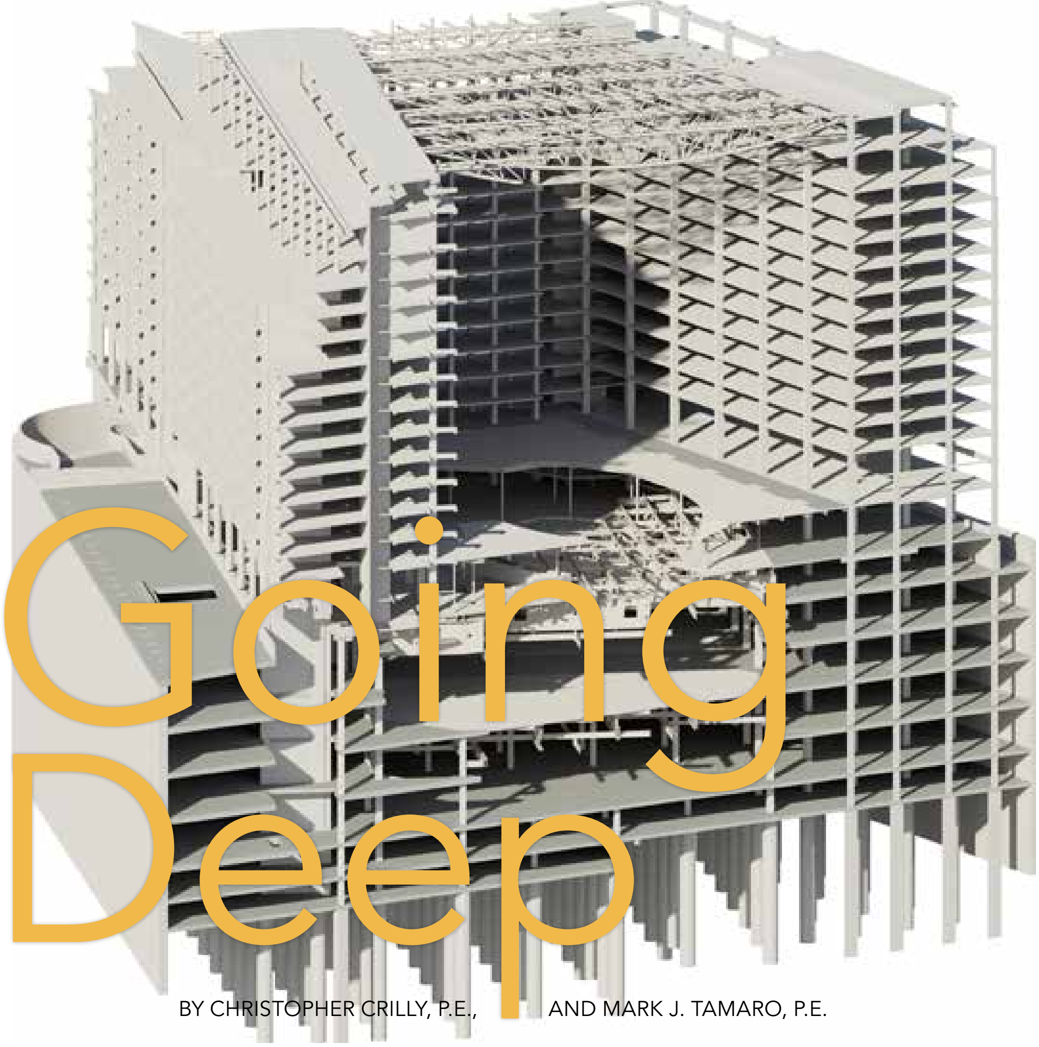

Going Deep

BY CHRISTOPHER CRILLY, P.E., AND MARK J. TAMARO, P.E.

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Top-down construction, supported by steel, facilitates Washington, D.C.'s deepest building.

THE NEW WASHINGTON Marriott Marquis hotel in Washington, D.C., has been a long time coming.

