

Second Opinion Leads to Substantial Savings in Evaluation and Repair of Marble Façade

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When it comes to considering a costly and disruptive façade replacement, it always pays to get a second opinion. That's what the condominium association at Water Tower Place in Chicago found when faced with ongoing façade deterioration.

Water Tower Place is a 74-story, 859-foot tall reinforced concrete building, constructed between the years of 1973 and 1976 (Figure 1).

Georgia Cherokee Solar Gray marble panels (1½-inch thick) and flush aluminum-framed windows envelop the tower exterior (Figure 2). The veneer panels on this portion of the high-rise are anchored to the reinforced concrete structure spandrel beams and columns. Approximately 16,000 marble panels clad the building, and 6,240 of those (39%) cover the condominium portion of the building.

In 1979, several marble panels fell from the lower commercial portion of the building during a severe windstorm. As a result of the damage and what was believed to be a kerf anchor failure, the entire building façade was investigated by a local engineering firm which concluded that the kerfs had insufficient outward load strength capacity. In 1980, at the conclusion of the review, mechanical repair anchors were post-installed at the lower half of the spandrel panels to supplement the kerf outward strength resistance.

Over the years, regular inspections and repairs were performed and additional anchors installed on many of the façade panels. In early 2006, helical anchors were installed in panels at the corners of the tower (designated higher wind zones). The engineering consultant involved in this work determined that the marble would continue to lose 3% of its original strength or more per year, and that the entire façade would require replacement in the near future.

The prospect of such a costly and disruptive undertaking led the Condominium Association and the building's other two ownership entities to seek a second opinion. The Owners retained a consortium of European experts who were completing a European Union-sponsored five-year study on thin-stone marble failures in Europe. These engineering

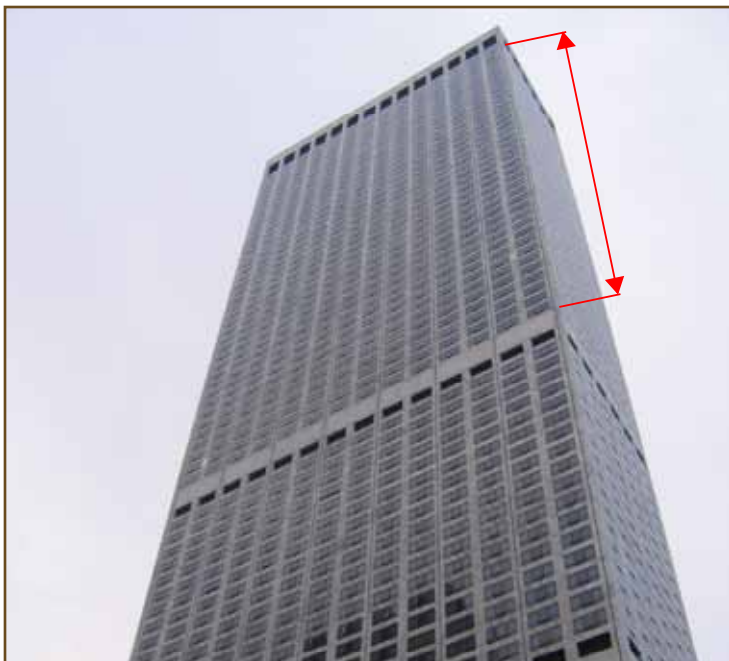


Figure 1: The condominium portion of Water Tower Place is from floors 33 to 72; the balance of the building is owned by commercial and hotel entities.



Figure 2: Overview of the metal-framed windows and marble panels.

and scientist professionals conducted a peer review of the prior investigations, and performed independent laboratory and field tests on the marble veneer and its connections. In general, the group determined that although the marble was losing strength, and would continue to do so, the loss was approximately 1% per year, or less.

Confronted with two analyses with broadly differing ramifications for the serviceable life of the façade, the Condominium Association hired Thornton Tomasetti in 2006 to be the engineering consultant for the condominium portion of the tower and to develop a preservation program for the marble façade.

