TSX BROADWAY—POST-TENSIONED TRANSFER GIRDERS AND THE RAISING OF THE PALACE

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TSX Broadway and the Palace Theatre Renovation together are a project with numerous complex pieces, each of which required unique design approaches to solve complex problems using time-tested techniques but applied in innovative ways. The Palace Theatre and the former DoubleTree Hotel sat at 1568 Broadway, the intersection of 7th Avenue and 47th Street, in the heart of Times Square in New York City (Fig. 1).

In its original configuration, the Palace Theatre was located at the base of the DoubleTree Hotel. It occupied the first seven floors and a majority of the cellar floor. When the DoubleTree Hotel was built in the 1990s, its support system spanned over the existing Palace Theatre with a series of tri-level steel trusses that were tight to the top of the existing theater. This truss system was supported by four "super columns" that formed a tabletop over the existing theater with the space on the west side reserved for stairs and elevators to access the hotel floors above.

The streetscape of Times Square is one of the most valuable retail spaces in the world. A theater is open for business both in the afternoon for matinees and in the evening. Even during a theater's business hours, its street-front is not generating revenue from the passersby. The owner sought to find a way to monetize this prime location for the greater part of the evenings. Razing the theater was definitely not an option. The Palace Theatre (Fig. 2) is world famous, and its interior is a historic landmark. However, retail space also had to be created, which led to an unprecedented design decision: lift the theater, in its entirety, approximately 30 feet (9.1 m) higher than its current street elevation to allow for two levels of column-free ground-floor retail. To add to the complexity, most of the cellar space was already used by the theater but was inadequate to support the needs of a modern theater. Adding space was a necessity but the only direction of expansion available was to go deep into the supporting rock.

Zoning laws have evolved in the 30 or more years since the construction of the Double Tree, and any new construction that was not a renovation would require conformance with the new requirements resulting in less hotel space. To qualify as a renovation, 25% of the existing building area had to be maintained as is. Every new structural element that was added took away from net area. "New" area could not replace "old" area since "old" area had to always be retained. Ergo, the new subcellar had to be carved out of rock while the existing theater and the

Fig. 1—TSX Broadway

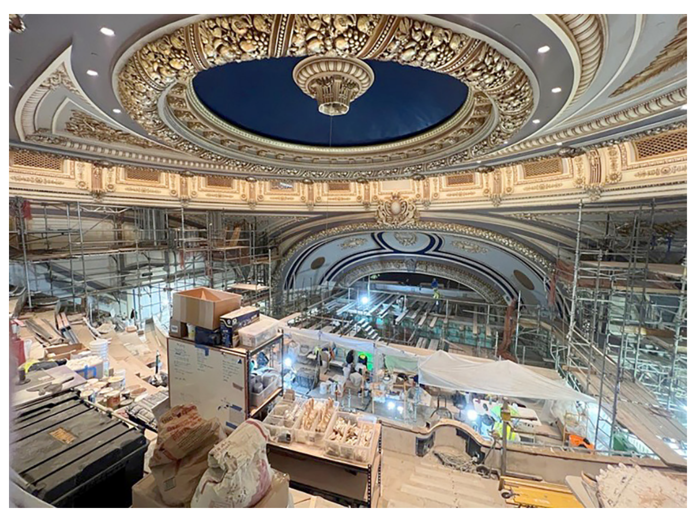

Fig. 2—Palace Theatre renovation

retained portion of the hotel tower remained above.

This project focused on maintaining all the existing systems in place while adding a transfer system above a 110-year-old theater, ensuring that the transfer system could be built without any temporary support or shoring, and designing the transfer system so that it would not continually deflect while a 42-level cast-in-place concrete hotel tower was placed on it.

THE NEED FOR A NEW TRANSFER SYSTEM

As the existing steel transfer trusses were located directly above the existing theater, no theater raising would be possible without first creating a volume of space to raise the theater into. Therefore, the existing system had to be removed, and a new system had to be designed and built at least 30 ft (9.1 m) higher than the existing one.