Shortly after the opening of the Walter E. Washington Convention Center in 2003, planning began for the construction of an adjacent hotel, with the two buildings sharing an underground loading dock. But it wasn't until five years later that the current site was finalized and building design commenced.

Once completed, the four-star hotel will feature 1,175 rooms, a 30,000-sq.-ft grand ballroom, two 10,800-sq.-ft junior ballrooms and 53,000 sq. ft of meeting space. The design team resorted to using unconventional design solutions in order to fit the full building program on the available footprint and within the city's building height restrictions; it shares a triangular-shaped block with the historic AFL-CIO headquarters building and the Pepco electrical utility substation building. The resulting design called for extending the construction 100 ft below grade, which is deeper than any other building in the city. While the 15-story building is primarily framed with concrete, its seven below-grade levels relied on more than 5,100 tons of structural steel to facilitate the top-down construction method.

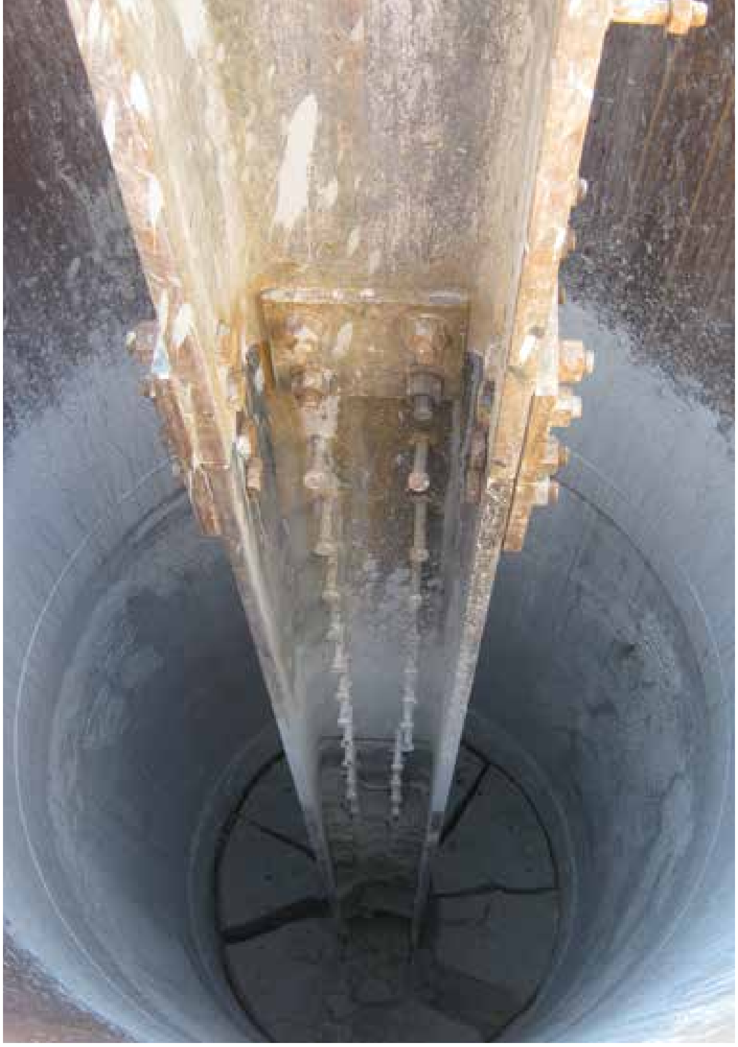
▲ A Revit model of the building.

Top-Down

The tower structure is polygon shaped and surrounds a full-height atrium that is topped by a skylight half an acre in area and supported by long-span steel trusses. The structure below grade consists of 18-in. and 24–in. thick concrete flat plate slabs with mild steel reinforcing. The slab thicknesses were governed by the in-plane shear stresses caused by the lateral earth pressures. At certain critically stressed locations, the concrete alone had insufficient capacity. Here, wide-flange structural members and steel plates were cast into the slabs, which are supported by concrete-encased composite steel columns. The steel column sections were embedded in 5-ft to 7-ft diameter concrete drilled shaft foundations extending to various elevations within disintegrated, weathered and solid bedrock.

