The Tallest Modular Tower Design Using a Performance Based Approach

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ABSTRACT

When completed in 2019, 2044 Franklin St. in Oakland, CA will be the tallest structure in the US utilizing *Prefabricated Prefinished Volumetric Construction (PPVC)* as an alternative to conventional construction methods. A vertically integrated team of AEC professionals in collaboration with manufacturing experts designed a modular system suitable for a mixed-use tower of this size. The structural engineering team developed a unique structural system comprising a steel plate shear wall (SPSW) core supplemented with buckling restrained brace (BRB) outriggers to provide a ductile response to severe West Coast earthquakes. This structural system not only complied with the architectural and rentable space requirements, but also helped to maximize off-site construction and to minimize the on-site labor work.

The use of Performance Based Design in conjunction with full-scale testing of some of the critical structural components helped to establish a compatible lateral story drift ratio of up to 2.75% at MCE level for the modular gravity space frame. The primary energy dissipation mechanism is envisioned to be a combination of steel plate shear wall post-buckling tension field action and BRB yielding. The proposed structural system translated to substantial material savings up to 30% and improved the construction speed by 6 months compared to conventional structural systems and construction methods.

INTRODUCTION

Despite being one of the largest sectors of the global economy, the construction industry has suffered for decades from remarkably poor productivity relative to other sectors. Highrise buildings as a sub-category of the construction industry have consequently inherited this trend and evolved slowly over past decades. Based on a recent study by the McKinsey Global Institute (Figure 1), the productivity rate in the construction industry has decreased in the past 50 years. Agriculture and manufacturing industries have experienced quantum leaps in contrast.

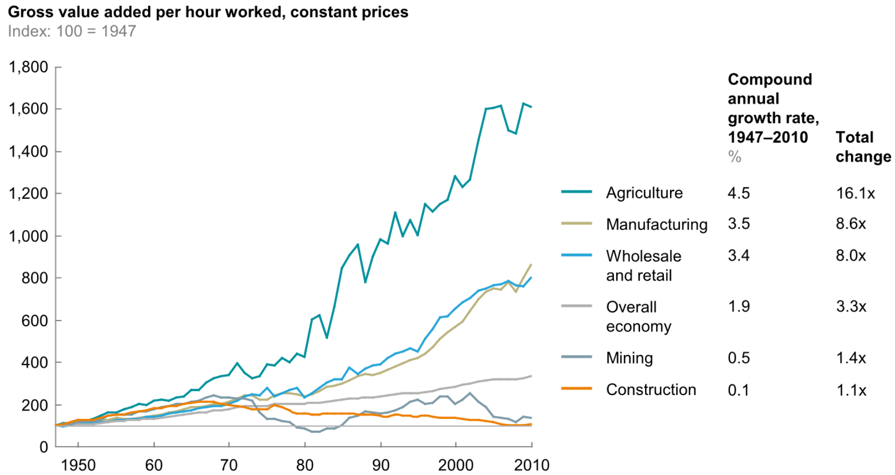


Figure 1. Productivity Trend Across Different Industries (Source: McKinsey)

The current highrise construction industry lacks efficiency because key factors affecting any given project's success, such as labor costs or local regulations are outside of the owner's control. By incorporating advanced manufacturing into the AEC industry, the design and construction processes can be completed in a highly controlled environment conducive to higher precision, better finishes and significant overall cost savings.

The 2044 Franklin St structure (Figure 2) uses state of the art engineering techniques and is the first highrise structure that offers faster construction for both gravity framing and the seismic force resisting system (SFRS), while maintaining high performance in a severe seismic zone. Modular highrise using volumetric construction has been attempted previously using traditional structural systems such as braced frames in the 32 story Atlantic Yards project in New York and concrete core in the 29 story SOHO tower in Darwin, Australia. However, it has proven problematic with the use of volumetric construction with higher tolerance demands.

Figure 2. 3D Rendering © SDG/ RAD Urban

The combined use of a SPSW system with volumetric construction is projected to reduce the construction schedule by at least 6 months compared to a traditional concrete core system (Figure 3). Using a SPSW core as opposed to traditional concrete core reduced the structural occupied area by 2%, allowing for more architectural flexibility and additional rentable area.

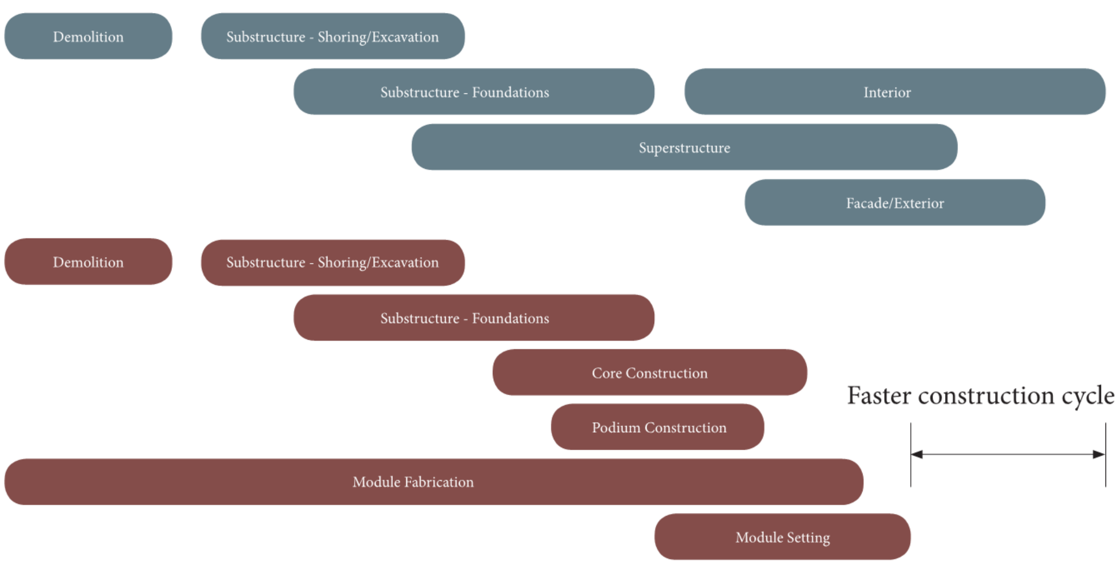


Figure 3. Construction Schedule Benefits (Top: Conventional Construction Process, Bottom: Volumetric Construction) © RAD Urban

STRUCTURAL SYSTEM

2044 Franklin St is located at the intersection of Franklin St., 21st St. and Webster St. in Oakland, CA. The site is approximately 25,000 SF and consists of two buildings. Building one is a 29 story, mixed-use tower with six lower levels designated for commercial use and the upper 22 stories are residential units. Building two is a 3 story low-rise. The ground floor consists of 4,000 SF of retail space. Level two and three include town homes. Building two uses conventional construction methods.

Super-Structure

The proposed tower superstructure system is a non-prescriptive design approach. The SFRS consists of ductile steel plate shear walls supplemented by BRB outriggers on level 7 and level 30 (Figure 4). Steel plate shear walls are expected to range between 5/8" and 1/4" with yield strength of 36 ksi. There are partial-height and full-height openings in the steel plate shear walls for doorways and corridors. Sufficient local boundary elements are implemented around the openings to provide necessary strength and stiffness and to enhance ductile behavior.

The gravity system of the superstructure is a combination of conventional composite steel framing and volumetric steel modules (Figure 5 and Figure 6). The floor plates on the lower office levels consist of perimeter steel girders that span between W14 columns and W18-W24 secondary composite beams spaced at 10 feet O.C. spanning between the primary W24 girders and the central cores. The composite steel-framing and slab system typically consists of a 3" lightweight concrete-fill over a 3 1/4" metal deck for a total depth of 6 1/4".