Marble Façade Design and Construction

Four sizes, or types, of marble veneer panels clad the high-rise portion of the building (Figure 3). The structural frame here is comprised of upturned reinforced concrete spandrel beams, typically 12.5 inches wide by 32 inches deep, which are covered with four 33 inch tall panels per scaffold bay (Type F panels), as well as a smaller panel at the corners of the building (Type B panels). The columns are typically 10 inches or 14 inches deep by 48 inches wide, covered by 49-inch wide marble panels at the center of each bay (Type C panels). At each end of the bays, adjacent to where the building scaffold tracks are located, two vertical panels were utilized to cover the columns (Type E panels). All panels are typically separated from one another, and from the window framing, by 3/8-inch thick sealant joints.

When the building was constructed, marble panels on the high-rise portion were anchored to the reinforced concrete structure only around the panel edges, using stainless steel relieving angles as kerfs at the bottom, and straps with stainless steel dowel pins at the top and sides. A nominal 3-inch cavity, which is partially filled with urea formaldehyde foam insulation, separates the marble veneer from the reinforced concrete backup and concrete masonry infill.

The marble panels and connections appeared to be originally designed to resist 33 pounds per square foot (psf) wind pressure loads, per the 1973 Chicago Building Code. Due to the possibility of new wind dynamics caused by more recently constructed neighboring buildings, Thornton Tomasetti, together with the consulting engineer for the other two owners, suggested that an updated wind tunnel analysis test be performed for the entire building. The results from the test, which was completed in May 2007, indicated maximum pressures of up to 90 psf for the building's exterior cladding.

Furthermore, it was determined that any modifications to the marble anchorage needed to account for other environmental conditions, such as moisture and temperature cycles.

Existing Repairs: Post-Installed Anchors

Prior repair programs included the installation of numerous supplemental anchors in the façade panels. The first repair program was conducted in 1980, following the severe windstorm in 1979. As part of these repairs, mechanical restoration anchors were post-installed in all of the spandrel panels. The majority of the corner panels were also pinned with this type of anchor in 1980. One drawback of this type of anchor is its ability to resist outward (suction) wind loading only. Thornton Tomasetti's review of the façade noted that several of the mechanical anchor heads appeared to have loosened over time.

Moreover, Thornton Tomasetti observed that the helical anchors installed in early 2006 provided little out-of-plane support for the marble panels because they engage only a relatively small contact area of the stone. The lack of ability of the anchor to engage the stone material was evident in the slippage observed in test specimens.

Deterioration of the Marble Panels

As part of the initial review of the building, it was observed that a number of the veneer panels were displaying signs of inward or outward bowing. Other consultants previously reported that the degree of bowing and the amount of strength loss have a direct correlation. However, after testing of the marble strength was completed, this correlation was proven to be inconclusive, as none of the consultants for the façade were able to correlate the magnitude of panel bow with the magnitude of strength loss in the material on this building. Additionally, while many panels are bowed, especially on the West side of the building, most of the panels located in the condominium portion of the building are within the allowable tolerance for thin marble veneer panels ($\frac{1}{8}$ -inch bow displacement over a 4-foot length), as established by the Marble Institute of America (MIA) in the *Dimension Stone Manual v.6 (2007)*.

As part of the analysis, the experimental results from material strength testing performed previously by others were studied, and then the results most appropriate for the structural review of the marble façade panels were selected.

During the analysis of the testing performed by others, it was assumed that the panels utilized for experimental testing were selected randomly and evenly from the façade to represent the overall condition of the marble on the building. To utilize the experimental strength values for the structural review of the panels, Thornton Tomasetti used ASTM E122 to identify more reliable "true" strength averages (μ) from the measured strength averages (\bar{x}), by taking into account the specimen sample sizes (n) and the measured average strength variations ($s \approx \sigma$). As shown in *Table 1*, this typically resulted in corrections of the average measured strengths.

After review of all the available data and testing results, the rate of strength loss of the stone was determined to be close to an average of

Table 1: Summary of the marble and connection strength test results.

