Solutions to Structural Distress in the South Tower of the Milwaukee City Hall

Part 2

By
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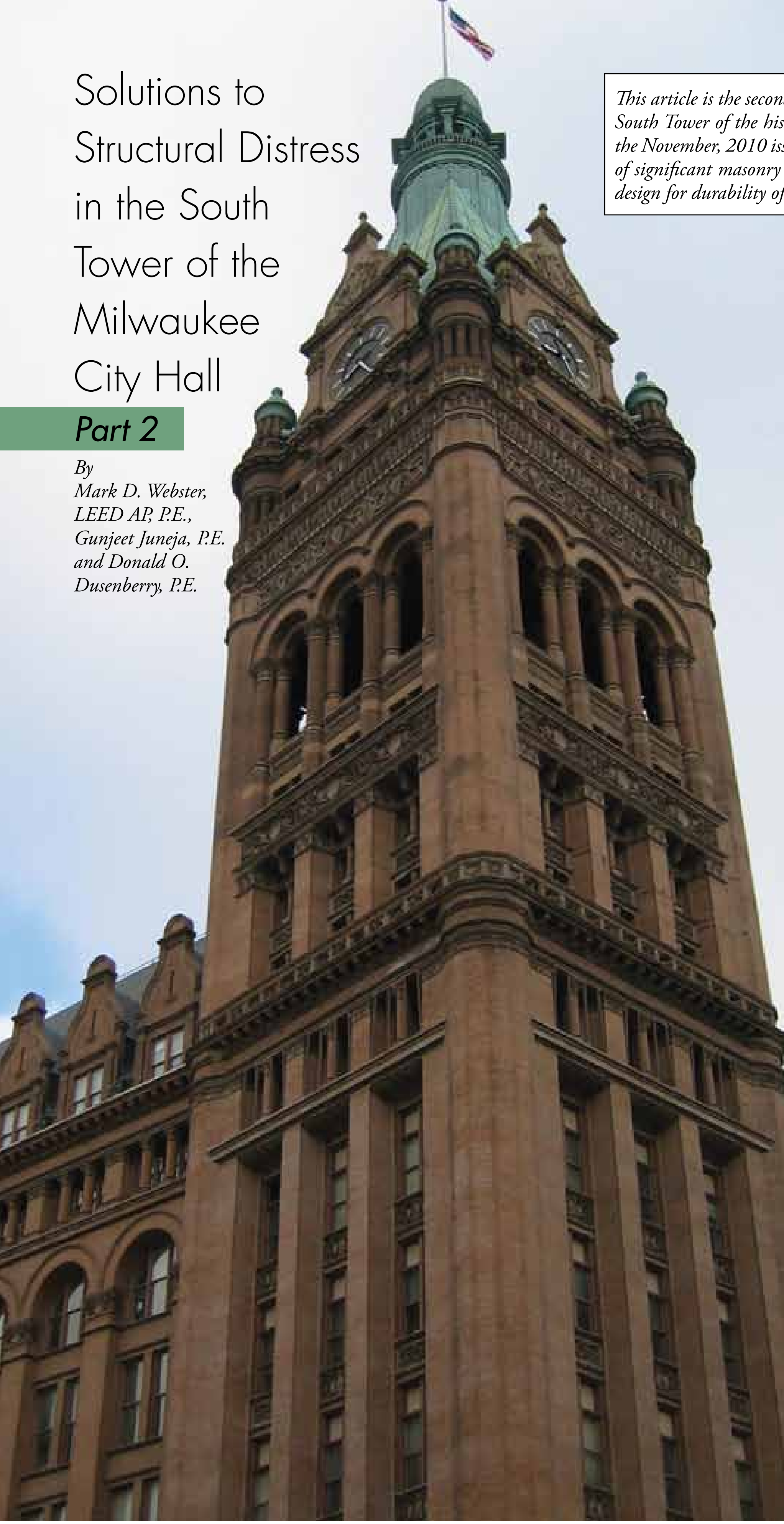

Figure 1: Milwaukee City Hall's South Tower.