The transfer system would need to support 42 floors of hospitality space and would need to span about 130 ft (39.4 m) to safely clear the existing theater. It had to allow for the retention of the existing floors, and it had to be built while the existing tower above was still being demolished. It could not have any significant deflect under load and had to stay almost totally flat under its final fully loaded state. All this on one of the busiest corners of the world.

TRANSFER STRUCTURE OPTIONS EXPLORED

Several different structural systems were explored for the transfer structure.

A structural steel truss scheme seemed most logical. Due to the limited area around the site, given its location in Times Square, it was not feasible to build the trusses on the ground and lift them as one full truss. No tower crane that could fit on the site had the lifting capacity and reach to erect this system. As such, the trusses needed to be built, element by element, within the existing structure. Because of this, the structural steel was discarded as it was deemed too time-consuming, costly and dangerous to public safety.

A conventionally reinforced concrete solution using standard deformed bars was explored.

There was no lay-down space on site. The mild steel reinforcement for such a system could be pre-built and lifted into place with the tower crane. Every idea from high strength reinforcement bars to 14 ksi (97 MPa) concrete was fully designed. But questions remained. How does one support the wet weight of concrete over an old theater? What holds the formwork in place? Where do the temporary shores go? And above all, how much additional self-levelling concrete is required on 42 floors as this system deflects? A transfer system camber would help with the truss but the new floors in the lower half of the tower would crack as the support deflected.

Enter the post-tensioned (PT) transfer system with multi-stage erection and staged prestressing.

OVERVIEW OF PT TRANSFER SYSTEM WITH STAGED PRESTRESSING

The system, as conceptualized, would have three girders. Each would be about 5 ft (1.5 m) wide and about 40 ft (12 m) deep and would be capable of spanning anywhere from 130 to 135 ft (39.6 to 41 m) (Fig. 3). The three girders would be tied to each other at their ends with similarly dimensioned cross girders. They would be tied at their bottom by a cast-in-place concrete slab and at their top by a cast-in-place concrete mat that would form the base for the hotel tower. This base would give the hotel design architects complete freedom to design the hotel tower and place the hotel columns and vertical circulation systems anywhere they wished. The four existing "super columns" would be extended the extra height to meet the higher positioned transfer system. The four super columns would be laced together in the cross direction to create two laterally braced frames.

THE EVOLUTION

The PT girders vary in width between 4.5 ft (1.4 m) and 5 ft (1.5 m) wide and are approximately 41 ft (12.4 m) deep. Each girder is approximately 140 ft (42.4 m) long with a 130 ft (39.4 m) long clear span.

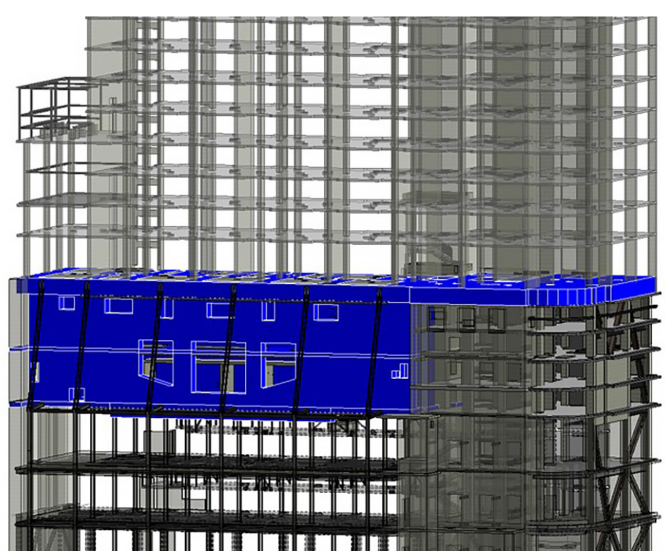

Fig. 3—Transfer structure with building

Each girder contains five groups of PT strands. These PT strands are tensioned in a phased manner during construction with the first group of strands being tensioned to counteract the self-weight of the girder itself and the next four sets of strands each tensioned every time 10 floors of structure are constructed.