Marble Panel Component and Load Type	Exp. Avg. \bar{x}	Std. Dev. $s (\approx \sigma)$	No. of Tests n	"True" Avg. μ
10" Kerf Strength (lbs)	655	292	55	537
Pin Strength (lbs)	295	77	52	263
Pin Strength (lbs)	382	121	18	296
Helifix Anchor Strength (lbs)	918	236	8	668
Flexural Rupture Stress – Flexure w/Shear (psi)	1204	195	18	1066
Flexural Rupture Stress – Pure Flexure (psi)	718	137	492	699

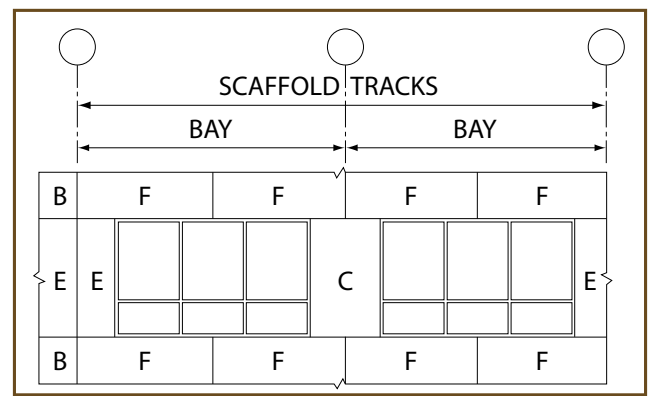


Figure 3: Partial elevation indicating panel types.

1% per year. The rate of the stone strength loss is expected to decrease with time. If the initial flexural rupture strength of approximately 1,100 psi to 1,200 psi for newly quarried marble is compared to the recently identified strength of approximately 700 psi for the stone on the building, the loss in strength over the past 30 years appears to be slightly more than 1% per year. Therefore, the Condominium Association was advised that an assumed strength degradation of 1% per year could be used to identify the serviceable life of the façade. The reduced strengths, over time, of the stone and connections have been summarized in *Table 2 (page 24)*.

The results of the wind tunnel study, dated May 2007 by The Boundary Layer Wind Tunnel Laboratory, were used for the analysis of the marble façade panels. The outward (suction) wind pressures ranged from 30 psf to 90 psf for the condominium portion, depending on the panel location.

Structural finite element models were utilized to represent the marble façade panels, their existing support conditions (steel dowel pins, kerfs, and post-installed anchors), as well as the proposed repair anchors. For the structural review of the panels, Thornton Tomasetti used the projected flexural rupture stresses, as well as pin and kerf support force capacities, with an applied factor of safety of two. The capacities were determined from testing results by others between 2004 and 2005, reduced by the estimated strength loss of 1% per year. Capacities were determined for the estimated strength loss up to the year 2025.

The marble panel and support geometries were modeled with SAP2000® structural analysis software. The marble panels were represented by shell elements, with a modulus of elasticity of 8,000 ksi and coefficient of thermal expansion of 8.3×10^{-6} in./in./°F, based upon the typical published values of these properties. The steel pin, kerf, and previous repair anchors were represented in the model with frame elements.

Structural Analysis Results and Discussion

The analysis of the façade resulted in a determination that the distress in the existing panels is a result of two load-deformation mechanisms: in-plane and out-of-plane.

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Table 2: Summary of the projected strength of the marble and connections.

Marble Panel Component and Load Type	Strength Test Results			Projected Strength at 1% per Year Reduction					
	Years 2004 – 2005			Year 2010		Year 2012		Year 2025	
	μ	$\mu/2$	$\mu/5^*$	μ	$\mu/2$	μ	$\mu/2$	μ	$\mu/2$
10" Kerf Strength (lbs)	537	269	107	505	252	494	247	424	212
Pin Strength (lbs)	263	132	53	247	124	242	121	208	104
Helifix Anchor Strength (lbs)	668	334	134	628	314	615	307	528	264
Flexural Rupture Stress – Flexure w/Shear (psi)	1066	533	213	996	498	968	484	786	393
Flexural Rupture Stress – Pure Flexure (psi)	699	350	140	627	314	603	302	447	224

* – Factor of safety recommended by MIA for new marble construction.