This article is the second of a three-part series on the rehabilitation of the South Tower of the historic Milwaukee City Hall. Part 1, published in the November, 2010 issue of STRUCTURE®, addressed the investigation of significant masonry cracking in the structure. Part 3 will discuss the design for durability of the reconstructed masonry.

After the completion of the investigation outlined in Part 1 of this three-part series, the City of Milwaukee elected to proceed with design for the repairs of the South Tower of Milwaukee City Hall *(Figure 1)*. National engineering firm Simpson Gumpertz & Heger Inc. (SGH) teamed with architect Engberg Anderson and engineer Bloom Companies, LLC for the repair design. The City of Milwaukee quickly established a goal for the repairs to last 100 years, with regular maintenance for wear and tear of the materials. The City awarded the construction contract to J.P. Cullen & Sons, Inc. of Janesville, Wisconsin.

Finite Element Analysis

SGH performed finite element (FE) analyses to determine the most likely causes of the observed cracking patterns in the tower and to help design the structural repairs. These analyses included global models evaluating the tower steel structure and the masonry walls, and component models to study the effect of brick pointing on a wall section and the performance of critical masonry piers.

Steel Structure FE Model

The interior steel structure (the core truss, *Figure 2*) of the tower extends from the 10th floor to the lantern at the apex of the roof, 86 feet above the 13th floor. The truss was analyzed using a two-dimensional model representing one plane of the three-dimensional trusswork. This model calculated the load distribution through the truss under the weight of the roof and wind loads acting on the roof, and provided the reactions of the steel structure on the masonry for the design of masonry rehabilitation.

With the exception of the 13th-floor plate girders that span between the core truss and the masonry walls and the steel columns immediately above, all members of the core truss were modeled to carry only tension and compression. The plate girder members were modeled as beam elements to capture the important bending behavior of the girders and the effects of the eccentricity of the connection of the diagonal truss members and the plate girders.

The steel structure is supported on two steel trusses spanning diagonally across the tower

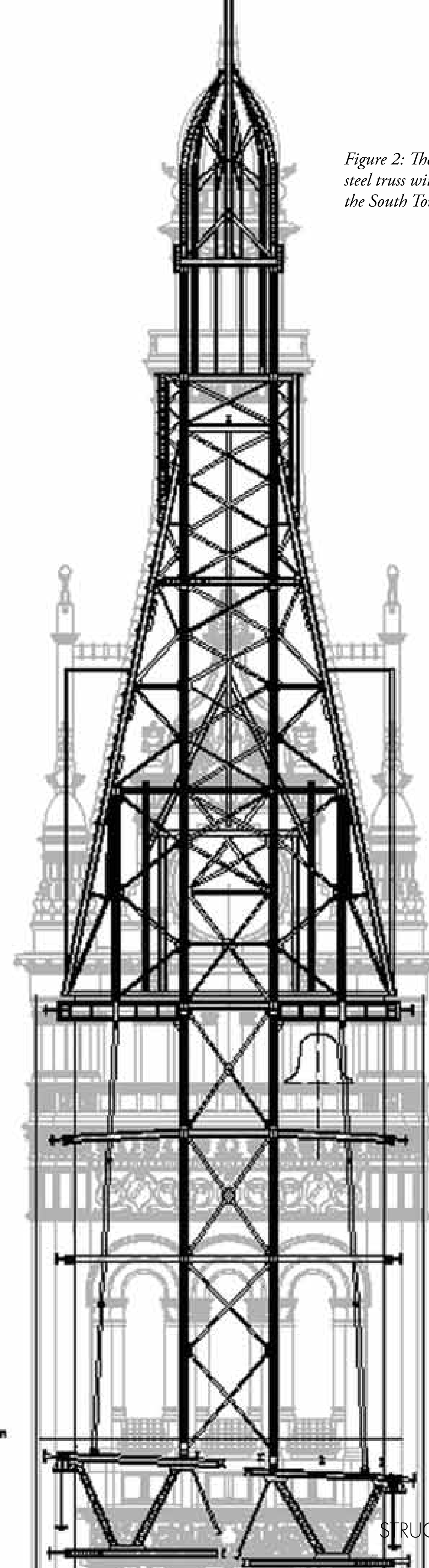

Figure 2: The core steel truss within the South Tower.

at the 10th floor. The flexibility of this support was accounted for using elastic springs at the base of the analytical model. The spring stiffness was calculated based on the elastic deformation of the diagonal truss considered as a simply supported beam resting on the masonry. Rotational restraints were used at the ends of floor beams where they are embedded in the masonry walls.

