The Resilience-Based Design of the 181 Fremont Tower

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The 181 Fremont Tower, located in downtown San Francisco adjacent to the new Transbay Transit Center, will arguably be the most resilient tall building on the West Coast of the United States when completed in 2017. At that time, it will be the second tallest building in San Francisco (802 feet). The tower was designed to exceed CBC-mandated (*California Building Code*) earthquake performance objectives for new tall buildings by following a "resilience-based-design" approach. It was designed to achieve immediate re-occupancy and limited disruption to functionality after a 475-year earthquake (i.e. functionality is reestablished once utilities are restored) by targeting specific design criteria to achieve a "Gold" rating as outlined in the REDi Rating System (REDi, 2013). The US Green Building Council (USGBC) announced that satisfaction of the REDi guidelines may be used as a means to obtain the recently adopted LEED Resilient Design Pilot Credits. To achieve a "Gold" rating required that the structure is designed to remain essentially elastic (under 475-year shaking, which is approximately the recurrence interval for the CBC design basis earthquake, or DBE) and that the non-structural components are designed for more stringent force and displacement requirements relative to the code. The owners, Jay Paul Company, were also provided with additional recommendations for implementation of preparedness measures that would aid in achieving the enhanced performance objectives.

Background of Code-Based and Performance-Based Design

Tall buildings in San Francisco and other West Coast cities are typically designed and assessed using a performance-based design approach (following the Pacific Earthquake Engineering Research Center (PEER) Tall Building Initiative guidelines or similar), primarily to circumvent height restrictions for certain lateral systems contained in the building code. Often, the earthquake performance objectives adopted are no more stringent than those outlined in modern building codes (i.e. low probability of collapse in an MCE, see ASCE 7-10) even though the loss of occupancy or functionality (in lower intensity shaking) in even a single tall building could have significant economic and societal repercussions.

In many cases, building owners are unaware of these potential consequences. They assume that meeting the minimum requirements of the building code will preserve their investment. This is a misconception of the code intent, shared by the public. PEER (Holmes et al., 2008) conducted a survey of building owners which found that their expectations for performance did not align with those outlined by the code. Tipler et al. (2014) found that tall reinforced concrete core-wall buildings, designed to meet state-of-the-art performance-based design guidelines (PEER-TBI, 2010), are still expected to suffer 15% financial loss and almost two years of downtime after 475-year earthquake shaking. This discrepancy suggests that our building codes are not aligned with public expectations.

Figure 1. The 181 Fremont Tower with the Transbay Transit Center in the foreground.

Resilience-Based Design

The owners of the 181 Fremont Tower envisioned a high-performance building, one which would incorporate innovative one-of-a-kind design strategies for sustainability (targeting LEED Platinum) through water savings and energy efficiency. They recognized that resilience is a natural extension of their enhanced sustainability objectives. After they had found that typical earthquake performance objectives did not align with their vision of a high-performance building, they elected to pursue a design strategy presented by the structural engineers to achieve "beyond code" earthquake resilience objectives.

Achieving such high-performance targets requires a holistic "resilience-based-design" approach, which identifies and attempts to mitigate all threats that may hinder re-occupancy and functionality objectives through enhanced design of both structural and non-structural components, and pre-disaster contingency planning. To supplement this approach, a site-specific loss assessment (based on FEMA P-58 (2013)), developed specifically for REDi, was used to verify the success in the design and planning measures to achieve the higher performance objectives. This method explicitly estimates downtime associated with specific recovery states such as re-occupancy and functionality, considering building damage, utility disruption, and external factors such as "impeding factors" (which delay the initiation of building repairs, e.g. the time it takes for a contractor to begin repairs).