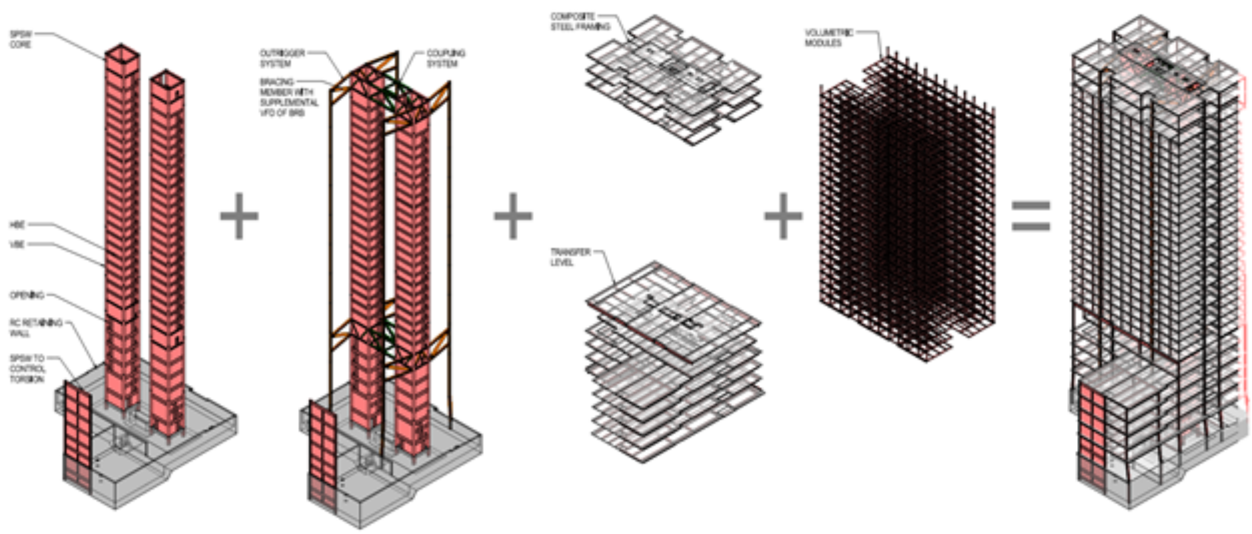


Figure 4. Exploded View of Structural System © RAD Urban

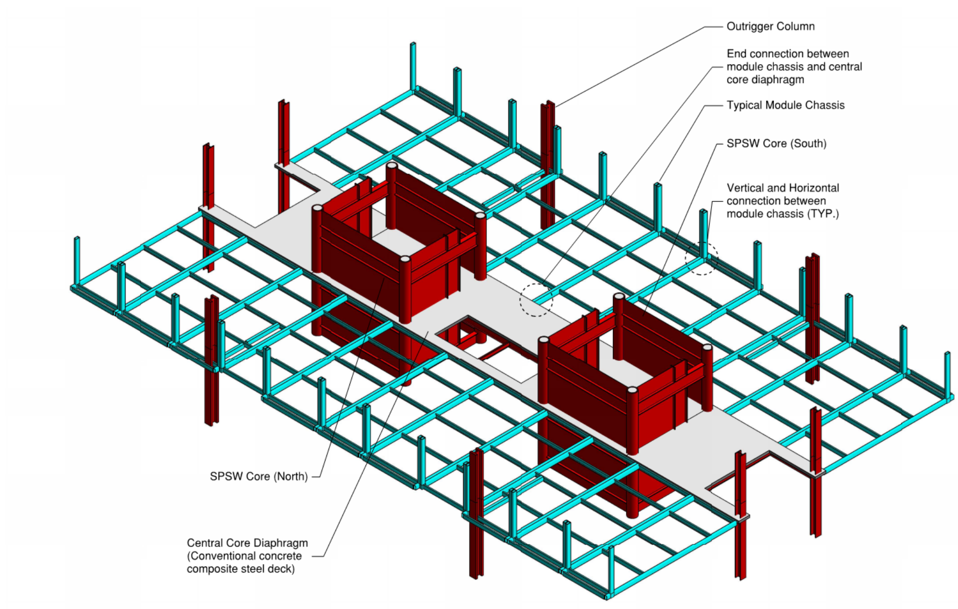


Figure 5. Typical Floor Framing (Volumetric Construction) © RAD Urban

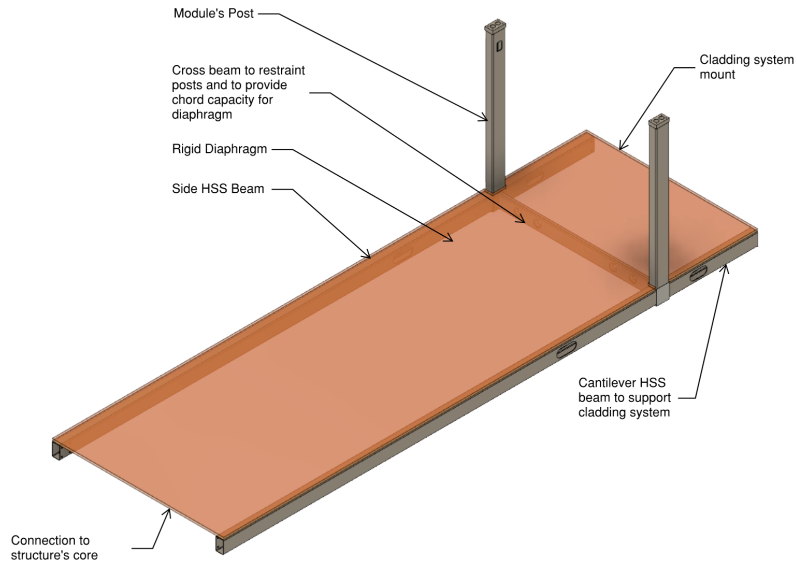


Figure 6. Typical Module Chassis © RAD Urban

Sub-Structure

A 14’ tall basement is the only subgrade level. The primary use of the basement level is resident parking but heavy mechanical equipment and water tanks may also be located here.

The foundation system will consist of an 8-10 ft thick cast-in-place reinforced concrete. A conventional cast-in-place 18-inch-thick reinforced concrete perimeter wall surrounds the foundation and the building footprint.

USE OF PERFORMANCE BASED DESIGN (PBD)

Several code exceptions related to the structure require the Structural Engineer of Record (SEOR) to utilize performance-based design rather than code-based design. The code exceptions of the project are summarized below:

- Building Height
- Non-traditional Volumetric Modular Structures for residential levels
- Steel infill plate aspect ratio
- Mass Irregularities
- Re-entrant Corners and vertical geometry irregularities

As part of PBD process, an independent Structural Peer Review Panel (SPRP) was introduced into the project, which provided invaluable feedback and comments to the SEOR. Through the PBD process, the team reduced the structural material usage to satisfy projected global performance objectives (Table 1). Both material and geometric nonlinearity were incorporated in the modeling phase to capture a more realistic structural behavior during severe seismic events (Figure 7). The energy dissipation mechanism of the lateral system was optimized under a suite of seven ground motions to enhance the overall structural performance.