In recent years, the majority of the “new” small cracks located along the top of the spandrel panels (at or near the dowel pin supports) appear on the south elevation of the building. These vertical cracks can largely be attributed to thermal contraction and expansion of the façade panels. This mechanism was further confirmed by the finite element analysis of the panels subjected to in-plane thermal loading; the analysis results indicated relatively high tensile stress concentrations at or near the dowel pin locations.

The analysis considered both wind pressures and suctions applied to the marble panels, as updated by the wind tunnel test results, which resulted in out-of-plane moments, shears, reactions, and deflections.

As noted in the literature, bowing of marble panels can be attributed to anisotropic thermal and moisture expansion/contraction cycles, which

cause permanent strain in the stone microstructure. The panel bow is generally a result of movement at the marble grain boundaries which is independent of mechanical stresses in the material.

Some of the results of the June 2007 analysis are included in *Figure 4*, which displays results incorporating Dead, Thermal (+90°F), and Wind (55 psf) Loading. The following cases are compared:

- Case A: the original design and installation of the marble panels
- Case B: the marble panels with mechanical restoration anchors installed in the 1980s
- Case C: recommended repairs, with installation of 4 restoration anchors (missing one original pin)
- Case D: recommended repairs, with installation of 4 restoration anchors (missing two original pins)

The stress concentrations in the stone are visible at the kerfs, pins, and anchors. Based on these results, Thornton Tomasetti determined that the installation of repair anchors that have capacity to resist both wind suction and wind pressure loads would provide adequate support for the marble panels.

Repair Strategies

After demonstrating to the Condominium Association’s Façade Committee that the strength of the marble panels would be adequate for wind loads provided supplemental anchorage was installed, the search commenced for a repair anchor with the unique anchor qualities required for this project.

Because no anchor was found to have all the required design and installation properties, the possibility of working with a manufacturer to customize an anchor with the necessary characteristics was explored.

The structural analysis of the stone panel behavior concluded that a rigid out-of-plane connection is favorable for wind pressure resistance. A flexible repair anchor, previously suggested by others, would not attract enough of the load relative to the existing pins/kerf, which

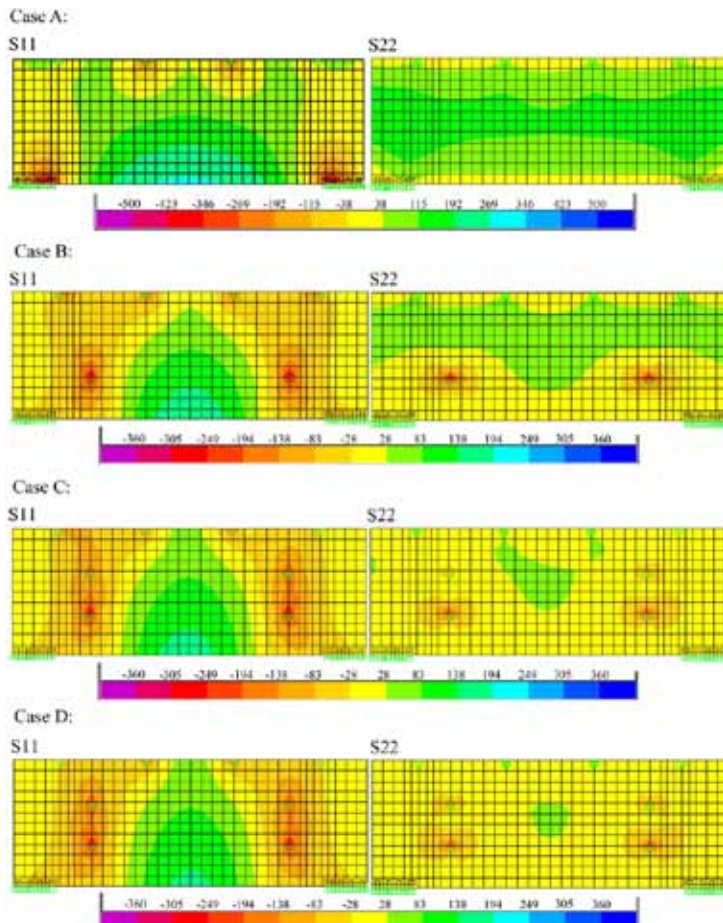


Figure 4: Analysis results for Panel Type F.