Structural System Description

Arup is the Structural Engineer of Record and Geotechnical Engineer of Record for the 181 Fremont Tower. The building has a steel core and perimeter framing with composite floors of concrete on steel framing. Above ground, it is 56 stories tall and the spire rises to a height of 802 feet. The lower 37 levels are offices and the upper levels are condominiums. It will be the tallest mixed-use building on the West Coast when completed. Below ground, a 5-story concrete basement is supported on a concrete mat with 5-foot to 6-foot diameter concrete piles socketed into bedrock more than 200 feet below the ground surface. The piles control total and differential settlement due to the soft Bay Muds and other soft soil layers immediately below grade, as the originally developed downtown site was reclaimed from San Francisco Bay (specifically, Yerba Buena Cove). The seismic system consists of a dual system including a mega-frame.

Figure 2. Rendering of the structural system.

Figure 2 shows the structural system with the mega-frame highlighted in yellow. Mega-columns and mega-braces, which resist the global lateral loads, make up the system. In the office levels, steel mega-braces span from ground level to Level 20 and from Levels 20 to 37. Between Levels 37 and 39 (which house amenities and a mechanical level), an inverted chevron braced frame provides continuity of lateral forces from the condominium levels to the office levels. Large W14 sections make up the perimeter braces above Level 39. In general, vertical axial loads are resisted by welded steel box mega-columns (in the condominium levels, the columns are W14s), which vary in dimension but are commonly 36 inches by 36 inches at the base of the tower. Concrete fills the box columns up to Level 21. The mega-columns at the base support the entirety of the perimeter gravity loads; a transfer truss, which spans to the mega-columns, supports the perimeter frame system at Level 3. Since most of the lateral force is resisted at the base of these mega-columns, the designers could consider an uplifting solution to reduce column tension demands. A secondary system of special moment frames (generally W24s) transfer individual floor demands up or down to the mega-node locations (at Levels 3, 20, and 37). The small footprint of the building (approximately 120 feet x 90 feet at the base and tapering to 95 feet x 80 feet at the roof) did not allow for a core system in the office levels, so the lateral system is located entirely on the perimeter. In the condominium levels, there is a braced core of buckling restrained braces (BRBs) that act as a secondary system between the mega-nodes at Level 39 and the roof.

Structural Design

A key component of resilience-based design is to limit the damage to structural elements to essentially elastic or better. Any structural damage requiring significant repairs could cause the building to receive a Restricted (yellow) or Unsafe (red) post-earthquake placard which would impede the ability of residents/tenants to re-occupy the building or resume business operations. All of the structural elements were designed to remain essentially elastic under 475-year earthquake shaking; the nonlinear response history analysis verified this objective.

For higher intensity shaking beyond the DBE, maximum-considered earthquake (MCE) structural actions in each component were either designated as *deformation-controlled* or *force-controlled*, following the methodology of PEER-TBI (2010). Acceptance criteria for deformation-controlled actions were adopted to limit the amount of damage at MCE. Force-controlled actions were