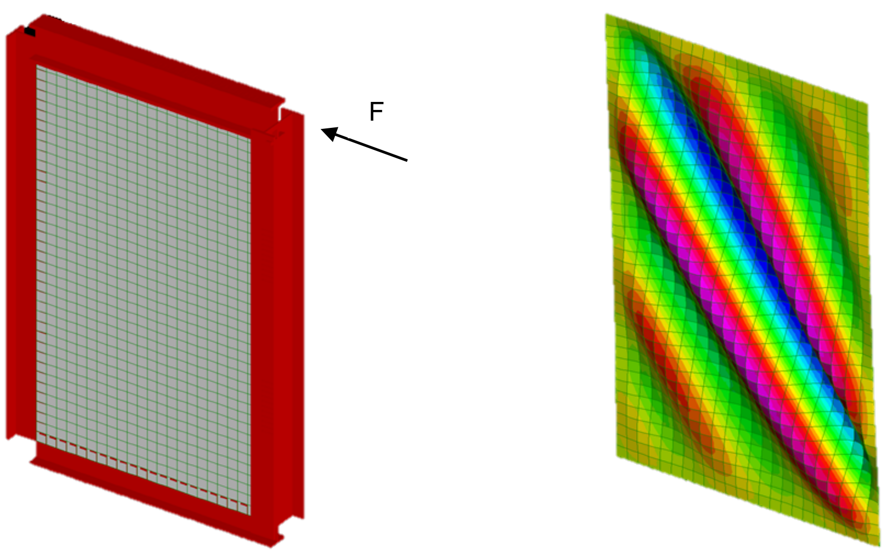


Figure 7. Steel Plate Shear Wall Tension Field Action Modeled in Strand7

Table 1. Global Performance Objective

Performance Level	Performance Objective	Design/Analysis Procedure	Analysis Method
Service Level Earthquake (43-year mean return period 50% probability of exceedance in 30 years)	*Immediate Occupancy:* Minimal structural damage that is repairable. Possible disruption of non-essential services.	1. Secondary serviceability check that uses a 50% exceedance in 30 yr. spectrum. 3. Strength Demands per PEER TBI Document (2010) (D+.25L+E) 4. Inter-story Drift Limited to 0.5% per PEER TBI (2010). The buckling threshold for steel infill plates varies across the height based on plate thickness and height. This is shown in Volume II, Schematic Design Calculation package.	• The analysis is based on linear Response Spectrum Analysis (RSA). • The steel infill plates are modeled using *Orthotropic* shell elements (due to low buckling threshold for upper thin plates). • The appropriate stiffness modifiers are used for concrete elements and un-topped corrugated steel deck. • 2.5% damping
Design Earthquake (475-year mean return period 10% probability of exceedance in 50 years)	*Life Safety*: Moderate damage to structure that may require extensive repairs	1. Site Specific response spectrum developed for 10% probability of exceedance in 50 years. 2. Strength demands per load combinations of IBC 2012 and ASCE 7-10. 3. System over strength will not be used (instead will be verified by MCE level)	• Linear Response Spectrum Analysis • using *orthotropic* shell elements (SPSW is expected to buckle) and • appropriate stiffness modifiers for un-topped corrugated steel deck. • Steel plates were modeled as shell elements, with rotation and stiffness modifiers applied. • 5% damping
Maximum Considered Earthquake (2475-year mean return period 2% probability of exceedance in 50 years)	*Collapse Prevention:* Extensive damage to the structure that may or may not be economically feasible to repair. Collapse is prevented.	1. A suite of 7-time history pairs scaled to the MCE response spectrum according to the methods contained in ASCE 7, Section 17.3.2 as recommended by the LATBC Consensus document (2005) Section 2.3.1.3. "1.5 Mean was used element. 2. Load Combination per PEER TBI Document (2010) (D+.25L+E) 3. 5% accidental eccentricities not required 4. Inter-story Drift Limited to 2.75%	• Nonlinear Response History Analysis (NLRHA) • Steel Plate strips modeled using strip method per AISC 341-SDG20 and calibrated to Driver. et al test results. • un-modeled equivalent Viscous Damping is 0.5% plus a small amount of Rayleigh for numerical stability of the P3D model.

The selected global structural performance metrics and local member behavior are shown below to demonstrate that performance objectives were met (Figure 8 to Figure 14). The performance-based design metrics aided in the volumetric module design process since the modules must maintain compatibility with the lateral system as force-controlled elements.

The hysteresis behavior for the shearwall plates and BRBs were critical for accurate prediction of the structural response due to ground motions. The shearwall plate modeling is based on the “strip” approach per AISC Design Guide 20. When a “strip” is unloaded after an incursion into the inelastic range, a “strip” exhibits residual deformations and tensile stress will not develop in the next cycle in the same direction until a minimum deformation is reached. BRB hysteresis behavior, however, must exhibit non-symmetric bilinear hysteresis behavior with no observable loop pinching.