Cross-sectional properties were calculated from the structural shapes specified on the drawings for the steel tower, and from field measurement of some members.

The steel truss supports 3-inch-thick terra cotta roof tile and copper cladding, the weight of which we modeled as concentrated masses at nodal locations. Other miscellaneous weights, such as the lantern weight at the top of the core truss, were also included. The weight of the floor at various levels was modeled as concentrated masses at the beam-column intersection nodes.

Figure 3: FE mesh of the tower masonry.

The wind load was calculated in accordance with *Minimum Design Loads for Buildings and Other Structures* (ASCE 7-02), using 90 mph as the basic wind speed for Milwaukee.

The steel structure incorporates tie rods that appear intended to help resist tower overturning forces between the 12th and 13th floors. The tower was analyzed with and without these tie rods. The results show that the tie rods do not significantly influence the stresses in the members or the tower deformations.

The steel stresses throughout the model were found to be within allowable limits, based on standard design practice at the time the tower was built.

Masonry Wall FE Model

The reactions from the steel truss analysis were applied to a FE model of one side of the masonry tower from the 9th to 13th floors. The effect of the adjacent perpendicular walls at the tower corners was modeled by applying a symmetric boundary condition about a 45-degree plane through the center of the corner pier. *Figure 3* shows the FE mesh of the masonry tower model.

The cross-sectional properties of the wall and piers were computed, incorporating the offsets in the wall at different levels. The various colors in *Figure 3* represent the different section and material properties used in the model. SGH analyzed the wall for gravity loads including the reactions from the steel tower. The resulting stress values were used to design the repairs at and above the 11th floor.

Elements of the Structural Design

Figure 4 (page 32) schematically shows the locations of some of the major structural elements that we discuss below.

Durability

SGH addressed the durability goals of the project by careful selection of systems, materials, and details. Much of the tower structure, both interior and exterior, is exposed to the elements and subject to temperature extremes, rain, snow, and humidity, at heights where the wind can drive precipitation into every corner, so it presented exceptional durability challenges.

Galvanized steel was specified for all new structural steel. As a value engineering step, the final construction utilized a durable three-coat paint system, comprised of an inorganic zinc-rich primer followed by an epoxy intermediate coat and finished with a polyurethane coat. SGH specified stainless steel or hot-dip galvanized components for all post-installed anchors and threaded rods. For concrete elements, epoxy-coated reinforcement and wire-welded fabric was specified for all applications, and a minimum concrete strength of 5,000 psi.

Ring Beam

The structural analysis found excessive horizontal tension stresses in the perimeter masonry at the 13th level. These walls supported four solid masonry gables, incorporating 15-foot diameter clocks, and massive solid masonry corner turrets, as well as a portion of the load from the sloping steel-framed roof.

To resist the stresses at the 13th level, we designed a reinforced concrete "ring beam" that is 1 foot 3 inches wide by 4 feet 6 inches deep. Because the gable walls and corner turrets were in poor condition, and constructing the ring beam required removing the masonry at this level, it was determined that the solution that best matched the economic and durability goals of the project was to remove and reconstruct the masonry from the 13th level up, incorporating modern materials and systems while maintaining the historical appearance of the tower.

The ring beam was designed using forces derived from the computer model of the steel roof, as well as loads from the 13th floor and the clock gables.

The ring beam at each face is supported by the corner turrets and by four intermediate piers, which also required reconstruction due to extreme deterioration.

Corner Turrets

At each corner of the tower, the ring beams frame integrally into the massive concrete cores of the reconstructed corner turrets. The solid concrete cores are over 7 feet in diameter and nearly 11 feet tall.