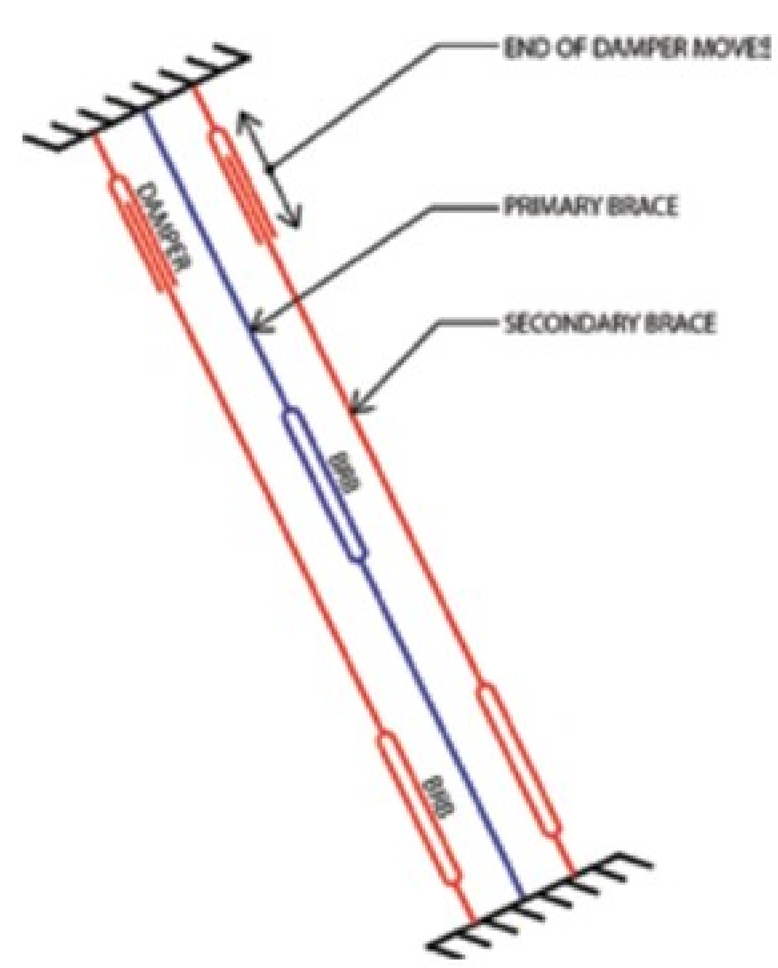

Figure 3. Schematic of damped mega-brace system.