The majority of PT strands are draped to match the natural sag of a catenary. The transfer system occupies floors 12 through 16 of the building, with the existing steel truss transfer system occupying the sixth to ninth floors and acting as the support and safety shield. As no shoring or falsework could extend through the theater, which is directly below the transfer system, the new PT transfer system used the existing steel truss transfer system to support its load (primarily wet weight of concrete) during construction until it achieved strength. As there was no existing steel transfer truss near the new center post tension girder, it would have required an extensive shoring system to support the self-weight of the middle concrete girder. To avoid the need for an extensive temporary shoring system, the middle girder contains additional straight strands in the first tensioning stage and half the strands in this first tensioning group were stressed early, to partially balance part of the girder self-weight thereby bypassing shoring requirements. These early strands are straight, instead of draped, as the PT girders are constructed in four lifts, each approximately 10 ft (3 m) tall. They could not drape these strands as they would terminate in the top portion of the girder (which was not yet built) and therefore could not be tensioned early. The need to pour the girders in discrete "lifts" was due to the constraints of limiting the weight of the placement (Fig. 4 and 5).

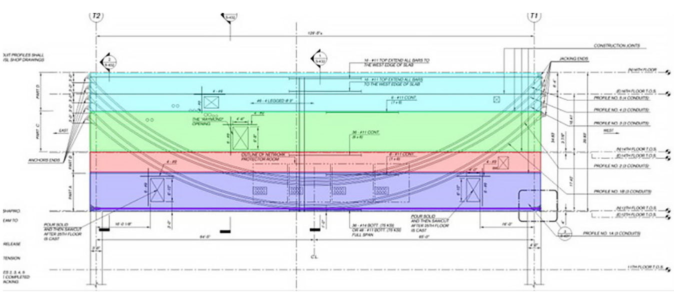

Fig. 4—PT girder elevation

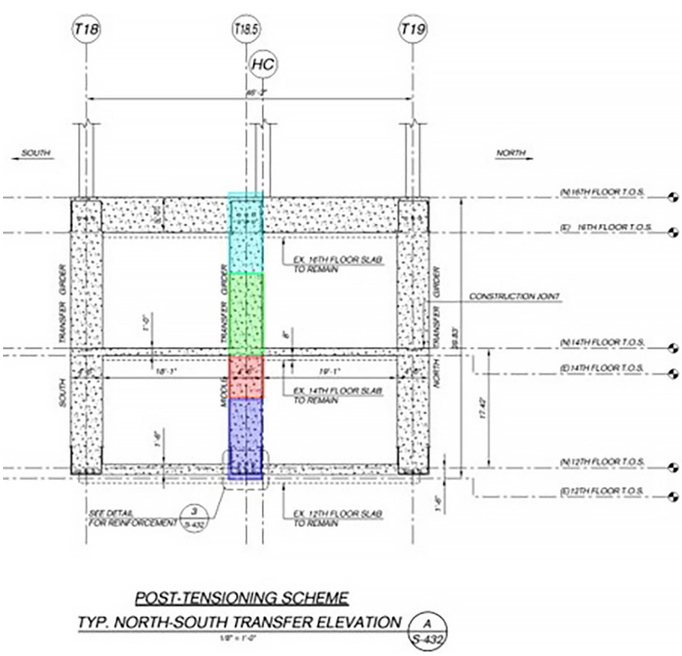

Fig. 5—PT girder section

The three PT "super girders" are supported by matching multistory reinforced concrete girders at their ends with the total load bearing on four laced "super columns." The new super columns supporting the new transfer system use the original steel super column that supported the old transfer system and are reinforced with high strength 80 ksi (552 MPa) reinforcing bars and a 14 ksi (97 MPa) concrete encasement.

As the existing foundations of two of these super columns were bearing directly below the existing cellar, these had to be jacked and re-supported to a lower bearing area with the use of permanent caissons. The final load in each super-column is approximately 25,000 kip (110,000 kN).

CONCERNS ABOUT ALL THINGS BEING LEVEL

There were significant concerns raised during the design and construction process about the potential deflections associated with the long span transfer. Techniques such as cambering the girders, and building areas of the floor high can be effective in helping resolve the permanent elevations of the building but they are challenging with high rise construction, as trades need to start working simultaneously from the bottom up, well before the full building is topped out. This means that the building, if constructed with a steel or conventional concrete transfer structure, would continue to deflect over the course of construction. This becomes problematic for floor finishes and façade installation because the levelness of the tower slabs continually change as weight is added to the finished support system from the ever-rising building above.