The *ACI 301 Specifications for Structural Concrete Checklist* states that heat of hydration should be considered for elements with minimum dimensions over 2.5 feet. Because the turrets are large by this standard, special procedures were specified to address the potential for excessive heat gain in these elements. The use of up to 30% flyash or 50% slag replacement of cement was permitted to reduce the heat of hydration. A maximum differential concrete temperature of 35 degrees Fahrenheit and a peak temperature of 135 degrees Fahrenheit was specified.

To verify that the contractor's mix design would meet these goals, SGH required the contractor to submit thermal and strength analysis of the mass concrete mix, including heat-of-hydration analysis of the cement, concrete strength tests, adiabatic heat signature tests on 6-inch by 12-inch cylinders, and simulation studies.

The mix design for the 5,000 psi turret concrete included 615 lb/cy of cementitious materials, 32% of which was a combination of Type C fly ash and slag, as well as a set-retarding/water-reducing admixture. The contractor engaged a consultant to perform heat analyses of the pour, which predicted a maximum temperature of 130 degrees Fahrenheit and a maximum temperature differential of 32 degrees Fahrenheit, assuming a placement temperature of 50 degrees and ambient temperature ranging from 45 degrees to 50 degrees.

There were some concerns about the actual pour, because the delivered temperature of the concrete was 70 degrees Fahrenheit and the air temperature was below 40 degrees, but temperature sensors cast into the northwest turret recorded a maximum temperature of 115 degrees Fahrenheit in the middle of the turret and a maximum temperature differential of about 30 degrees Fahrenheit, satisfying specified goals. No significant cracking was observed in the concrete when the contractor removed the forms.

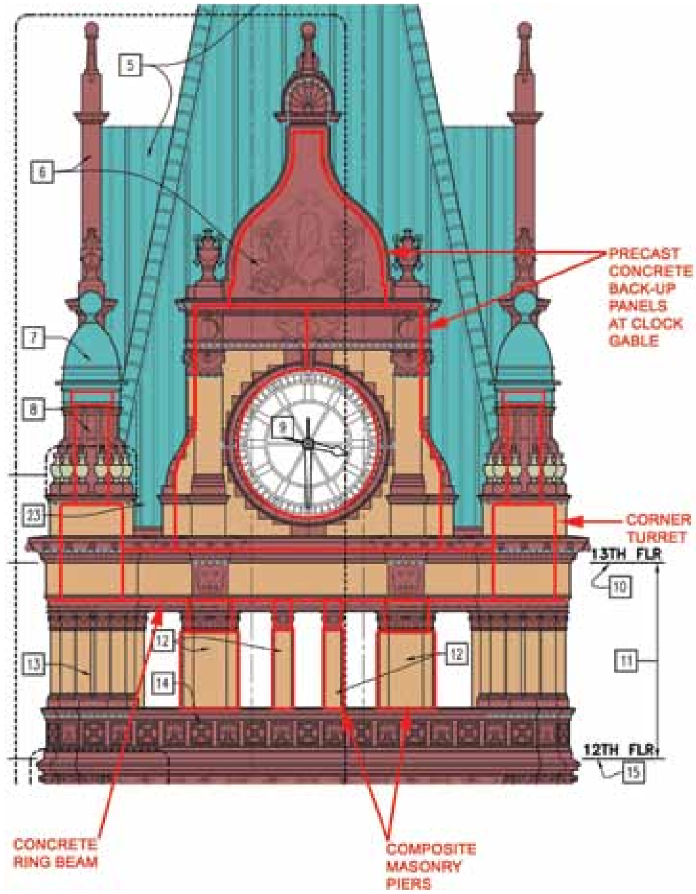

Figure 4: Schematic diagram showing location of major new structural elements.

Clock Gables

The clock gables were originally constructed using solid mass masonry with embedded structural steel framing. Precast concrete panels attached to new structural-steel framing were used to reconstruct the gables. This solution offered structural system continuity with the steel framing in the core truss, eliminated any embedded structural-steel framing that would be vulnerable to future hidden corrosion, and provided a stable backup surface for the brick veneer cavity wall system used to face the new gables.