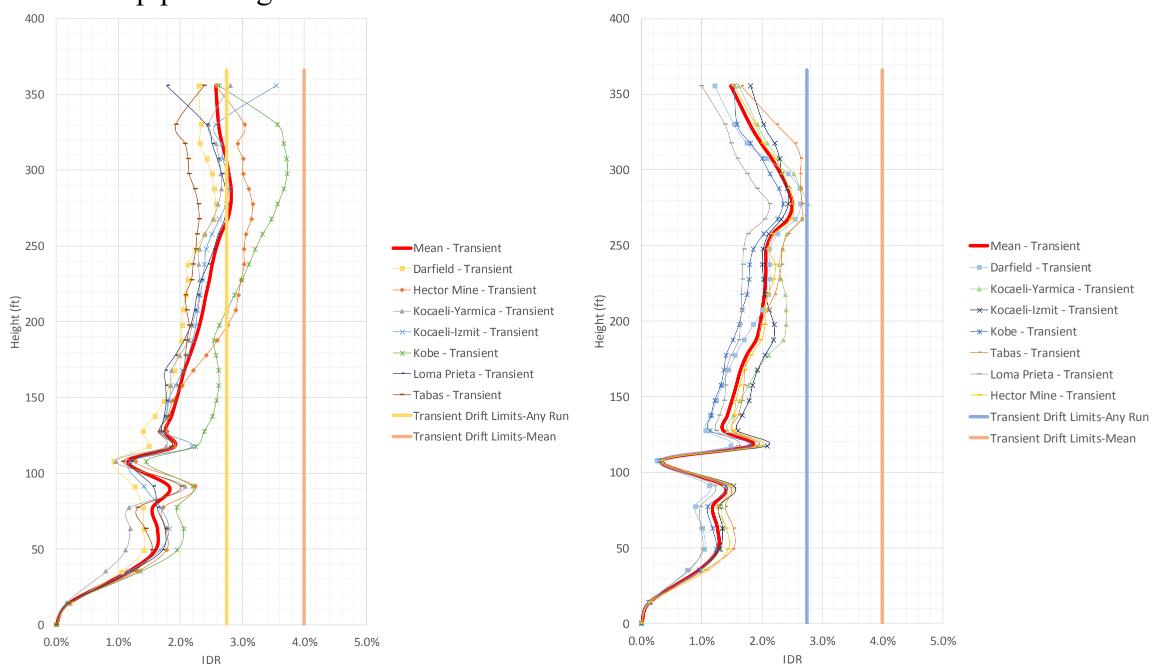


Figure 8. E-W (left) and N-S (right) Direction Transient Inter-Story Drift Ratio

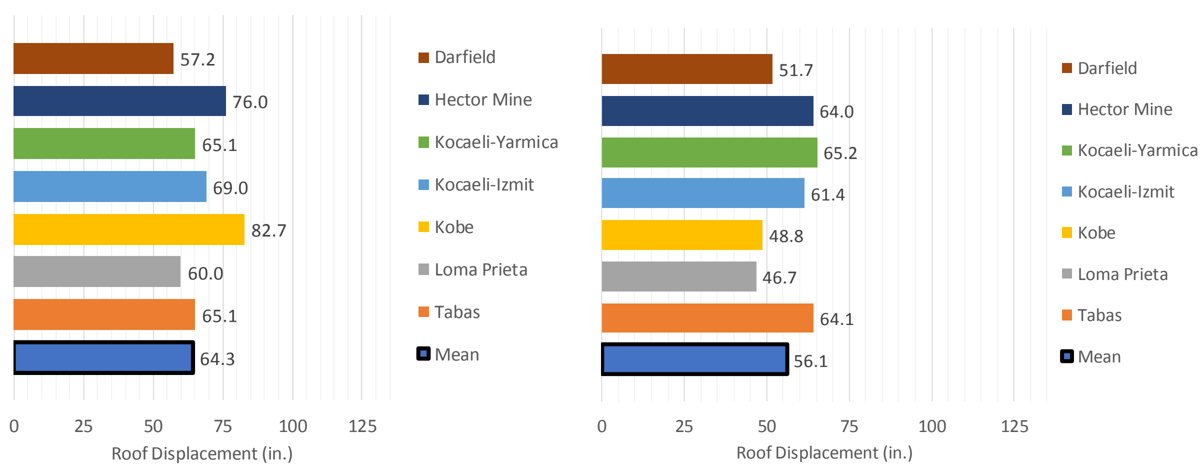


Figure 9. E-W (left) and N-S (right) Direction Roof Displacement

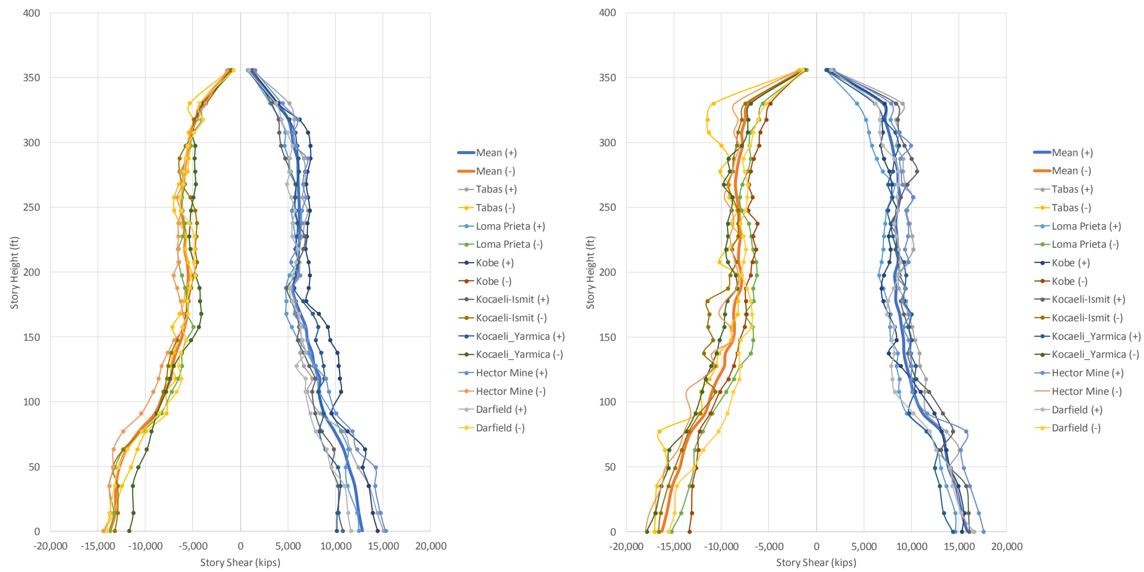


Figure 10. E-W (left) and N-S (right) Direction Story Shear

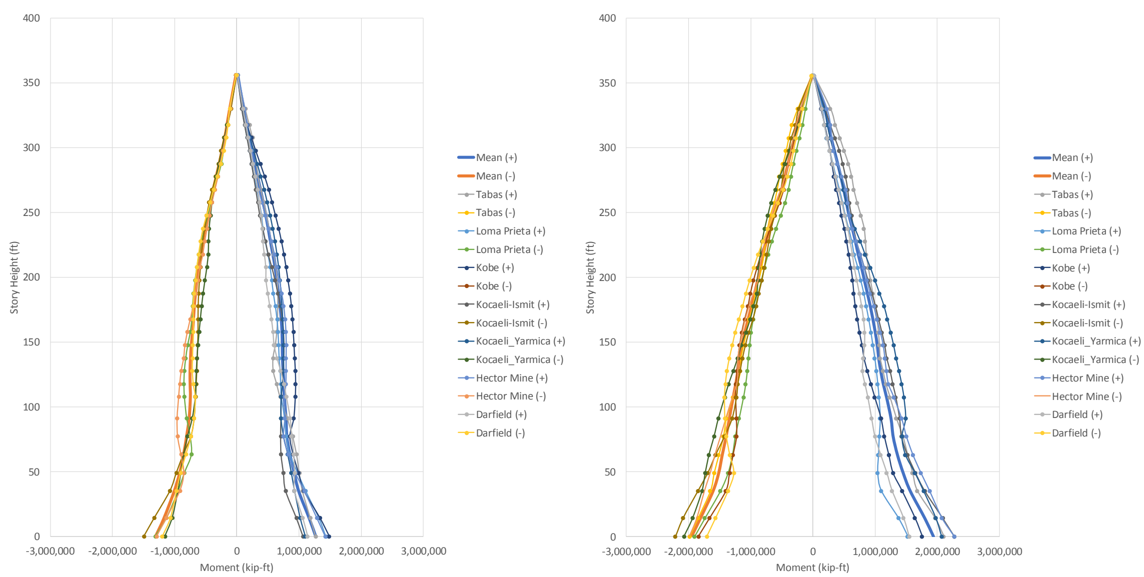
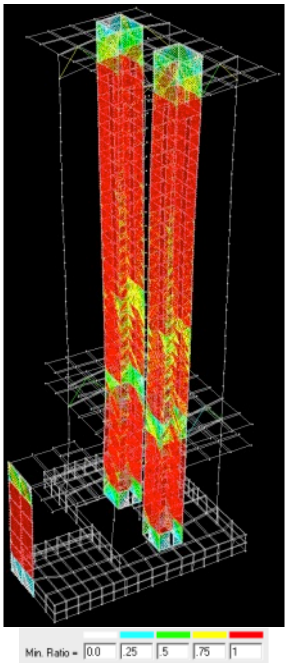
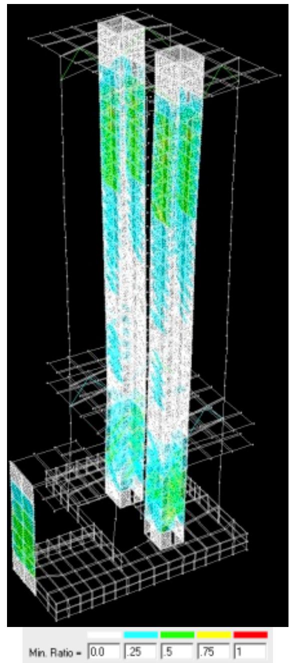


