HANGING OUT in Salt Lake City

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An elaborate erection and jacking scheme involving an innovative roof-top truss helps pull together the Utah capital's newest Class A high-rise.

FROM THE OUTSIDE—and from three directions—111 Main might look like any other high-rise.

But from a steel design and construction perspective, the project was on a whole new level when it came to erection challenges. For one thing, the 25-story, 502,000-sq.-ft Class A office tower, located in the heart of downtown Salt Lake City, extends 46 ft over an adjacent structure. All 18 perimeter columns are hung from roof trusses framed with jumbo W14 shapes (with heavy node weldments) that rest on six structural spherical bearings at the top of the concrete core, a new scenario for the erection engineer and steel erector.

Tightening Up

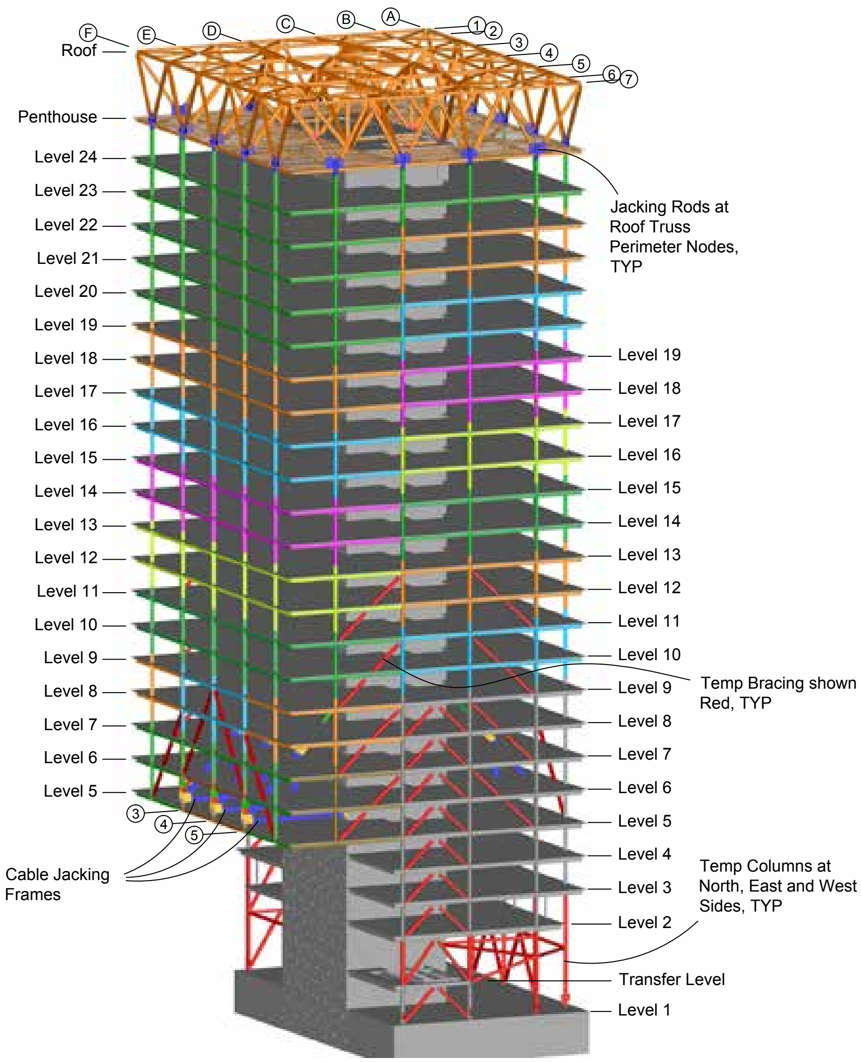
Temporary columns were the obvious solution for the north, east and west sides, but the south side required a different solution—one that would allow steel erection to start at level 5, which would then serve as the support base for the floors above.

The design team at architect and structural engineer Skidmore, Owings and Merrill, LLP (SOM) envisioned a "saddle cable system" that would employ cables through the core and act to balance the placement of construction loads while eliminating any eccentricity on the core that would affect the stress, strain and creep of the reinforced concrete core walls. The challenge for the steel erection team was to come up with a practical solution of framing and cable connections that allowed the tensioning of the cables to account for sag and stretch, as the cable loads increased while the building was being erected. Working with erector SME, erection engineer Hassett Engineering developed a steel-framed concept that minimized the cable length, thereby minimizing cable sag and stretch.

