# Seismic Performance of Slender Core-Only Towers

By Mark Sarkisian, S.E., Eric Long, S.E., and David Shook, P.E.

Figure 1. 500 Folsom Tower.

**500 FOLSOM** is a new residential high-rise providing needed housing in the densifying urban fabric of the Transbay District of San Francisco. The site was originally part of the Embarcadero Freeway, connecting the Golden Gate Bridge and Bay Bridge, that was heavily damaged in the 1989 Loma Prieta earthquake. The site has been rejuvenated by the San Francisco Transbay Redevelopment Plan and Essex Property Trust. The architectural and structural designs were collaboratively conceived by Skidmore, Owings & Merrill LLP. The design was further enhanced through alliances with small business design collaborators Fougeron Architecture and STRUCTUS. The design of 500 Folsom is a good example for designers of slender-core only towers in high-seismic zones.

Massing maximizes the site's housing potential with a 42-story, ¾ million-square-foot, 537-unit apartment tower. The large podium is 85 feet tall with a 120- x 90-foot tower footprint rising to 420 feet (*Figure 1*), a floor height of 9.25 feet is used to achieve the preferred density of units. A significant tenant amenity at the podium roof provides sweeping views of the city. The architectural design gives the distinction of shifting blocks balanced with verticality, achieved with energy-reducing shading fins.

The structural system is entirely reinforced concrete, for cost-efficiency, and facilitates a 3-day floor-to-floor leading core concrete construction cycle. Spans were coordinated with the architectural design to facilitate a thin 7-inch-thick post-tensioned slab supported by conventional concrete columns and a core-only lateral force-resisting system. The concrete core had a dimension of 33 feet by 52 feet. Wall thickness ranges from 36 to 24 inches. This system rests on a 6-story below-grade basement, which is founded on a 10-foot-thick reinforced concrete mat foundation over dense sands, rock, and soil-improvements.

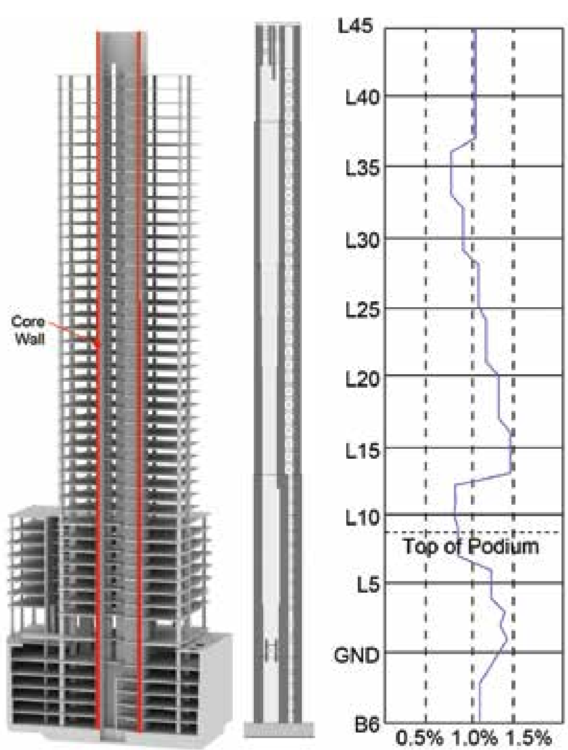


Figure 2. Building cut-away, shear wall elevation, and wall vertical reinforcement ratio.

## Background

The building code requires buildings over 240 feet in height to have a dual lateral force-resisting system, which includes a moment frame. The added moment frame is a redundancy provision, in part due to the limitation of code-prescribed design methods that utilize linear methods such as response spectrum. Performance-based seismic design (PBSD) guidelines by PEER/TBI have become the standard for tall building design on the West Coast. These provisions create a rational method for validating seismic force-resisting systems that take specific exceptions to the building code. For 500 Folsom, the dual-system requirement and a slight reduction in vertical reinforcement at the hinge zone where the only exceptions.