The phased PT transfer system eliminated most of these concerns. As the draped strands are tensioned, they induce an upward force on the girder which creates an upwards deflection. With the full 42 floors of the hotel structure being supported by the transfer system and based on the five staged PT, the maximum expected deflection was 1/4 in. (6.4 mm) downward and a positive (upward) induced deflection of 1/4 in. (6.4 mm) upward. The total expected movement for the entire system was calculated to be about 1/2 in. (13 mm) for the entire transfer span. Considering the clear span of 130 ft (39.4 m), the system deflects 1/3 120th of the span or 1/6 240th of the span when considering only the final downward deflection.

DESIGN PROCESS

The design process started with a simple hand drawn section and elevation of the PT girders. A spreadsheet was used to calculate the natural drape of the catenary and the points plotted to review geometric constraints and clashes. The concern of clashes was especially important at the mid-span bottom of the girder, and at the girder ends where the tendons terminate, and access was needed for jacking. Once the general geometry was set, each girder was idealized as a simple beam and equivalent moments and shears were calculated. The moments were balanced using simple calculations. Once the various conditions were checked, the girder was drafted to scale, and detailing began. After the "hand calculation," two analysis models were created using different software to provide "reassurance" and validate the concept. The modeling included a staged model in which the girder was built and tensioned stage by stage and stresses/deflections were stored for each subsequent step. The hand calculations for the stresses were remarkably close to the final results found by the complex analysis models (and took a fraction of the time). Finite element staged computer models were used to approximate the movements and deflections of the girders (Fig. 6).

The design of the PT transfer system involved coordination and feedback from many parties until the final design was developed. The initial design was developed by Severud Associates, the Engineer of Record (EOR) for the project. Pavarini-McGovern, the construction manager, arranged for a series of design charrettes that were held with various key players, who each used their expertise to help refine and develop the design. Howard I Shapiro and Associates, the Shoring engineers, consulted on the coordination of multiple shoring systems to ensure that the sequencing was feasible without overstressing the existing structure below. Methods for reducing the loading to structure, such as the early PT sequence for the middle girder, were proposed and integrated into the design. VSL/Structural Group, the specialty PT subcontractor, reviewed phasing of the construction and helped introduce methods to resolve congestion. For example, the initial design used tendons with a maximum of 31 strands per tendon. VSL proposed the use of up to 43 strands per tendon, with a total jacking force of up to 2,017 kip (8,972 kN) (Fig. 7 and 8). This meant that