The temporary framing—referred to as "jacking trusses"—was designed to jack up the building columns while simultaneously

The south side of the building cantilevers 46 ft over an adjacent structure.

All images courtesy of Hassett Engineering unless otherwise noted.



The temporary truss scheme.

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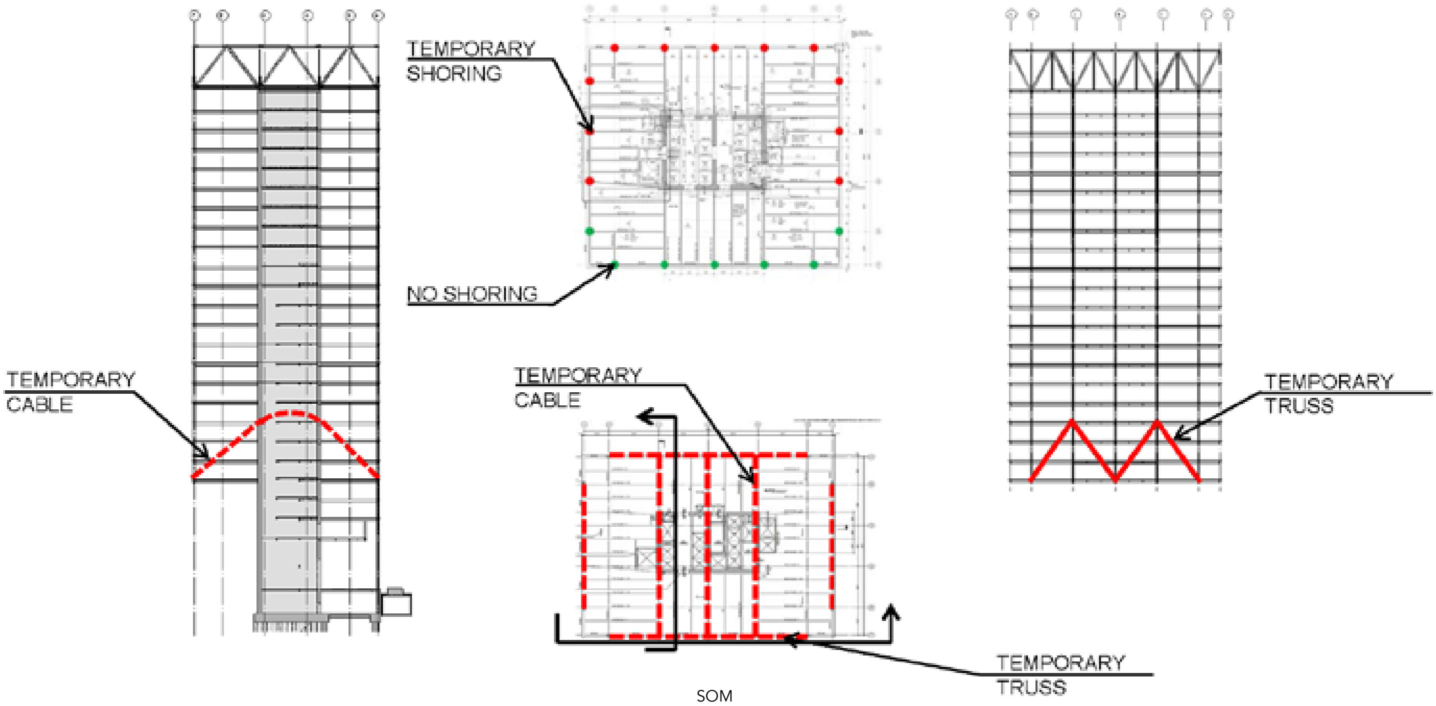


The construction sequence of the saddle cable system.

tensioning the cables. Column lines 3, 4 and 5 would be framed with jacking trusses and the saddle cables supporting the south side, and anchored with similar framing on the north side to columns that extended down to the foundation.

After extensive review of available cable types, 4-in.- diameter ASTM A586-04 were chosen. The cables were pre-stretched to provide an effective modulus of elasticity of 23,000 ksi and rated with a breaking strength of 997 tons (which translated to an allowable capacity for construction loading of 997 kips). There were two cables per grid line for a total of six, and they were installed through the 30-in.-thick concrete core walls on a circular radius in a lubricated steel conduit. Column lines 2 and 6 were coupled to lines 3, 4 and 5 through temporary bracing on the south face of the building between levels 5 and 9, and were consequently supported by the jacking trusses as well. Lines 1 and 7 had a different problem: There was no core to anchor through, so a braced frame was designed to carry the hanging and the associated lateral component.