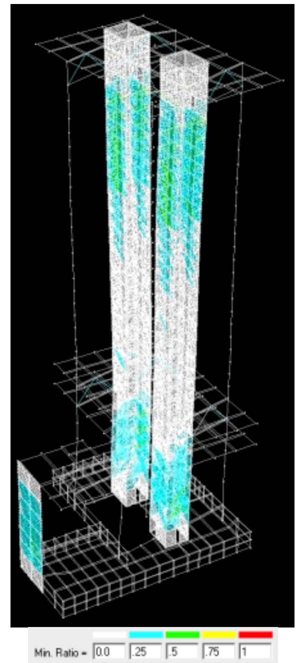
Figure 11. E-W (left) and N-S (right) Direction Overturning Moment



All Immediate Occupancy



All Life Safety



All Collapse Prevention

Figure 12. Tabas Ground Motion Yielding at the End of the Ground Motion

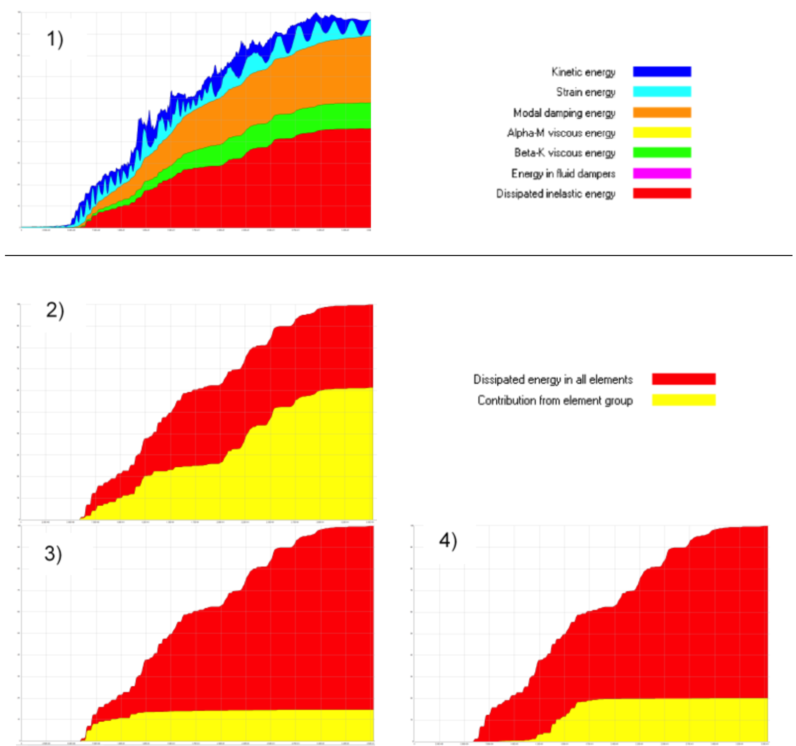
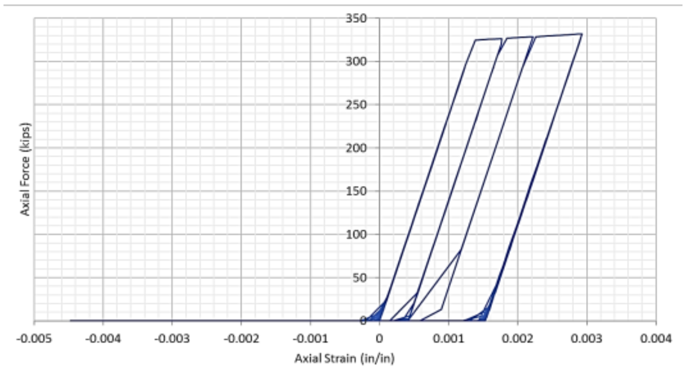


Figure 13. Tabas Ground Motion: 1) Total Energy Balance, 2) BRB Inelastic Energy Balance, 3) E-W Strip Inelastic Balance and 4) N-S Strip Inelastic Energy



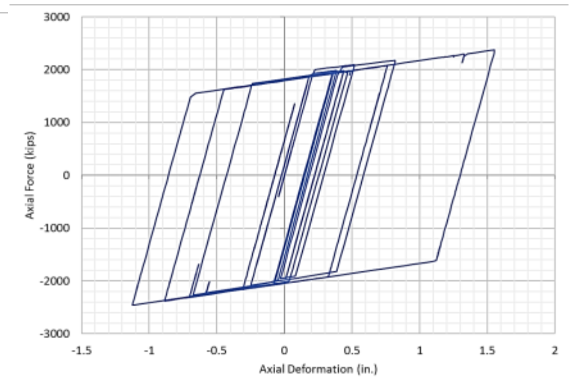


Figure 14. Typical SPSW Strip (left) and BRB (right) Hysteresis Loop

FULL SCALE TESTING AND CALIBRATION

Due to the uniqueness of the connections and assemblies used in the volumetric modules, especially in a high-rise context, it was essential to use both experimental and analytical models for structural evaluation. The models together verify the module's performance and provide confidence in the module's structural integrity under severe seismic excitations. Monotonic and cyclic tests of several structural sub-assemblies, agreed upon with the SPRP, were conducted in collaboration with independent structural testing laboratories (Figure 15 to Figure 22).

Once experimental results were gathered and processed, analytical models in both Strand7 and Perform 3D were calibrated to more accurately match the experimental results.

Figure 15. Full-Scale Column Splice Flexure (Strong-Axis) Test Setup
(Max. 2.75% MCE Drift) © RAD Urban

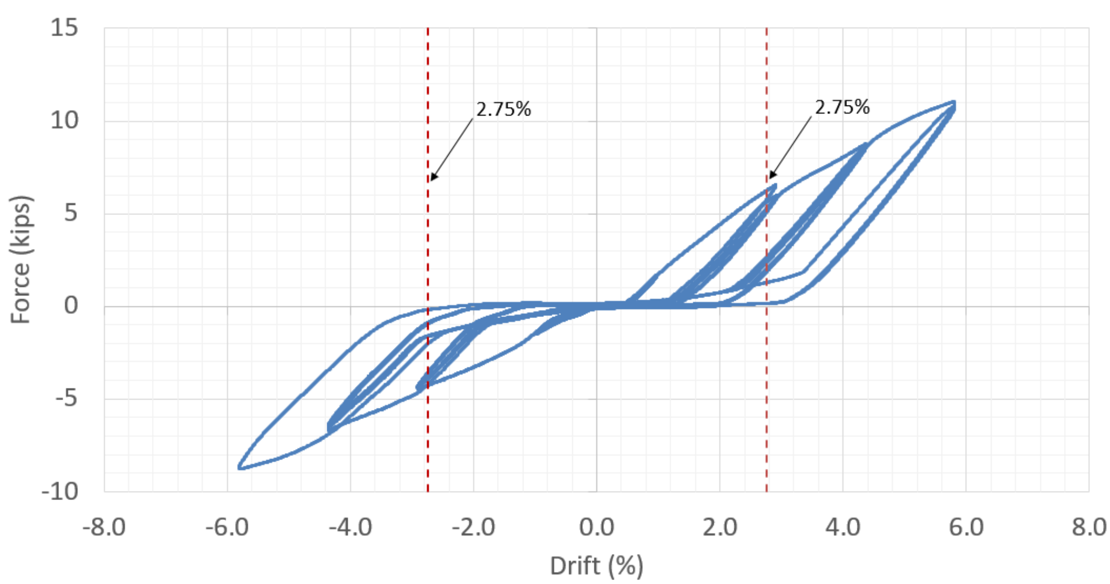


Figure 16. Full-Scale Column Splice Flexure (Strong-Axis) Test Hysteresis © RAD Urban

Figure 17. Full-Scale Column Splice Flexure (Weak-Axis) Test Setup (Max. 2.75% MCE Drift) © RAD Urban