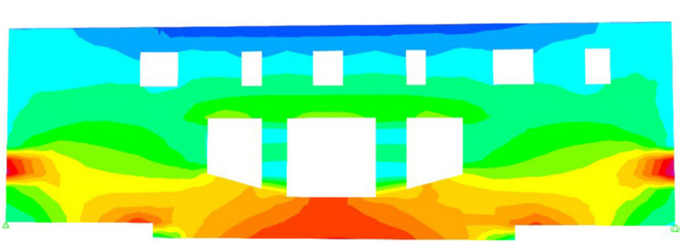

Fig. 6—Stresses in wall

Fig. 7—Jacking operations

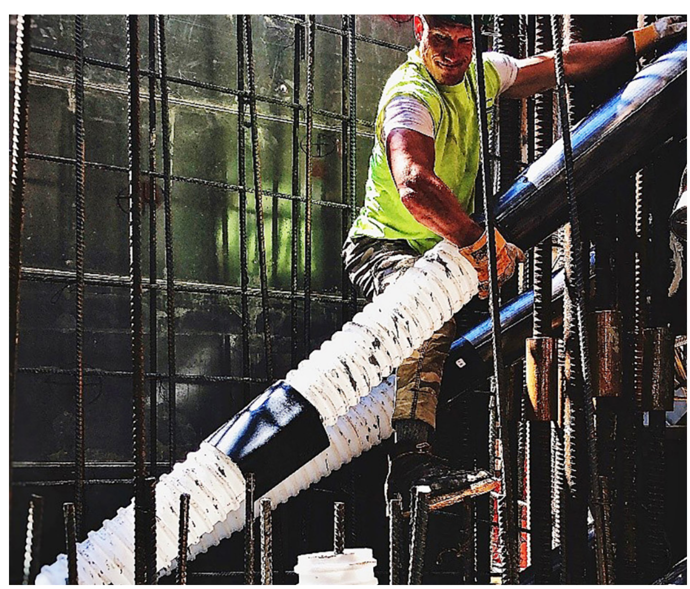

Fig. 8—PT conduits at end

special jacks needed to be brought in from overseas and had to be calibrated at Lehigh University's laboratory as few laboratories in the United States could handle jacks of this size and power. Sorbara, the concrete subcontractor, coordinated amongst their various suppliers and was critical in discussions related to the logistics of placing 14 ksi (97 MPa) concrete, 15 floors (185 ft [56 m] above street level) in the air on one of the busiest corners in the world.

THE NUMBERS

The final design of the PT girders consisted of the three girders each being tensed in five stages, with the center girder having half of the first stage tensioned early to resolve the shoring requirements as previously mentioned. The girders had two to four tendons per stage with each tendon having 31 or 43 strands each (Fig. 9). Therefore, each girder has approximately 500 strands.

Fig. 9—PT end fittings

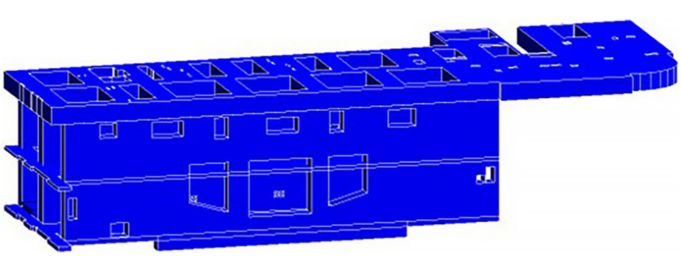

Fig. 10—Transfer structure

Each seven-wire strand is 0.6 in. (15 mm) in diameter with an area of 0.217 in.^{2} (140 mm^2). This generated a total jacking force of approximately 22,000 kip (97,860 kN) with a drape of approximately 40 ft (12.2 m), inducing an upward-induced moment of approximately 880,000 k-ft (1,193,100 kNm) (Note: numbers are approximate and do not include losses). The complexity of the staged tensioning required that the transfer girders be checked at the many combinations of the different concrete placement heights, and the induced forces each time the girder was tensioned, as areas would see both tension and compression at different stages.

ELECTRICAL VAULTS AND PENETRATIONS

As is often the case, especially in a New York City structure, no space in a building is left unused. As such, the space between the post-tensioned transfer beams was planned to be general mechanical space and also house the transformers and switchgear for the electrical service. The general mechanical space created the need for some louvers scattered within the elevation of the transfer structure. The transformers created a significantly more complex issue. The transformers required large louvers for their free air requirements, but also had minimum opening size requirements to allow for the easy replacement of the transformer should one fail. Conceptual rigging plans were created showing the pathway to bring up a replacement transformer to the 12th floor of the building and the openings in the transfer structure were sized to accommodate the unit replaced and also allow for the required free area for ventilation. This required extensive coordination with engineers from all disciplines and also required numerous meetings with the electrical supplier to coordinate this unique layout of openings. In addition, the PT system had to accommodate not only the effects of a catastrophic failure of the transformers but ensure that the system remained intact and maintained its full structural integrity since it was supporting a 42-level occupied space above it (Fig. 10).

MOCKUPS

There were many concerns related to the feasibility of the actual installation and placement. While 3D models of the concrete reinforcement were prepared for all areas of congestion, there was no room to allow for a problem to occur during construction. With a tight project schedule and the complexity of constructing these transfer girders on floors 12 through 16 of a building being simultaneously retained and demolished, everything had to fit correctly and effortlessly the very first time. Trucks mixing 14 ksi (97 MPa) concrete cannot be delayed or returned to the mixing plant. As such, a full-scale mockup was built for one portion of the PT girder where it was most congested.