Transferring load from the temporary shoring system to the roof hat truss once the structure was complete was another challenge. The initial concept was to pull up the perimeter columns using jacks at the hat truss nodes. Through collaborative meetings between SOM, SME and Hassett Engineering, this scheme was changed to a "lowering" of the columns at the base. The solution allowed gravity do the work as opposed to pulling up, fighting against gravity and the stiffness of the building itself. With this scheme, the perimeter of the building needed to be erected to a higher point than the final theoretical elevation. Since the core does not move during load transfer, the slab and floor beams would rotate about the core walls. Concrete pour strips were implemented adjacent to the core, and special pinned beam connections were developed to allow rotation without adding extra stresses into the system.

As is typical for a building of this type, the permanent columns are designed largest at the top where the tension is the highest, and smaller at the bottom. During construction, however, the columns were supported from below and saw large compressive forces most notably in the lower columns. Through collaboration with SOM, the columns were checked and resized as necessary to handle the temporary construction loads.

Tier by Tier

Erection of the composite steel construction followed a typical sequence of two stories at a time, tier by tier, and concrete slabs were poured via normal sequencing four floors (give or take) behind the erection. The column lift (via jacking) was estimated to require ⅜ in. at every tier, accounting for deflection due to two slabs and two floors of steel. The jacking frame was designed to deflect by rotating about a pin connection at the core. The two 500-ton jacks at each column were designed to push down on the jacking frame while pushing up on brackets welded to the flanges of the columns. This would provide the elevation adjustment for the cantilevered south side throughout the project. Loads were measured on the jacks and compared to the theoretical loads, which were calculated using an ETABS model.

However, after a few tiers were erected, the jack readings were showing higher loads than expected. Unable to capture

➤ The cable jacking frames and building core.

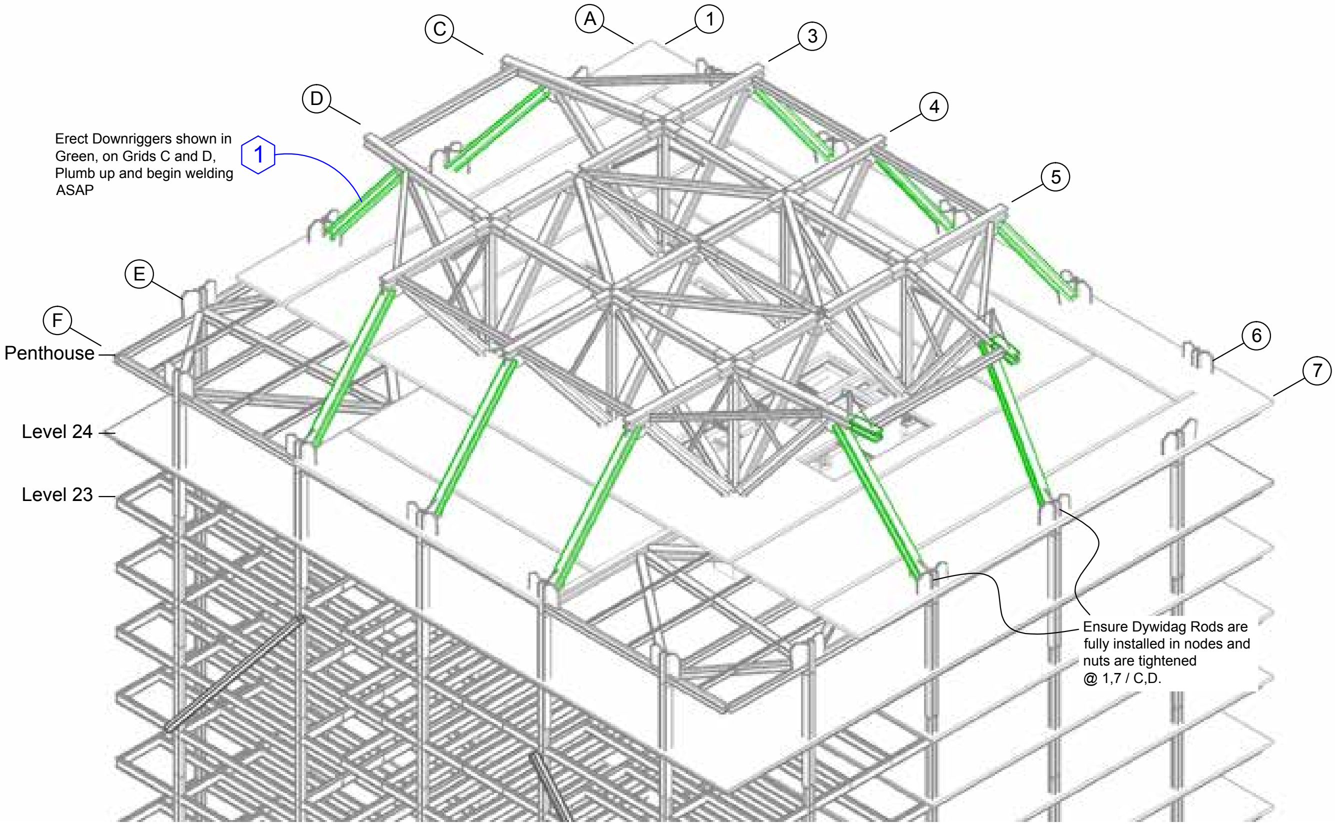
the behavior in the ETABS model, the team determined that some load transfer (by plate action of the slabs) was occurring between jacking lines 3, 4 and 5 and braced frame lines 1 and 7. Connections for the jacking trusses and the bracing for lines 1 and 7 were reinforced to accommodate the additional loading, and the jacking magnitude at each adjustment was minimized to keep the floor elevations within acceptable tolerances while ensuring that the jacking frames would not be overstressed.