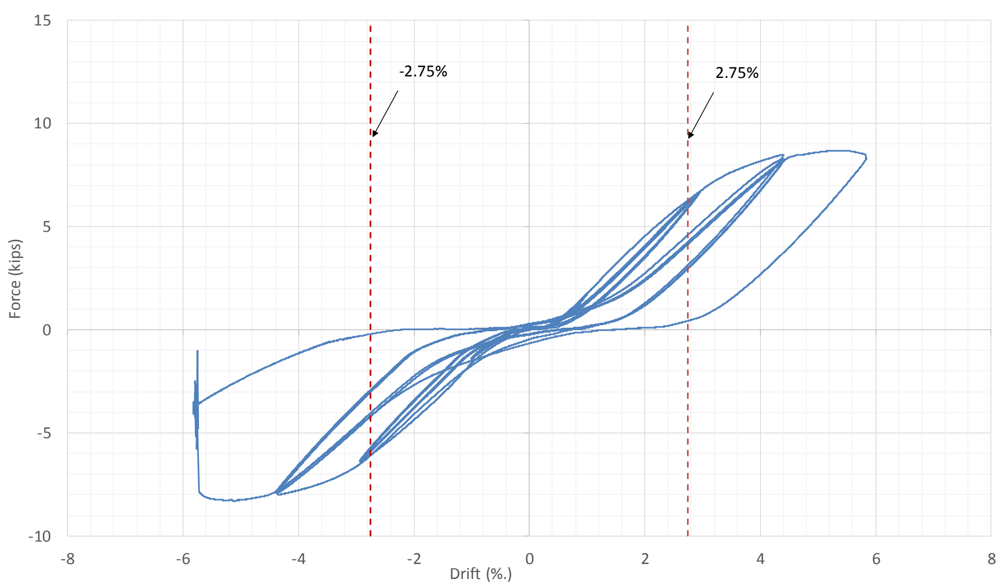


Figure 18. Full-Scale Column Splice Flexure (Weak-Axis) Test Hysteresis © RAD Urban

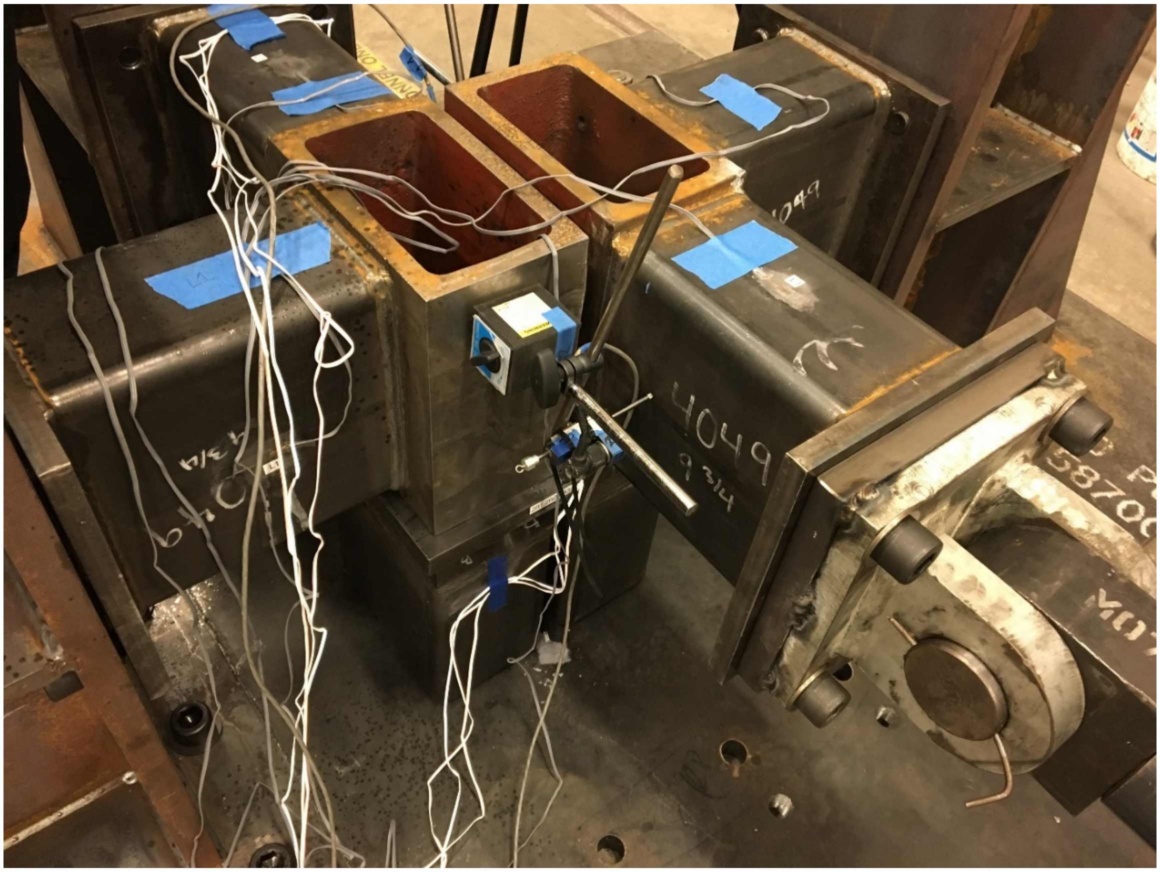


Figure 19. Full-Scale Module-to-Module Connection Shear Force Test Setup
© RAD Urban