The mockup was 8 ft (2.43 m) long and 8 ft (2.43 m) wide and the full depth of the actual girder (Fig. 11). The reinforcement bars and conduits representing the PT strands were placed in the mockup to match the actual design. One wall of the formwork was replaced with plexiglass so we could observe the flow of the concrete. As the concrete used was self-consolidating concrete, vibration was not required but verification of a smooth flow around all the reinforcement was critical. The mockup was built in a yard in New Jersey and the concrete truck arrived within 30 minutes of being batched. As the travel time to site can vary significantly based on traffic, the truck was held for additional time to simulate the actual expected field conditions and delays in Times Square. A total of 18 Hilti Concrete Sensors were placed in a pre-planned grid pattern throughout the mockup to monitor concrete temperatures during the placement. A warm summer day was chosen to represent the expected New York summer weather.

The results showed that the congestion and workability of the concrete achieved the design intent. However, concrete temperatures were borderline and approached allowable limits. During the mockup, concrete temperature reached 163°F (72.8°C), exceeding the allowable 160°F (71.1° C) threshold. Sensors showed that the temperature gradient within the girder cross-section also exceeded the upper bound. The concrete mixture already used 40 percent cement replacement and had concrete additives to slow the heat of hydration.

Fig. 11—PT girder mockup

To further reduce the concrete temperature, the use of liquid nitrogen to cool the aggregate was tested on a concrete sample. It lowered the temperatures of the concrete mixture by approximately 10°F (5.6°C) thereby meeting design requirements. This mixture was then used in several concrete placements for elements of similar thicknesses to the PT girders so that additional temperature monitoring could be done (Fig. 12 and 13).

MONITORING

Due to the dimensions of the PT girder system, it was classified as mass concrete and care needed to be taken to ensure that the overall and relative temperatures stayed within allowable limits. More than 150 Hilti Concrete Sensors were installed throughout the girders and thick slabs to allow for real time monitoring of the temperatures within the girder via Bluetooth connection to the sensors.

CONSTRUCTION

In general, the pre-planning and mockup proved to be extremely helpful in resolving issues in advance. The PT conduits, reinforcing bar, embedments, and so on, all

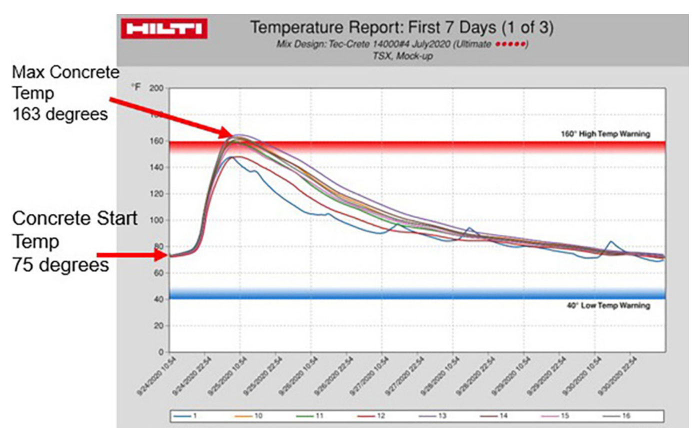

Fig. 12—Concrete temperatures before nitrogen

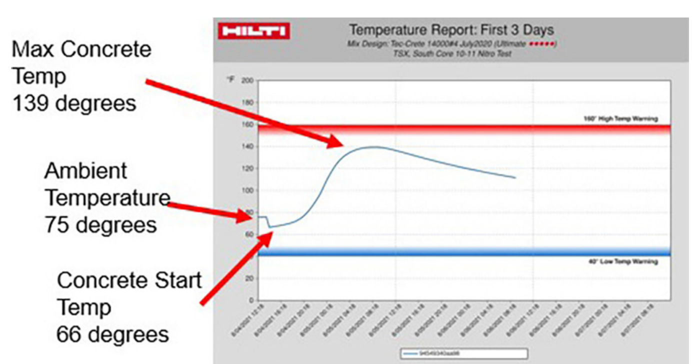

Fig. 13—Concrete temperatures with nitrogen

Fig. 14—PT girder construction

fit with minimal issues and adequate workability (Fig. 14 and 15). There were some issues encountered during construction which the team had to resolve.