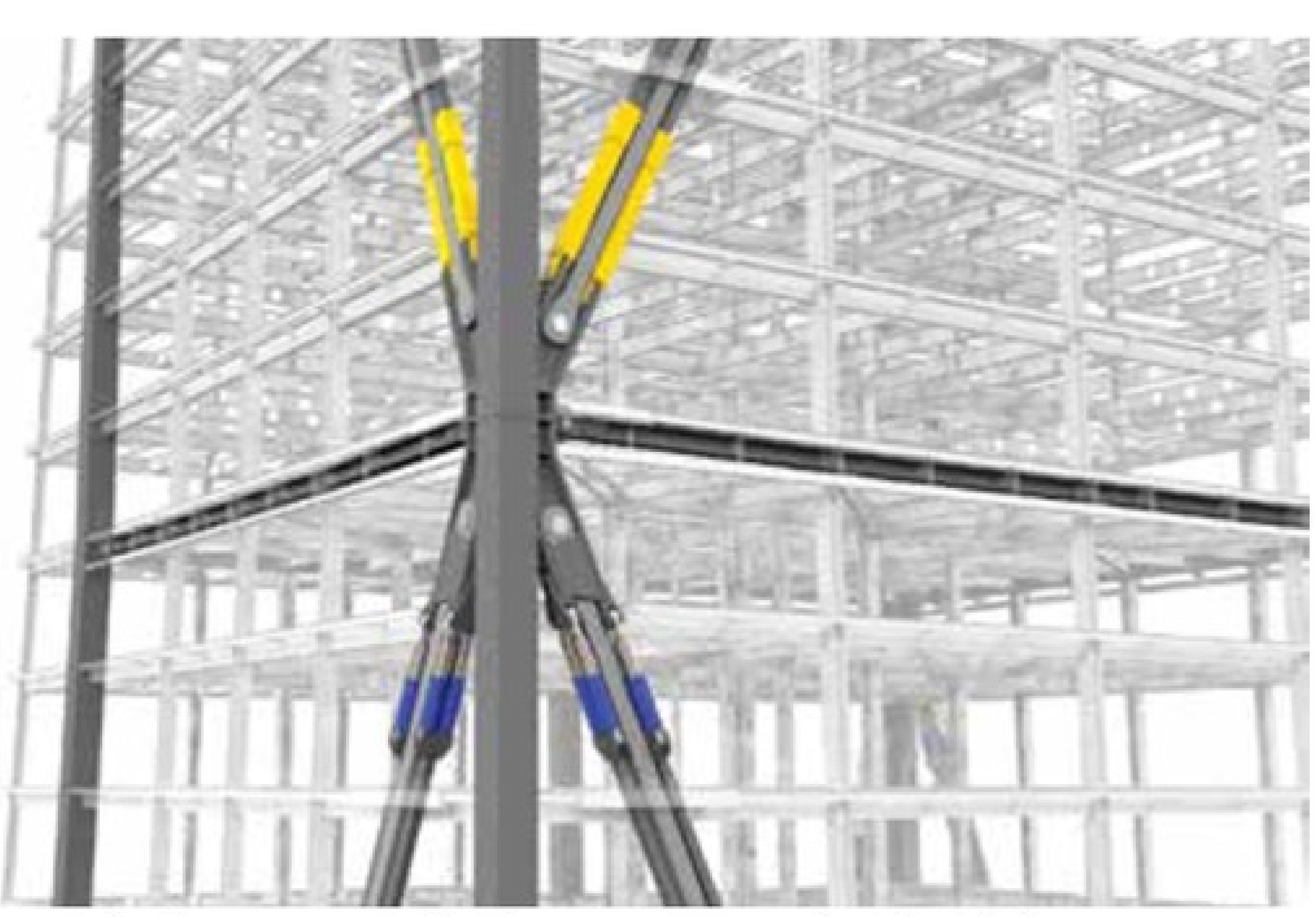

Figure 4. Close-up view of the dampers integrated within the brace system.

Figure 5. Construction workers are lowering the mega-column base over the shear key.

typically designed for 1.5x mean MCE demands determined through non-linear response history analysis.

Damped Mega-braces

Since the tower is slender and lightweight, wind accelerations (particularly near the top) posed a potential issue due to stringent wind acceleration criteria in the condominium levels (at the fundamental period of the building of 7.5 seconds, approximately 10 milli-g peak acceleration under 1-year winds and 20 milli-g under 10-year winds). The traditional approach for mitigating wind vibration is to incorporate a tuned mass damper (TMD), typically at or near the roof level of tall buildings. TMDs are not always an optimal choice since they are costly, heavy, relatively large, take up valuable real estate in the most desired locations and increase the gravity loads on the structure. They are also not reliable in reducing seismic demands.

Therefore, an innovative viscous damping system was developed to fit within the architecturally expressed mega-brace design. The damped mega-brace system generates approximately 8% of critical damping, which has a significant effect in reducing both seismic and wind forces, particularly for a tall building that has very low (~2%) inherent damping. This freed-up space originally reserved for the TMD and allowed the owner to create an additional residential penthouse.

The mega-brace system is three braces in one (*Figures 3* and *4, page 43*). The middle (or "primary") brace is a steel box section and the two outer (or "secondary") braces are comprised of built-up plates attached to two viscous dampers at one end. As the building flexes laterally in a wind or earthquake event, large (elastic) strains develop in the very long primary braces. The result is approximately 6 inches of lengthening or shortening in the primary brace between the connected nodes. Since the secondary braces are connected to the same mega-nodes via dampers, this relative movement is utilized to activate the dampers and dissipate energy. The system was tuned to optimize the wind performance. However, the damping additionally benefitted the seismic response of the tower by reducing the earthquake demands across several modes of vibration. This contributed to keeping the structural system elastic in the 475-year earthquake.

For the mechanism described above to function properly, the relative movement of the mega-braces in the axial direction was allowed to slide freely as they cross each floor plate but was restrained from buckling in all other degrees of freedom by incorporating low-friction PTFE bearings with excellent fatigue properties. This served to limit the wear from approximately 1500 km of anticipated travel distance.

Buckling restrained braces (BRBs) were also introduced into the load path of the mega-braces to act as a fuse in MCE shaking, protecting the primary and secondary braces, mega-columns, and dampers from damage.

Uplifting Megacolumns

The mega-columns were designed to uplift slightly (approximately 1 inch) at their bases in the MCE to significantly reduce the tension demands in the foundation and the mega-columns. The bases of the mega-columns were pre-tensioned with anchor rods extending below into the foundation so that uplift would not occur in wind or smaller earthquake events. Uplift would occur at a plane located just above the ground floor elevation, below the mega-column base plate and above a steel cruciform that rests upon the concrete pilaster embedded into the basement walls. A shear key (essentially a solid steel cylinder, similar to a pin in a BRB) was devised to transmit shear across this plane from the columns into the foundation in the event of uplift (*Figure 5*).