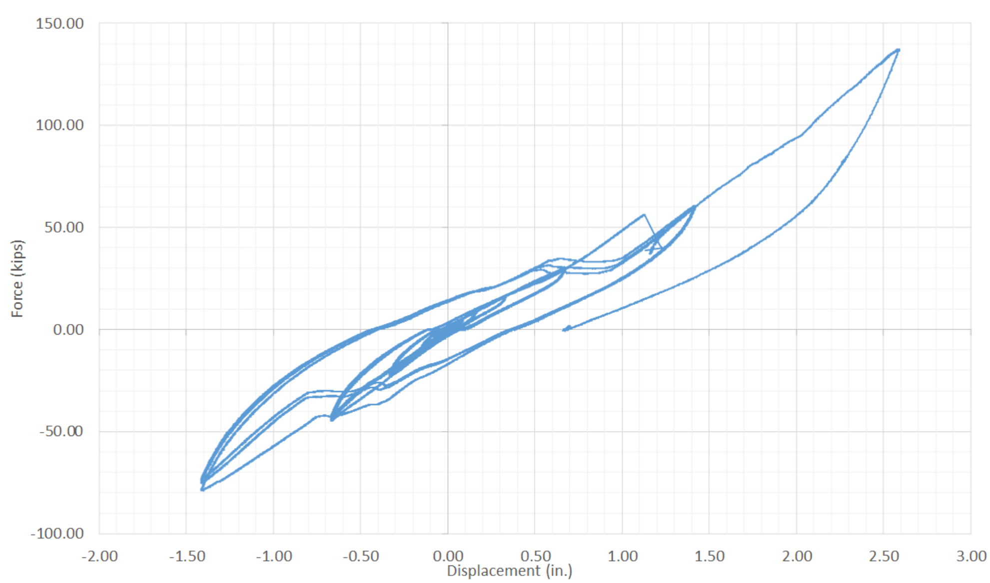


Figure 20. Module-to-Module Connection Shear Force Hysteresis © RAD Urban

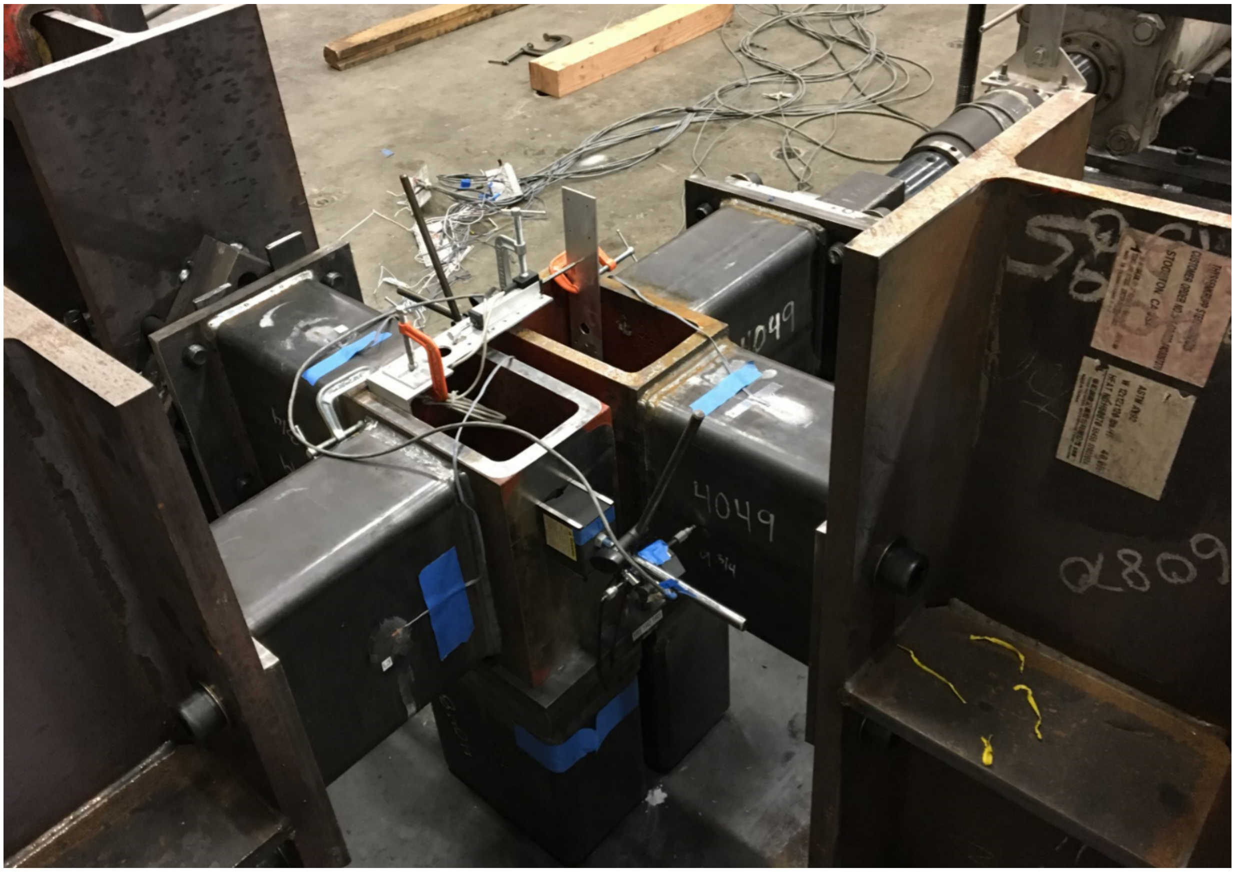


Figure 21. Full-Scale Module-to-Module Connection Axial Force Test Setup © RAD Urban

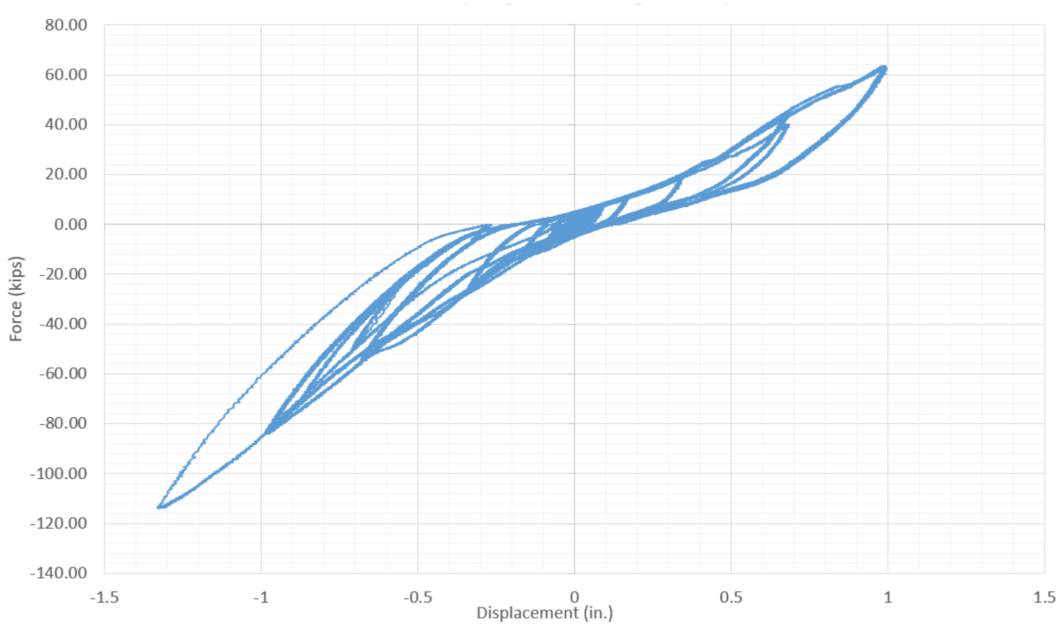


Figure 22. Module-to-Module Connection Axial Force Hysteresis © RAD Urban

Perform 3D Finite Element Analysis (FEA) Results

Perform 3D was used to assess the modular system's structural behavior when combined with the tower's lateral system of SPSW cores and BRB outriggers. Acceptance of the modular system is confirmed if the discrete modular connections remain force-controlled under an MCE event and are accurately calibrated to the experimental results. It was decided to analyze the modular system by using the sub-structuring method instead of modeling a superstructure with 22 modular levels. The subassembly approach offered reduced computational runtime of up to one week and faster module design iterations without losing much accuracy.

The subassemblies are modeled using combinations of line elements and semi-rigid hinges. Semi-rigid hinges represent the discrete modular connections, which are represented analytically as combinations of moment, shear and axial hinges in a parallel configuration (Figure 23).