As with any high-rise project, the team had to pay close attention to the "differential shortening" or compression strains of the perimeter columns relative to the core structure. But in the erection scheme for this project, the columns were in compression, then finally in tension, and the floors lost elevation due to the deflections of the roof truss. Anticipating this in the design phase, the team performed analysis to determine what, if any, column length changes would be needed. Column lengths per tier would be required to be ⅛ in. shorter than theoretical. However, SME's field experience told us that there would be approximately ⅛ in. of weld shrinkage at the column splices. Therefore, the columns were detailed to their theoretical lengths, thus eliminating complexity in the modeling for the column shop drawings.

Since the columns would compress during shored construction, then stretch and deflect (due to hat truss deformation) after hanging, a specific, unique "set-high" elevation was required at each column of each floor that would vary at any given stage of construction. The erection plan also required the bottom level of each column line erected to an initial set-high elevation, varying between 3 in. and 4.5 in., to account for deformation of the temporary steel during construction, final column stretching and final hat truss deflection.

Similarly, the perimeter nodes of the roof hat truss were erected to a "tip-up" elevation between 1.5 in. to 2 in. to account for the final truss deflection alone. To complicate matters, the schedule required the curtain wall glass panels to be installed during steel erection, prior to jacking down the building and transferring the building weight from the shoring to the roof truss. The glass panels were designed to allow greater than normal tolerances due to movement after installation. As such, general contractor Okland Construction performed extensive survey monitoring on a weekly basis so that fine adjustments to the preset elevations of each column could be made in order to ensure the integrity of the panels. The target maximum differential vertical movement between adjacent columns was +/-½ in.

➤ A jacking frame and column bracket.



The main axial members of the hat truss (in green).

Making Adjusments

Erection proceeded quickly up to level 24, with the south side column elevation adjustments made close to the theoretical estimate of ⅜ in. per tier. As the stretch of the cable continued, it would slip through the core, occasionally breaking friction with a noticeable bang. As the roof truss began erection with the heavy members and nodes, the concept was to start at the core, landing the nodes on the structural spherical bearings and work out towards the four perimeter column lines. This procedure allowed welding to start at the core and work outward, thus allowing relatively symmetrical and unrestrained weld shrinkage. In addition, the main axial members of the trusses could soon be able to support their own weight, carrying load from the perimeter toward the core.