Non-Structural Design

The performance of non-structural components is crucial to achieving the immediate re-occupancy and functionality objectives in the 475-year earthquake. Following the REDi guidelines, the enhancements incorporated in the 181 Fremont Tower design include:

- In tall buildings, elevators are crucial for not only continuity of operations but also for re-occupancy. To have the utmost confidence that at least one elevator would function after the 475-year earthquake, the guide rails and support brackets of one of the elevators that stops at every floor were upgraded to satisfy California hospital requirements (CBC, 2010). The 181 Fremont Tower is the first tower in the United States to utilize an elevator as a designated evacuation route.

Figure 6. The façade system was racked to high levels of drift and proven to remain weather-tight.

- Noting the devastating consequences of stair failures in the Christchurch, NZ, earthquakes in 2010 and 2011, the structural engineers specified enhancements for the design-build stairs to accommodate more movement and sustain less damage at MCE level relative to the requirements of the CBC (which references ASCE 7, Chapter 13). For stairs which relied on bearing support, Arup specified a minimum horizontal bearing seat of 1.5x mean MCE displacements. The stairs were also required to retain their ability to carry dead and live loads under MCE level demands with minimal damage. A protective 'moat' was allowed for, within the stair shaft, to mitigate any partition wall damage and ensure pressurization.
- The façade was designed and tested to remain air- and weather-tight after the 475-year earthquake. A full scale, three-story performance mock-up was tested and showed that the façade for 181 Fremont Tower is air- and water-tight up to drift limits of 2%, which far exceeds the expected drifts for 475-year shaking (*Figure 6*).
- Additional limitations were applied to R_p factors for the anchorage design of non-structural components and distribution systems to keep them essentially elastic under 475-year shaking. Further, a plan to confirm that the installation of non-structural components conformed to the drawings and specifications was agreed upon with the contractor.

As noted above, the accelerations are expected to be low relative to shorter buildings. Therefore, mechanical and other equipment are not anticipated to be damaged. However, while critical life-safety systems are required to be certified, the design team was encouraged to specify other seismically certified, non-essential equipment where possible. Also, there are emergency back-up systems which are designed to keep the essential functions of the building, including elevators, running for 8 hours after an earthquake.

Organizational Resilience through Preparedness

There are a number of other earthquake preparedness and planning measures recommended to the owner and design team based on the REDi guidelines. These recommendations include:

- Retain a qualified pre-certified professional who can perform an inspection quickly after an earthquake (for example, the City and County of San Francisco have developed a Building Occupancy Resumption Program (BORP) to facilitate this). A quick response aids in avoiding delays to re-occupancy.
- Due to the likelihood of utility disruption, maintain "hard" backup security measures (i.e. keys in addition to access codes) to ensure that non-tenants cannot access the building if the power goes out.
- Train and certify on-site facilities personnel to re-start elevators since they are required, by code, to incorporate a shake-actuated shut-down mode. Otherwise, it could take weeks for external vendors to re-start them.
- Plan for natural gas shut-off.
- Incorporate recommendations in an *Owner's Guideline to Earthquake Resilience* for anchorage of heavy or mission-critical building contents, enhanced partition details, and food and water storage.

Figure 7. At each end of the mega-brace, the connections are pinned. At the lower end (shown), BRBs are introduced into the load path of the secondary braces.

Conclusions

While a resilience-based design approach is intended to extend beyond the typical purview of the structural engineer (to non-structural performance, contingency planning measures, and identification of threats outside the building envelope), skilled structural engineers are uniquely qualified to provide such expertise to owners and other stakeholders who demand "beyond code" performance.

In the future, it is hoped that enhanced re-occupancy and functionality objectives will be commonplace for tall buildings in high seismic regions because life-safety is no longer good enough for communities to achieve recovery quickly in the aftermath of a large earthquake. In the meantime, early adopters are critical in demonstrating that more resilient buildings can be designed and constructed for a little-to-no-cost premium and, by doing so, bring greater value to our community.▪

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