As required by the *San Francisco Building Code*, a peer-review panel was formed with experts in reinforced concrete analysis (professor), design (practitioner), and seismology. The panel is responsible for reviewing the design criteria, analysis results, and final design of the seismic force-resisting system, including drawings and calculations. Presentations are made to resolve comments by the peer-review panel. The deliverable of the peer-review panel is a letter to the City of San Francisco, giving a summary of their findings.

## Design and Performance

The core-only approach has been analytically verified to provide adequate lateral system strength and stiffness on many projects. It is proportioned to meet all non-exempted seismic requirements using code-prescribed linear response spectrum analysis. The behavior observed in response spectrum analysis is favorable.

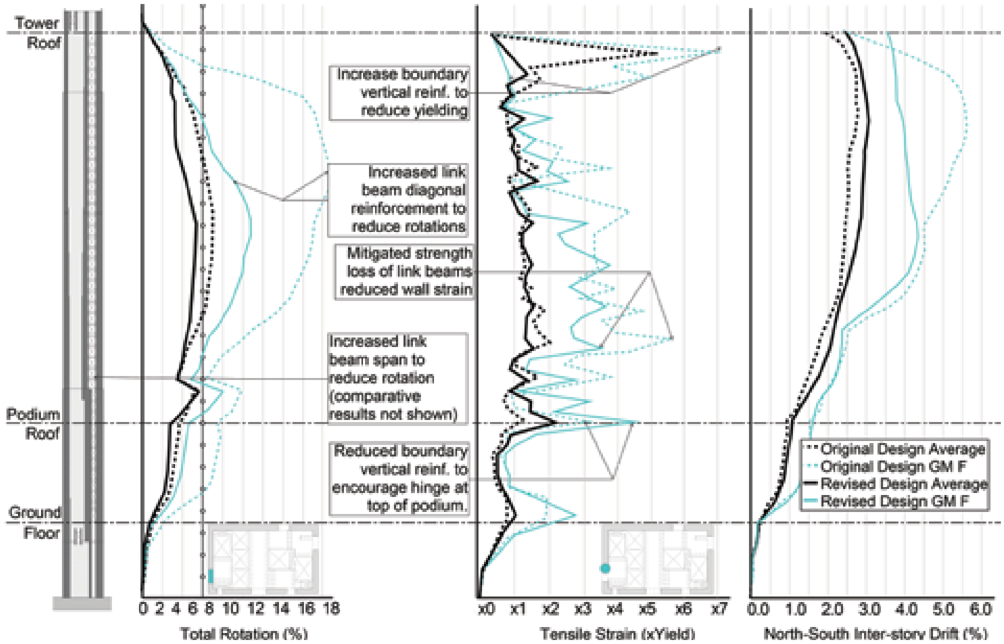

*Figure 3. Design revisions and improved performance.*

PEER/TBI requirements are intended to meet minimum building code provisions with a few enhancements considered appropriate for tall building design. ASCE 7-16, *Minimum Design Loads for Buildings and Other Structures*, has formalized the use of nonlinear response history analysis (NLRHA) for taking exception to building code requirements under the guidance of a peer review panel, much like PEER/TBI, but only meeting the minimum code requirements. A comparison of three significant differences between ASCE 7-16 and PEER/TBI are:

- ASCE 7-16 Chapter 16.4.1.2 permits the average of peak drift results under MCE-level demands to be 4% for a shear wall building. PEER/TBI requires the average of peak drifts to be 3%.
- ASCE 7-16 has no limit for individual ground motion results, but PEER/TBI limits individual ground motion results to 4.5% drift or less.
- ASCE 7-16 has no residual drift limits. PEER/TBI limits average residual drifts to 1% and individual ground motion residual drifts to 1.5%.