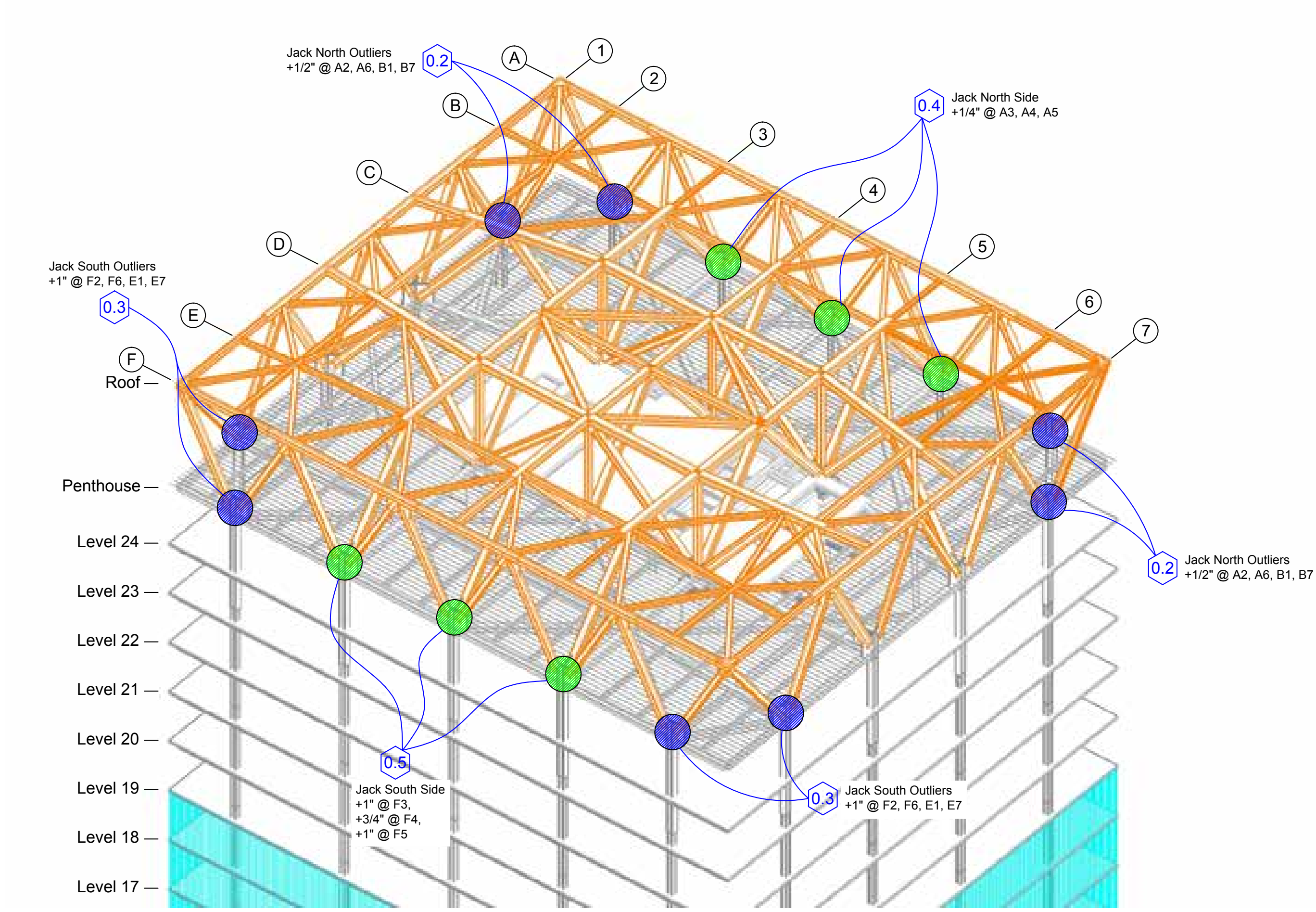
Jacking rods were installed on the bottom chord of the roof truss as a backup plan to make further elevation adjustments, which would involve pulling up the columns and pulling down the trusses. This backup scheme proved to be very useful in adjusting the floor elevations, as the floor elevations at the south side and outlier columns were lower relative to the other grid lines. These "outlier columns" were columns outside of the main truss lines and were supported by perimeter trusses off the main trusses at the roof. Because the measured loads during jacking indicated that more jacking would stress the saddle cables beyond their intended design loads, the decision was made to leave those three columns relatively low.

Relying on the carefully developed survey data provided by Okand Construction, and correlations made with ETABS erection sequence modeling, adjustments at the roof were done in four phases:

1. The four outliers on the north were jacked ½ in.; these column grids then could provide some pretension for the supporting hat truss, acting as an abutment
2. The four outliers on the south were jacked 1 in.
3. The columns on line A at 3, 4 and 5 were jacked ¼ in. to provide further pretension for the next jacking.
4. The columns on line F at 3, 4 and 5 were jacked 1 in., ¾ in. and 1 in., respectively. By making these small final adjustments at the roof level, the roof trusses were effectively preloaded and the erection team was able to set the geometry closer to the theoretical elevations prior to jacking down the building

Once the columns were welded to the nodes at the main truss lines (3, 4, 5, C and D) and the rods were engaged at the outlier columns, the building was ready to be lowered into position. These outlier columns were expected to deflect more than the main truss line columns. Therefore, they were left un-welded to the nodes so that they could be jacked later for a final floor elevation adjustment at those grids.