Figure 5: Expansion sleeve anchor designed by CTP and Thornton Tomasetti for panel support against wind pressure and suction.

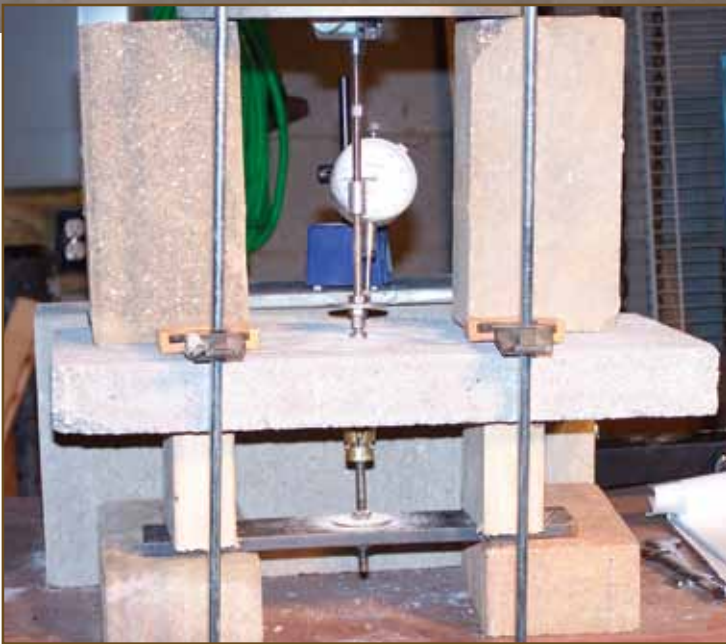


Figure 6: Anchor testing apparatus.

would result in overstresses and further cracking at the panel mid-spans, as well as at the pin and kerf supports. Thornton Tomasetti proposed using a mechanical anchor with an expansion sleeve, which was co-developed with Construction Tie Products (CTP), for resisting inward and outward wind loading (Figure 5).

Due to the low axial stiffness of the helical anchors and their susceptibility of slipping through the stone, all of the panels with helical anchors, post-installed in 2006, were presumed to be unreliable.

Thornton Tomasetti directed CTP to develop a testing assembly that would represent the interaction of the anchor and the stone (similar to ASTM C1354 testing). The anchor was tested in the stone with the reaction base spacing set at three times the anchorage depth (in this case it was 4½ inches from the test specimen), which generated an “anchorage load” value (Figure 6).

The anchors were loaded up to approximately 1,100 pounds without failure, and they exhibited an average axial stiffness of approximately 37 kips/inch.

The Condominium Association and Thornton Tomasetti have been able to increase the durability and longevity of the façade by the implementation of a repair and maintenance program. The additional years of prolonged service life will be utilized to increase the reserve fund of the Condominium Association in the event of a future need to replace the façade. ■

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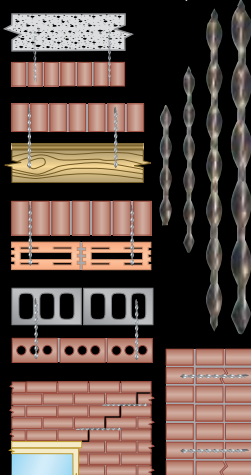
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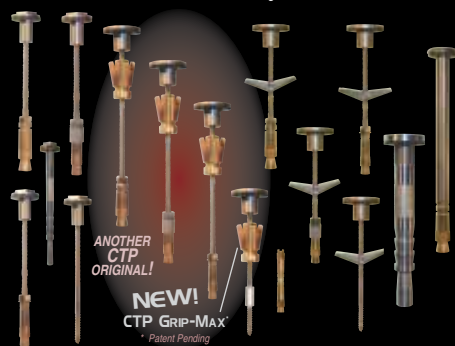
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