Traditional tiebacks could not be used because of the hotel's proximity to the Pepco building, AFL-CIO building and the underground convention center loading dock. Rakers and cross-lot bracing were also not feasible. The size and frequency of steel required for either of these systems would prohibit construction activities within the site. On top of that, the design also had to resist the very high soil and hydrostatic forces due to the depth of excavation.

After careful consideration of all support of excavation systems, the team opted for top-down construction with below-grade floor



▲ Wide-flange and plate steel slab reinforcement.

▲ Typical below-grade top-down steel column.

slabs serving to brace the perimeter slurry wall. This construction method requires the below-grade columns to be installed first, and steel was the best choice for the columns since they could be spliced together on-site, tied to the rebar cage for the drilled shafts, lowered to the shaft bearing elevation and held in place while the concrete for the drilled shaft was placed by tremie method under a bentonite slurry. The steel columns were encased after successive slabs were cast and excavation below progressed downward, and were optimized to support only the construction loads and building self-weight expected to be in place prior to encasement. The composite section was then designed for the full building design loading, which occurred in the final condition. This reduced the tonnage of the top-down columns by 100 tons.

Below-Grade Ballrooms

All of the ballrooms are below grade, which created challenges that could only be solved with structural steel. The two-story-deep grand ballroom is stacked on top of the two-story-deep junior ballrooms, and the deep volume of the ballrooms created significant openings through the below-grade diaphragms. These large openings, combined with several others required for vertical transportation and mechanical shafts, left little structure to resist the in-plane diaphragm forces caused by the high soil and hydrostatic forces. In many areas, there is only 10 ft to 15 ft of slab between openings or openings and the perimeter slurry walls.

Thornton Tomasetti performed extensive non-linear finite element analysis to determine the in-plane axial, flexural and shear stresses at hundreds of sections throughout each of the below-grade levels. In many locations, concrete and conventional mild steel reinforcement was inadequate to resist the design forces, and making the slab thicker than 24 in. was not practical. At sections where the

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Steel plate girder and grand ballroom steel truss floor framing.

11-ft-deep ground floor transfer girder.

Erection of the first plate girder prior to the beginning of top-down construction.

shear stress exceeded the ACI limit of $10\sqrt{f'_c}$, steel plates ranging in thickness from ½ in. to 2 in. were added to resist the design level shear. Load transfer to the plates was accomplished through the use of headed studs embedded in the slab. At areas of high axial and flexural stresses, W14 wide-flanges were embedded in the slab to act as compression and tension reinforcement. These members were enclosed by mild steel reinforcement and lined with headed studs for confinement and development.

The large open ballroom spaces also created the need for long-span framing to support the floors above and below. Again, structural steel was the ideal solution. The two 10,800-sq.-ft junior ballrooms are located on either side of a 27-ft-wide grand hallway, which allows for two rows of columns to support the floor of the 30,000-sq.-ft grand ballroom directly above. The grand ballroom's long-span floor is created by the column-free junior ballrooms on either side of the grand hallway below. This floor area was framed by 9-ft-deep steel trusses at 27 ft. on center spanning 80 ft in the opposite direction with steel beam and slab on metal deck infill. Not only did this infill framing need to support ballroom loading, but it also needed to be stiff enough to limit vibrations from music or dancing and support operable partitions for the junior ballrooms below.