Once the roof welding and inspections were complete, the building was ready to be jacked down and the load transferred to



Pre-jacking adjustments to the hat truss.

the two-way roof "hat" truss system. A hydraulic control system was placed on level 5 at the south (line F) to control the six jacks at the three jacking frames, and another hydraulic control system was placed at level 1 (ground level) to control the 22 jacks and 11 columns. As the jacking-down process proceeded, surveyors were placed at level 5 and at the roof and were in radio contact with Okland's superintendent. SME, Hassett Engineering and SOM had continuous communication between the ground and level 5 jacking, giving the go-ahead for each simultaneous jacking-down of ⅛-in. increments. The glass curtain walls and MEP systems were being monitored by their respective subcontractors as well. As the load was transferred to the hat truss and the load on the temporary cables was reduced, cable slippage back through the core was observed as the cables occasionally broke friction with noticeable bangs, as with initial erection.

Winding Down

Jacking-down was performed in small increments until an accumulation of 1 in. at the ground and ¾ in. at level 5 was achieved. Column elevations and jacking loads were then recorded, reported to the team and reviewed; upon approval, jacking-down would recommence. A three-to-four ratio was used since previous fine adjustments at the roof jacking rods had preloaded the south side more than the north, resulting in less required jacking-down at the south for full load transfer. Furthermore, in order to achieve the required ¾ in. of jacking-down at level 5, a total jack movement of 3.5 in. was required. This was due to the jacking trusses moving upward (since they were unloaded) at the same time the building was moving downward. After each 1-in. lowering step, jacking forces and survey elevations at levels 5, 15 and 24 were reported.

Elevations and loads were compared to theoretical predictions, and the curtain wall deflections were checked to be within tolerance before proceeding to the next step. This process was repeated until the jacks were unloaded and the building was fully supported by the hat truss above. Total jacking-down at the columns on the ground varied between 3 in. and 4 in., and total jacking at level 5 was about 8 in. for a column movement of approximately 1.7 in. Cable movement through the core was between 3 in. and 3.4 in.

Further adjustments to column elevations were inevitable since the floors would continue to deflect due to the pouring of the level 25 concrete slab, the removal of the temporary braces and the completion of the façade and remaining dead load. The worst predicted deflection locations were at the outlier columns, as they were supported by the more flexible parts of the hat truss. After load transfer, about ¼ in. of jacking was performed at the roof level on the four outlier columns on lines A and F, north and south, respectively. Removing the temporary braces or releasing the connections caused a redistribution of forces throughout the building and resulted in further deflection, predominantly at the outlier columns on the south, where the braces were retaining

A tension release at a braced end-plate connection.

much residual tension load. This was done by loosening the tension rod nuts in the end-plate connections of the braces, with a resulting gap spread of between $\frac{1}{16}$ in. and $\frac{3}{16}$ in. between end plates.

Finalizing the roof slab pour only added to the deflections. In response, and in anticipation of the remaining façade load and other dead loads, a final adjustment of 0.4 in. was performed at the two south outliers on line F in order to bring the floor elevations closer to theoretical. These outlier columns were then welded off, essentially securing the building into its final state and intended structural system load path. The façade, having survived several changes in elevation and geometry, was given a final adjustment.

The steel structure of 111 Main was built from the ground up and went from being supported at the bottom levels to hanging from the hat truss above—basically $\frac{1}{8}$ in. at a time—proving that steel was ideal not only as a framing system, but also in terms of constructability for an especially intricate erection operation. ■

Owner
City Creek Reserve, Inc., Salt Lake City

General Contractor
Okland Construction Company, Salt Lake City

Architect and Structural Engineer
Skidmore, Owings and Merrill, LLP, San Francisco

Erection Engineer
Hassett Engineering, Inc., Castro Valley, Calif.

Steel Fabricator, Erector and Detailer
SME Steel Contractors, Inc., AISC CERTIFIED FABRICATOR
West Jordan, Utah

Jacking adjustments at the roof node.