After completion of the code-based design, it can be beneficial to the design team to create summarized plots describing wall vertical reinforcement ratio over the building height (*Figure 2*). This information was compiled for all vertical reinforcement in the core and link beam shear strength. Gradual changes in core-wall vertical reinforcement are essential. Also, consistent link beam strength over the height of the tower is necessary to avoid local concentrations.

Traditionally, designers have preferred energy dissipation in core wall buildings to primarily be from a *hinge* (a focused area of vertical wall reinforcement yielding) complemented by the yielding of link beams. When evaluating the seismic performance of slender core-only residential buildings using NLRHA, many designers have not observed this behavior. For slender core-only buildings, it is common for most of the energy dissipation to come from link-beam yielding and minimal energy dissipation to come from shear wall vertical reinforcement yielding. Typical residential towers stand unique from typical office towers in that the floor-to-floor heights of typical residential towers are noticeably lower. This results in shallow link beams (9.5 feet is a typical residential tower floor height, and 13 feet or more is a typical office building floor height). The shallower link beams, common in residential towers, provide lower wall-to-wall coupling and result in higher link beam rotational demands. The lower cumulative strength of the shallower link beams over the height of the building limits the formation of a concentrated wall *hinge* at the dynamic base of the core wall. While the behavior is certainly acceptable, it is different than anticipated by the code-prescribed design procedures.

Furthermore, many structural designers using NLRHA have observed higher shear demands than prescribed by response-spectrum code design. This finding is partially attributed to a code-based design methodology that presumes all modes experience uniform energy dissipation. As affirmed by NLRHA, the deformations associated with higher modes often experience less energy dissipation and, therefore, higher shear demands, often 2-4 times response-spectrum code-based designs. In the most recent version of ACI 318-19, *Building Code Requirements for Structural Concrete and Commentary*, this effect has been addressed by a *dynamic amplification factor* for shear in slender shear walls.

As part of the PBSD process, a detailed nonlinear response history analysis was conducted. The analysis was conducted using PERFORM 3D and includes detailed nonlinear modeling of shear walls, link beams, and slabs represented as equivalent frames. A robust set of 22 MCE-level linearly scaled ground motions with a high level of dispersion was developed. Conditional mean spectra were used for each set of 11 ground motions to target short period and long period demands. At the primary period of the structure (5.5 seconds), some ground motions were 120% to 170% of the MCE target spectra. In other words, at the fundamental period of the tower, some ground motions were up to 70% higher than the MCE level demands. This is not unreasonable since MCE is not an absolute maximum, and it is not unreasonable to consider a few individual ground motions greater than MCE with some below the MCE, such that the average meets the MCE.

## Analysis Results

Upon review of analysis results, ground motions within 120% of MCE at the target period performed well, and behaviors were well aligned with expectations resulting from response spectrum results. Isolating results of ground motions less than 120% of MCE, all key behavior results such as drifts, link beam rotations, slab rotations, and wall shear were reasonable. The difficulty arose from 3 ground motions that were between 120% and 170% of MCE at the fundamental period. These ground motions caused unacceptable link beam rotations and drifts.

500 Folsom is a compelling case in that the minimum requirements of ASCE 7-16 could be considered satisfied with these responses but would not have met the requirements of PEER/TBI without a revision to the code-based reinforcement design. Since the criteria

was PEER/TBI, designers evaluated a variety of design alterations which included:

- Widening openings on the east and west sides of the core to reduce link beam rotation magnitudes.
- Increase link beam reinforcement from $6\sqrt{f'_c}$ to the code maximum of $8\sqrt{f'_c}$. This change increased the link beam diagonals from #11s to #14s.
- Increase vertical reinforcement in the upper floors near the roof solid wall regions to reduce yielding.
- Reduce vertical reinforcement near the top of the podium to help encourage a more distinct plastic hinge in the wall vertical reinforcement in the higher magnitude events.