Three 6-inch precast panels were specified to frame the face of each gable *(Figure 5)* and two panels for the cheek walls, nominally reinforced with #5 bars at 12 inches on center each way at panel mid-thickness. The panels were designed to resist their self-weight, out-of-plane wind, and seismic forces, providing connections with slotted and oversized holes between the precast and steel to isolate the panels from other loads that were intended to be supported by the steel framing.

Cintec Ties

To address vertical cracking and spreading of the tower walls at the 11th floor, the installation of three horizontal 54-foot long, Cintec 1-inch diameter deformed stainless-steel rods were specified in each masonry face. The rods were sized using the tensile stress results from the FE analysis of the masonry wall. In the Cintec system, the rods and associated fabric socks are inserted into cored holes, drilled horizontally in the plane of the walls at mid-thickness, and grouted. The fabric socks prevent uncontrolled dispersal of the grout while "keying" into voids and irregularities in the base material.

Pier Reconstruction

Each tower face had four solid brick masonry piers that needed to be reconstructed at the 12th story: two round piers approximately 1 foot 9 inches in diameter, and two roughly rectangular piers with overall dimensions of approximately 5 feet 6 inches by 2 feet 6 inches. The replacement piers were designed as reinforced composite masonry elements to strengthen them beyond what was inherent in the original unreinforced brick piers. The piers are reinforced with #6 vertical bars and #4 ties.

In these composite masonry elements, the veneer brick served as formwork for the grout. The brick was constructed in two-foot lifts. After the mortar set, the brickwork was braced as needed and filled with masonry grout. The grout needed to be sufficiently fluid to fully engage the perimeter brick and carefully consolidated to ensure composite action with the brick.

Figure 5: View of the ring beam, corner turrets, and clock gable precast backup during construction.

Construction Sequence

The construction sequence for the tower reconstruction presented special challenges. The top story of masonry had to be completely rebuilt, while the sloping roof structure above remained supported and capable of resisting live, snow, and wind loads. In the construction documents, a specific demolition and construction sequence was recommended that would maintain the structural integrity of the tower during construction, as follows.

1) Remove the clock gable and corner turret masonry down to the 13th floor.
2) Remove the steel elements that were embedded in the clock gables and the steel framing between the gable faces and sloped roof framing.
3) Repair or replace remaining corroded steel members as required.
4) Install Cintec anchors at the 11th floor.
5) Reconstruct the floor system at the 12th story. The replacement floor framing was designed to carry shoring loads needed to complete the following steps.

On one side of the tower at a time, complete Steps 6 through 10, allowing at least one week of curing time at a given side before proceeding to the next side:

6) Shore the central core truss and the 13th floor down to the new 12th floor.
7) Remove all the masonry at the 13th-story and 12th-story piers, including one contiguous corner turret.
8) Reconstruct the 12th-story piers and corner turret up to the bottom of the ring beam.
9) Cast the new concrete ring beam and remaining portion of the associated corner turret.
10) Resupport all roof framing and floor framing on the newly constructed ring beam as required.
11) Reconstruct clock gables.

The contractor elected to modify the proposed sequence, for scheduling reasons, by constructing the ring beams at the 13th floor before constructing the 12th-story piers. Since the piers support the ring beams, SGH was especially concerned about establishing tight joints between the tops of the piers and undersides of the beams. Following constructive dialog with the contractor, a sequence was settled upon in which the contractor constructed each composite masonry pier to the bottom of the terra cotta capital, cast in place the concrete backup for the capital, leaving a 2-inch gap below the ring beam, and then finally filled the gap beneath the ring beam with dry-pack.

Conclusion

The South Tower of the Milwaukee City Hall had serious structural damage that related to its original design, but had been aggravated by decades of exposure to a very aggressive environment. Using a combination of tailored construction materials and techniques, structural repairs and reconstruction procedures were developed that were designed to revitalize and extend the useful life of the South Tower of the magnificent Milwaukee City Hall, while preserving its important historical features.▪

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Photos courtesy of Simpson Gumpertz & Heger Inc. (SGH)