The 225-ft by 135-ft grand ballroom is situated partially under the main lobby and atrium at level one and partially under the 15-story guest room tower along the east, west and south sides. The framing supporting the structure over the grand ballroom consisted of eight built-up steel plate girders, spaced at 27 ft, spanning 135 ft and infilled with composite steel beams and slab on metal deck. The original framing design consisted of heavy steel trusses in lieu of the plate girders. Through the design-build process, Canam-Structal (the steel fabricator) suggested replacing the trusses with the built-up plate girders to reduce cost and increase constructibility. The girders along the east and west sides support four equally spaced tower columns, along with multiple smaller plate girders also transferring tower columns. These end girders were 11 ft deep with 4-in. by 68-in. flanges and 2-in.-thick webs. The six girders in between support a single tower column 17 ft, 6 in. from the south end. The center girders were 10 ft, 4 in. to 10 ft, 6 in. deep, with flanges ranging from 2 in. by 34 in. to 3 in. by 42 in. and 1.5-in.-thick webs. Along with the steel framing at the ground level above the grand ballroom, most of the remaining ground floor is also framed with structural steel to accommodate complex geometry and large slab steps as well as to support a number of smaller transfer columns. In addition to the eight large girders spanning the grand ballroom, there were approximately 35 additional built-up steel plate girders that ranged from 3 ft deep to 6 ft deep, with spans that extend anywhere from a few feet to 30 ft transferring other tower columns located outside the grand ballroom footprint.

Controlling Deflection

The most complex structural challenge created by the ground-floor framing over the grand ballroom was controlling deflection to limit settlement and tilt of the tower floors that it supports. It was not economical or logistically feasible to design these girders to limit their deflection under tower dead load to an acceptable limit. The design build team came up with two unique solutions to control the tower settlement as construction progressed. The first scheme, which applied to the end girders on the east and west sides, was to preload the girders and pre-deflect them to the anticipated deflection caused by the self-weight of the tower prior to beginning construction of the superstructure. This was accomplished by us-

The congested site in the middle of D.C. included tower cranes, drilled shaft rigs, excavation rigs and crawler cranes.

Ground floor transfer girders above, ballroom trusses below.

Looking down into the ground-floor atrium.

ing hydraulic jacks and tension rods and jacking against four steel columns directly below the girders, which were embedded in concrete caissons socketed into bedrock. After jacking, the tension rods were locked off and the jacks removed. As the construction of the superstructure progresses and the girders are loaded, the stresses in the tension rods are relieved. At completion of the superstructure, the stresses will be minimal, at which point they can be removed. The columns used as reaction to the jacks are not necessary in the ballroom, but they are required to support the lower levels and were installed as part of the top-down construction. Therefore, they were well suited to use as reaction elements and could be removed from the ballroom once the jacking sequence was complete.

The second scheme for tower settlement control applied to the middle girders, which also support a single line of columns along the south end. At these locations, preloading the plate girders was not feasible because columns did not exist directly below the girders. Instead a scheme was developed that included building steel jacking shoes with tapered shims that would sit on top of the plate girder and directly underneath the tower columns. As each level of superstructure was constructed, the plate girders and floors deflected a small amount. When incremental settlement of the second floor reached a predetermined limit of ⅛ in., the columns were jacked upwards until the second floor was back to design elevation. Each floor above the second floor was constructed to the design story height rather than design elevation so that only the second floor elevation needed to be monitored. To help limit stresses induced on the flat plates due to column settlement, a hinge was detailed into the slab at the theoretical inflection point near the next back-span column. The final column jacking scheme design and execution was a complete team effort between the contractor and design team, with significant input from LPR Construction, the project's steel erector.

Although predominantly a concrete structure, the Washington Marriott Marquis would not have been possible without the use of a significant quantity of structural steel. The hotel is scheduled to open next spring. MSC

Owner
Quadrangle Development Corporation and Capstone Development

Operator
Marriott International, Inc.

Design-Build Contractor
Hensel Phelps Construction Co.

Design Architect
TVS Design

Architect of Record
Cooper Carry

Structural Engineers
Thornton Tomasetti/A+F Engineers

Steel Team

Fabricator and Detailer
Structal, a division of Canam Steel Corporation, Point of Rocks, Md. (AISC/NSBA Member/AISC Certified Fabricator)

Erector
LPR Construction Company, Loveland, Colo.
(AISC Member/AISC Advanced Certified Steel Erector)