CONSTRUCTION ISSUES—TEMPERATURES

Based on the original project schedule and planning, the placement of concrete for the PT transfer system should have occurred in the winter. However, due to Covid-related delays, the actual placements occurred in the summer months. While the mockup was done on a hot day, the actual pour day for one of the concrete lifts occurred on a record-breaking high temperature set of days for New York City. Despite all the planning and preparation, some concrete in local zones had larger differential temperatures than specified and the design team chose to address the issue rather than consider it an anomaly. A series of cores were taken from this area and subjected to a testing program to ensure concrete integrity. At the completion of the testing, the capacity of this concrete was determined, and a protective coating was added to the face of the concrete to prevent any potential water infiltration in the future, which could potentially damage the concrete.

CONSTRUCTION ISSUES—REMOVAL OF SHORING BELOW

As the existing structure was used to support the wet weight of concrete from the new transfer system, it had to hold substantial loading from the transfer structure.

Fig. 15—PT girder overall

During the demolition of the existing structure below after the new transfer was installed, the existing structure was found to have stresses in it from the shoring loads imposed. Because of the way that the shoring was installed, they could not simply lower the shoring and back it off. To alleviate the load on the shoring below, for one of the stages, the tendons were stressed a few floors early to "lift" the PT girders off the shoring below. While tensioning early was a simple fix, it was not a simple design decision to make. The girders were being monitored regularly for deflection and the building was being monitored locally for deflection and elevation changes due to shortening, creep and shrinkage. All this data had to be reviewed and assessed to correlate the movements to date and ensure that the early tensioning would relieve the stress from below without overstressing the top of the girders in tension. After careful analysis by the EOR, the decision was made to tense the PT girders early for one of the stages. The net result was that the tensioning process relieved the loads on the shoring below and there were no issues with overstressing the top of the girder, a simple and effective solution to the issue.

RESULTS—DEFLECTIONS

The results of the PT girder system deflections during construction showed that the settlement of the middle of the box girders was found to be less than 0.25 in. (6.4 mm) for the span. This meant that there were no issues in the project related to floor leveling or facade installation issues due to the transfer system. The construction teams were extremely pleased (even surprised) at the ability of the transfer system to minimize movement, shortening "finishing" time and making floor placement in the hotel easier. The transfer girders would "lift" approximately 1/8 in. (3.2 mm) after being tensioned and then continue to deflect until they were tensioned again. When comparing the analysis model to the measured deflections, the overall total deflection observed was slightly less than the total expected based on the analysis model. The amount that the transfer system "lifted" based on the tensioning was also slightly less than the analysis model predicted. Both of these observations are likely due to fixity within the system that was not included in the analysis model. Overall, the modeling results closely matched the measured movements in the field.

CONCLUSIONS

The introduction of the PT transfer girder system for this project, though highly unconventional, was extremely beneficial to the overall project from a cost, schedule and performance perspective. This structural system was able to save months of time from the schedule for the installation of these transfer floors and the minimal deflections associated with the system saved significant time and money for the project. The observed movements of the transfer systems closely matched the analysis models. The flexibility of using a concrete system in terms of mechanical, electrical, and plumbing (MEP) coordination was also valuable to the project.

What began as a one-off transfer system idea has now become our "go-to" solution for high-rise towers requiring column-free podiums.

ACKNOWLEDGEMENTS

We would like to acknowledge the contributions of Severud team members Steve Reichwein, Dr. Mohamed Arafa and Meghan Krupka who were instrumental in the design of the project. We would also like to acknowledge the contributions of Pavarini McGovern (CM), Mancini Duffy (AOR), Sorbara (Concrete Subcontractor), and VSL/Stuctural Technologies (PT Specialty Subcontractor), in addition to the many other subcontractors and engineers who contributed to this project.

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