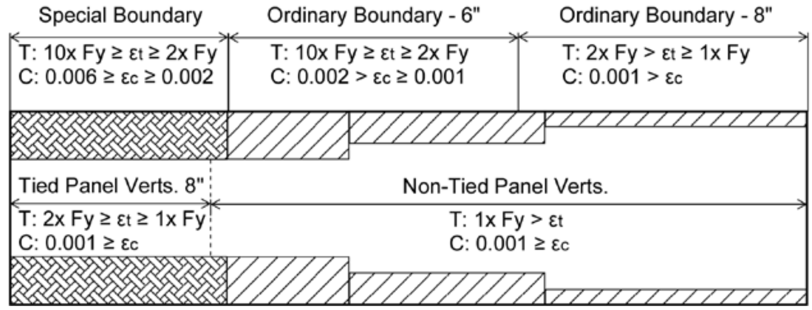


*Figure 4. Proposed mapping of NLTH analysis result to ACI 318-14 Chapter 18.10.6.4 boundary detailing provisions.*

These targeted changes resulted in overall cost reductions and performance improvements. These changes did not significantly change the response to ground motions under 120% of MCE. Still, the changes did result in significant improvements for ground motions greater than 120%, as shown in *Figure 3, page 33*. These results indicate the importance of designing with NLRHA using prescribed guides such as PEER/TBI. Through the incorporation of NLRHA, important design improvements were identified to achieve the design intent conforming to PEER/TBI and the *San Francisco Building Code*.

The high levels of dispersion in the ground motion demands is an important consideration for resilience as well. Both ASCE 7-16 and PEER/TBI permit the use of spectral matching. As often applied, spectral matching reduces demands to MCE level demands at all periods, limiting designer understanding of performance at higher levels of shaking, which may occur. Resilient designs should continue to generally perform well even beyond MCE level demands but, without evaluations beyond MCE, this behavior cannot be confirmed. The higher levels of dispersion using spectral scaling in the 500 Folsom design allowed designers, through a few modest changes to the design, to achieve reasonable performance beyond MCE level demands and reduce costs.

## Shear Wall Boundary Confinement

Nonlinear wall elements in NLRHA output strain at numerous locations of the core walls on all floors. This information cannot be obtained with response spectrum analysis but can be greatly beneficial in specifying boundary zone detailing. At the time of design, ACI 318-14 prescribes the type and location of boundary elements based on a simple shear wall, but core walls are much more complex. Mapping of compression and tension strain demands to boundary zone types could be immensely helpful in increasing resilience while reducing costs.

ACI 318-14 Section 18.10.6.4.2 provides boundary zone detailing requirements for three different types of boundary zones: special boundary zone, ordinary boundary zone with a 6-inch spacing of ties, and ordinary boundary zone with an 8-inch spacing of ties.

1) Special Boundaries (ACI 318-14 18.10.6.4):
   Bar buckling restraint in tension and ductile confinement in compression.
   T: 10x yield ≥ $\varepsilon_t$ ≥ 2x yield
   C: 0.006 ≥ $\varepsilon_c$ ≥ 0.002
2) Ordinary Boundary with 6-inch Spacing (ACI 318-14 18.10.6.5-Yielding):
   Bar buckling restraint in tension and very modest confinement in compression.
   T: 10x yield ≥ $\varepsilon_t$ ≥ 2x yield
   C: 0.002 > $\varepsilon_c$ ≥ 0.001
3) Ordinary Boundary with 8-inch Spacing (ACI 318-14 18.10.6.5-No Yielding):
   Limited bar buckling restraint in tension and no confinement in compression.
   T: 2x yield > $\varepsilon_t$ ≥ 1x yield
   C: 0.001 > $\varepsilon_c$

   <u>Tensile Strain Limits</u>
   Unrestrained bar = 1x yield
   8" tie spacing = 2x yield
   6" tie spacing = 10x yield
   <u>Compressive Strain Limits</u>
   Ordinary (8") Limit = 0.003/2*/1.5** = 0.001
   Ordinary (6") Limit = 0.004/2* = 0.002
   Special Limit = 0.013/2* = 0.006
   *Per Wallace, 2007
   **Force-controlled action