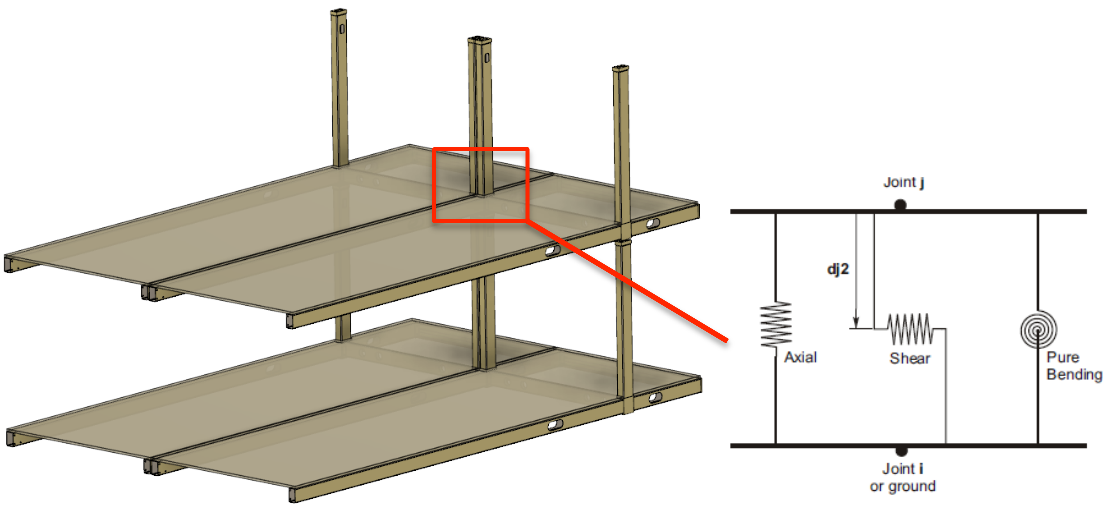


Figure 23. Multi-linear springs in between module stacks (Source: CSI)

Strand 7 FEA Results

Critical connections in-between modules were modeled and studied thoroughly in Strand7 (Figure 24). This was essential to establish loading protocols for full scale testing.

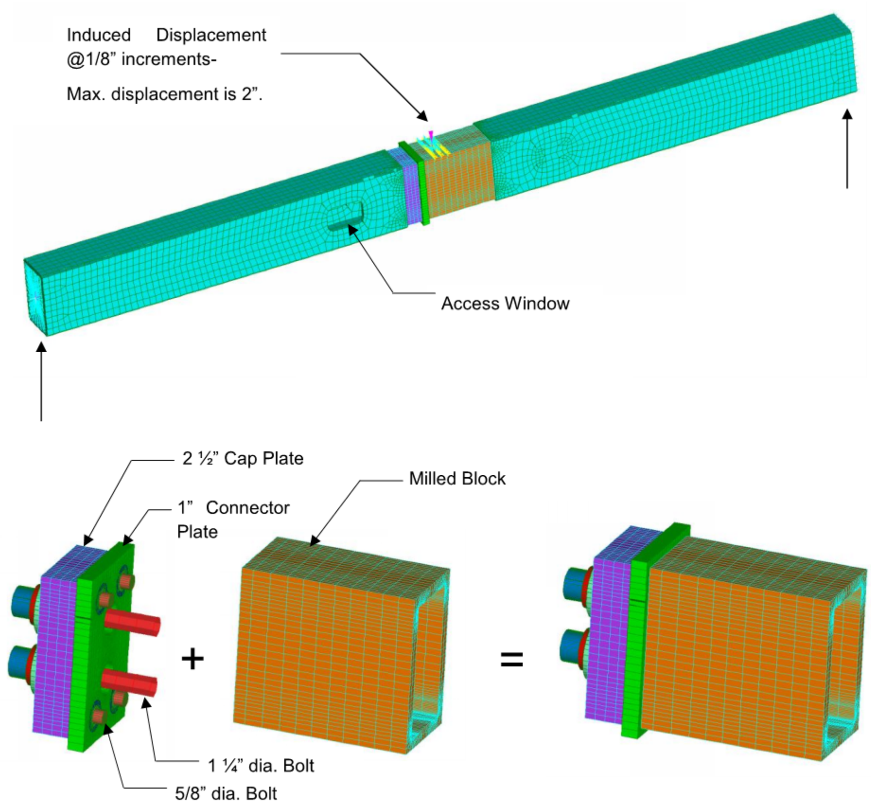


Figure 24. Module Chassis' Column Splice FEA Study – 3 point flexural test (Strand 7)

ENERGY DISSIPATION OPTIMIZATION

As mentioned previously, the tower relies on three inelastic energy dissipation mechanisms in response to seismic actions: 1) SPSW infill plate tension field action, 2) buckling restrained brace axial yielding, and to a lesser extent, and 3) SPSW horizontal boundary element (HBE) moment hinging. Non-linear response history (NLRH) parametric studies revealed that the optimal SPSW configuration utilized a tiered design of HBE and infill plate sizes based on yielding behavior. BRBs were configured in single diagonals at the outrigger levels to act as a "structural fuse" under an MCE event and prevent overstressing of outrigger columns. The single diagonal configuration is primarily based on the relative stiffness between the BRB and SPSW, as well as truss bay dimensions as evidenced in simplified models.

Parametric studies of smaller 1D and 2D models were used to optimize the BRB and SPSW structural behavior (Figure 25). Under a mock cyclic loading protocol it was possible to tune the energy dissipation of the system by adjusting the relative stiffness of the SPSW and BRB elements.

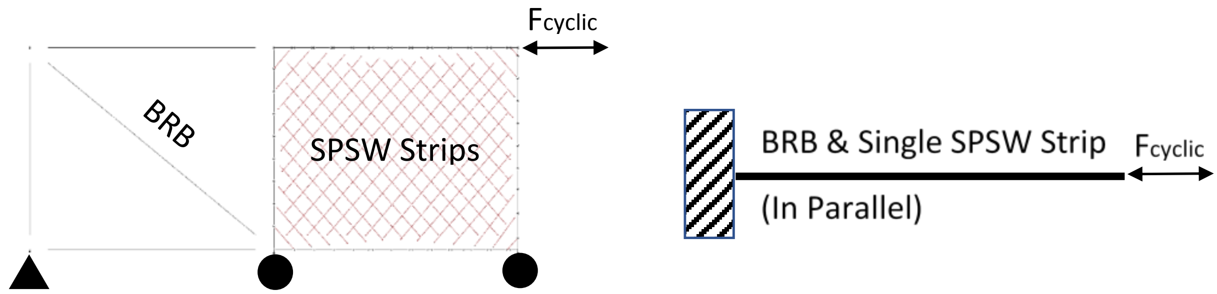


Figure 25. A two-dimensional P3D model considering adjacent bays of BRB and SPSW elements (left) and a one-dimensional P3D model connecting a BRB element and SPSW strip element in parallel (right).

Following the 1D and 2D studies, the full 3D superstructure was subjected to ground motions to study the effects of BRB configuration and number of elements on inelastic energy dissipation. The team determined that strips govern when subjected to significant seismic excitations of a given time history. This response is due to the strips' tension-only behavior where tensile stresses further develop only after surpassing residual deformations per AISC Design Guide 20. BRBs, however, tend to govern the energy dissipation typically after significant seismic excitations because BRB hysteresis account for bi-directionality, nearly symmetric hysteresis loops and no strength degradation. Based on these observations, the team adopted the following changes to optimize energy dissipation while balancing other global structural considerations, which include:

- Tiered vertical boundary elements (VBEs)
- Tiered SPSW steel plates
- Single BRB diagonal configuration for critical bays in the outrigger and coupling truss

CONCLUSION

The presented volumetric construction technologies and processes improved the construction speed and reduced the cost of construction. The unique structural system used in 2044 Franklin St will be a key contributor to the project success. Steel as the base material, in contrast to wood or concrete, allows for tighter tolerances and a relatively lightweight structure with a steel tonnage at quantities comparable to highrise structures in non-seismic zones. Careful consideration of factory and on-site tolerances were made to allow for compatibility between the two separately fabricated components. Construction efficiency will be achieved through offsite manufacturing that reduces lead times via concurrent scheduling. Because most of the fabrication will occur offsite, onsite construction will be limited considerably. Overall, construction costs may be reduced by up to 20%. Such savings would verify that volumetric construction is a cost-effective alternative to conventional construction. 2044 Franklin St could be used a paradigm for future modular highrise developments.

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