil garage was originally scheduled to be demolished and replaced in 5 years, which would have been a tremendous financial burden on the hospital operations, but the repairs were conducted in 8 months, saving the owner time and money. The repairs, maintenance, and protection of the garage as developed by the engineer and constructed by the restoration contractor is the first step in assuring that this facility remains serviceable for the next 25 years. The cost savings to the hospital while allowing this facility to remain occupied is a tremendous benefit to the users of this health organization.

Location: Towson, MD
Owner: Greater Baltimore Medical Center
Engineer: Morabito Consultants, Inc.
Contractor: Concrete Protection & Resotration, Inc.
PT Supplier: DYWIDAG Systems International
Submitted by: Concrete Protection & Restoration, Inc.

Jury Comments:

- The use of post-tensioning for repairs and strengthening extended the service life of this structure for another 25 years, as opposed to complete replacement.
- The repair work presented was just so great and really showed off the power of post-tensioning.

2017 AWARD OF MERIT: TRUCK MAINTENANCE FACILITY

In 2008, planning began for Teck's off-road truck fleet expansion at the Fording River Mine in Eklford, BC, Canada. Two existing facilities had been constructed under DSI Canada Civil, Ltd.'s direction in 1997 and 2001. This new facility was to accommodate the even larger, newer off-road vehicles when scheduled for either repair, cleaning, or simply a change of tires.

It was determined that the final structure configuration would be not less than four bays 66 ft (20 m) in width, with a minimum clear height of 75 ft (23 m), incorporating 75 ton (68 metric ton) overhead cranes with interstitial bays of 30 ft (9.1 m) to permit access by specialized equipment to manipulate and change the 12 ft (3.6 m) diameter tires used by these off-road earthmovers (Fig. 1).

Additional requirements were the provision for one of the truck bays to be dedicated as a wash bay. This wash bay was to be equipped with high-pressure water cannons capable of dislodging the accumulated mud and debris from the trucks prior to servicing. This is an essential process in the servicing of these trucks, which can

Fig. 1—Teck truck maintenance facility.

accumulate a significant amount of mud and rock (up to 10 tons [9.1 metric tons]) and prevent access to the undercarriage, which can be dangerous for shop personnel.

The previous facility (built in 2001) was fitted with in-floor heating and constructed as a single pile-supported, simply reinforced slab, but because of the combinations of heavy loading and repeated cycling of the heating system, it was determined that the in-floor heating required a better solution (Fig. 2). To add complexity, the facility was also to offer new space for soundproof management offices located in the upper half of the interstitial bays.

The two previous truck facilities and the remainder of the mine structures occupied the most desirable and readily accessible and constructible areas of the mine site. The previous facility's foundations, based on driven piles, had experienced some difficulty due to the presence of deep, undetected underground rifts and crevices in the underlying rock, where the depth of the rock surface varied significantly and required many on-site modifications.

Once the decision had been made to implement a large raft foundation, Golder Associates Ltd. was charged with providing a comprehensive investigation of the subsoil conditions. Their investigation determined that the rock surface at the proposed location varied significantly with a large rift at least 25 ft (7.6 m) in depth over one-third of the site.

Mine tailings containing only a very small percentage of coal residue were provided to support the raft slab. This raft slab had to be capable of providing sufficient stiffness to remain effectively level over the wildly varying depths of compacted tailings, which were compacted to 98%+ density. The tailings were reinforced with geogrid and compacted in 8 in. (200 mm) lifts. This provided a stable base and economical solution to the subsequent raft slab installation.

The slab itself consisted of a base slab 20 in. (510 mm) in thickness, which has no other reinforcement but the uniform, equally spaced two-way post-tensioned grouted tendons, (Fig. 3) centered in the slab with little or no deviations to produce the maximum post-tensioned force possible (Fig. 4).

The wash bay incorporates a fully integrated large tank (approximately 20 x 100 ft [6.1 x 305 m]) to both store the water resulting from the wash operation and also to partially contain some of the residue that is dislodged from the truck carriages. The larger mud and rock debris is cleaned off with a front-end loader.