Based on performance, a mapping of NLTH results and ACI 318-14 design provisions are utilized to distribute the three types of boundary zone types throughout the tower (*Figure 4*). The boundary zone type above was mapped to each boundary on each floor based on average tension and compression strains under MCE demands. Generally, special boundary zones were used between ground and podium hinge and also lined the east and west core openings to the roof. Ordinary boundaries with 6-inch and 8-inch spacing were used through the height of the tower, with select zones near the two-third-point of the height having special boundary elements.

## Conclusions

The design and verification of the 500 Folsom seismic force-resisting system with NLRHA revealed the importance of conducting a detailed nonlinear analysis even if code-provisions are satisfied. Furthermore, component behaviors can be understood in greater detail, producing greater resiliency and reducing costs.▪

The online version of this article contains additional graphics. (**www.STRUCTUREmag.org**)


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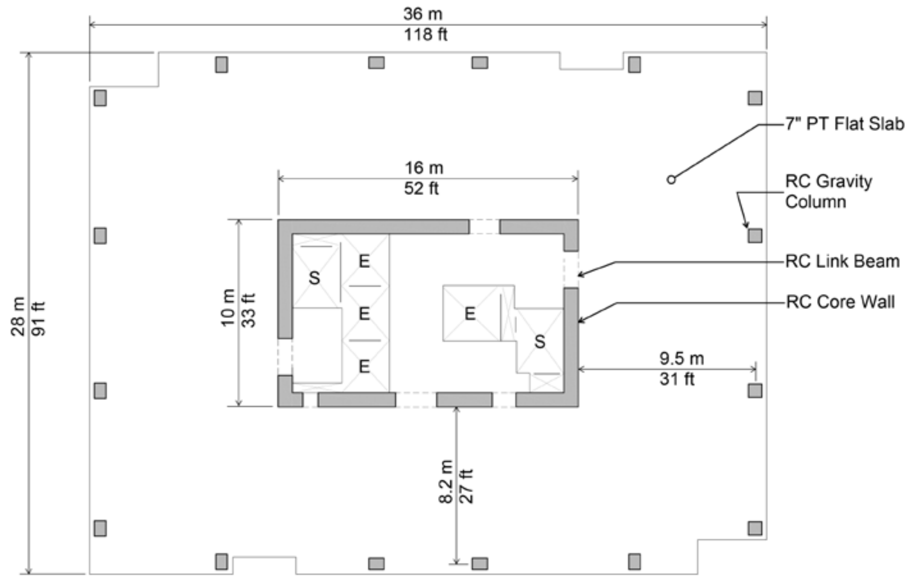
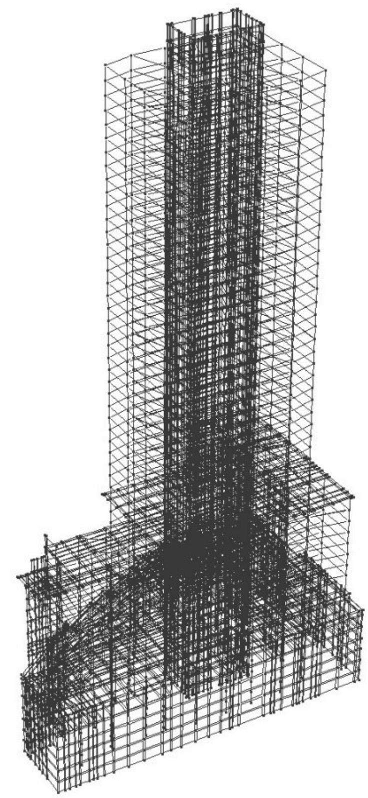



*Typical tower floor plan.*



*3-D view of Perform3D model.*