This tank uses three forms of post-tensioning (Fig. 5). The base of the tank consisted of post-tensioning tendons

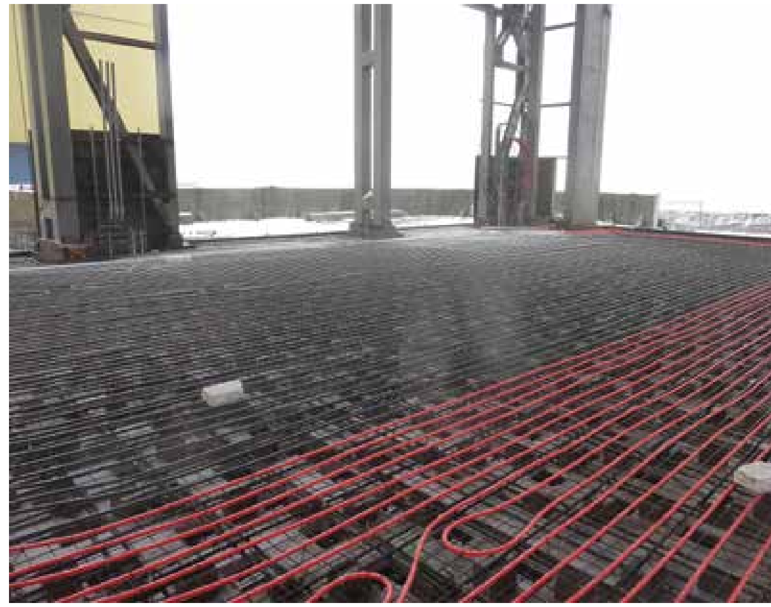


Fig. 2—Heated slab.

Fig. 3—Bonded PT anchorages.

Fig. 4—Two-way grouted PT base slab.

Fig. 5—Tank under construction.

Fig. 6—Walls of tank with vertical PT bars and horizontal encapsulated unbonded tendons.

in the longitudinal direction because of the ability to follow the base contour. Stressing bars of 1.4 in. (36 mm) diameter were used transversely across the base and for the vertical walls of the tank due to the ability to be both self-standing and to be accurately tensioned over short lengths. Horizontally, the walls were post-tensioned using 0.6 in. (15 mm) fully encapsulated unbonded monostrand tendons placed in pairs, one on each side of the vertical 1.4 in. (36 mm bars) (Fig. 6). All post-tensioned elements were grouted following the tensioning operation except for the encapsulated unbonded tendons. All walls and the tank base are a nominal 23.6 in. (600 mm) in thickness for simplicity and water retention requirements. The average compression from post-tensioning is equal in all three axes of the tank. The entire tank is equally supported on the compacted tailings and is fully integrated with the raft slab (Fig. 7).

The top surface of the tank is equipped with slotted square cast steel grates each weighing some 6 tons (5.4 metric tons) and is designed to provide a uniform load-carrying surface. The steel grates are each fully removable using overhead (waterproofed) cranes, allowing the tank to be occasionally cleaned of debris.

The walls of the wash bay are lined with 11.8 in. (300 mm) prestressed solid concrete planks incorporating viewing/lighting windows capable of resisting impact produced by debris being dislodged by the water cannons. Entrance to the area is limited and carefully monitored. Additionally, the wash bay incorporates an exterior, fully heated post-washing/dripping pad.

Beyond the wash bay, there is also a heated bay with a heated floor capable of handling two trucks and used as a warming area for the trucks prior to the cleaning operation (Fig. 8 and 9).

This pad is partially articulated perpendicular to the building to allow for minor movements as a result of its summer/winter exposure. It is of a similar nature to the wash bay slab being fully post-tensioned in two directions and incorporating steel rails to minimize damage created by the large front-end loaders equipped with chains. This slab is capable of accommodating significant vertical rotational movement.

There are many more aspects and benefits to the building, but without the use of post-tensioning for the base slab, this structure would have never been constructed or been constructed as economically and successfully.

One of the most beneficial aspects of this method of construction, other than the very stable slab, is the more effective organization of activities. The normal construction of in-slab heating requires up-front integration of the mechanical and electrical trades. This is often time-consuming, generally contrary to general construction practice, and the results can often be unsatisfactory. By constructing the base slab separately, it allowed the superstructure to proceed immediately following stressing of the tendons and the addition of a secondary slab with electrical components in addition to the anticipated mechanical components made it easier to coordinate the construction activities. An additional site benefit is provided in the ability to construct the secondary slab under cover or even during inclement weather. The secondary slab can thus be virtually crack-free with fewer joints. The base slab is jointless thanks to post-tensioning. The ability to make necessary repairs or modifications without altering the main structure is a definite plus.

Fig. 7—Tank fully integrated with raft slab.

Fig. 8—Warmup slab under construction.

Location: Elkford, BC, Canada
Owner: Teck
Engineer: J.R. Spronken & Associates
Contractor: Graham Construction
PT Supplier: DSI Canada Civil, Ltd.
Submitted by: DSI Canada Civil, Ltd.

Jury Comments:

- This is not your typical slab-on-ground!
- The use of post-tensioning made possible the wash bay and other building features and all of it supported on a large raft-type post-tensioned foundation.

Fig. 9—Completed warmup slab.